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Investigate Live Load Distribution and Stability of Prestressed Concrete Girders During Construction

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Abstract

Prestressed concrete girders are widely used in bridge applications in the state of Texas. While the girders have traditionally been utilized for simply-supported bridges, continuous-girder applications have been used recently by employing spliced-concrete girders that allow significant increases in the spans. However, the larger spans introduce increased girder slenderness that can introduce potential stability limit states that historically have not been an issue with simple-span systems. In addition, the larger spans of the girders can also have an impact the live load distribution of the girders. This report documents a comprehensive investigation on the stability behavior of prestressed concrete girder systems throughout the erection and construction process and also considers the live load distribution of the in-service bridges. Full scale laboratory experiments were conducted to measure the flexural and torsional behavior of the girders in both the uncracked and cracked state. The results from the laboratory tests were utilized to validate three-dimension finite element models that were used to consider girder stability throughout the erection and construction stages of the deck. Because cracks that might occur during transportation and erection are often closed from the prestressing, cracked section properties were utilized to assess the girder stability. The impact of diaphragms on the stability girder behavior and also live load distribution were studied. Recommendations are provided for diaphragm bracing, stability evaluation, and live load distribution in the girders.

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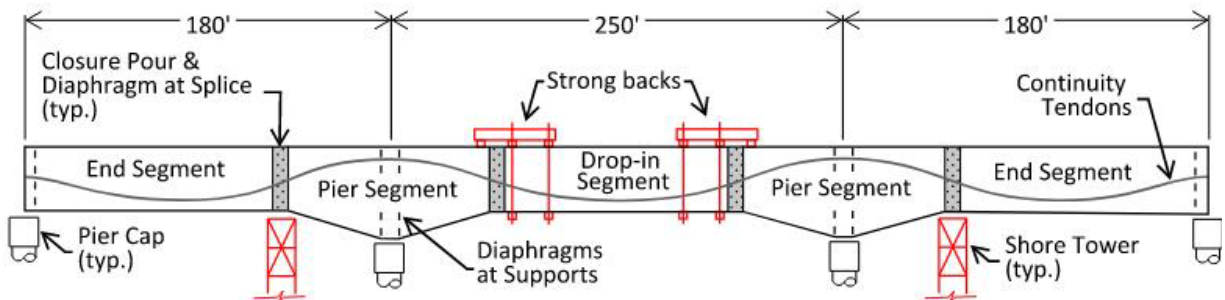
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Chapter 1. Introduction

Prestressed concrete girders are the most widely used structural system for bridge applications in the state of Texas. While the girders have traditionally been utilized for simply-supported bridges, continuous-girder applications have been used recently by employing spliced-concrete girders such as the applications shown in Figure 1.1. The spliced-girder applications generally make use of a constant depth prestressed girder (with ducts for post-tensioning) in the positive moment regions, along with a tapered section (with ducts for post-tensioning) at the interior piers. Although precast elements are used for the girder segments, during construction, a cast-in-place section is utilized to complete the splices. Once the girders are fully erected and the splices are cast, continuity tendons are threaded through the ductwork and post-tensioned, creating a continuous girder. While the spliced girder allows longer spans, another major benefit is the improved efficiency in moment distributions and stiffness created by the girder continuity.



(a) Main span of bridge under construction, Ge et al. (2019)



(b) Schematic of spiced girder bridge components

Figure 1.1. Construction of spliced girder bridge in Round Rock, TX

Cast-in-place concrete diaphragms are provided at intermediate locations (usually at the splice locations and support regions for the continuous girder systems), shown in Figure 1.2. During construction, the precast section requires temporary support while the splices and diaphragms are constructed. The temporary support can take the form of shore towers or strong backs that suspend a precast girder in the positive moment region from the cantilevered support sections. The bridge in Figure 1.1 made use of strong backs for temporary support; however, shore towers are also effective, depending on the availability of right-of-way beneath the bridge. There are advantages to both shore towers and strong backs. Shore towers generally provide the most stable platform that can provide both vertical and lateral stability; however, such temporary supports are time-consuming to erect and dismantle and require access beneath the bridge. If shore towers are utilized, the girder self-weight is supported by the shore tower during splicing activities, and the load is transferred into the continuous girder once the girders are post-tensioned and the towers are removed. Strong backs provide vertical support and depend on the cantilever pier section for lateral and torsional restraint, but have the significant advantage of not requiring access immediately below the girder. The strong backs are supported by the previously erected girder before the continuity splice is made, which creates additional load cases to be analyzed.



Figure 1.2. Cast-in-place diaphragms at splice locations

The use of spliced girders allows the clear spans for prestressed concrete girder bridges to be significantly increased compared to traditional applications of non-spliced precast concrete girders. However, with increases in span lengths, several questions arise regarding the design and construction behavior of these systems. In general, the spliced girders result in longer spans and also result in increases in girder slenderness. One area of uncertainty pertains to ensuring the stability of the girders throughout all stages of erection and construction. The necessity and design requirements of the diaphragms are directly related to the girder stability. In addition, another area

of uncertainty is the live load distribution behavior of a completed bridge with girder span lengths and boundary conditions outside of existing code specifications. This report documents Texas Department of Transportation (TxDOT) Project 0-7115, which focuses on assessing the stability of the prestressed concrete girder bridge systems during construction as well as the live load distribution in the completed bridge. The research study includes laboratory experiments and computational investigations to develop design guidance for performance during construction and in-service. The original project scope also included field monitoring; however, that effort was removed from the study scope due to complications with construction timelines for bridges with applicable girder types within the project duration.

The efforts of the research study are divided into the following chapters of this report:

- Chapter 1. Introduction
- Chapter 2. Background and Initial Studies on Lateral Torsional Buckling
- Chapter 3. Full-Scale Laboratory Testing
- Chapter 4. FEA Validation Studies of Full-Scale Laboratory Testing Related to Stability and Live Load Distribution
- Chapter 5. Parametric Finite Element Analysis (FEA) on Stability
- Chapter 6. Parametric FEA Studies on Live Load Distribution
- Chapter 7. Design Methodology
- Chapter 8. Summary

Chapter 2. Background and Initial Studies

2.1. Current Practice for Prestressed Girders in Texas and Other States

The Texas Department of Transportation (TxDOT) has heavily utilized composite, prestressed I- and U-Girders on its system since the introduction of the structure type in the United States. These bridge types are usually modular in nature for designers and offer quality, repeatability, and economy for fabricators and contractors. The structures tend to have high durability and low life-cycle costs in the Texas built environment. This is partially due to few annual freeze-thaw cycles, limited use of deicing chemicals, and lack of damage to joints that might occur in colder states due to snow removal equipment (snowplow impact). Individual girder lengths are often less than 150 ft due to shipping, handling, and site restrictions. There are generally four applications of these girder types in the United States:

- 1) Simple span construction between supports with compression seal joints.
- 2) Simple span construction with a continuous deck and crack control joint – current TxDOT practice for span lengths less than ~150 ft. The girders are designed as simple spans for both dead load and live load.
- 3) Simple span for dead load between supports and continuous for live load over supports. Continuity is established by a reinforced diaphragm between girder ends and longitudinal reinforcing steel in the deck. The joints are at the end of units or at abutments. Depending on the thermal movements of the bridge, they are often made integral or semi-integral at the abutments.
- 4) Spliced girder construction (prestressed and post-tensioned) where the structure is continuous for deck placement and live load. Ideally, the structure is continuous for the girder self-weight, but this is complicated by the erection procedure and secondary force effects from the post-tensioning. The girder cast-in-place splices are placed near the dead load contraflexure points of the bridge. A continuity tendon(s) is then run throughout the bridge and post-tensioned.

Typical elevation view details of Applications 1 and 2 are provided in Figure 2.1 and Figure 2.2 – as presented by the Precast Concrete Institute (PCI) *Bridge Design Manual* (2011).

theoretical pin support, i.e., no flexural restraint is provided by the pier to the superstructure. Recent examples of Application 2, simple span and continuous deck, are shown in the following figures. This structural detail is often referred to as “poor-boy continuous girders.”



Figure 2.3. Common overpass in Austin, Texas, using simple span and continuous deck construction



Figure 2.4. Complex multi-level interchange in Austin, Texas, with large portions using simple span and continuous deck construction

Application 4, spliced concrete girder construction, is a relatively new application within the state of Texas. This application usually requires a significant amount of temporary works and phased-construction analysis. However, this application can result in significant increases in available span lengths for concrete girder systems. These systems have been successfully utilized to achieve longer spans for concrete girders. Most applications of segmental construction consist of

precasting the full cross-section of the bridge and post-tensioning the segments together to achieve longer spans. In spliced girder applications, the girder segments are cast in lengths that can be transported to the bridge site using conventional transportation modes. The individual segments are then spliced together by threading prestressing strands through ductwork in the precast segments. The current maximum span of this construction type in Texas is the Matagorda Bridge in Sargent Beach. The Matagorda Bridge main unit is a three-span bridge with the following span lengths: 195-ft – 300-ft – 195-ft. Figure 2.5 presents the entire structure and main navigation channel. The increase in span length has potential savings in these areas (project dependent):

- Foundations & Substructure Cost
- Right of Way
- Maintenance of Traffic
- Superstructure Cost
- Construction Schedule
- Maintenance & Inspection



(a)



(b)

Figure 2.5. Sargents Beach Bridge (a) layout of entire bridge, (b) main span of bridge during construction, Modjeski & Masters (2021)

The TxDOT current standards for spliced girder bridges allow the main span lengths for spliced concrete girders up to 340 ft. The *FHWA Post-Tension Box Girder Design Manual* (Corven, 2016) summarizes the range and application of concrete girder bridges and a summary plot is shown in Figure 2.6. TxDOT has sponsored a number of research projects investigating aspects of spliced concrete girder use. Hueste et al. (2012) investigated the girder behavior and provided analysis, design, and standard detail recommendations. Hueste also presented a typical construction sequence for these structures – see Figure 2.7.

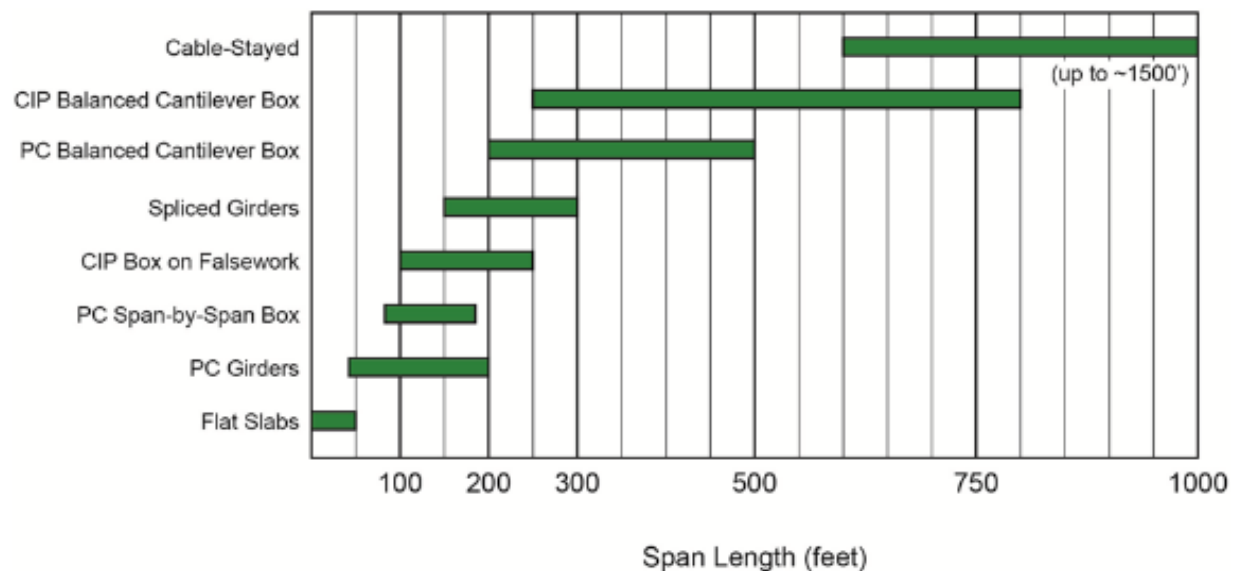
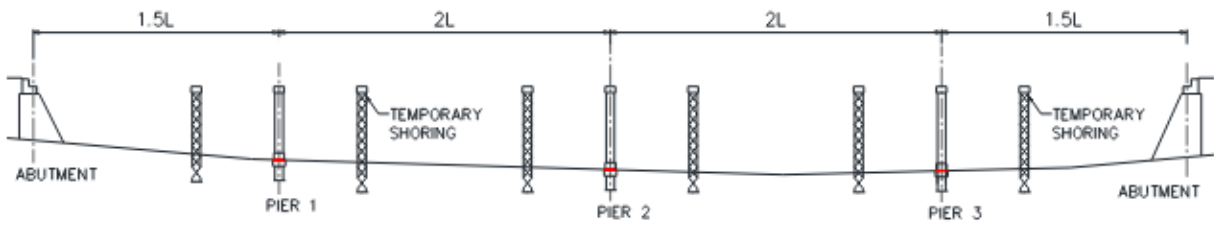
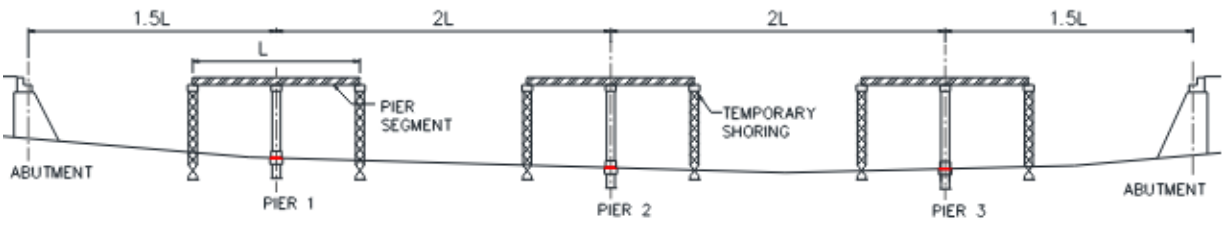


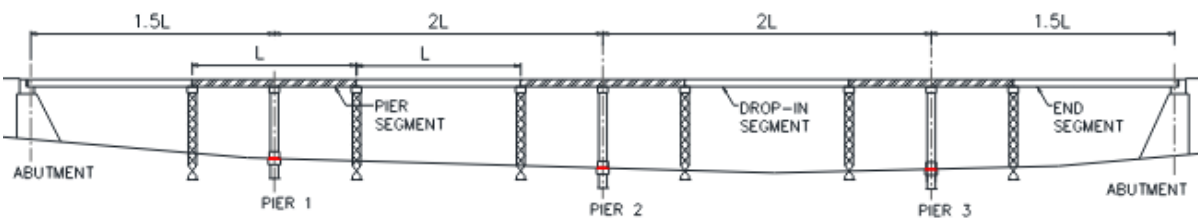
Figure 2.6. Typical span ranges for prestressed concrete bridge types, Corven (2016)



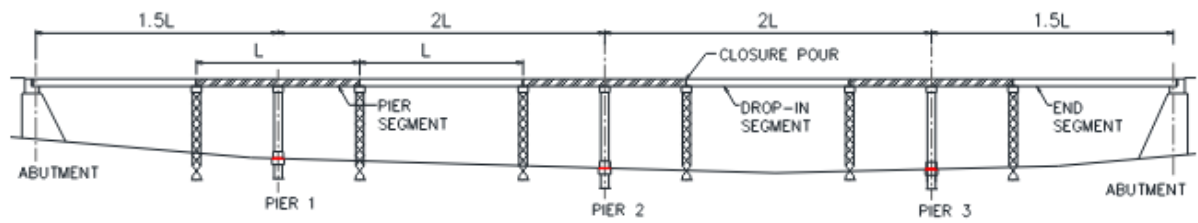
(a) Erection of Piers and Temporary Shoring Towers



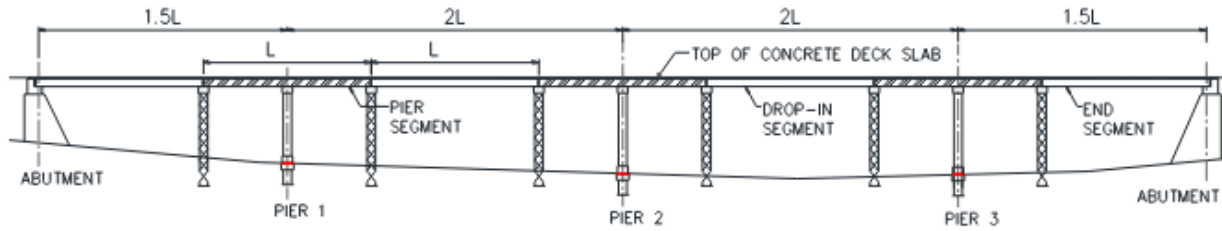
(b) Erection of Pier Girder Segments



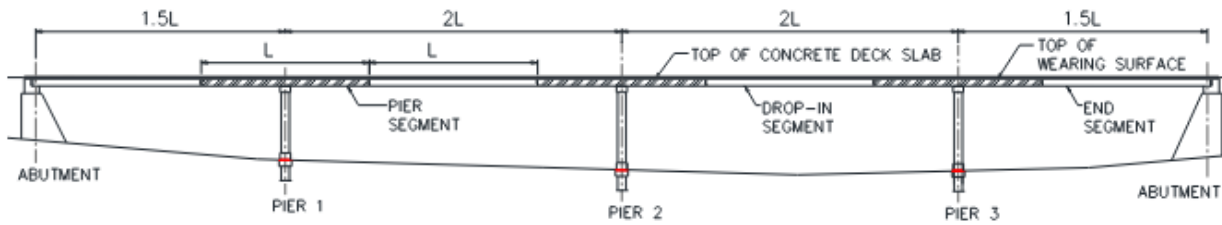
(c) Erection of End and Drop-in Girder Segments and Stressing Stage 1 Post-tensioning Tendons



(d) Casting Splices



(e) Casting Deck Slab and Stressing Stage 2 Post-tensioning Tendons



(f) Removal of Temporary Shoring Towers, Casting Barriers, Laying Wearing Surface and Opening Bridge to Traffic

Figure 2.7. Construction sequence of a continuous spliced girder bridge, Hueste et al. (2012)

Another noteworthy work is NCHRP Report 517, *Extending Span Ranges of Precast Prestressed Concrete Girders* (Castrodale and White, 2004). Like the later work of Hueste et al. (2012), they investigated the techniques and behaviors of spliced concrete girder bridges. NCHRP Report 517 did a broad international survey of the use of extended prestress girders or spliced girders. Table 2-1 shows a summary of the data for the U.S. and Canada.

Table 2-1 Use of extended span prestressed girder construction after NCHRP Report 517

Region	Total Bridges	Constructed by Decade					
		1950's	1960's	1970's	1980's	1990's	2000's
West	26	0	0	0	1	19	6
Midwest	20	0	0	2	5	8	5
Northeast	23	1	3	3	1	6	9
Northwest	41	0	2	3	7	23	6
Far west	20	0	1	6	0	5	8
Southeast	51	0	2	3	9	19	18
Total US	<u>181</u>						
Total Canada	67	0	0	3	31	24	9

Note: Information recorded through 2004

The state of Texas currently has more than nine extended-span spliced girder bridges in service or being built. These types of structures often require specialty equipment and experienced contractors who have worked in the state or region on similar projects. One major contributor to extending the span range of these structure types is tapering or haunching (increasing depth) the girder near the interior supports of the superstructure, much like what is done with steel welded plate girders. This technique was used on a bridge in Round Rock, Texas – schematic elevation shown in Figure 2.8.

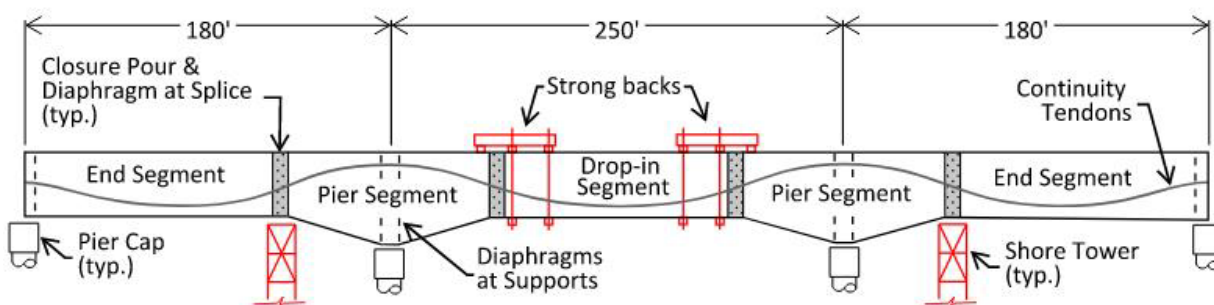
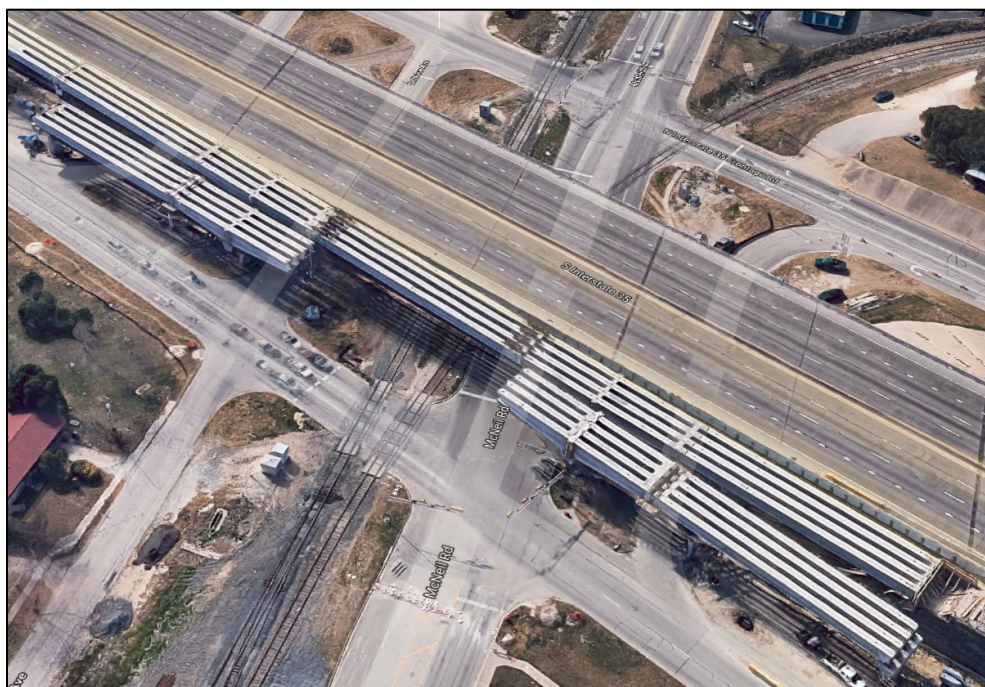


Figure 2.8. Round Rock, Texas bridge schematic layout

As presented in Figure 2.8, the erection procedure, temporary loading conditions, and falsework requirements can be very involved. Additional pictures during construction are shown in Figure 2.9.



(a) Elevation of main span during construction, Ge et al. (2019)



(b) Aerial view of the bridge during construction from Google Maps/Earth

Figure 2.9. Construction of the Round Rock Bridge and its complexity

The effect of non-prismatic (haunched) sections can significantly alter the stiffness of a structure and its response to loads. Figure 2.10 illustrates this for a very simple case including i) simple spans, ii) prismatic continuous girders, and iii) haunched continuous girders. The loading is a uniformly distributed load (UDL) of 1 kip/ft. No construction stage analysis or varying of member

self-weight is considered. The section is proportioned as a doubly symmetric I-Girder, and only the depth of the web is varied.

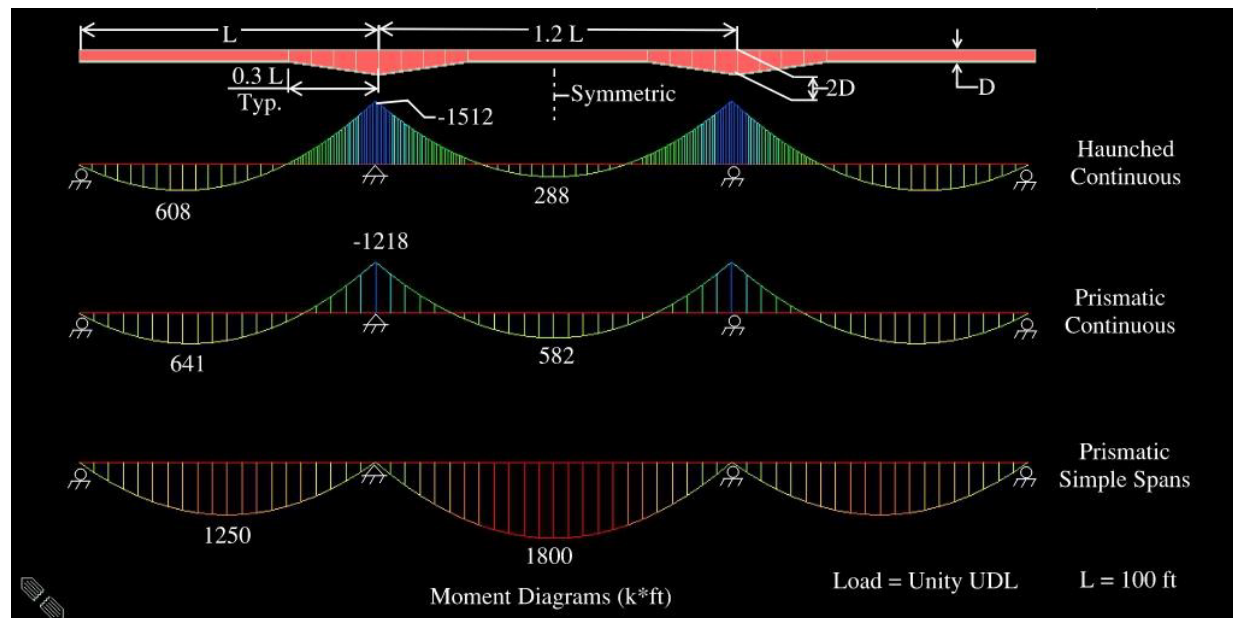
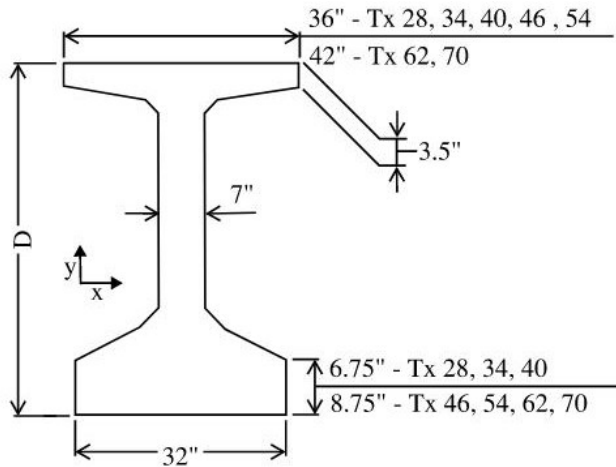


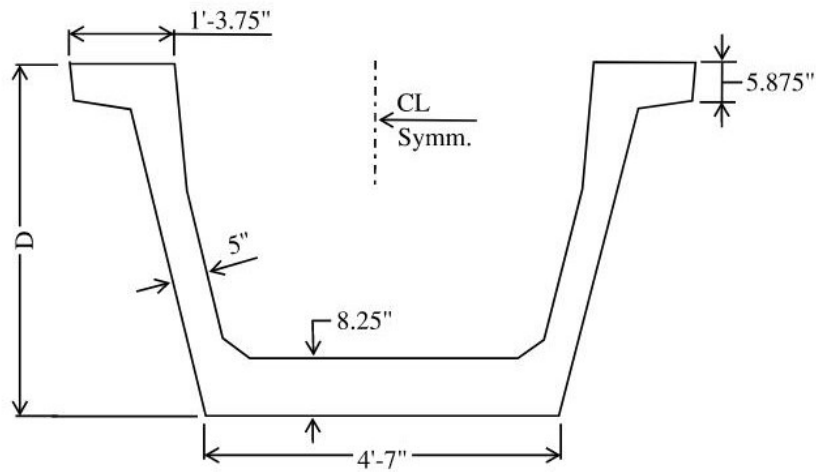
Figure 2.10. Moment diagrams of differing bridge continuity cases

Current TxDOT I and U-girder standard shapes and sizes are provided in Figure 2.11. Recent implementation of extended span lengths by TxDOT has resulted in the I and U-girder shapes and sizes shown in Figure 2.12. Where the STx denotes spliced concrete girders and HSTx denotes haunched spliced concrete girders. Comparing newly developed I and U-girder shapes with the historically used AASHTO Type I-VI Girders shows the general trend of wider and stockier flanged sections. This, in part, has led to increasing span lengths. Additional reasons are:

- Higher Concrete & Steel Material Strength
- Contractor and Engineer Familiarity
- Empirical Evidence of the Structural Performance
- Economics
- New Erection Practices
- QA/QC Improvements by Fabricators



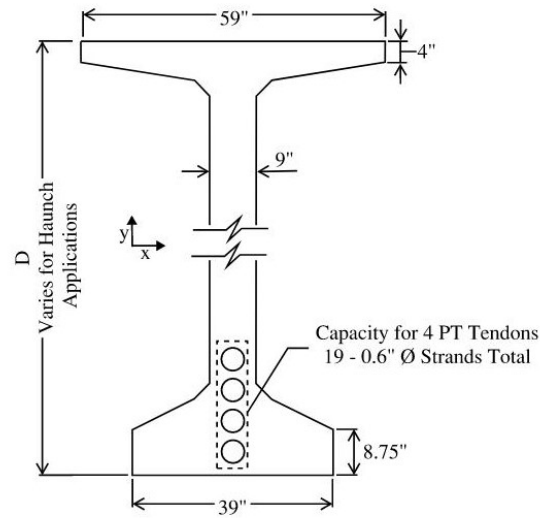
Tx I-Girder Dimensions



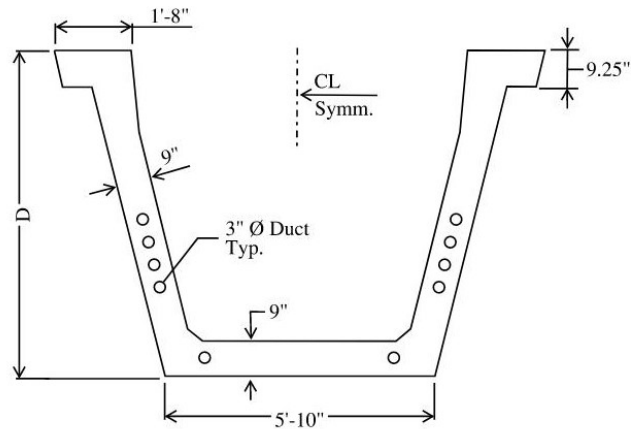
Tx U-Girder Dimensions

Girder Type	D in.	Area in. ²	I _x in. ⁴	Weight lb/ft
Tx28	28	585	52,772	630
Tx34	34	627	88,355	675
Tx40	40	669	134,990	720
Tx46	46	761	198,089	819
Tx54	54	817	299,740	880
Tx62	62	910	463,072	980
Tx70	70	966	628,747	1,040
U54	40	978	183,108	1,206
U40	54	1,120	403,020	1,055

Figure 2.11. TxDOT Standard. I and U-Girder shapes



Tx Long Span I-Girder Dimensions



Tx Long Span U-Girder Dimensions

Girder Type	D in.	Area in. ²	I _x in. ⁴	Weight lb/ft
Tx84	84	1,358	1,338,646	1,509
STx7	84	1,442	1,390,146	1,602
STx8	96	1,550	1,937,216	1,722
STx9	108	1,658	2,595,808	2,232
HSTx12*	147	2,009	5,600,633	2,442
HSTx14*	168	2,198	7,834,345	2,682
HSTx16*	192	2,414	10,983,036	1,842
U72-9	72	2,037	1,242,461	2,263
U84-9	84	2,260	1,862,691	2,511
U96-9	96	2,482	2,645,338	2,758

*Measured at pier

Figure 2.12. TxDOT long span I and U-girder shapes

2.2. Current Practices for Stability During Construction

2.2.1. Typical Causes of Failures and Examples

While many structures containing PS elements have been built and in service since the late 1940's in the U.S., there have been a number of failures and collapses of prestress girders in modern practice. The most critical period for these structural elements often occur in the following stages:

1. Releasing PS Strands and/or PT Jacking
2. Lifting Out of Forms
3. Transporting, Shipping and Lifting
4. Temporary Conditions (bearings, bracing etc.) and Loadings (wind, thermal, Const. LL).
5. Concrete Deck Placement and Equipment Loads

Once the bridge is constructed, and the girders are composite with the concrete deck, there is little concern over a conventional instability causing a collapse or ill-effect in the structure. Figure 2.13 through Figure 2.16 are examples of failures and collapses previously identified.



Figure 2.13. Failure during fabrication, VDOT (2022)



Figure 2.14. Lifting Failure, Fuente et al. (2018)

Likely Cause: Initial sweep of girder and lifting loop eccentricity from centerline of girder



Figure 2.15. Lifting failure, VDOT (2022)



Figure 2.16. Failure of erected girders, courtesy of TxDOT

2.2.2. Design Guidance for Stability During Construction

Most modern design practice for prestressed concrete girders during construction employ recommendations from Mast (1989, 1993) for the development of engineering expressions. The *PCI Bridge Design Manual* (PCI, 2011) provides guidance stemming from Mast's work. Chapter 8 of the manual states:

“Prestressed concrete members are generally stiff enough to prevent lateral buckling. However, during handling and transportation, support conditions may result in the lateral displacements of the beam, thus producing lateral bending about an axis.”

The conditions PCI describes are, in essence, an effect of bi-axial bending. The girder is located between two support points (either being lifted near the ends or placed on supports) and is under action from initial imperfections and/or environmental loads. The girder is also under the action of prestressing force. As the girder is being moved, it is in a quasi-static state. The failure initiates when the girder is deflecting linear-elastically and reaches a deformed position that exceeds a material stress limit. When the girder reaches this strength limit, the girder fails and subsequently collapses because it is no longer in stable equilibrium. The failure associated with the work from Mast is primarily associated with “roll stability” as opposed to classic member buckling, such as flexural buckling of columns or lateral torsional buckling of beams. These forms of buckling involve an instability where the stiffness of the member degrades as a function of the applied load

level. Member buckling (flexural buckling or lateral torsional buckling) can be found from closed-form solutions or using modern software packages that will perform either eigenvalue buckling analyses or a second-order analysis that considers non-linear geometry and imperfections. The application of these closed-form solutions and analysis methods to stocky, open prestressed concrete sections is of primary interest in this study. The following figures and expressions are provided in the *PCI Bridge Design Manual* (PCI, 2011) and were adapted from the work of Mast (1989, 1993).

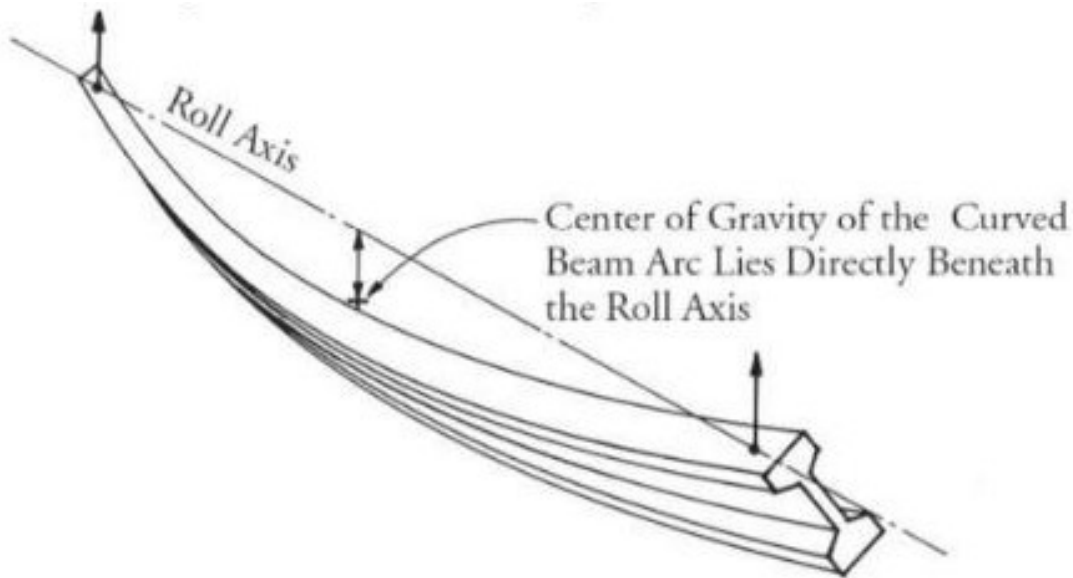


Figure 2.17. Isometric view of girder free to roll & deflect laterally, PCI (2011)

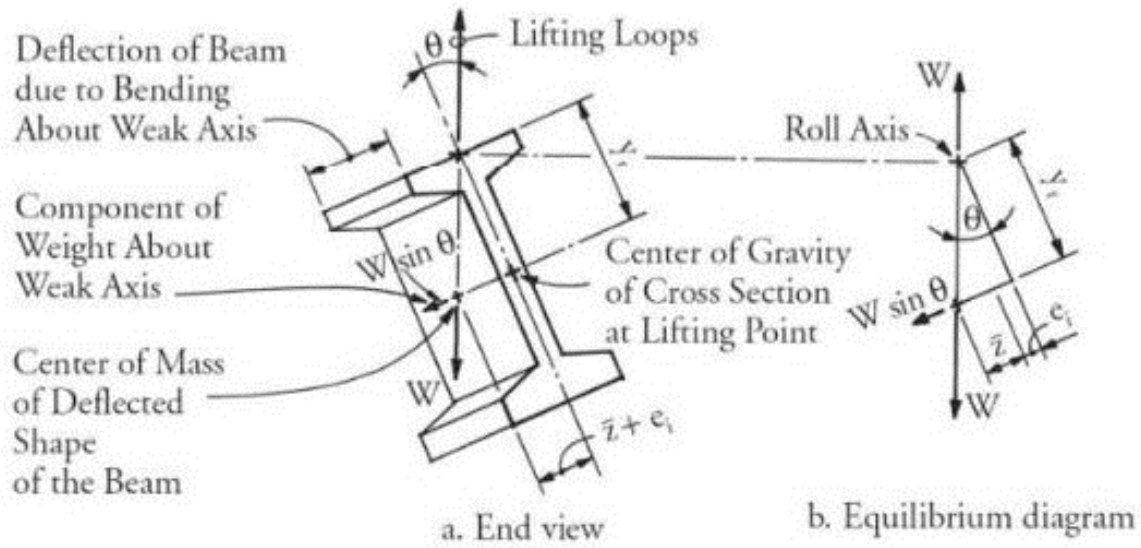


Figure 2.18. Tilted girder equilibrium, PCI (2011)

Where:

- e_i = the initial lateral eccentricity of the center of gravity (CG) with respect to the roll axis
- y_r = the height of the roll axis above the center of gravity of the beam
- $\bar{x}_o$ = the theoretical lateral deflection of the CG of the beam, computed with the full weight applied as a lateral load, measured to the center of gravity of the deflected arc of the beam
- θ_{max} = tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture
- l = length of beam
- a = overhang length at each end
- E = Young's modulus
- I = weak axis moment of inertia

$$\bar{z}_o = \frac{w}{12EI} [0.1l_1^5 - a^2 l_1^3 + 3a^4 l_1 + 1.2a^5]$$

The factor of safety against cracking, FSc, is given by:

$$FSc = \frac{1}{\frac{\bar{z}_o}{y_r} + \frac{\theta_i}{\theta_{max}}} \quad \text{where } \theta_i = \text{the initial roll angle, } e_i / y_r$$

PCI (2011) also provides guidance for girders placed on flexible support, e.g., bearings and trailers, as shown in Figure 2.19.

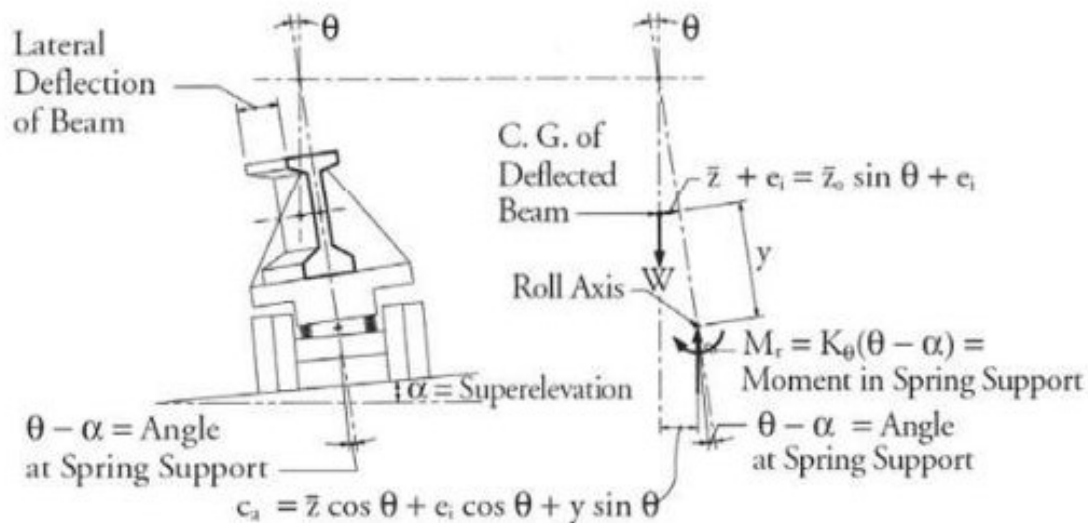


Figure 2.19. Equilibrium of girder on flexible support, PCI (2011)

PCI (2020) completed a user manual for calculating the lateral stability of concrete girders and a corresponding, robust spreadsheet. This publication further presented boundary conditions, sweep tolerances, lifting practices, and loadings (impact force etc.). However, the limit criteria is still a factor of safety for cracking, crushing, and rolling.

Additional efforts focused on stability of prestressed girders during construction were conducted as part of TxDOT Study 0-5706 (Yang et al. 2010). Although TxDOT details prior to the 1980s often utilized diagrams in PS concrete girder bridges, most simply-supported PS girders constructed in Texas over the past 30 years have not included any external braces between the supports. The impetus for the study came from problems encountered during placement of the concrete deck and the resulting torque on the fascia girders that can lead to excessive rotation on the bearings, as shown in Figure 2.20, where the girder has lifted off the bearing pad. The study focused on the behavior of the girders as a function of the loading that develops during construction, including the impact of loads from overhang brackets on fascia girders. The study included both full-scale laboratory tests and also parametric computational studies to develop design guidelines to ensure safe construction. The experimental studies focused on loading PS concrete girders to the point of tipping and included both rectangular and circular elastomeric bearings. Figure 2.21 (Yang et al., 2010) shows the experimental setup.

The work from Yang included details that are commonly used for simply supported girders in which reinforcing steel is welded to the R-bars (stirrups) that protrude from the top of the girders, as shown in Figure 2.23. The detail with the welded reinforcing bar works with the temporary

blocking that is often utilized in PS concrete girders as shown in Figure 2.21. However, while the blocking provides a reasonable load path with the fully placed deck concrete, unbalanced loading that often happens during concrete placement can result in some of the blocking members dislodging, as shown in Figure 2.22



Figure 2.20. Bottom of girder bearing rotation. Yang et al. (2010)



Figure 2.21. Experimental setup from Yang et al. (2010)



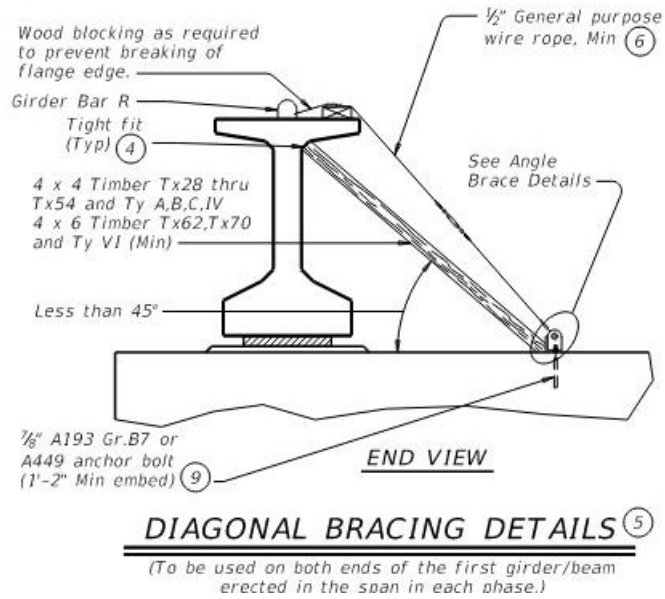
Figure 2.22. Underside of superstructure (looking up), temporary blocking, Yang et al. (2010)

2.2.3. Bracing and Diaphragm Requirements

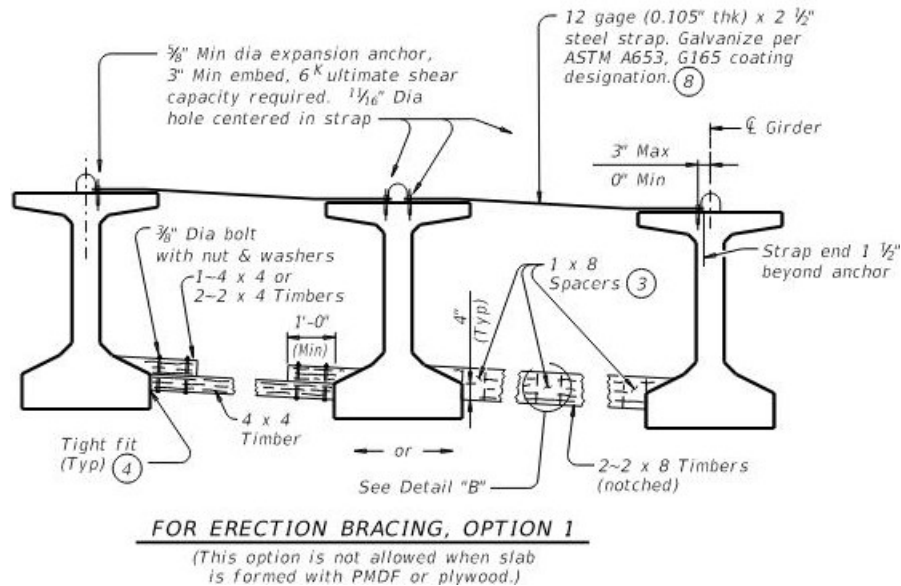
In recent years some DOTs have decided to remove requirements for permanent bracing or diaphragms on prestressed girder bridges. As noted in the past section, since approximately the 1980s, TxDOT has not required any permanent end diaphragms or intermediate diaphragms on simply supported prestressed girder bridges. On the continuous spliced concrete girders that have been recently constructed in Texas, *TxDOT Bridge Design Manual - LRFD* (2021) requires:

- Diaphragms at all bearing locations
- Full depth diaphragm at all splice and anchorage locations unless all the following conditions are met:
 - CIP splice details that do not promote poor consolidation or other constructability issues
 - Lateral stability using permanent and/or temporary diaphragms is analyzed during deck construction
 - Superstructure has demonstrated reliable lateral load distribution into the substructure with no adverse global or local effects
 - Live load distribution considers the lack diaphragms
- Intermediate diaphragm use is not mandatory

TxDOT requires a minimum amount of erection bracing, and the onus is always on the contractor and construction engineer to ensure stability. Details that are frequently used for prestressed girders are shown in Figure 2.23.



(a) At girder ends



(b) Between supports

Figure 2.23. TxDOT minimum bracing requirements, TxDOT Bridge Standards (2023)

TxDOT also employs the use of a dowel connection on exterior girders in bridge units. These details are part of the lateral load-resisting system. The dowel detail is presented in Figure 2.24.

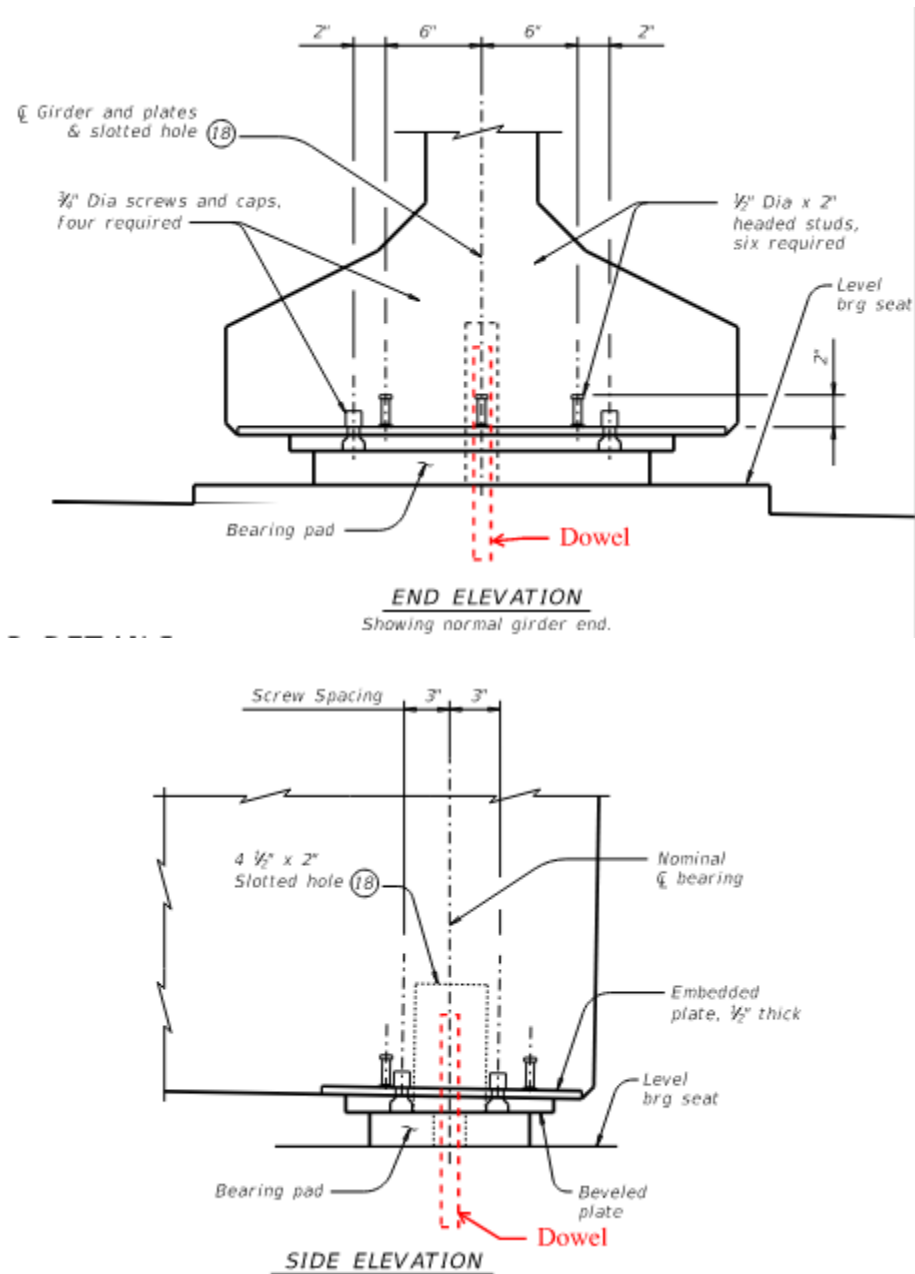


Figure 2.24. Dowel connection from the girder to the pier or abutment cap

Adapted from TxDOT Bridge Standards (2023), red markup by the current author.

Additional options for permanent/temporary diaphragms are the use of steel channels, bent plates, and cross frames. Some owners have also elected to allow the temporary diaphragms to be left in place. Situations where owners/contractors may tend to leave them installed are:

- Access limited areas (over railroads, water, active traffic beneath)
- Nonstandard roadway cross sections (varying cross slope, splayed girder spacing etc.)

Figure 2.25 is an example of a case where the superelevation varied across the bridge, and bent plate diaphragms were used.

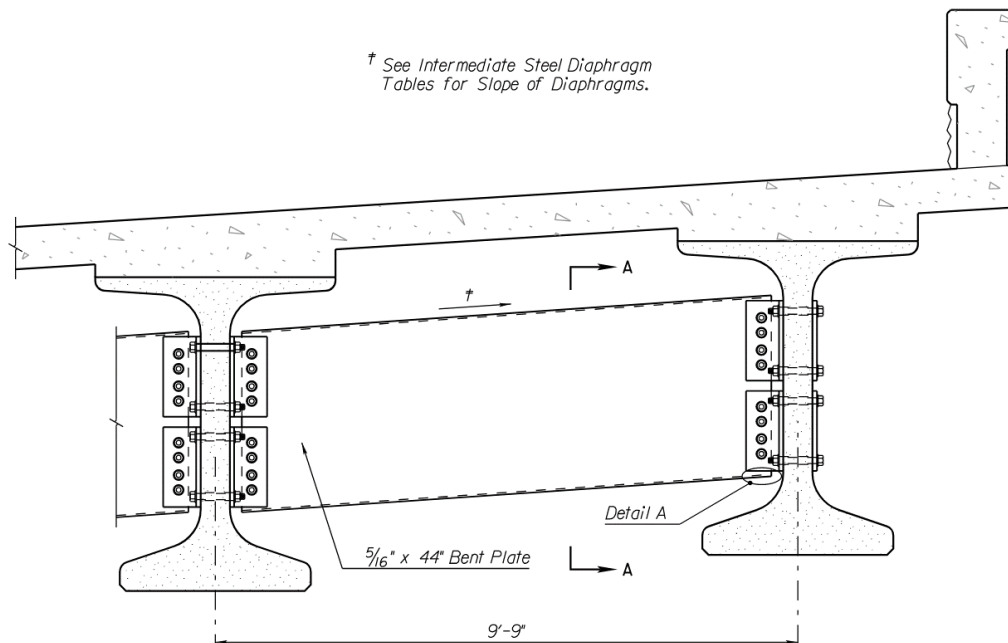


Figure 2.25. Bent plate steel diaphragm

Dupaquier et al. (2015) surveyed and reported that a number of owners have transitioned to only requiring temporary bracing. Those include Alabama, Florida, Kansas, and Texas. Dupaquier summarized the intermediate diaphragm use in the U.S. in Table 2-2.

Table 2-2 Summary of Intermediate Diaphragm Use in the U.S., Dupaquier et al. (2015)

No. of States	Intermediate Diaphragm Requirements
5	No Standard
4	Temporary Only
20	Concrete Only
13	Steel Only
8	Concrete or Steel

Many DOTs, like TxDOT, specify the minimum number of temporary brace points needed along a girder span based on the girder type, span length, and girder spacing. The normal ranges tend to be from zero braces required to every 1/8th point along the span. TxDOT Minimum Erection Bracing Standard Drawings require temporary bracing at both girder ends.

2.3. Recent Advances and Needs Related to Stability

2.3.1. Stability Theory Applied to Concrete Members

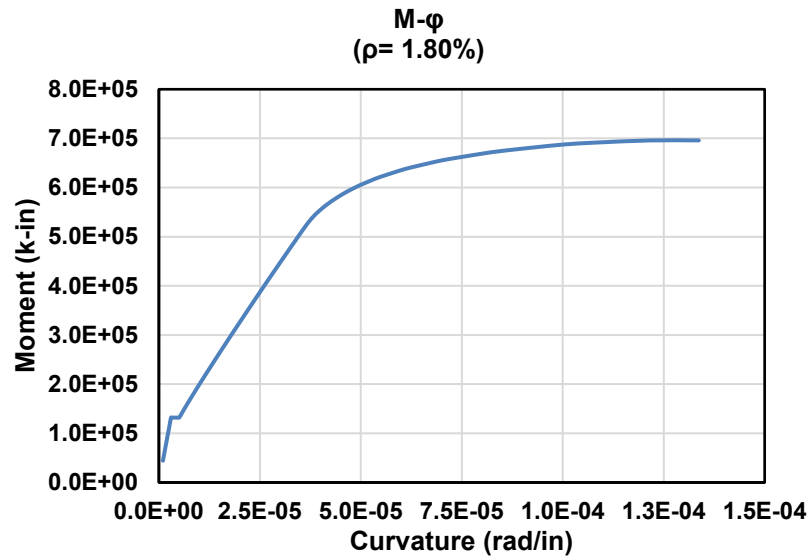
Both the concrete and the steel industry have long made use of and implemented column stability and buckling criteria. One factor complicating concrete column buckling analysis is the nature of its load deflection. As reinforced concrete members undergo flexure, the moment curvature relationship is not linear because the concrete is cracking and the flexural rigidity of the section changes. Most modern codes give simple expressions to account for the reduction in stiffness as a function of the reinforcing ratio (ρ) and the applied loading. Alternatively, a detailed moment curvature relationship can be developed. This is shown graphically in Figure 2.26.

Where: EI_e = Effective Flexural Rigidity
 EI_g = Gross Concrete Section Flexural Rigidity

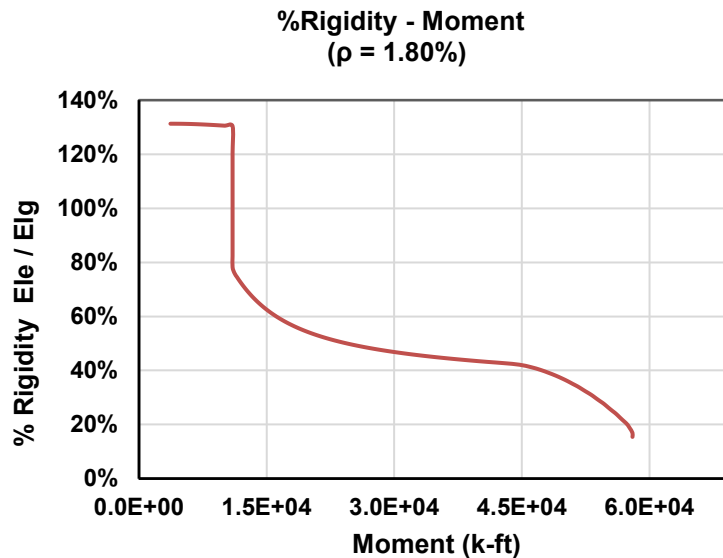
Although the buckling of a bridge girder would involve lateral torsional buckling, the impact of the flexural rigidity on the buckling performance can be understood by considering the flexural buckling behavior of columns. The buckling capacity of a column originally solved by Euler is given in Equation 2.1. It is easily observed that as EI decreases, so does the buckling capacity.

$$P_{cr} = \frac{\pi^2 EI}{L^2} \quad 2.1$$

Where P_{cr} is the critical axial load



(a)



(b)

Figure 2.26. (a) Typical concrete column moment curvature (b) effective flexural rigidity

During construction, prestressed concrete members may be impacted by reductions of flexural rigidity even though prestressing is usually proportioned to prevent tensile stresses under the construction conditions. The prestressing force is also thought not to contribute or diminish the members buckling capacity if the strands and tendons are in contact with the member along its whole length. This was first described by Lin and Burns (1981), and Libby (1990).

Unlike concrete columns, concrete beam stability is much less understood and codified. Partly because historically, concrete structures are usually controlled by cross-sectional strength or serviceability limits. The strength and service limits drive the proportioning of the members. The resulting members are usually very stocky and historically have not had instabilities associated with them, consisting of conventional member buckling. As demonstrated from some of the earlier figures, there have been a number of cases in which roll stability, either during lifting or other construction stages, has resulted in failures. Table 2-3 presents a selection of modern codes/design manuals treatment of concrete beam buckling.

Table 2-3 Modern design guidelines for concrete beam buckling

Industry Standard	Provisions for Concrete Beam Buckling
AASHTO BDS (2024)	No Explicit Guidance
ACI-318 (2022)	No Explicit Guidance
PCI Bridge Design Manual (2011)	No Explicit Guidance
CSA (2019)	No Explicit Guidance
Eurocode 2 (2023)	Flange to depth ratios of when it can be ignored

The primary way in which beams buckle is *Lateral Torsional Buckling (LTB)*. Timoshenko and Gere (1961) derived the expression for elastic lateral torsional buckling of a beam under uniform moment in Equation 2.1.

$$M_{cr} = \frac{\pi}{L_b} \sqrt{EI_y GJ + \left(\frac{E\pi}{L_w}\right)^2 I_y C_w} \quad 2.2$$

Where: M_{cr} = Buckling Moment
 L_b = Length Between Braces (Flexure)
 E = Young's Modulus
 G = Shear Modulus
 J = Saint-Venant Torsional Constant
 I_y = Moment of Inertia about Vertical Axis
 C_w = Warping Constant
 L_w = Length Between Braces (Warping)

This elastic buckling expression has mainly been applied to steel girder systems. The first term under the radical is referred to as *Saint-Venant Torsion*, and the second term is referred to as *Warping Torsion*. For thin-walled, open, and closed sections, the section properties are easily calculated with approximate methods. These approximate methods very closely approach the exact values for most steel shapes. Concrete shapes, however, are usually relatively stocky I-shaped or U-shaped girders. Studies by Yoo (2000) and Brice and Pickings (2021) have shown the approximate torsional constants can give errors on the order of 25% for some cases. Both studies used the finite difference method to calculate torsional constants. Prandtl's membrane analogy over these stocky shapes is helpful to consider and illustrates the complex, torsional behavior of the sections. Prandtl's membrane is an analogous bubble bulging on a cross-section. The local slope of the membrane corresponds to the magnitude of torsional shear stress at that location – see Figure 2.27. The warping coefficient must also be evaluated numerically for stocky sections.

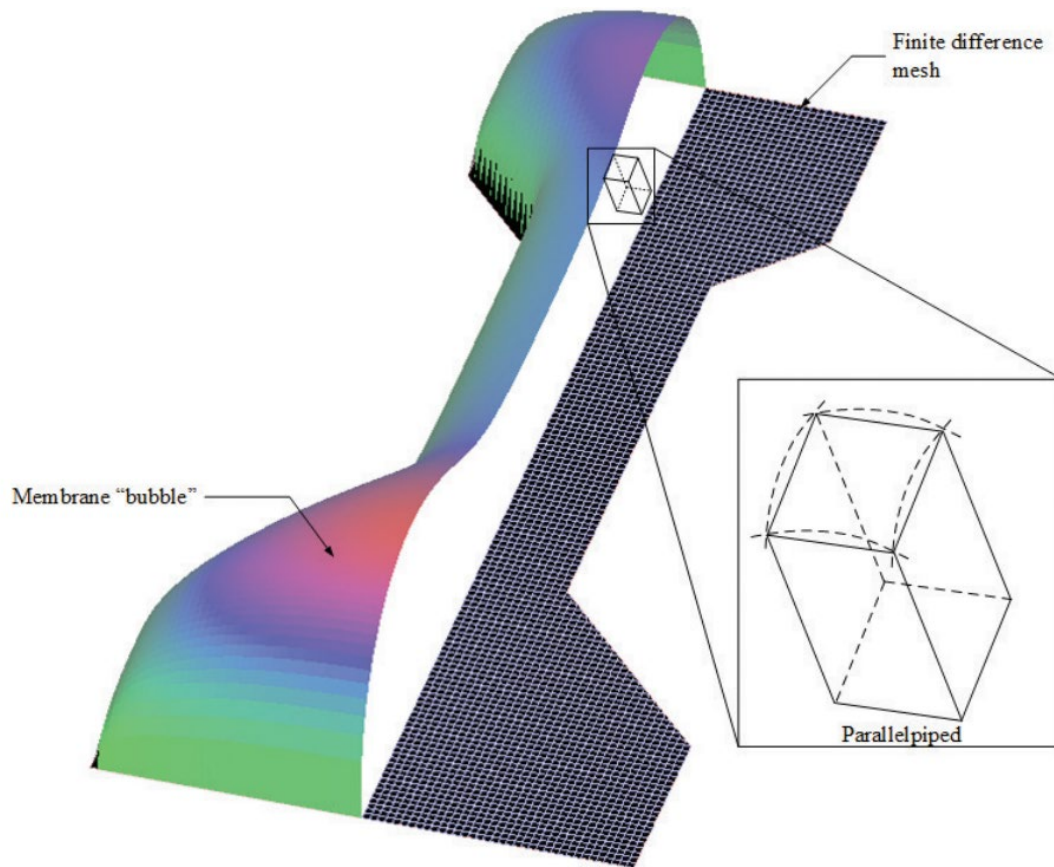


Figure 2.27. Finite difference solution of torsional constant, Brice and Pickings (2021)

Many prestressed girder types exhibit high degrees of monosymmetry in their non-composite state. The significance of monosymmetry related to stability is shown in Figure 2.28. Clearly shown in Figure 2.28, is the substantial effects of singly symmetric sections. The critical buckling load may be significantly affected if not considered.

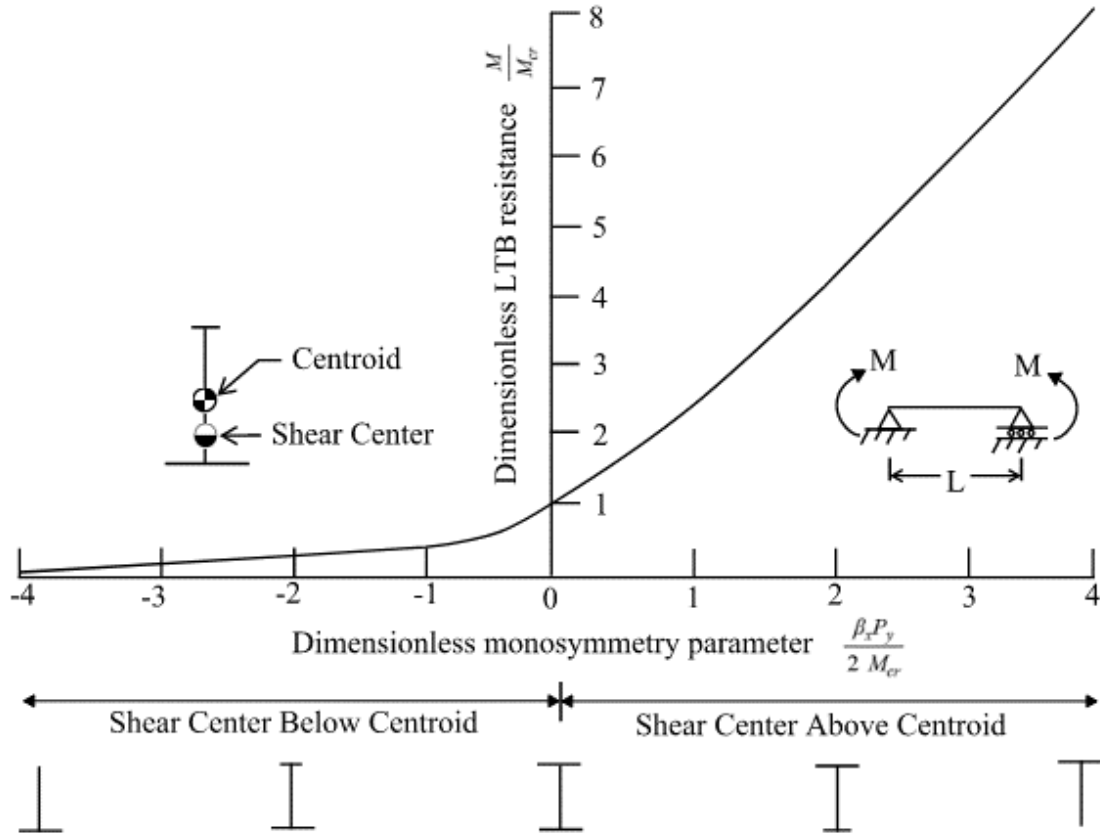


Figure 2.28. Range of monosymmetry and stability impacts after Trahair (1993)

Galambos (1968) presented the lateral torsional buckling solution for singly-symmetric sections in Equation (2.3). Galambos' solution includes the effects of singly-symmetric sections by including a monosymmetry constant, β .

$$M_{cr} = \frac{\pi}{L} \cdot \sqrt{EI_y GJ} \cdot \left(B_1 + \sqrt{1 + \frac{\pi^2 a^2}{L^2} + B_1 L^2} \right) \quad (2.3)$$

$$B_1 = \frac{\pi}{2} \cdot \frac{\beta}{L} \cdot \sqrt{\frac{E \cdot I_y}{G \cdot J}} \quad a = \sqrt{\frac{E \cdot C_w}{G \cdot J}}$$

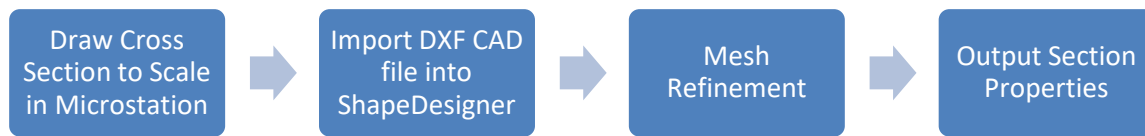
Where:

- M_{cr} = Buckling moment
- L = Length between braces (flexure)
- E = Young's Modulus
- G = Shear Modulus

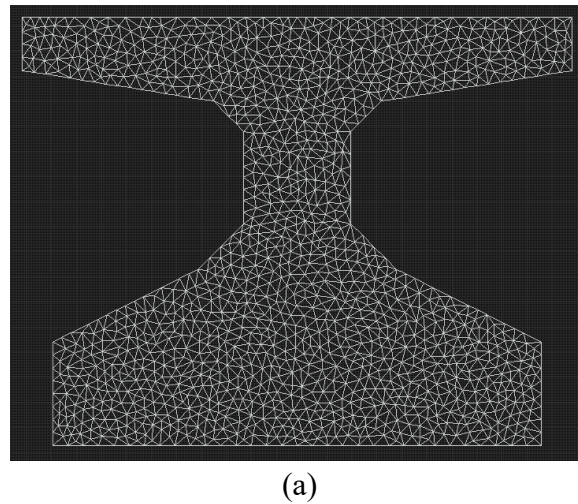
- J = Saint-Venant torsional constant
- I_y = Moment of inertia about vertical axis
- C_w = Warping constant
- β = Monosymmetry parameter

2.3.2. Study on Torsional Properties of Prestress Concrete Girders

An initial study in this report used the finite element software *ShapeDesigner SaaS* by MechaTools Technologies (2022) to calculate the torsional constants and monosymmetry parameter. All Tx. I-girder and U-girder properties were calculated via the following process:



All sections were analyzed as gross concrete and per TxDOT Standard Drawings. Example mesh density, resultant torsional stress, and warping bi-moment are shown in Figure 2.29 for a Tx 28 shape.



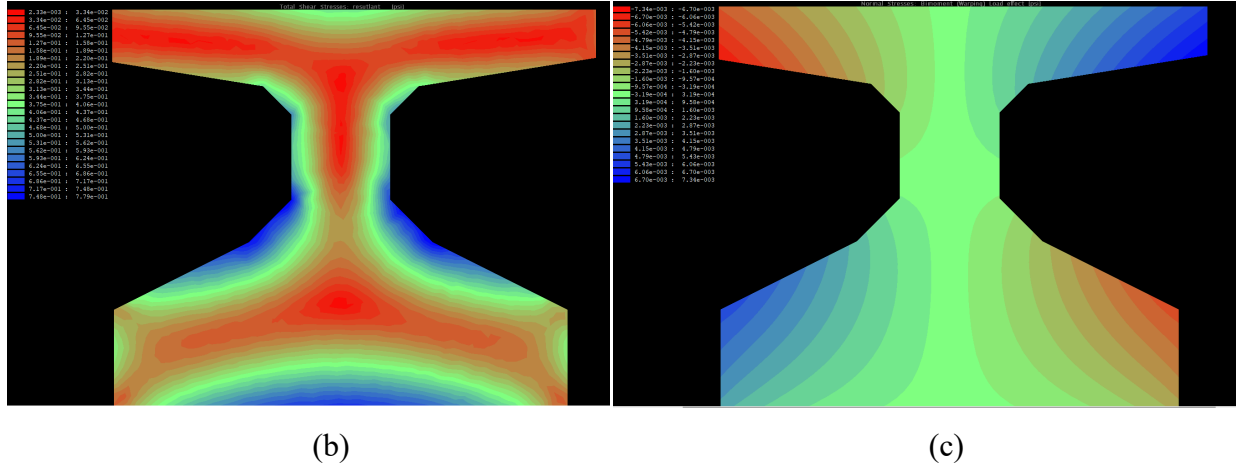


Figure 2.29. FEA modeling for Tx28 girder: (a) Mesh density; (b) Resultant torsional stress distribution; and (c) Warping stress distribution.

Table 2-4 compares the Saint-Venant torsional constant calculated by Brice and Pickings (2021) and the current study. There was very good agreement between Brice and Pickings (2021) and the current study. Additionally, the AASHTO BDS (2024) approximate equations labeled J_1 and J_2 are compared with the numerical solutions. The AASHTO approximate equations differ significantly from the numerical solutions, and depending on the assumptions used, can greatly affect structural analysis and capacity calculations.

Table 2-4 Comparison of Torsion Constants: Finite Difference (Brice and Pickings 2021) to Finite Element (RT) Solutions

Girder	J_{Brice}	J_{RT}	J_{Brice} / J_{RT}	J_1 / J_{Brice}	J_2 / J_{Brice}
Tx28	16,986	17,390	0.977	0.64	1.85
Tx34	17,660	18,069	0.977	0.65	1.7
Tx40	18,345	18,767	0.978	0.67	1.55
Tx46	25,692	26,214	0.980	0.75	1.34
Tx54	26,607	27,142	0.980	0.76	1.21
Tx62	28,148	28,743	0.979	0.76	1.17
Tx70	29,062	29,686	0.979	0.77	1.09
U40	16,885	17,430	0.969	0.45	n/a
U54	18,044	18,749	0.962	0.36	n/a

Note: Units are in⁴

$$J_1 = \frac{1}{3} \sum b \cdot t^3 \quad (\text{AASHTO C.4.6.2.2.1-1}) \quad (2.4)$$

$$J_2 = \frac{A^4}{40 \cdot I_p} \quad (\text{AASHTO C.4.6.2.2.1-2}) \quad (2.5)$$

Where: J_1 = Thin rectangle approximation

J_2 = General open section approximation

A = Area

I_p = Polar moment of inertia

The torsional properties were calculated and listed in Table 2-5 for the standard Texas prestressed concrete shapes and those used in spliced girder applications. The I-girders (except for the Tx54) exhibit a positive degree of monosymmetry, β . The U-girders on the other hand exhibit a negative degree of monosymmetry.

Table 2-5 Calculated torsional properties of Texas girder shapes

Girder	J ($\times 10^3 \text{ in}^4$)	C_w ($\times 10^6 \text{ in}^6$)	β (in)
Tx28	17.4	2.9	0.04
Tx34	18.1	5.2	0.91
Tx40	18.8	8.3	1.40
Tx46	26.2	12.5	0.30
Tx54	27.1	19.0	-3.91
Tx63	28.7	35.3	0.28
Tx70	29.7	47.6	0.29
STx7	52.7	168.4	9.6
STx8	59.7	231.5	10.1
STx9	62.7	304.7	11.5
TxU40	17.4	56.6	-44.6
TxU54	18.7	119.8	-54.7
TxU72-9	60.7	568.4	-71.1
TxU84-9	67.0	820.7	-80.2
TxU96-9	72.8	1116.1	-85.4

While it is not a direct correlation, it is helpful in interpreting monosymmetry values to consider the location of the shear center relative to the centroid, as shown in Figure 2.30. The shear center represents a point on or off the cross-section that, if a load is applied through the point, results in no twisting of the cross-section. Therefore, loads not applied through the shear center cause twist of the cross-section.

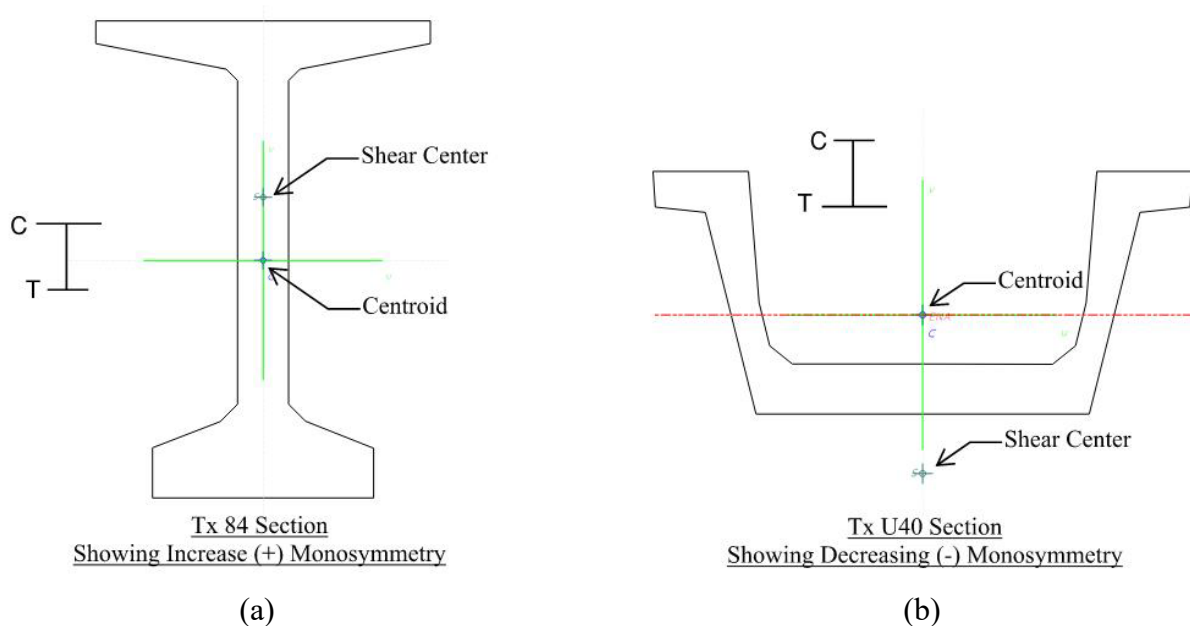


Figure 2.30. Example of shear center locations in I and U-girder.

2.3.3. Effects of Restraint and Continuity on Girder Buckling

Unlike steel girder bridges that generally have many intermediate diaphragms/cross-frames between supports, spliced girder concrete bridges have very few intermediate diaphragms. While the spliced girder bridges do not have as many intermediate diaphragms that improve stability, the spliced girder bridges are continuous, and the continuity does increase the stability of the system. The continuity adds stiffness and restraint to the continuous bridges that simple span bridge layouts do not.

This section presents the effects of continuity by analyzing differing end restraint cases of a W36x150 steel beam. This problem is very well characterized in the evaluation of stability behavior and can employ Timoshenko's exact equation when comparing the Finite Element (FE) solution. The W36x150 was modeled using linear-elastic steel properties and the FE solution used plate elements.

Example calculations and analysis for a W36x150 considering the effects of continuity:

Simply Supported Case, Uniform Moment -

Example Calculation Using Timoshenko's Exact Expression

$$f_y := 50 \text{ ksi} \quad E := 29000 \text{ ksi} \quad G := \frac{E}{2 \cdot (1 + 0.3)} = 11154 \text{ ksi}$$

W36x150 Section Properties

$$I_y := 270 \text{ in}^4 \quad J := 10.1 \text{ in}^4 \quad C_w := 82200 \text{ in}^6 \quad A := 44.3 \text{ in}^2 \quad S_x := 504 \text{ in}^3$$

$$L := 30 \text{ ft}$$

$$M_{cr} := \frac{\pi}{L} \cdot \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{E \cdot \pi}{L} \right)^2 \cdot I_y \cdot C_w} = 1104 \text{ kip} \cdot \text{ft}$$

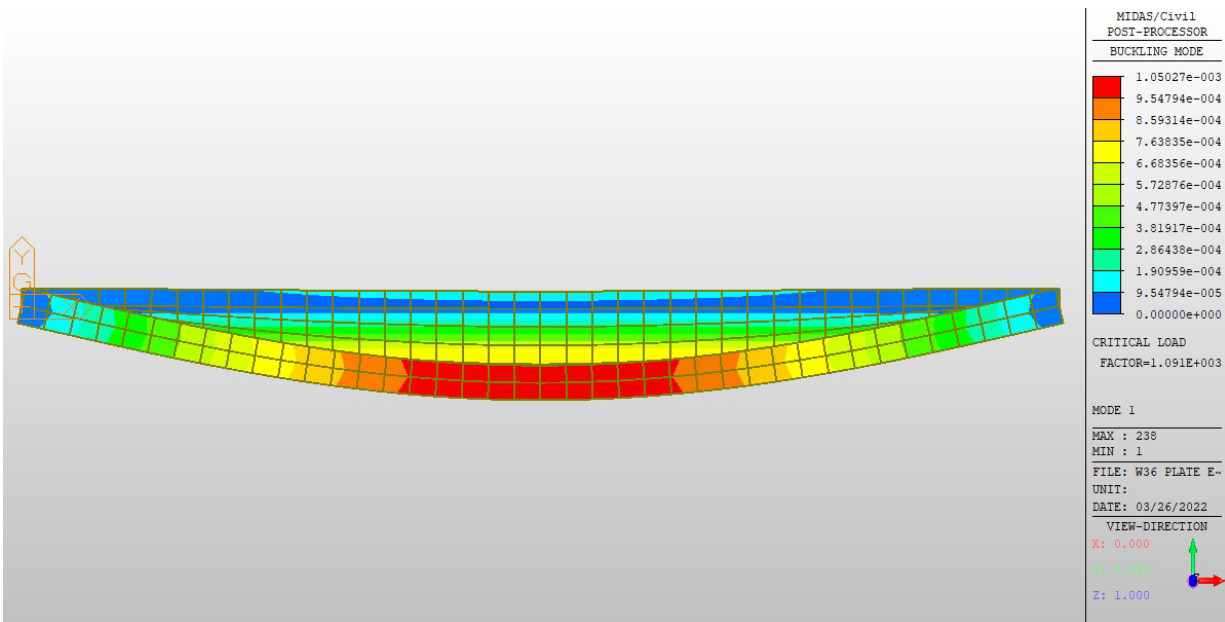
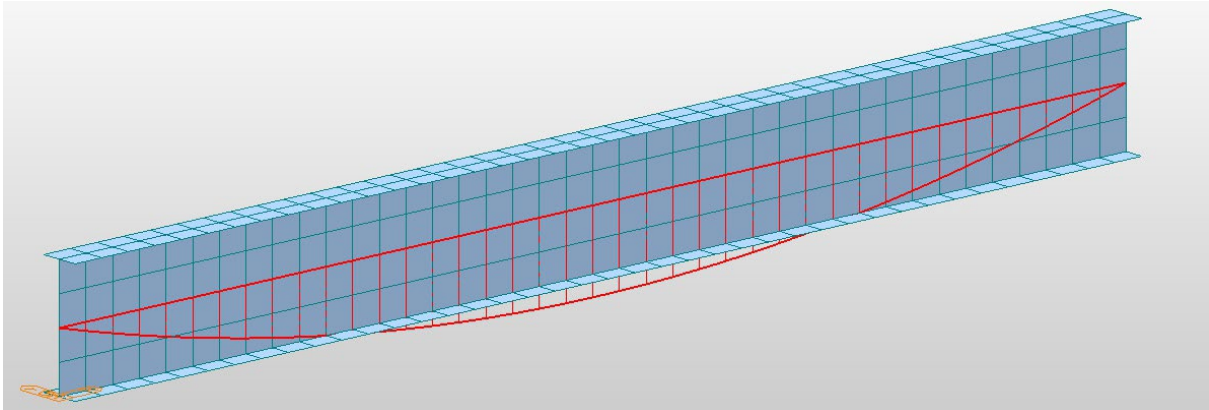


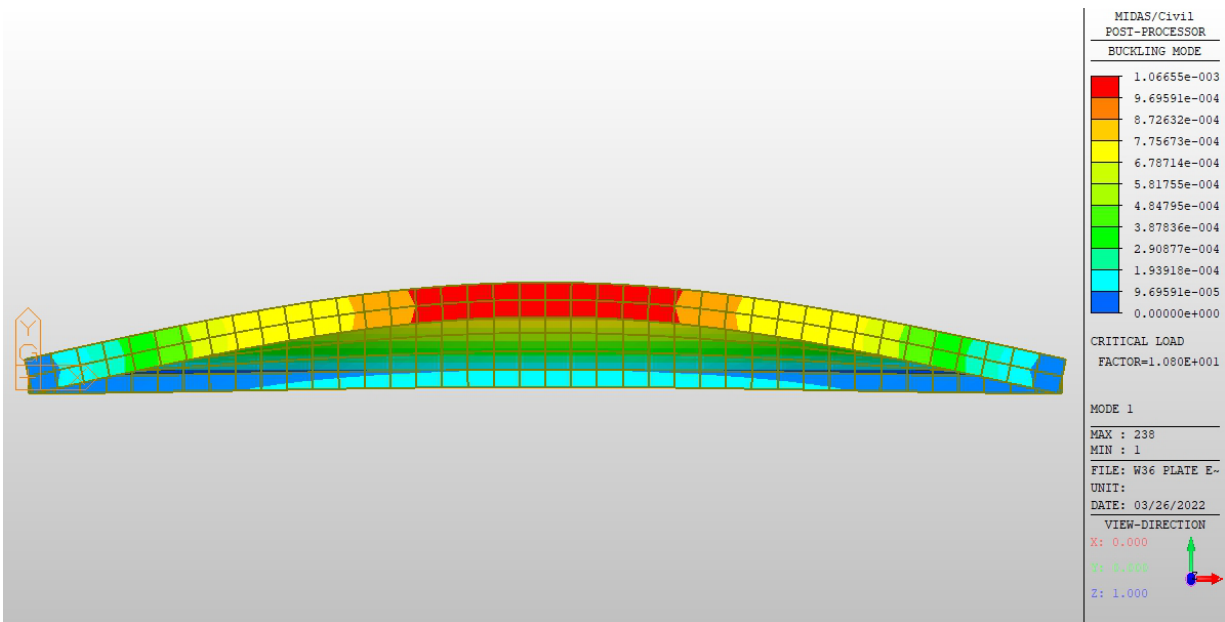
Figure 2.31. Plan view of simply supported girder with uniform moment

Eigenvalue, $M_{cr} = 1,091 \text{ kip} \cdot \text{ft}$

Simply Supported Case, Uniform Dead Load (UDL) at C.G. -



(a) Moment Diagram

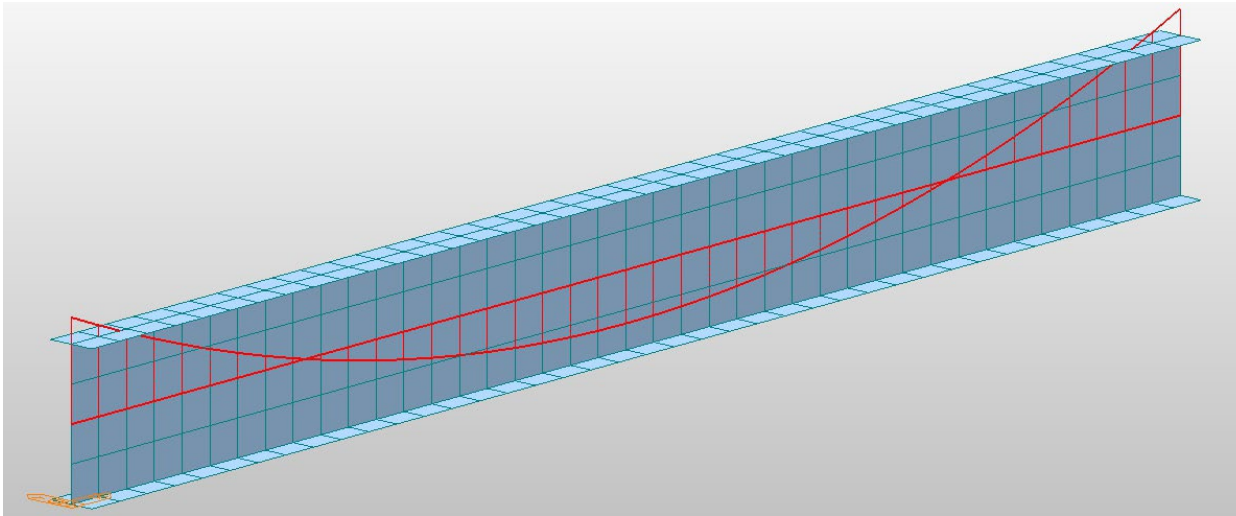


(b) Plan View Buckling Mode

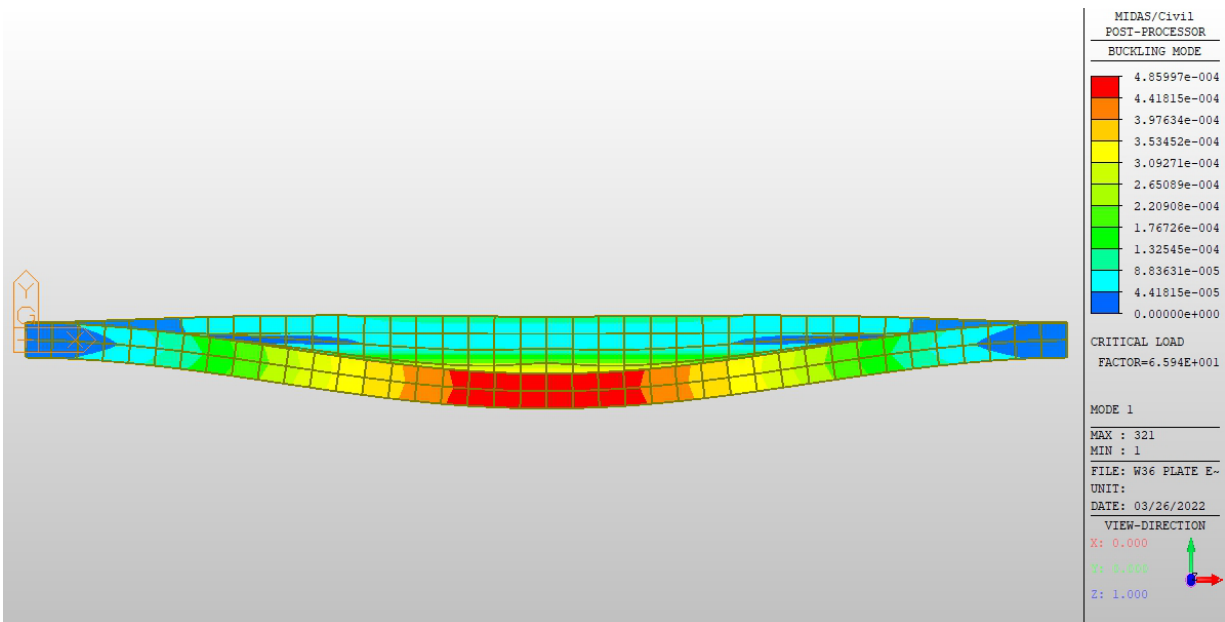
Figure 2.32. Simply supported girder with UDL at C.G.

$$\text{Eigenvalue Buckling, } M_{cr} = \frac{wl^2}{8} * 10.8 = 1215 \text{ kip-ft}$$

Fix-Fix Case, Uniform Dead Load (UDL) at C.G. -



(a) Moment Diagram

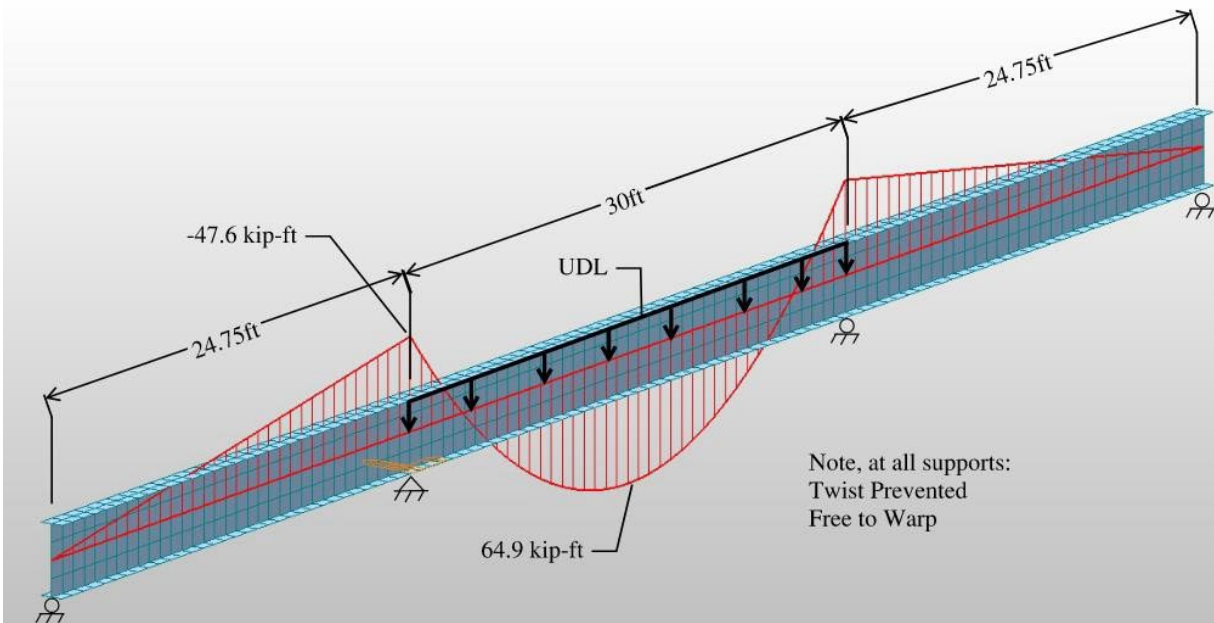


(b) Plan View Buckling Mode

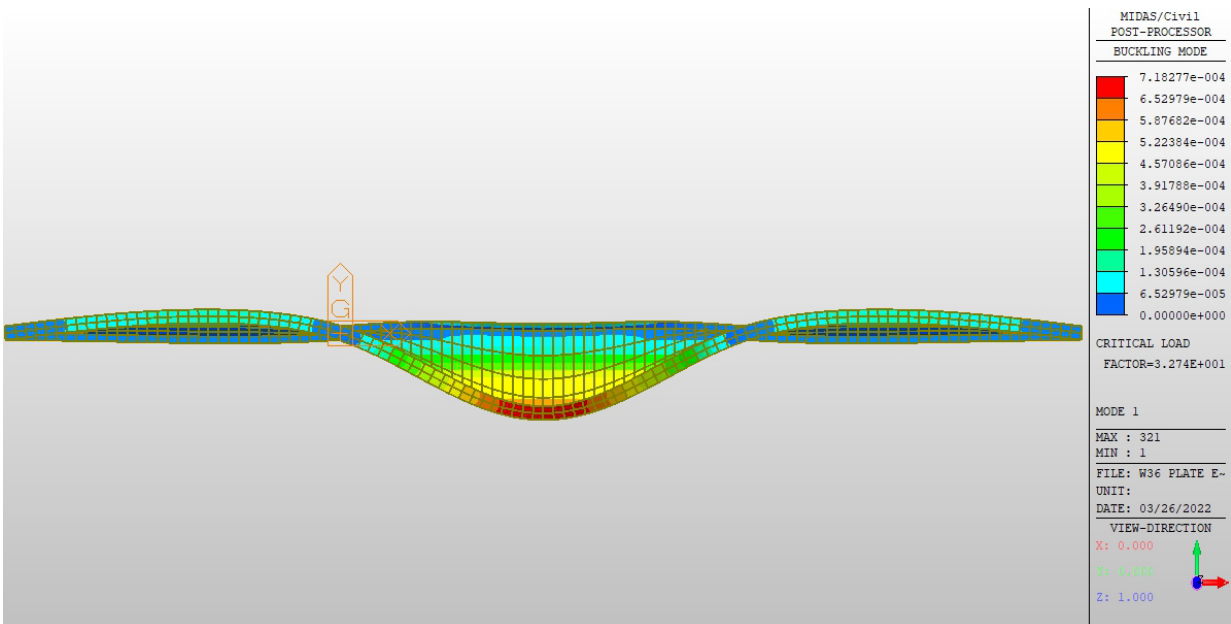
Figure 2.33. Fix-Fix girder with UDL at C.G.

$$\text{Eigenvalue Buckling, } M_{cr} = \frac{wl^2}{24} * 65.9 = 2471 \text{ kip-ft}$$

Continuous Case, Uniform Dead Load (UDL) at C.G. of center span only



(a) Moment Diagram



(b) Plan view buckling mode

Figure 2.34. Continuous girder with UDL at C.G.

$$\text{Eigenvalue Buckling, } M_{cr} = 64.9 \text{ kip-ft} \times 32.7 = 2122 \text{ kip-ft}$$

Table 2-6 - Summary of W36x150 continuity example

Case	M _{cr} (kip-ft)	% Fix-Fix
Simply Supported	1,215	49%
Fix-Fix	2,471	100%
Continuous	2,122	86%

The simple study of a W36x150 indicates the significance of a continuous girder on the stability of the bridge. When normalized against the Fix-Fix cases, the continuous girder was 86% of the capacity, and the simply supported case was 49%. During the course of construction, splice girder bridge components will have staged conditions that are simply supported, and then fully continuous for deck placement.

2.4. Research Related to Concrete Lateral Torsional Buckling

Hurff and Kahn (2012) presented findings for AASHTO Type I-VI girders comparing LTB and rollover limit states. Their findings are summarized in Table 2-7. These results included a 1.5 factor of safety and only considered self-weight. All girders studied had a minimum concrete strength of 6 ksi and were simply supported.

Table 2-7 - LTB & Rollover Comparison
After Hurff and Kahn (2012)

AASHTO Girder Type	Length below which lateral-torsional buckling does not occur (feet)	Length below which rollover about support does not occur (feet)
I	127	75
II	133	80
III	155	100
IV	175	110
V	197	135
VI	193	140

Hurff et al. (2012) tested slender rectangular prestressed concrete beams with differing amounts of prestressing and locations. They concluded that slender prestressed sections can be susceptible to LTB and sensitive to initial imperfections. Through this investigation they also analyzed post cracking behavior of the prestressed members and the corresponding biaxial bending stress in the compression zone of the section.

Consolazio et al. (2007) performed a study sponsored by the Florida DOT researching the behavior of bulb-tee sections frequently used by the state. The study showed similar trends to Hurff and Kahn (2012). One key distinction being that flange cracking under selfweight was considered. The following figure plots the capacity of the girders loaded by a uniformly distributed load (UDL).

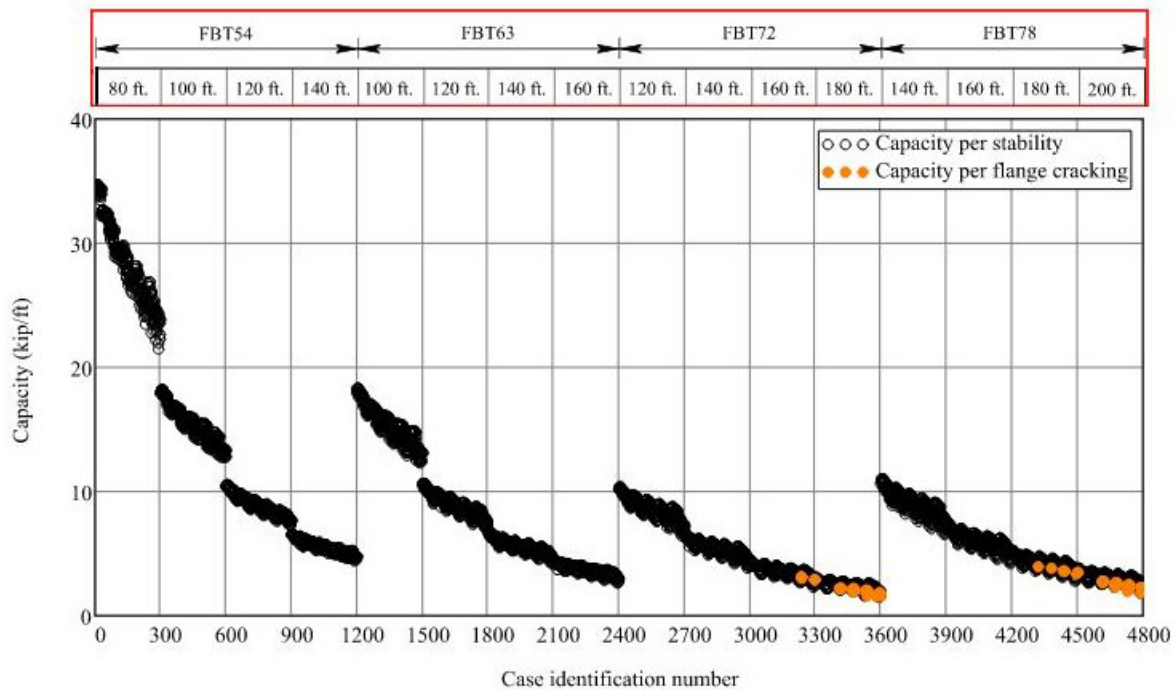


Figure 2.35. FDOT Bulbed Tee Stability and Cracking Capacity by Consolazio et al. (2007)

Red box added by current author to show case number nomenclature

Consolazio et al. (2007) performed the stability analysis in the commercial software program ADINA using generalized beam elements. The section properties for torsion and warping were calculated numerically in separate software. It can be seen in Figure 2.35 for longer span applications, spliced or non-spliced, LTB may control. Especially considering wet concrete weight and other environmental loads.

2.5. Live Load Distribution in Prestress Girder Bridges

Cross-frames and diaphragms are required in steel girder bridges; however as noted earlier – many states do not require these braces in simply-supported concrete girders. For straight bridges with little skew and negligible curvature, diaphragms are usually considered secondary members - meaning they are not explicitly analyzed. This analysis is referred to as line-girder analysis that utilizes distribution factors to approximate the interaction between adjacent girders in supporting the truck loading on the bridge. When there is significant skew and/or the curvature degree is high enough, the bracing members are considered primary members. The primary members are either explicitly modeled as members, or the effective stiffness input at correct locations. There are

generally three modeling techniques for the bracing treated as a primary member: 1) Grillage – girders, bracing and deck modeled as beam elements. 2) Beam and Eccentric Plate – girders and bracing modeled as beam elements, deck modeled as plate or shell element. 3) FEA – girders and deck modeled as shell or solid elements and bracing as beam elements.

For a simplified line-girder analysis, AASHTO BDS (2024) Chapter 4 provides guidance on the applicability of such an analysis. The distribution factors in AASHTO have the following limitations based upon the research utilized to develop the factors:

- 1) Span lengths less than 240 ft
- 2) Longitudinal stiffness is close to constant. However, the commentary states that for beams with variable moments of inertia, it may be based on average properties.

The distribution factors provided in AASHTO were developed in NCHRP 12-26 (Zokaie, 1993). The study did not consider the influence of diaphragms or braces on the interaction between girders. The large flexural and torsional rigidities of the CIP concrete diaphragms used at splice and bracing locations in spliced girders may have a significant impact. Moreover, the behavior may also be influenced by the non-prismatic sections.

The Michigan DOT sponsored a study to develop guidelines for the use of intermediate diaphragms on precast beams (Eamon et al. 2021). The work concluded that intermediate diaphragms do contribute to girder interaction and therefore would influence the distribution factor. Similar observations were made by Barr et al. (2001) and Cai et al. (2009) however, the aim of those studies was to discuss a more case by case discernment. Meaning, for specific bridge geometries or when load rating an existing structure, it is appropriate to use modified distribution factors.

Chapter 3. Task 3: Full-Scale Laboratory Testing

3.1. Experimental Specimen Details

The primary objective of the experimental testing was to assess the response of the prestressed concrete girders to various loading scenarios in both the uncracked and cracked states, so that accurate models were developed for the girders during construction and in-service. The test data served as calibration and validation ground truth for the appraisal of the numerical investigations. Although experiments on girders with spans of 150-200 ft, as found in spliced-girder applications, are desirable, practical limitations in the laboratory prevented such a scale. The primary limitations included available testing space and lifting/transportation abilities within the laboratory. The full-scale testing was necessary to validate Finite Element Analysis (FEA) modeling strategies to accurately capture the stiffness of these systems.

All experimental tests were performed at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin. The laboratory is served by two bridge cranes with respective lifting limits of 50 kips and 20 kips. In addition, the laboratory has two forklifts for moving pieces around the floor with respective lifting capacities of 28 kips and 7 kips. Therefore, the weight of test specimens for experimental testing was a major consideration. Because both I-shaped and U-shaped girders are utilized in Texas, the research team (RT) selected and designed specimens that represent the cross-sectional behavior of both shapes.

Based on the lab constraints, the selected prestressed girder specimens were:

- Girder 1: 60 ft Length Tx28 I Girder
- Girder 2: 60 ft Length Tx34 I Girder
- Girder 3: 35 ft Length Tx-U40 U Girder

Working with a local fabricator, the RT was able to locate and install Vibrating Wire Gauges (VWGs) at key locations within the girders prior to casting of the concrete. Locations were selected to provide strain profiles across the width and depth of the girders at various locations along the length. Figure 3.1 shows the basic cross-sectional geometry of the girders along with the VWG locations on the cross-section. The loading on the girders during experiments focused on: i) bending about a vertical axis, ii) lateral bending about the horizontal axis, and iii) torsional loading. The test sequences often combined lateral bending and torsion, as well as vertical bending and torsion. The VWGs were placed to capture the variation in axial strains due to combinations of the different types of loading. The basic design parameters for the test specimens consisted of the following:

Design Compressive Strength at Strand Release	$f'_{ci} = 6,500$ psi
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28-Day, Design Compressive Strength	$f'_c = 8,000$ psi
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6/10 in. Diameter Low-Lax Prestressing Strand Guaranteed Ultimate Tensile Strength (GUTS)	GUTS = 270 ksi
Prestressed Jacking Force per Strand	$F_{\text{strand}} = 44$ kips
Concrete Design Slump for Tx28 and Tx34 Girders	Slump = 9 in.

The Tx-U40 Girder required self-consolidating concrete (SCC) to minimize voids that might otherwise develop during casting as a result of the forming system that has a propensity to trap air. The Tx28 and Tx34 Girders were consolidated by conventional means via pneumatic metal form vibrators and insertion vibrators at top flange locations. The basic girder geometry and strand information are shown in Figure 3.1, and VWG gauge locations on the girder cross-section are provided in Figure 3.2. The gauges were located at the quarter points and at midspan of the girders. The instrumentation was installed after the reinforcing steel cages were tied and prestressing was placed; however, the formwork had not been installed. The specimens were fabricated by Bexar Concrete in San Antonio in August of 2022.

Note:

▲ Partial Length Debonded Strands

All Strands are ASTM A416

Low-Relaxation 0.6" $\varnothing$ Strands

Gr. 270. Each Strand Jacked to 44 kips.

⊙ Gross C.G.

$f'_{ci} = 6,500 \text{ psi}$

$f'_c = 8,000 \text{ psi}$

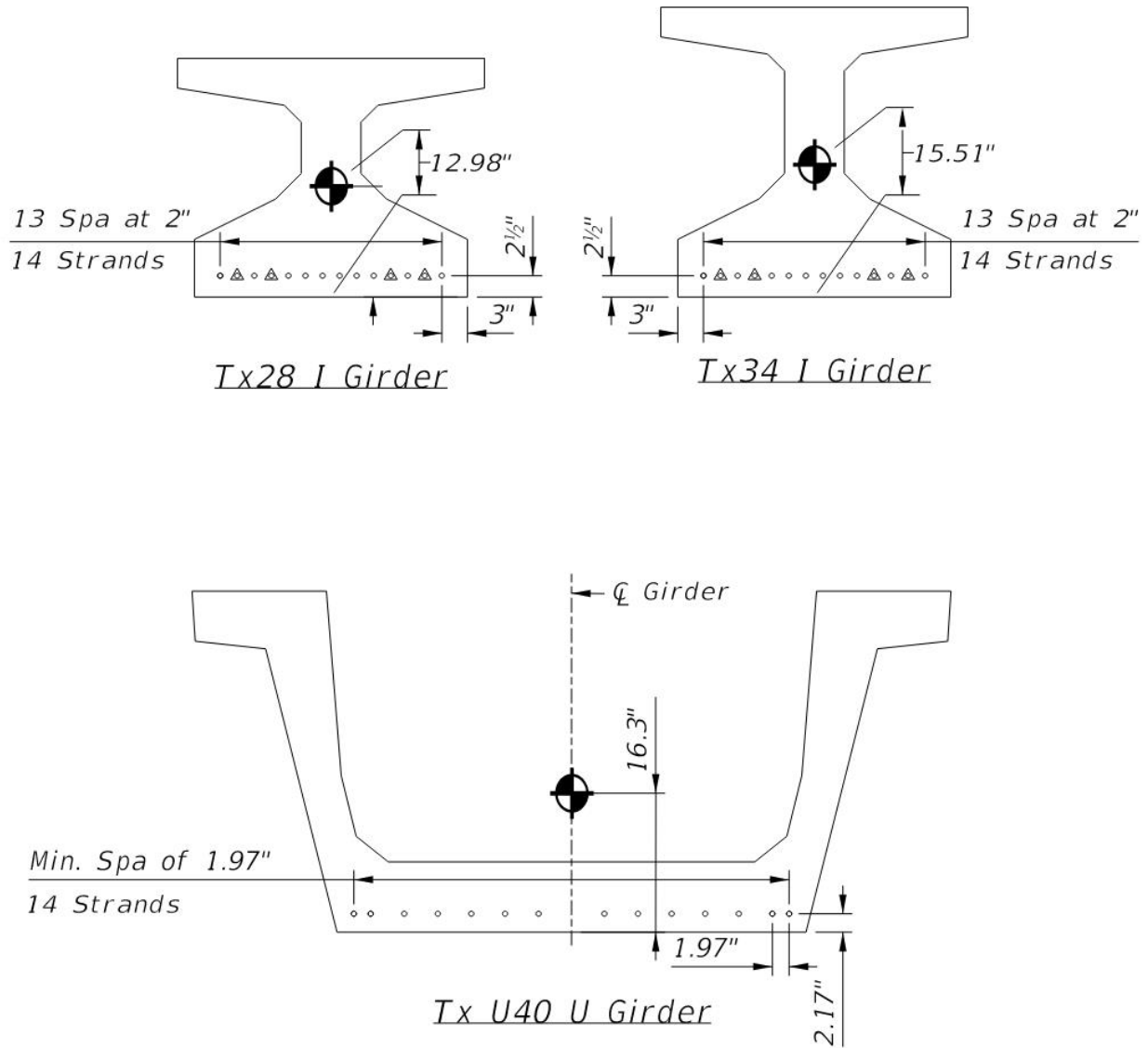


Figure 3.1. Prestress girder details

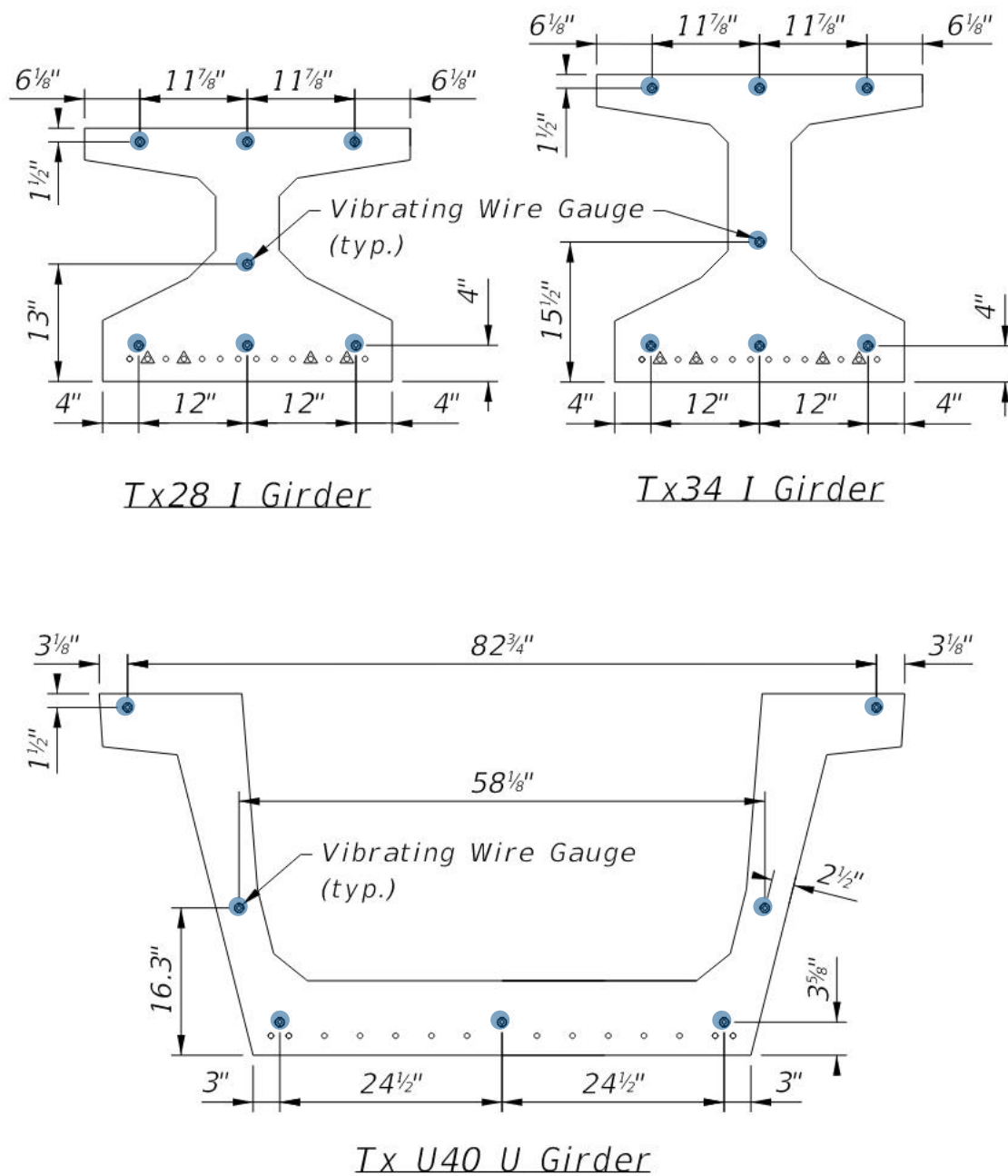


Figure 3.2. Specimen cross-sections with locations of vibrating wire gauges (VWGs) at the quarter points and midspan of the girders.

3.2. Tx-U40 Girder Laboratory Experiments

A variety of loading protocols were pursued to gain insight into the flexural and torsional rigidities of the girder, which were necessary for validating the finite element models used in the parametric FEA studies. The properties of the girders are desired in the “cracked” and “uncracked” states, as cracked concrete sections are accompanied by a change in stiffness, and stiffness directly affects stability. The term “cracked” excludes the common hairline and shrinkage cracks that develop during the curing process. In addition to the vibrating wire strain gauges installed at the quarter points and midspan of the girder, linear potentiometers (LPs) were employed to measure displacement at various locations along the girder length. Displacement measurements were taken at the end, quarter point, and midspan. The location and orientation of the displacement measurement sensors on the cross-section are shown in Figure 3.3. Vertical and lateral deformations were obtained by comparing the measurements from the sensors in the specific directions. The twist of the girders was also obtained based upon the differential displacements between two locations on the cross-section by subtracting the measurements in a given direction and dividing by the distance between the respective sensors.

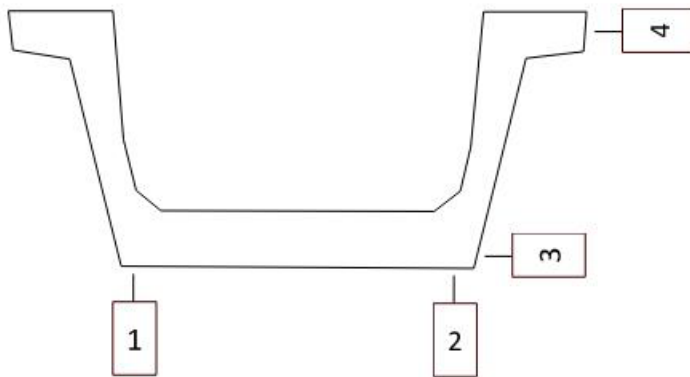


Figure 3.3. Tx-U40 displacement sensor locations on the cross-section

The following tests were performed on the Tx-U40 girder:

1. Lateral Bending (uncracked)
2. Bending (uncracked)
3. Combined Bending and Torsion (uncracked)
4. Bending into the cracked region
5. Combined Bending and Torsion (cracked)

Unless noted, all bending tests were performed with vertical loading. The results from all experiments presented in this tech memo represent deflections and strain changes only from the applied loading during the experiment and do not reflect the stresses and deflections that occurred

during fabrication, prestressing, or girder self-weight (resulting from the erection on both bearings). However, when estimating cracking loads, the effects of prestressing and self-weight were considered.

3.2.1. Lateral Load Test (Uncracked)

An overview of the lateral test setup for the Tx-U40 specimen is provided in Figure 3.4, and Figure 3.5. As noted earlier, the total length of the U40 girder was 35 ft. The span between the centerline of the supports was 32 ft to accommodate the 4 ft spacing between anchor points on the lab reaction floor. The girders were supported vertically at the ends using elastomeric bearings placed on wide flange sections that were anchored to the floor. Twist was restrained at the ends of the girders using the brackets shown in Figure 3.5. The brackets included threaded rods with rounded ends to minimize warping restraint. The threaded rods were adjusted to account for the geometry of the specimen and reacted on steel plates embedded in high-strength grout on the sides of the specimen at the desired lateral reaction points. The loading was applied at midspan and centered at mid-thickness of the girder bottom flange. Fixtures were fabricated and attached for a proper application of the lateral load, as detailed in Figure 3.6.

To avoid cracking the girder early in the experiments, the maximum magnitude of the lateral load was 20 kips. Three iterations of each test were conducted to ensure repeatability in the tests. The experimentally observed load-displacement response is shown in Figure 3.7, which provides a graph of the general load-displacement behavior at midspan. While there are slight fluctuations in the load-deflection plot, the behavior was essentially linear, as anticipated in the uncracked condition. The slight fluctuations are likely a product of the resolution of the instrumentation or minor restraints in the actuators or girder response. The strain profiles from the VWG gauges for the lateral loading up to 20 kips are plotted in Figure 3.8. The variation in linearity is likely due to the relatively small load levels, and therefore, the strain measurements may be impacted by the resolution of the instrumentation as well as localized effects around the sensors.

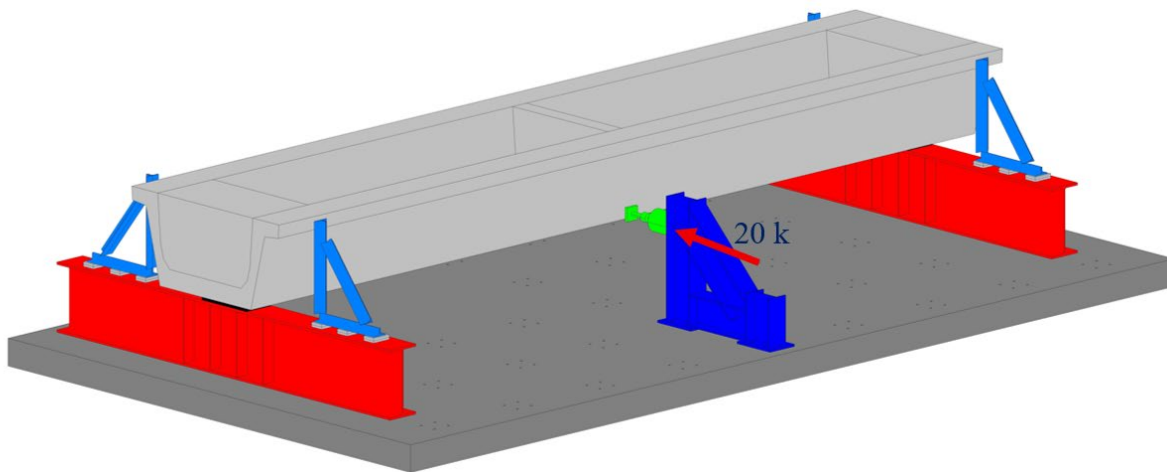


Figure 3.4. Overview of the lateral load test setup for Tx-U40 specimen



Figure 3.5. Lateral load test of Tx-U40 specimen

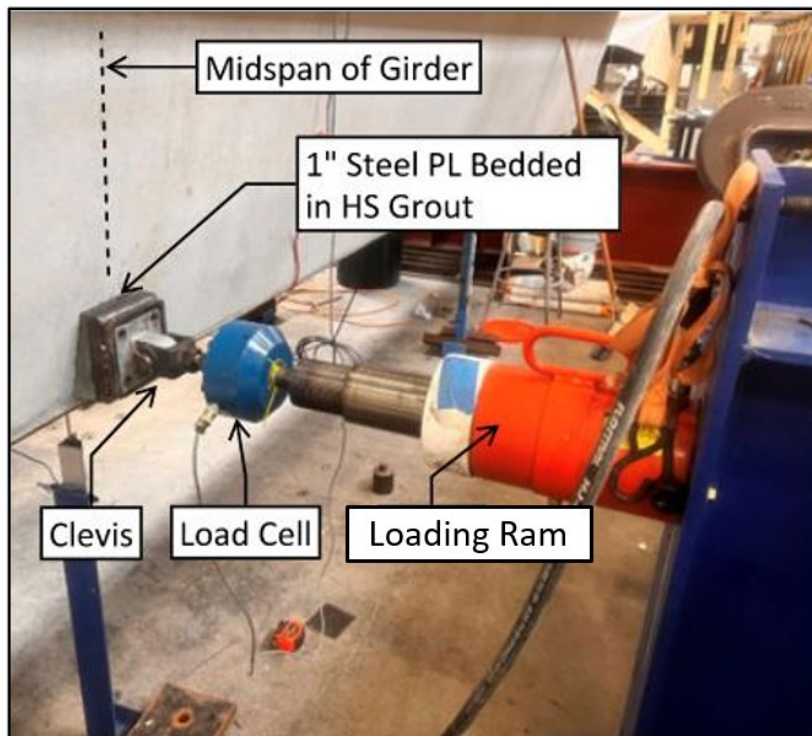


Figure 3.6. Close-up view of the lateral load application Tx-U40 specimen

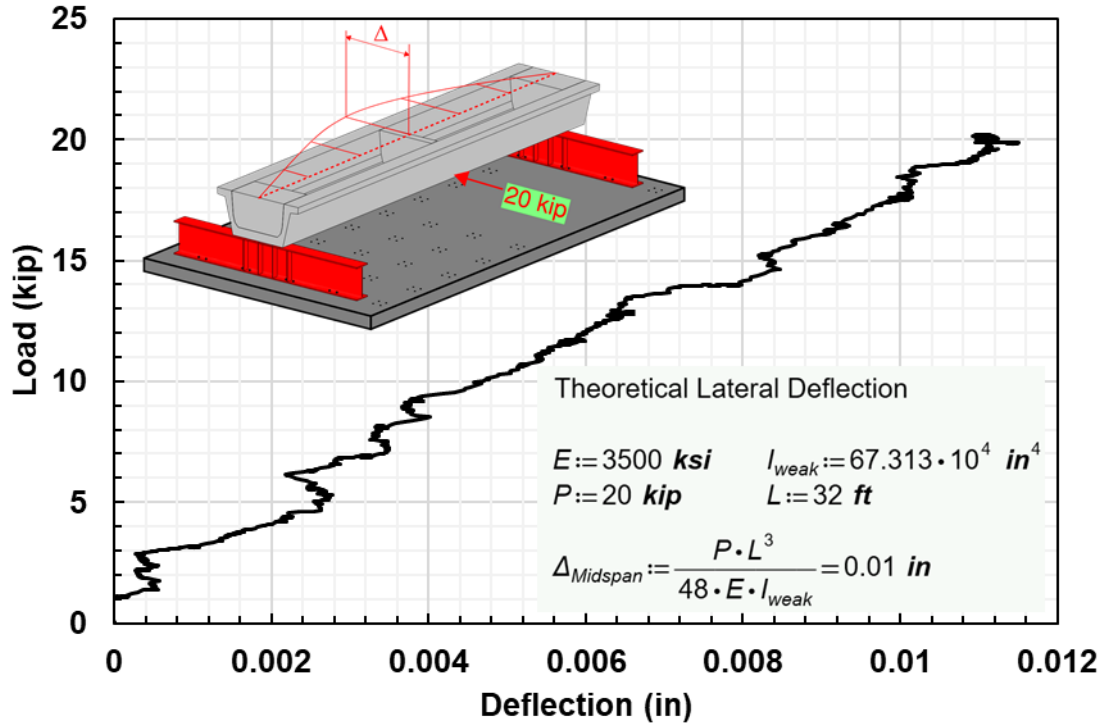


Figure 3.7. Experimental load-displacement results for Tx-U40 specimen subjected to lateral load

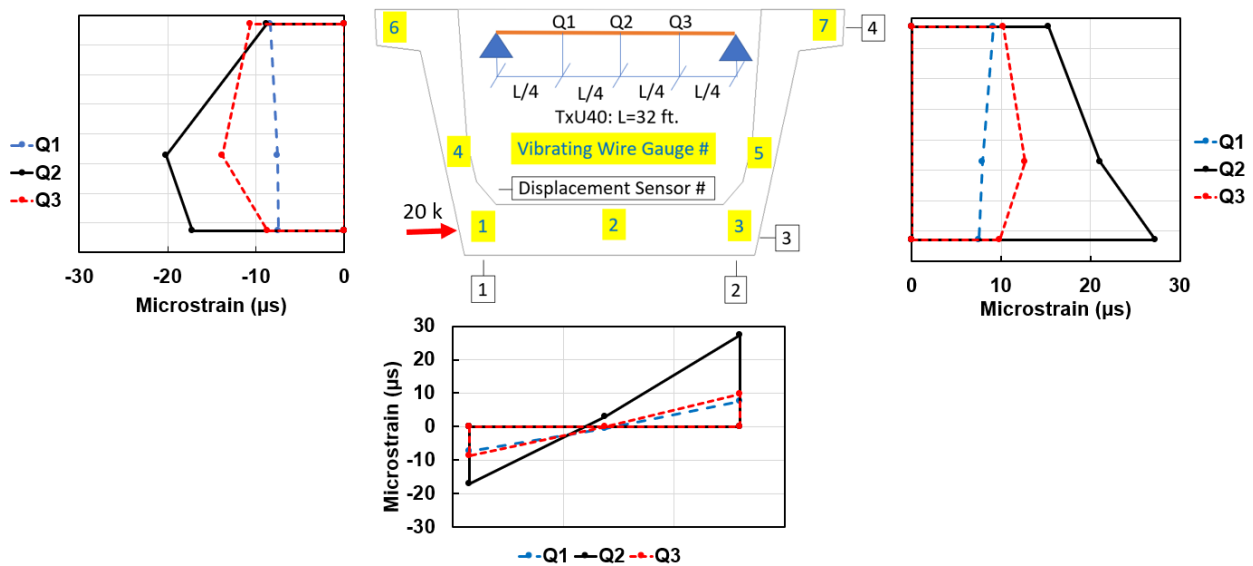


Figure 3.8. Sensor layout and strain profiles observed at 20 kips of lateral load for Tx-U40 specimen

Figure 3.9 through Figure 3.11 provide graphs of the load-deflection responses for three cycles of monotonic loading up to 20 kips. The displacements were measured at one end of the girder, the first quarter point (Q1), and the midspan of the girder (Q2) measured along the girder length (as indicated in Figure 3.8). Due to the symmetry of the girder and loading, the behavior at the

opposing support is assumed to be similar. The VWG strain readings for these cycles are shown in Figure 3.12 through Figure 3.14. The VWGs measured strains in the cross-sections at the two quarter points, Q1 and Q3, and the girder midspan (Q2).

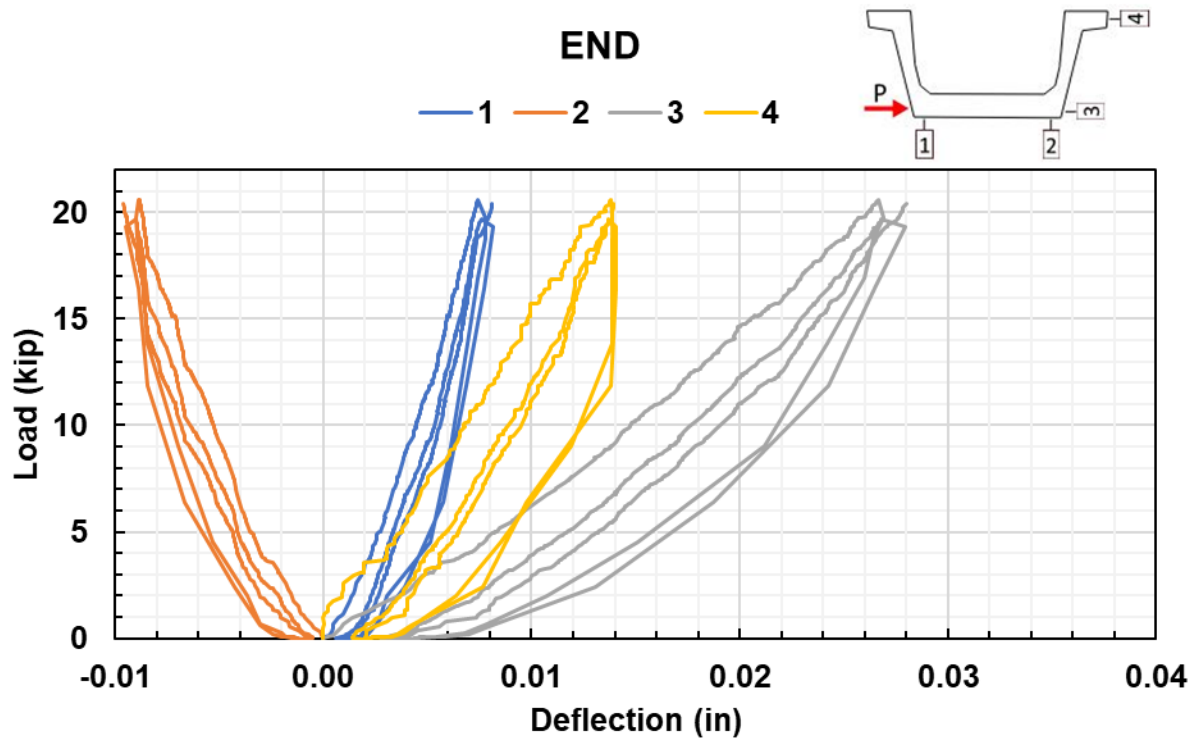


Figure 3.9. Tx-U40 lateral test load-deflection at girder end

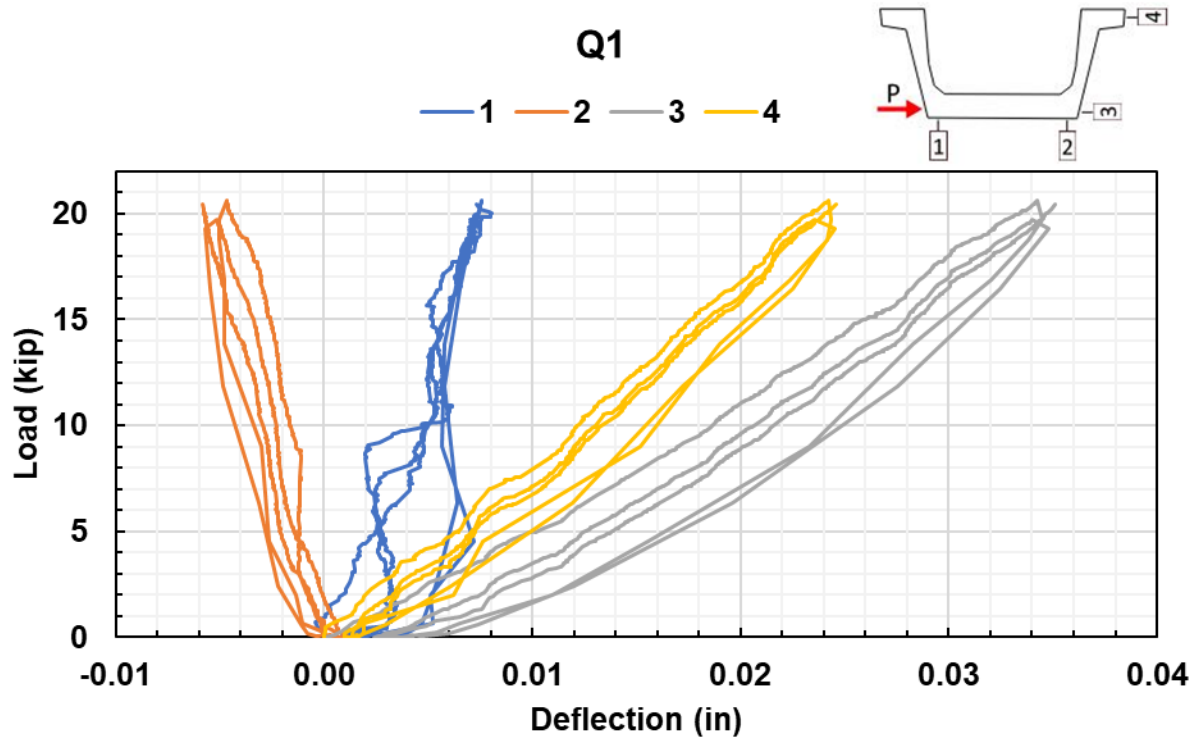


Figure 3.10. Tx-U40 lateral load-deflection at first quarter point (Q1)

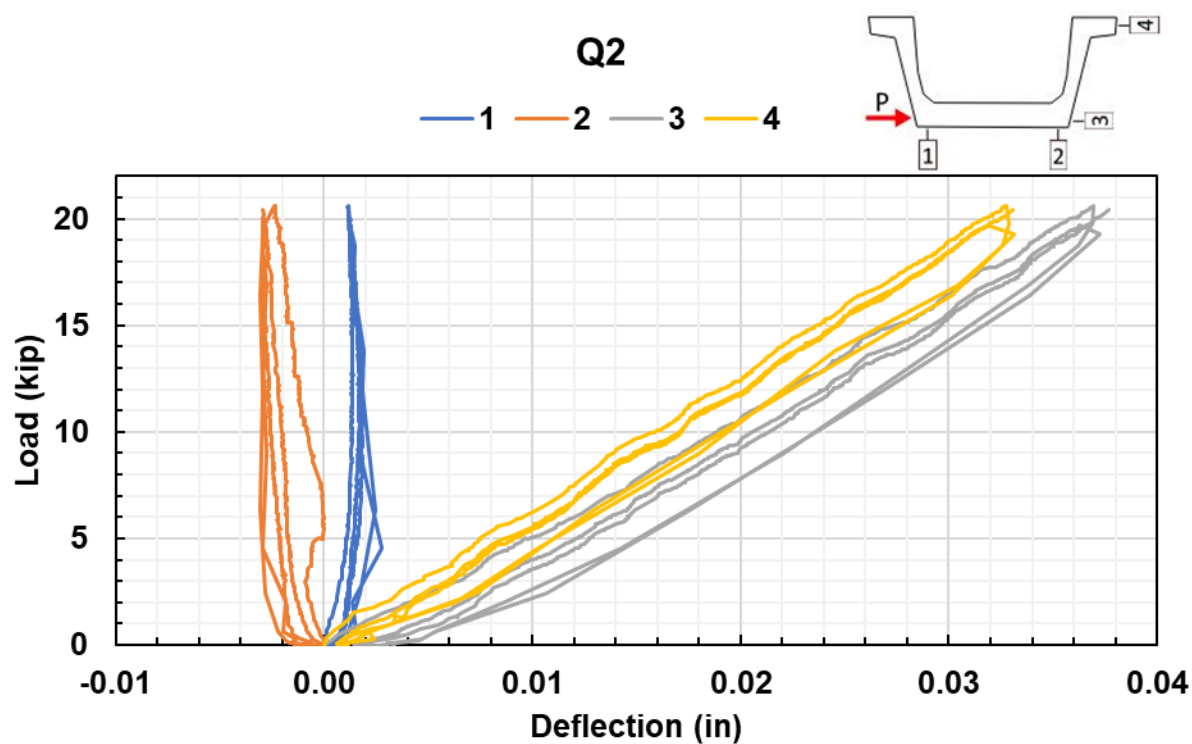


Figure 3.11. Tx-U40 lateral test load-deflection at midspan (Q2)

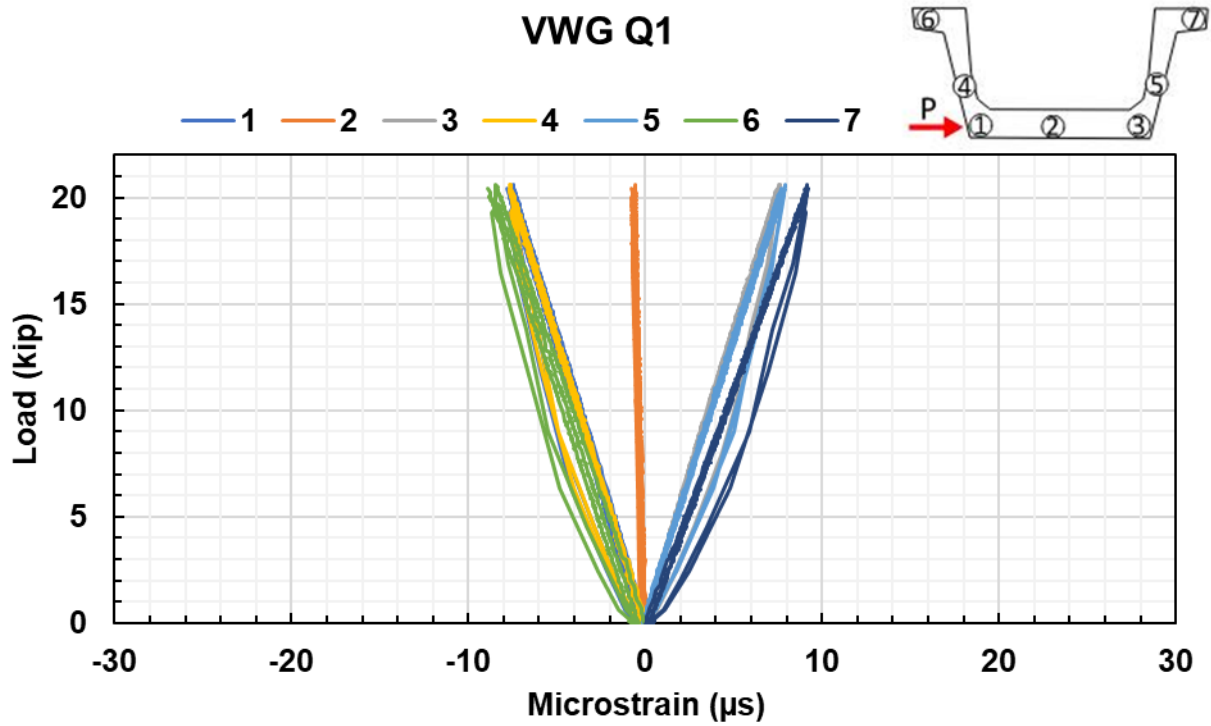


Figure 3.12. Tx-U40 lateral test graph of load-strain at first quarter point (Q1)

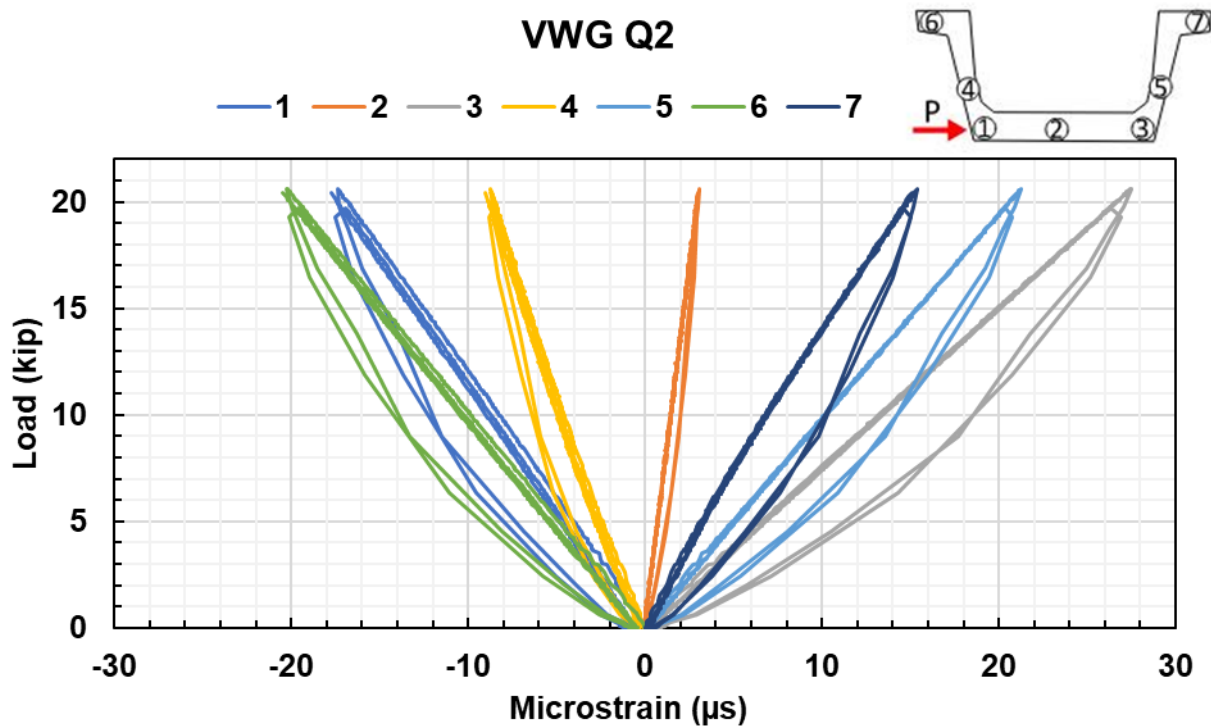


Figure 3.13. Tx-U40 lateral test graph of load-strain at midspan (Q2)

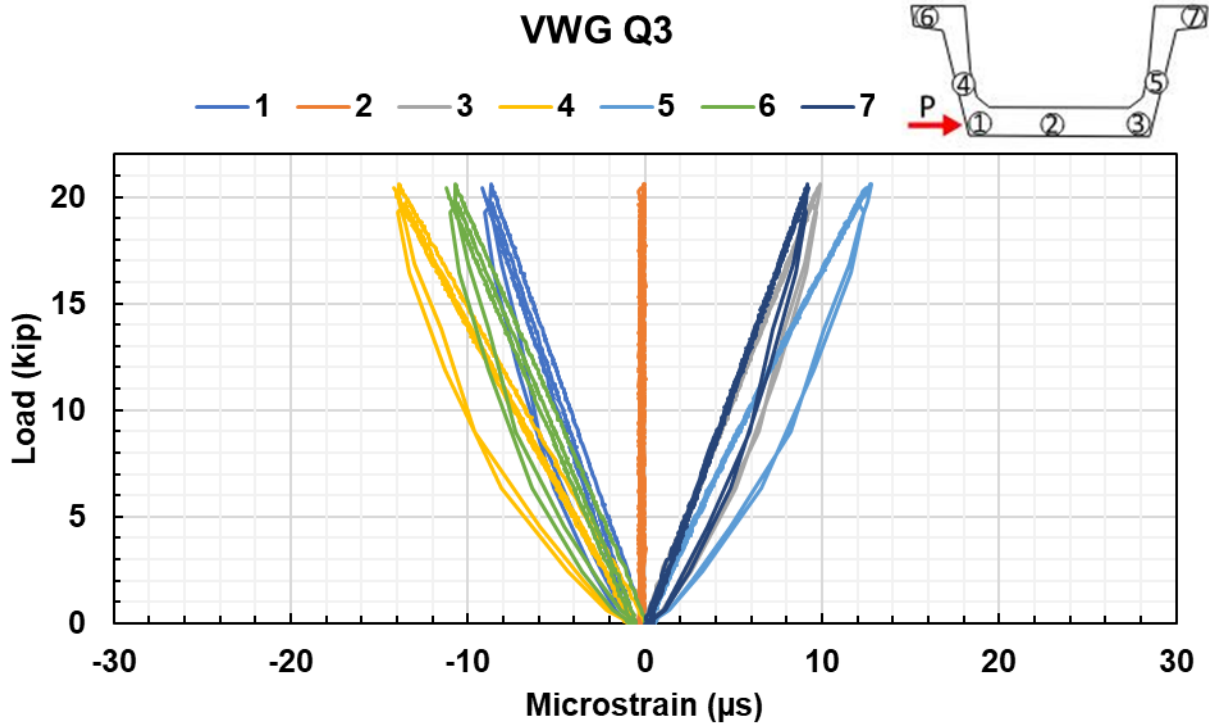


Figure 3.14. Tx-U40 lateral test graph of load-strain at second quarter point (Q3)

As mentioned previously, the primary objective of the first lateral load test was to determine the section properties of the uncracked specimen. The specimen was subjected to a maximum lateral force of 20 kips, which was a level deemed not to cause significant damage/cracking. As shown in the figures, the deflection, strain, and load histories can be characterized as linearly elastic without incurring cracking caused by external lateral force.

3.2.2. Description of Vertically Applied Loads with Gravity Load Simulators

Following the completion of the lateral load tests, the Tx-U40 girder was elevated to provide clearance for the gravity load simulators (GLSs), which were used to apply vertical loads to the girder. The girders were elevated by stacking three W30x173 sections on top of one another at both ends of the span, as depicted in Figure 3.15. Gravity load simulators are loading mechanisms that simulate gravity-based vertical loading through a hydraulic actuator. The GLS frames include needle bearings and provide a negligible amount of lateral restraint. Therefore, the girders are generally free to displace laterally as the girders experience potential instabilities or torsional deformations when the loads are applied eccentrically. The two GLS frames were fabricated by Wongjeeraphat (2011), with load capacities of 160 kips, and can accommodate a lateral movement of approximately 6 inches. An overview of the experimental setup for the vertical load tests, including both concentric and eccentric loading configurations, is shown in Figure 3.15, and Figure 3.16. The loading was applied at midspan. For the case with concentric loading, the point of load

application on the girders was applied directly above the centroid of the girder, with the load point reacting on the internal diaphragm of the U-girder. The fixture at the load point consisted of a hardened round bar welded to a plate that was connected to the load beam. The longitudinal axis of the round reaction bar was oriented along the longitudinal axis of the girder to accommodate the twisting of the U-shaped girder, as shown in Figure 3.17 and Figure 3.18. For cases with combined bending and torsion, the loading was applied eccentrically to the geometric centroid. For these cases, the point of load application was shifted three feet laterally and was applied over one of the webs of the girder.

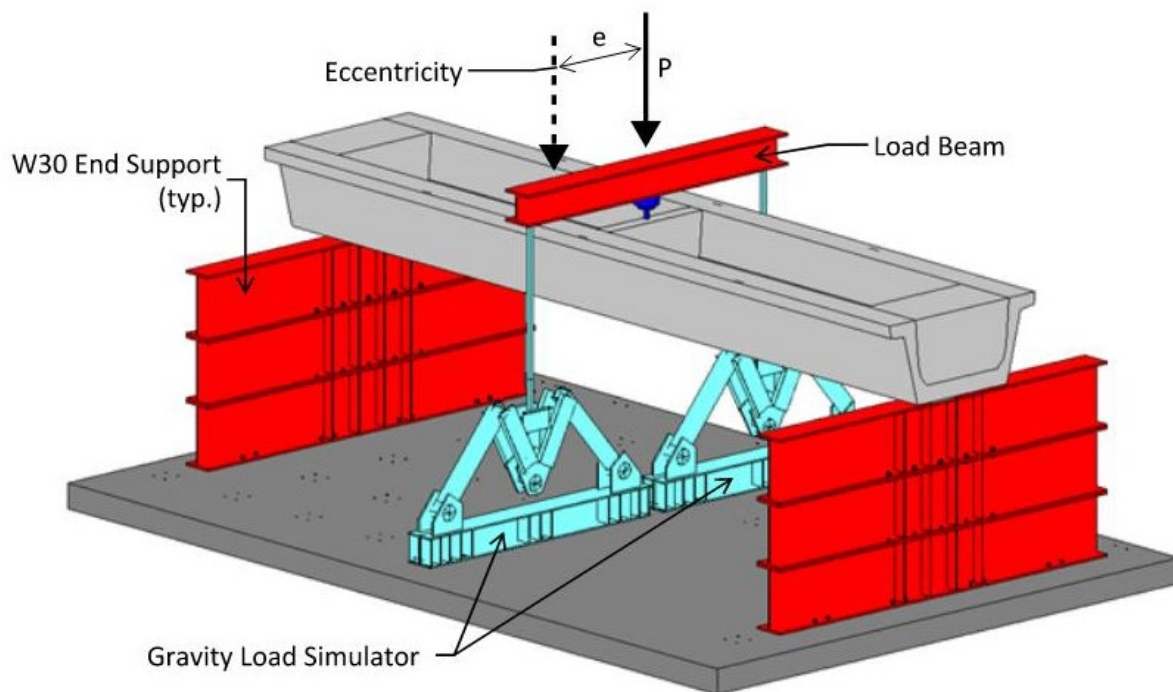


Figure 3.15. Overview of the vertical load test setups for the Tx-U40 specimen



Figure 3.16. Vertical load test of Tx-U40 specimen

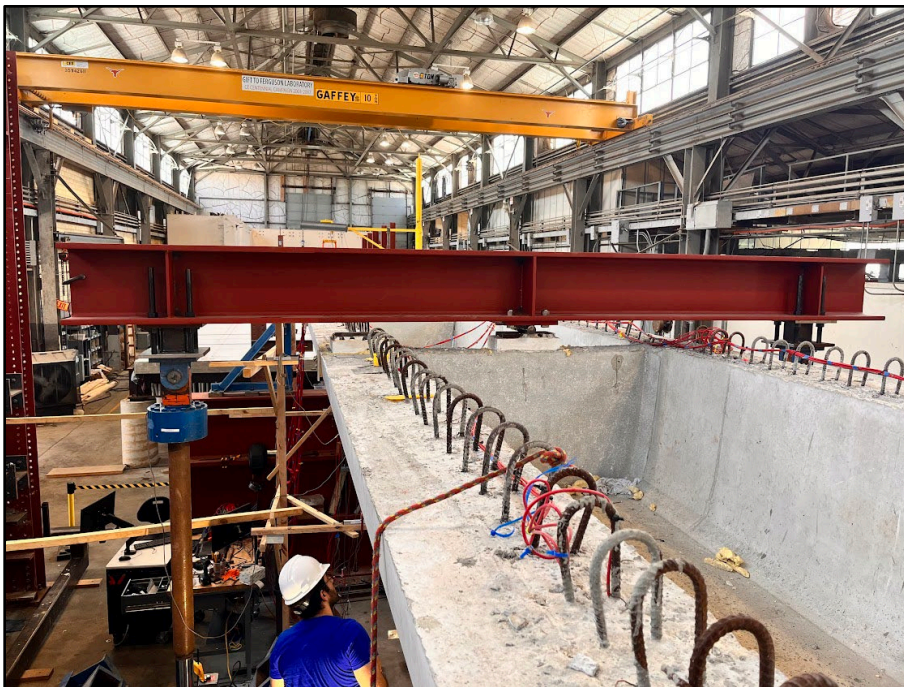


Figure 3.17. Load beam positioned at midspan of Tx-U40 specimen

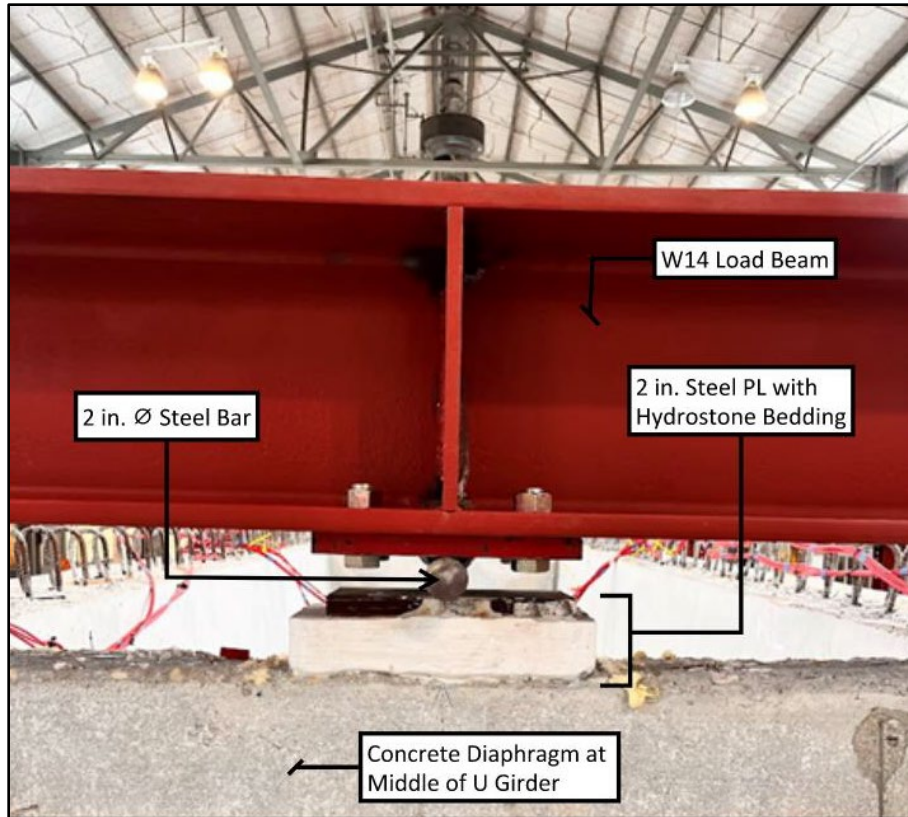


Figure 3.18. Elevation of vertical load application on Tx-U40 specimen

3.2.3. Combined Bending and Torsion Test (uncracked)

Prior to the experiments with combined bending and torsion, a vertical load test was performed with a maximum force magnitude of 50 kips. The behavior with loading up to 50 kips is captured in the section related to concentric loading in Section 3.2.4. Where the behavior in the elastic region was captured when the girder was loaded to cracking.

The strain profiles from the VWG gauges for a 50-kip load with a three-foot eccentricity are plotted in Figure 3.19. The strains on the girder web directly below the load are noticeably larger than the other web, as is expected. In addition, the strain profile from the gauges on the bottom flange has larger strains beneath the “loaded” web.

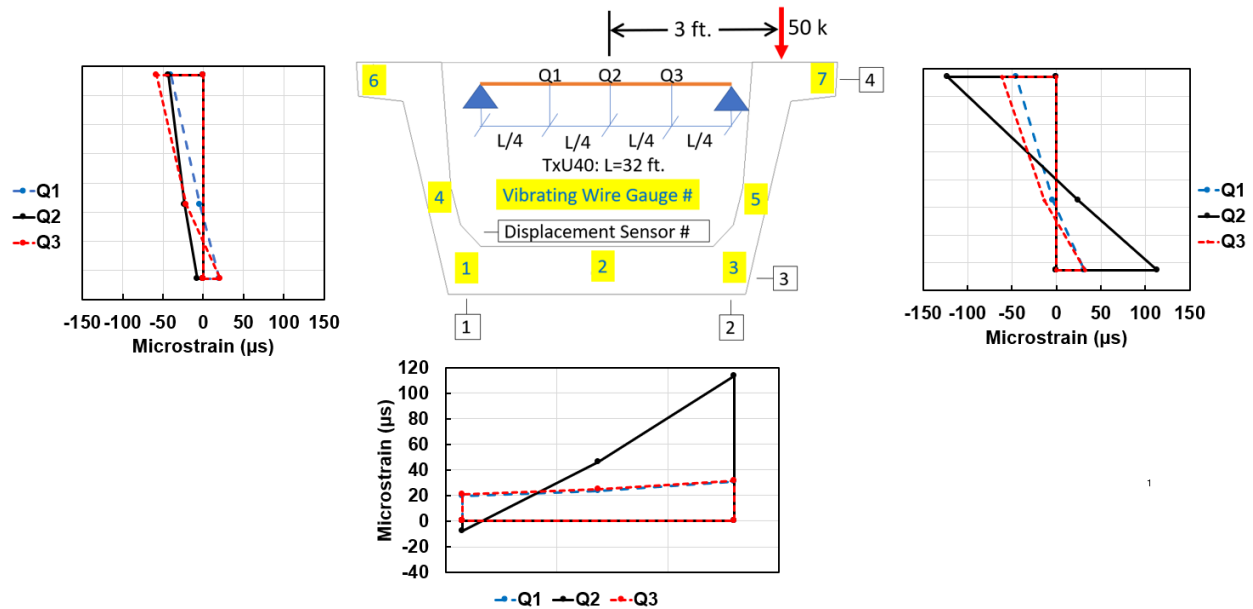


Figure 3.19. Eccentric load strain profiles and sensor layout for the Tx-U40 at 50 kips

Figure 3.20 through Figure 3.22 present graphs of the load-deflection responses for three cycles of monotonic loading up to 50 kips at an eccentricity of three feet. The displacements were measured at the end, the first quarter point (Q1), and at midspan (Q2). The VWG strain readings for these cycles are shown in Figure 3.23 through Figure 3.25. The VWGs measured strains in the cross-sections at Q1, Q2 (midspan), and Q3. All of the displacement graphs exhibit a bilinear curve with a noticeable change in slope at approximately 10 kips during the unloading branch of the cycle. This change in slope is likely due to seating on the elastomeric bearings as well as settling in the end brackets that resist twist. At the end of the girder, linear potentiometer “3” experienced electronic problems with the readings following the first cycle of loading. Therefore, only one cycle of loading is shown in Figure 3.20. All other curves show three cycles of loading. Linear potentiometer 4 also showed a few oscillations in readings in one of the cycles. These slight changes in behavior were deemed to be unrelated to the girder behavior and were likely due to simple electronic/mechanical issues with the linear potentiometer. The results generally showed good repeatability.

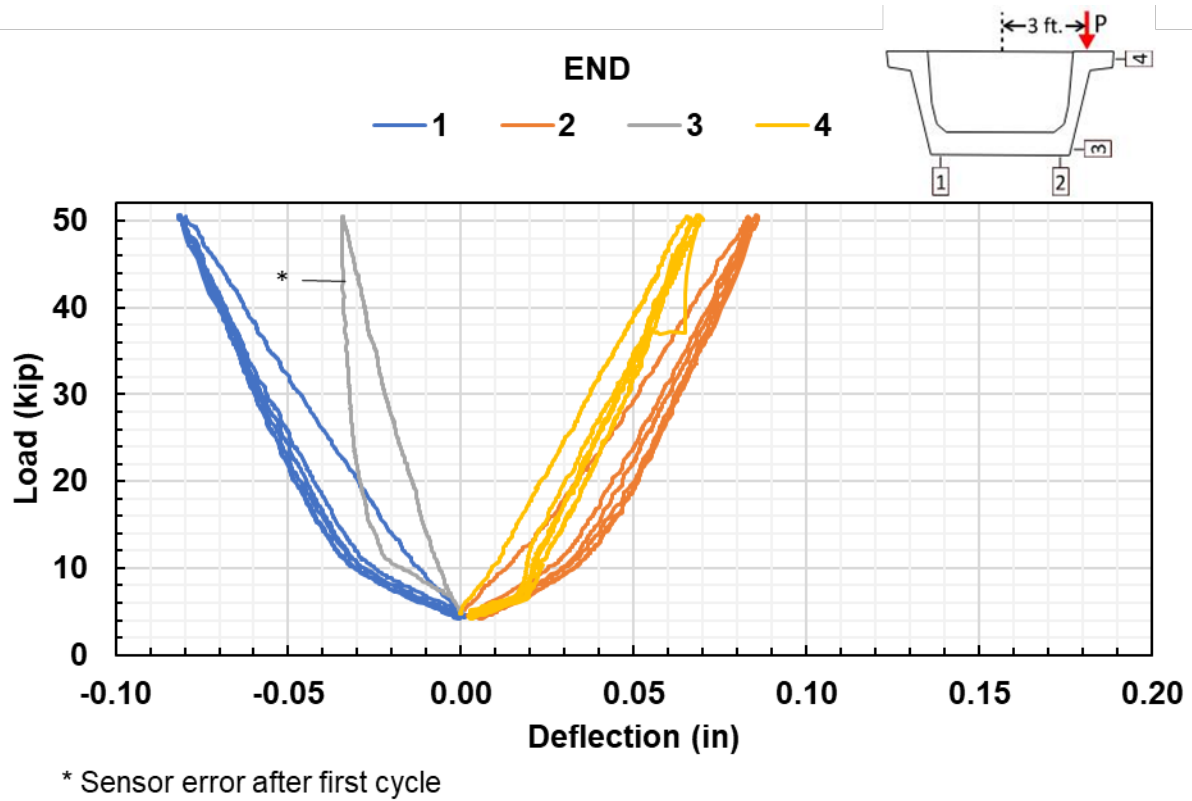


Figure 3.20. Tx-U40 combined bending and torsion test graph of load-deflection at girder end

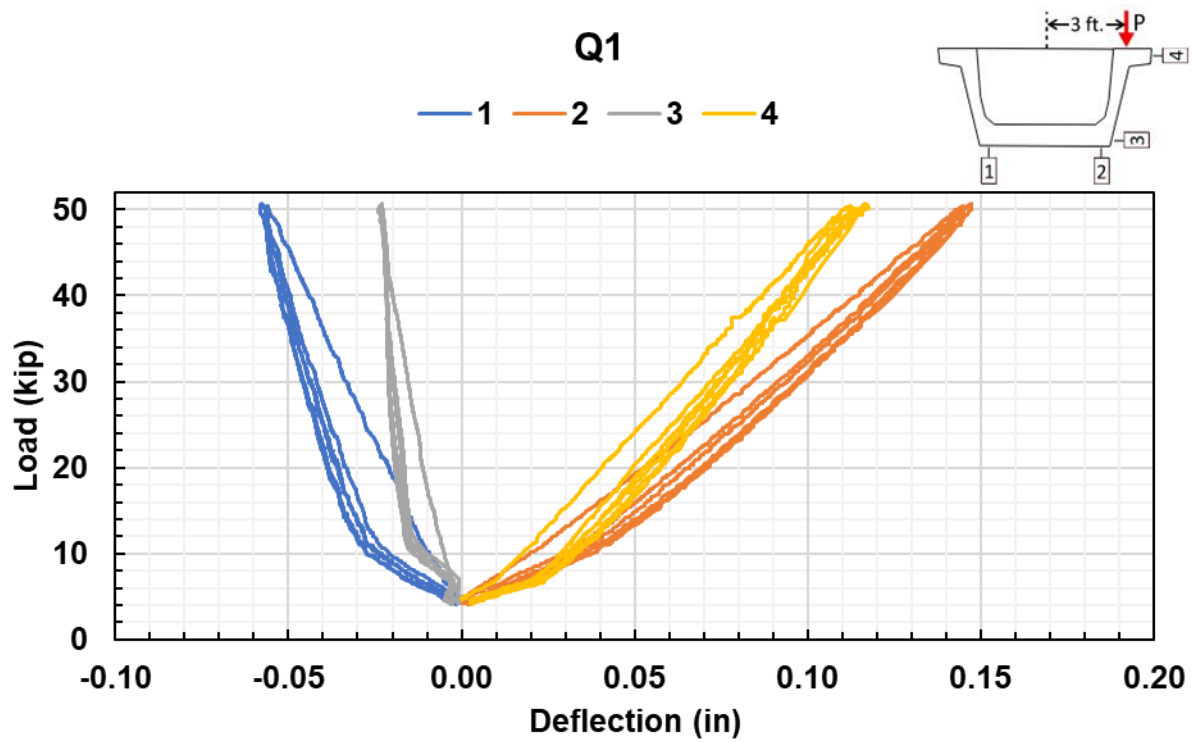


Figure 3.21. Tx-U40 combined bending and torsion test graph of load-deflection at first quarter point (Q1)

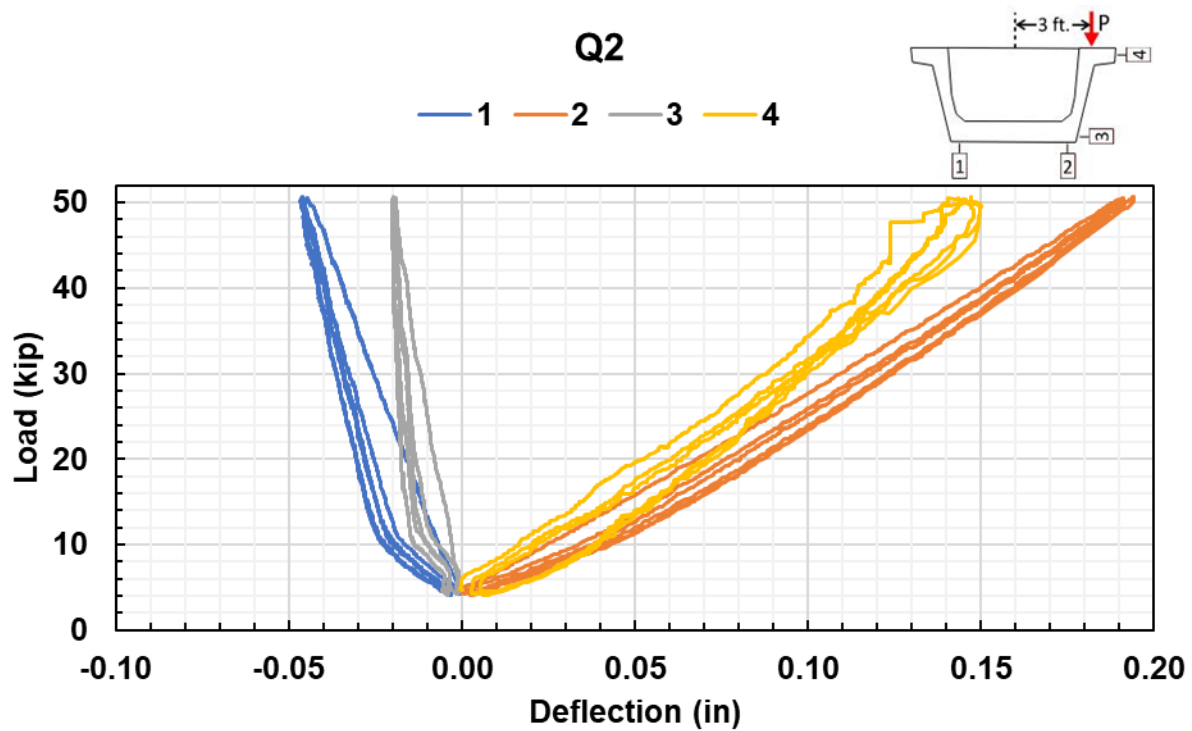


Figure 3.22. Tx-U40 combined bending and torsion test graph of load-deflection at midspan (Q2)

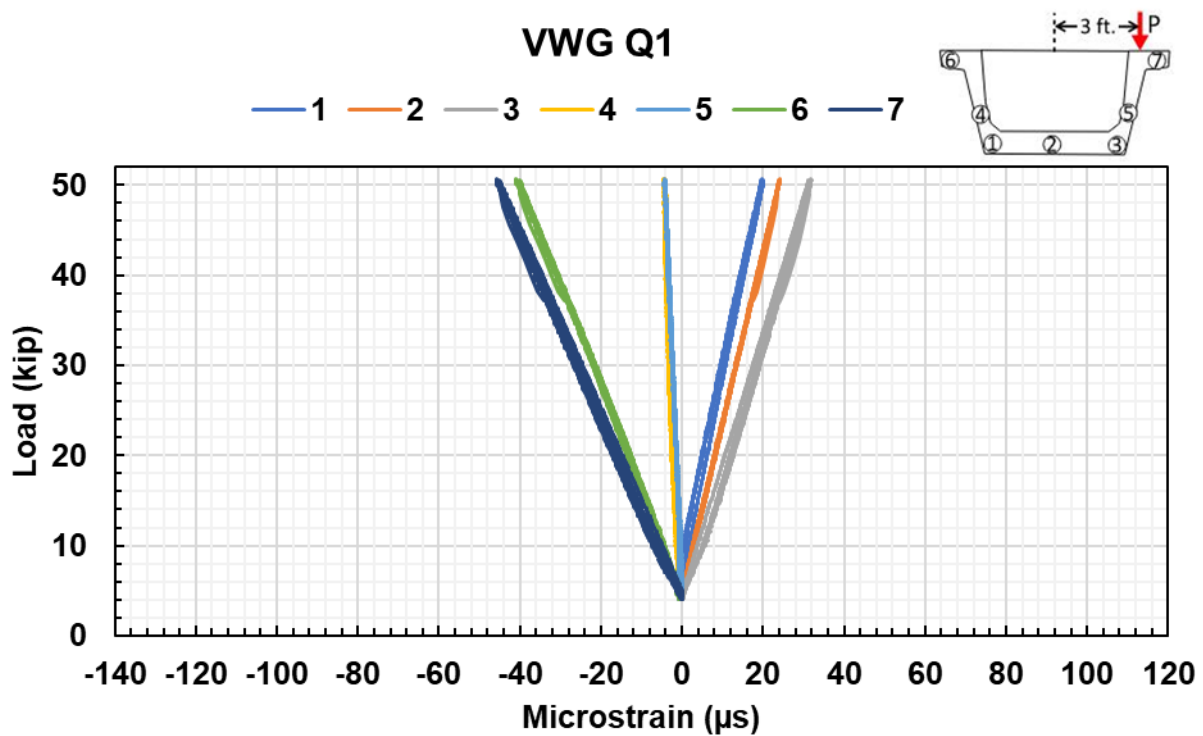


Figure 3.23. Tx-U40 combined bending and torsion test graph of load-strain at first quarter point (Q1)

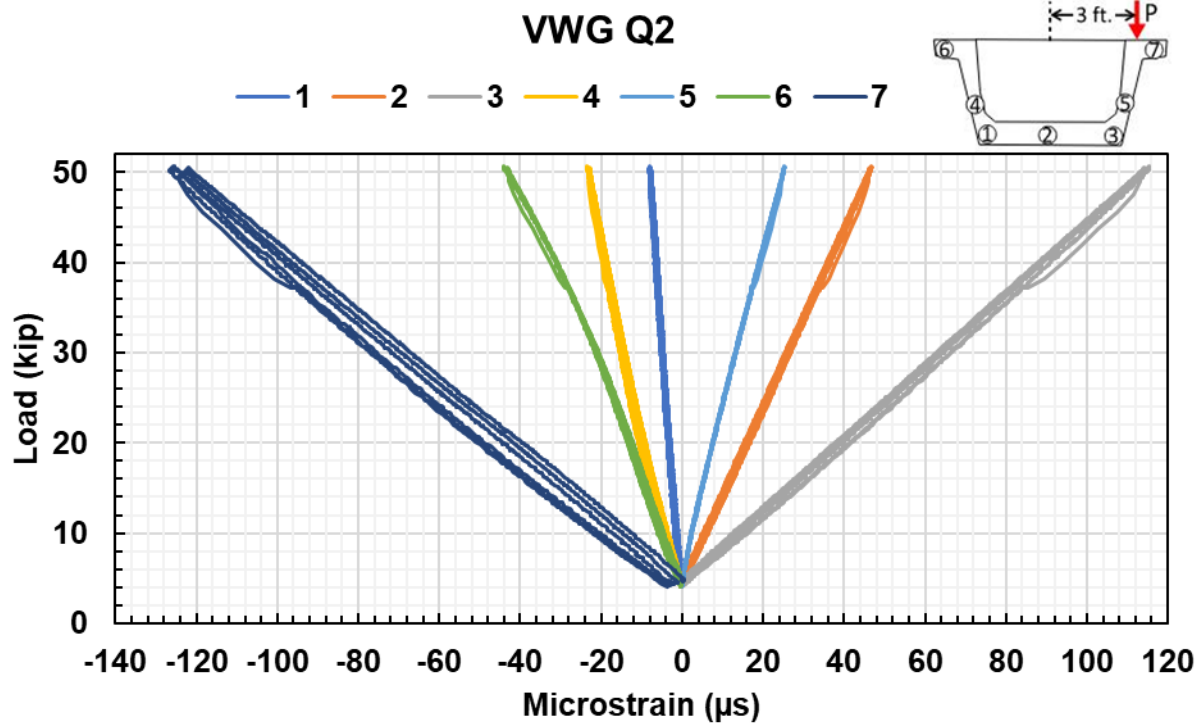


Figure 3.24. Tx-U40 combined bending and torsion test graph of load-strain at midspan (Q2)

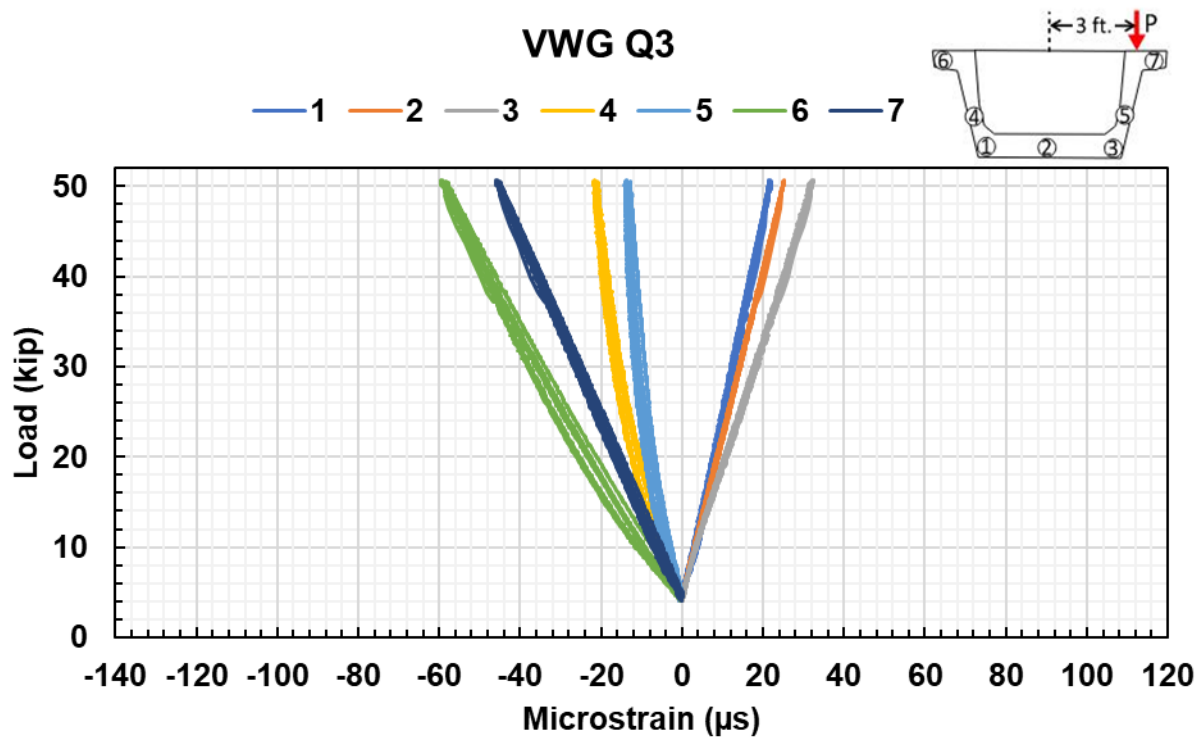


Figure 3.25. Tx-U40 combined bending and torsion test graph of load-strain at second quarter point (Q3)

The maximum applied force in the tests was 50 kips to prevent cracking of the concrete during these early tests. The deflection, strain, and load histories demonstrated a linearly-elastic behavior, with no crack initiation, verified through visual inspection and data measurements.

3.2.4. Bending Test into Cracked Region

Following the lateral load tests and the combined bending and torsional test, the girders were loaded concentrically to load levels corresponding to where cracking occurs. As noted earlier in the discussion of the testing procedure, the concentric load was applied along the girder centerline on the diaphragm at midspan. The load levels were increased into the cracking region of the concrete. A total of three cycles of loading were conducted. The primary rationale for the test was to obtain data on the moment-curvature behavior in both cracked and uncracked regions to evaluate section properties that can be used in the parametric FEA studies. The data from the experiments also provides insight into the impact of the prestressing on the cracked section behavior. Cracks in pre-compressed regions will generally close when the applied loads are removed. The experiments provide data on the behavior for subsequent loading. The girders were intentionally cracked in strong axis flexure. The behavior of the cracked girders in subsequent combined bending and torsion could then be evaluated.

The strain profiles from the VWG gauges with a 220-kip midspan load are provided in Figure 3.26. The strains at midspan (Q2) are significantly higher than at the quarter points (Q1 and Q3) since the girder was cracked at the midspan region. There was no observed cracking at the quarter-point locations, which is consistent with the relatively small strains. The strains along one of the webs are slightly kinked – perhaps from a slight misalignment of one of the VWGs. The strains along the width of the bottom flange are also slightly kinked instead of constant. The difference in the middle gauge may be from the loads applied at the diaphragm that might have some local effects on the strain gauge directly below the point of load application in addition to the variability of the crack pattern.

The moment-curvature response at midspan is provided in Figure 3.27. There is a noticeable difference between the first cycle of loading and the subsequent cycles of loading. The first cycle of loading is linear until approximately 1,200 kip-ft, and there is a noticeable change in slope once the concrete had cracked. The initial stiffness on subsequent load cycles was similar to the uncracked behavior; however, the nonlinearity began at a lower load level of approximately 800 k-ft. The change in slope at an early load level is consistent with the reduction in stiffness from cracked section properties. There was good repeatability between load cycles 2 and 3.

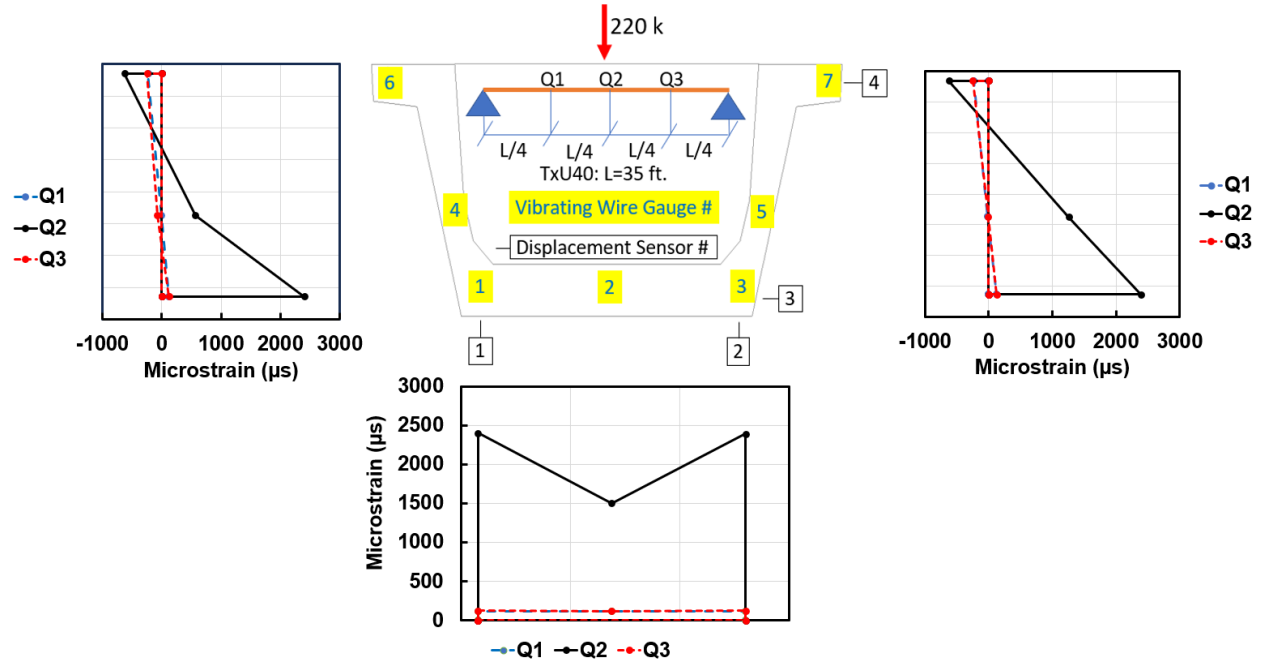


Figure 3.26. Tx-U40 220-kip concentric load strain profiles and sensor layout

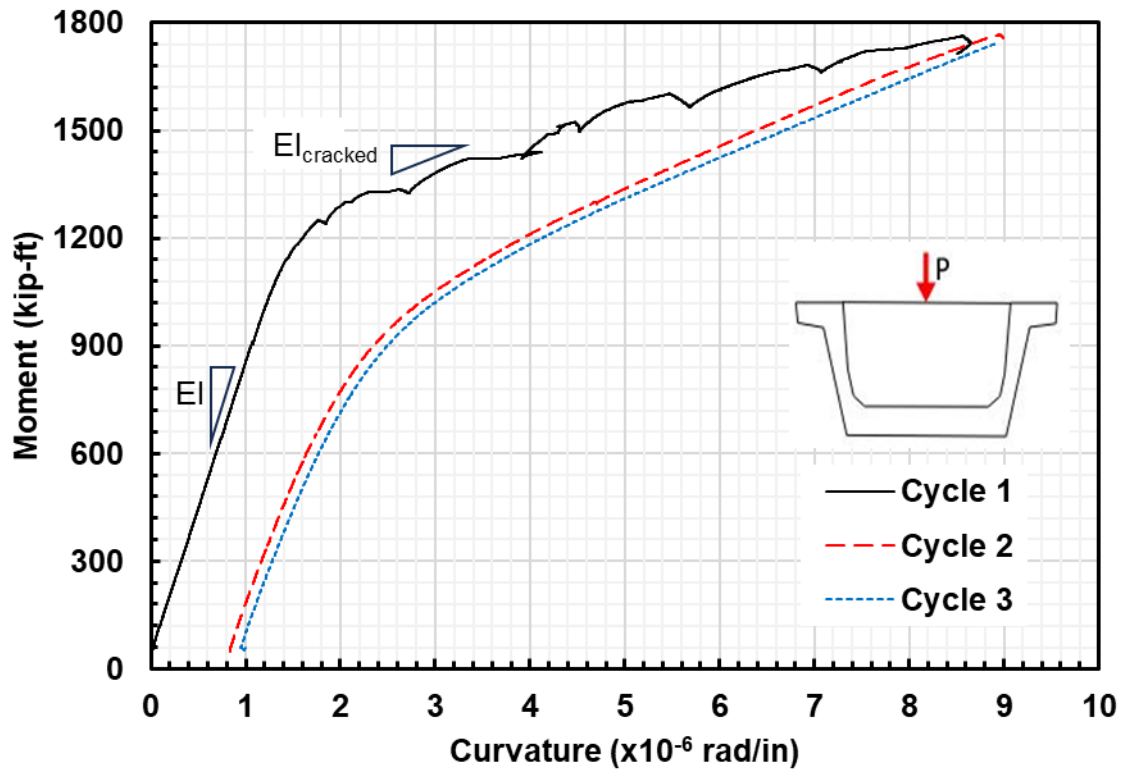


Figure 3.27. Moment-curvature response at midspan obtained from experimental test results of Tx-U40 specimen

Figure 3.28 through Figure 3.30 provide graphs of the load-deflection responses for three cycles of monotonic, concentric loading with a maximum applied midspan load of 220 kips. The displacements were measured at the end, first quarter point (Q1) and midspan (Q2). The graph of the support deflections in Figure 3.28 shows that there was no significant lateral deformation/twist at the support (Linear Potentiometer (4)), as expected. The deflections represented from the measurements given by Linear Potentiometers (LPs) 1 and 2 represent the compressibility of the elastomeric bearings. The difference in vertical deformation from the two sides of the bottom flange demonstrates a small amount of twist, likely caused by some small torsion from the applied loading. The torsion may have resulted either from small girder imperfections (sweep or twist) or positioning tolerances on the load point. While the difference between the deflection readings from LPs 1 and 2 appears significant in the graph – the relative magnitudes of the displacements are small.

Figure 3.29 and Figure 3.30 show that there was very little twist or lateral deflection at the quarter points or midspan as evidenced by the relatively small deformations measured by LPs 3 and 4. The small differences in the LP readings at the top and bottom of the section may be due to a rigid-body rotation, which would be similar to what was observed at the two LPs at the ends of the girders. As expected, the maximum vertical displacement occurred at midspan. Although nonlinear behavior in the deflection graph was observed at both the quarter point and midspan, the nonlinearity is more significant in the midspan deflection in Figure 3.30, which is in the vicinity where the girder cracked. As noted earlier, there was no cracking observed at the quarter points.

The VWG strain readings for the load cycles are shown in Figure 3.31 through Figure 3.33. The VWGs measured strains in the cross-sections at the first quarter point (Q1), midspan (Q2), and the second quarter point (Q3). A comparison of the strain variations at the two quarter points from the graphs in Figure 3.31 and Figure 3.33 shows very similar behavior, which is to be expected due to the symmetry about midspan.

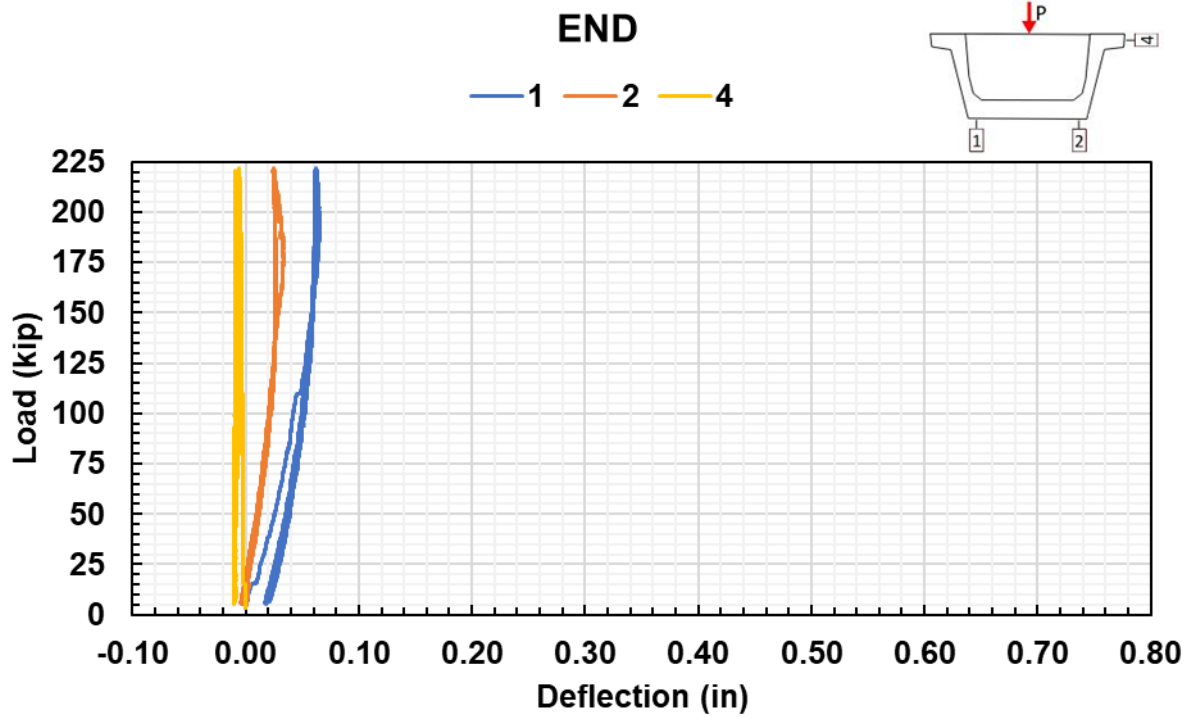


Figure 3.28. Tx-U40 bending to cracking test graph of load-deflection at girder end

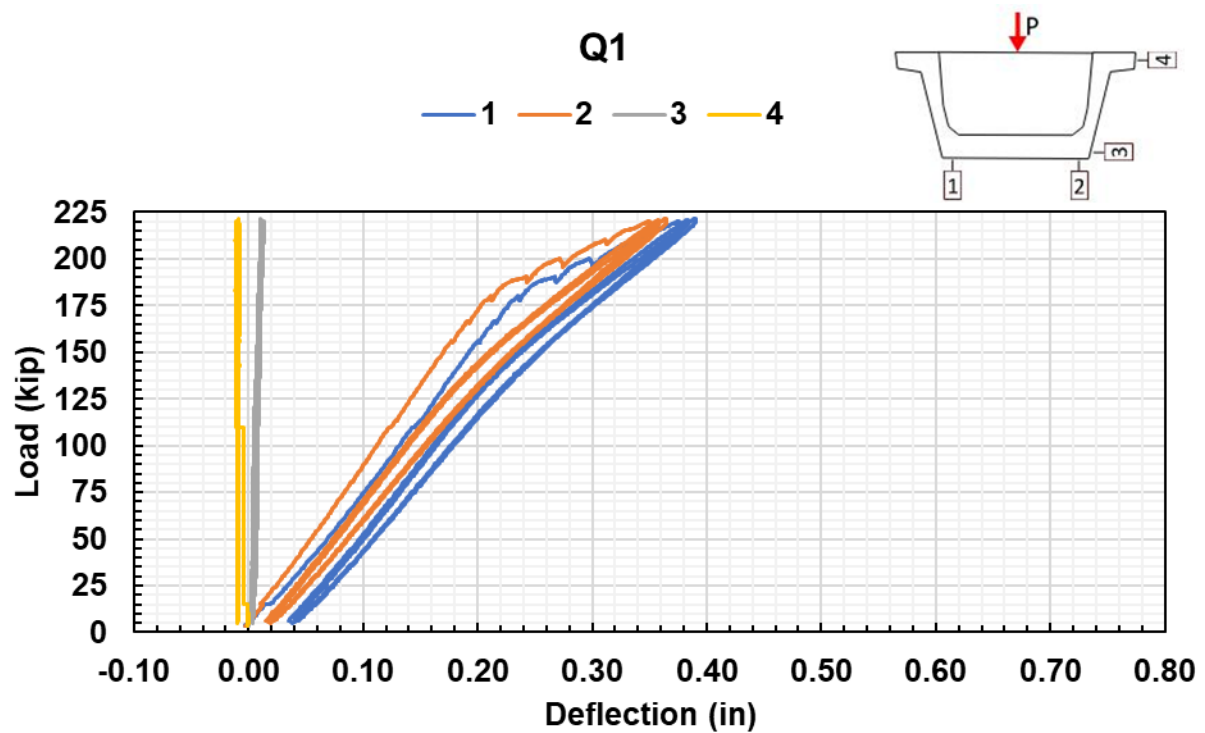


Figure 3.29. Tx-U40 bending to cracking test graph of load-deflection at first quarter point (Q1)

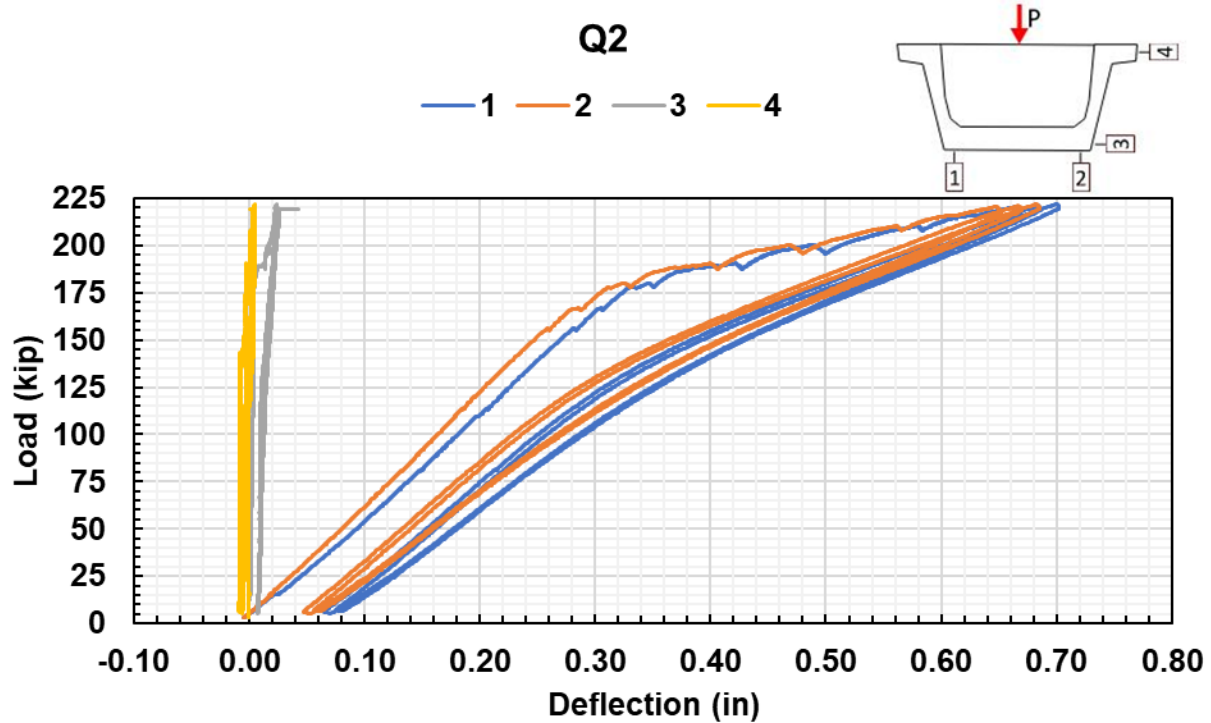


Figure 3.30. Tx-U40 bending to cracking test graph of load-deflection at midspan (Q2)

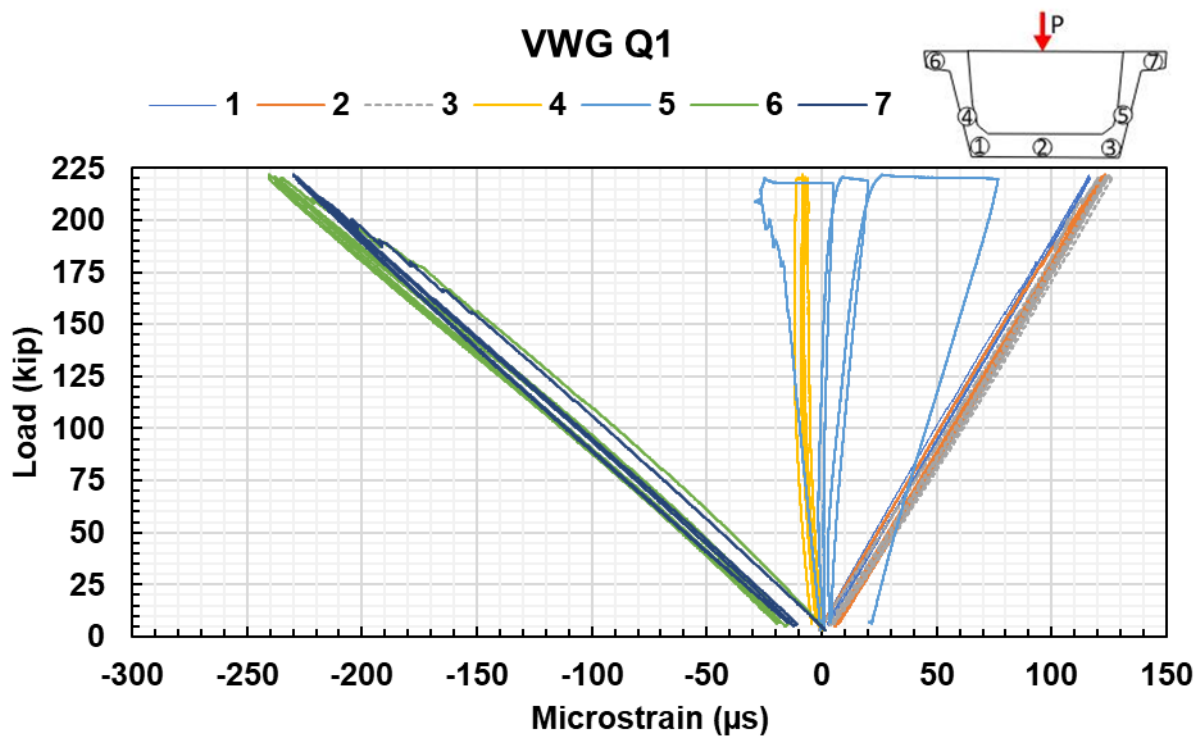


Figure 3.31. Tx-U40 bending to cracking test graph of load-strain at first quarter point (Q1)

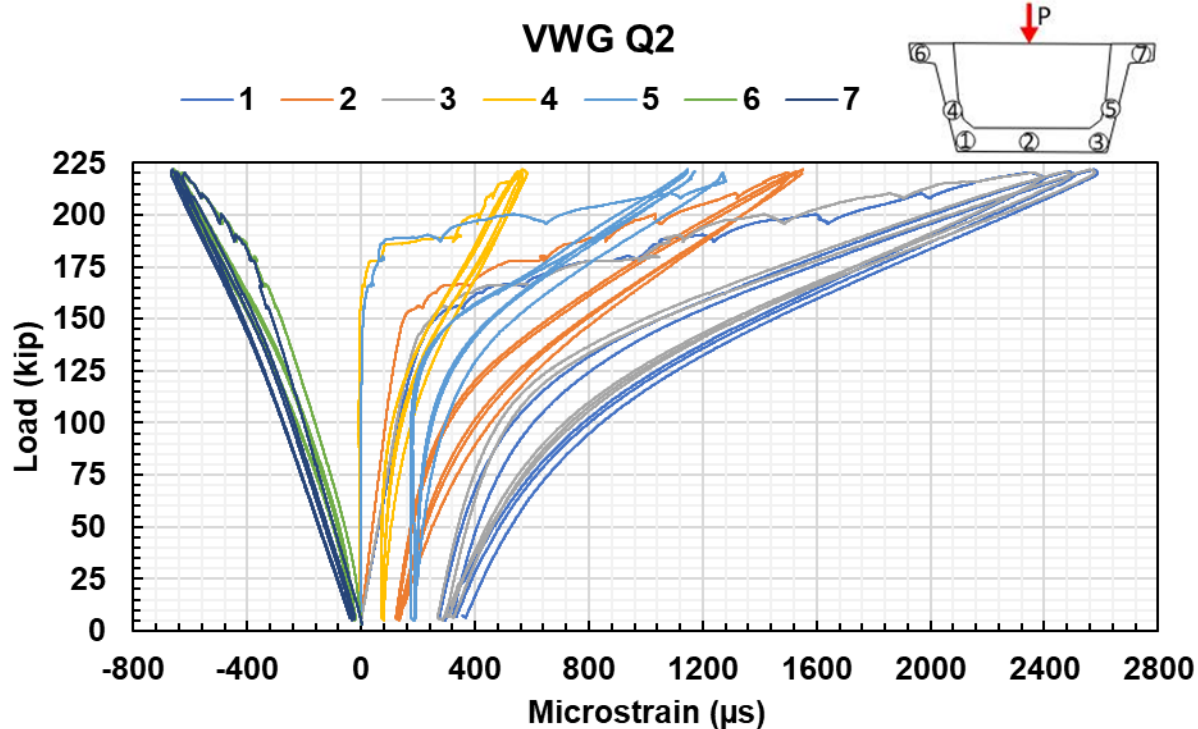


Figure 3.32. Tx-U40 bending to cracking test graph of load-strain at midspan (Q2)

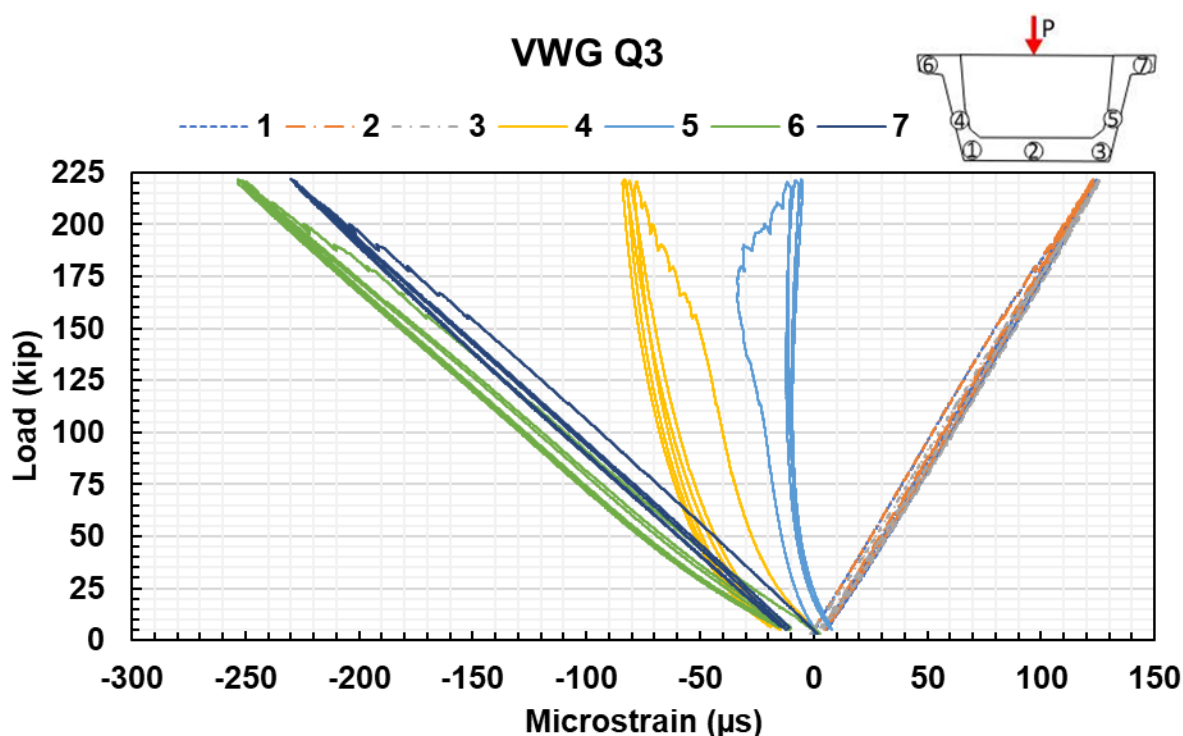


Figure 3.33. Tx-U40 bending to cracking test graph of load-strain at second quarter point (Q3)

The maximum applied vertical force was selected as 220 kips to enter the nonlinear region and capture the cracked sectional response, enabling subsequent test protocols for combined bending

and torsion in the cracked state. The deflection, strain, and load histories were characterized by a nonlinear response. Crack trajectories during the test are shown in Figure 3.34. An important outcome of using VWG gauges is that the finite element models were used to predict the strain at the VWG locations, and this can be compared throughout both linear and nonlinear responses. Therefore, the data can be used to validate the section properties in the uncracked and cracked states, as well as the damaged material models, which are used in the nonlinear range of material behavior.

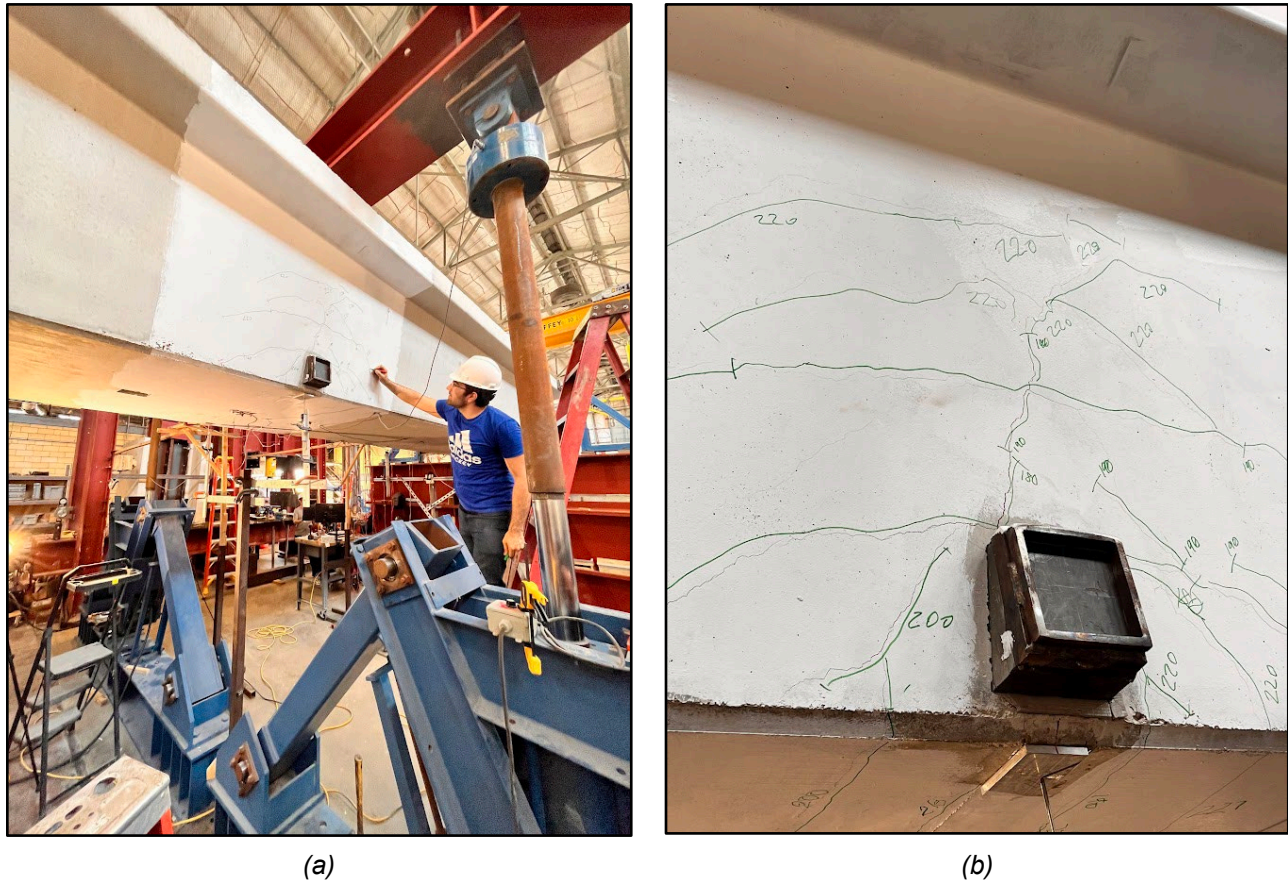


Figure 3.34. (a) Crack marking during testing; (b) Final crack trajectories at midspan

3.2.5. Combined Bending and Torsion Test after Cracking Load

Following the cracking of the section, the load case of combined flexure and torsion was repeated to investigate the impact of cracking on the torsional properties. While the maximum applied load on the uncracked section outlined previously in Section 3.2.3 was 50 kips, the load levels in this report section were 90 kips. The eccentricity was still 3 ft to match the previous value used. Therefore, the ratio of flexural and torsional moments was essentially the same. The strain profiles from the VWG gauges for a 90-kip load are plotted in Figure 3.35. The strain magnitudes with the 90-kip load are obviously larger than for the case with the 50-kip eccentric load. The strains along

the bottom flange show that cracking around midspan resulted in significantly larger strains than those measured at the quarter points where the section was uncracked.

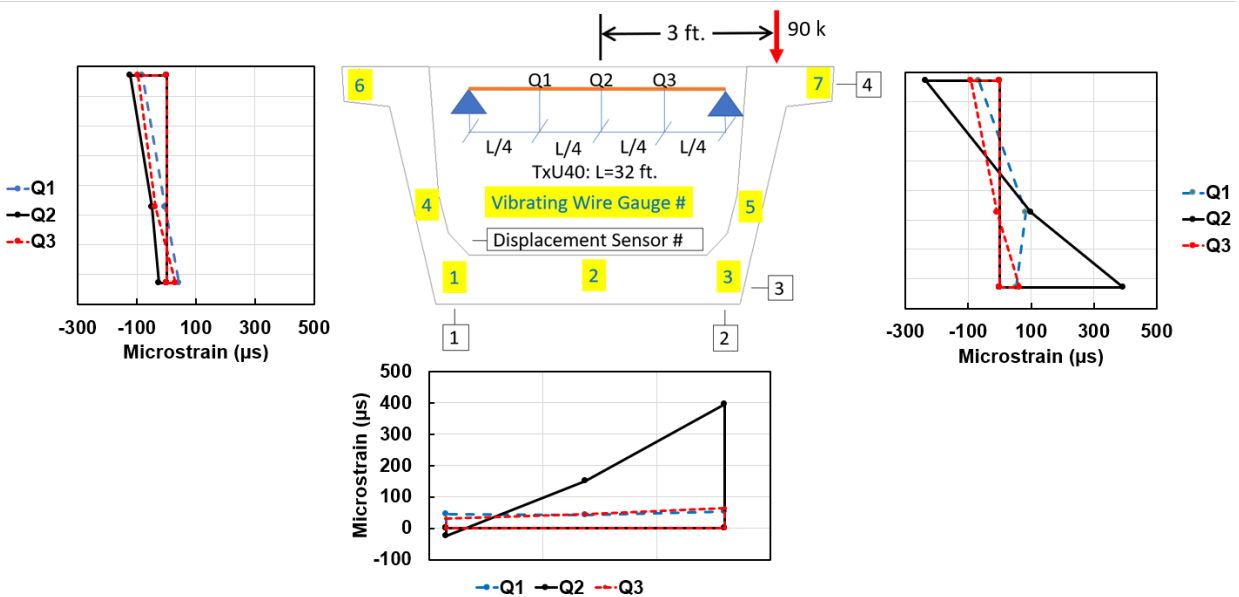


Figure 3.35. Tx-U40 90-kip eccentric load strain profiles and sensor layout

Figure 3.36 through Figure 3.38 provide the experimentally observed load-deflection responses for three cycles of monotonic loading up to 90 kips at an eccentricity of three feet. The displacements were measured at the end, first quarter point (Q1), and midspan (Q2). The curves presented for the support deformations in Figure 3.36 show large deformations that occurred due to a yielding in the end brackets restraining twist at the supports. The rounded threaded rod gouged into the embedded steel plate and caused combined axial and flexure forces in the threaded rod. Prior to the bending and yielding of the threaded rod, the support deformations were relatively small. The yielding of the end brackets at the supports resulted in a rigid-body rotation of the entire girder, as evidenced in the graphs of the girder deflections in Figure 3.37 (first quarter point) and Figure 3.38 (midspan). Prior to the end bracket yielding, the torsional deformations in the girder were behaving relatively linearly with only a small reduction in stiffness.

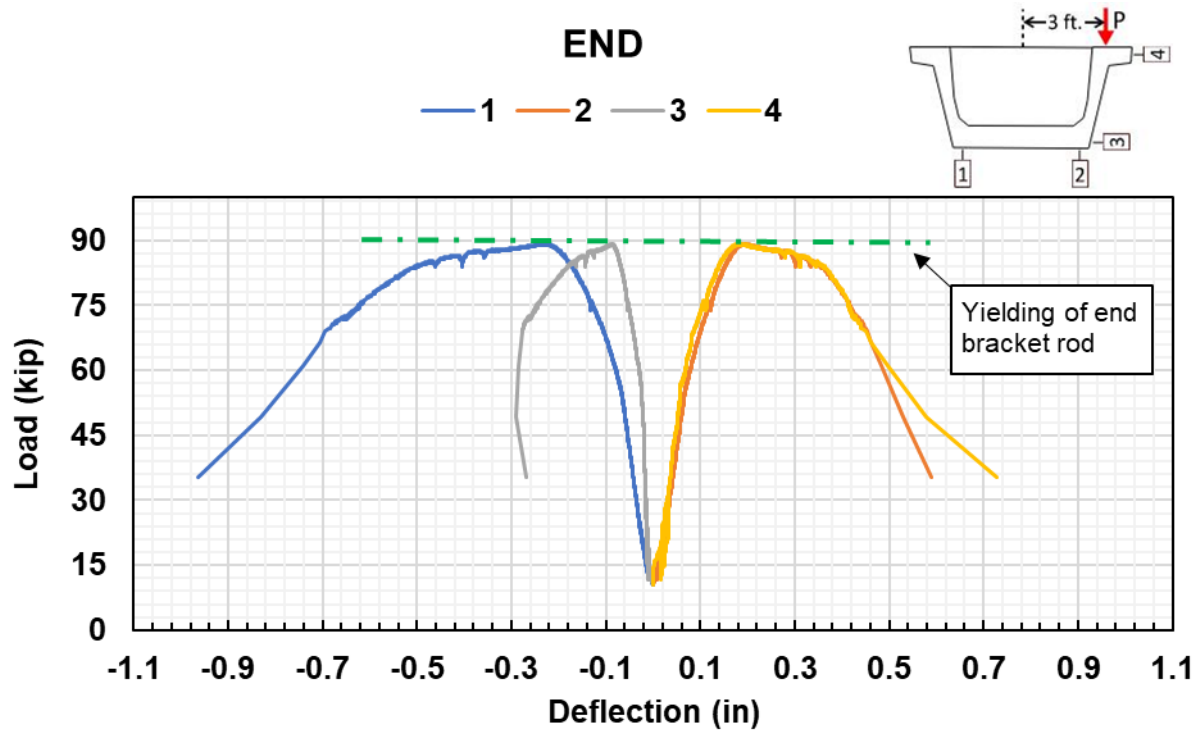


Figure 3.36. Tx-U40 combined bending and torsion test after cracking graph of load-deflection at girder end

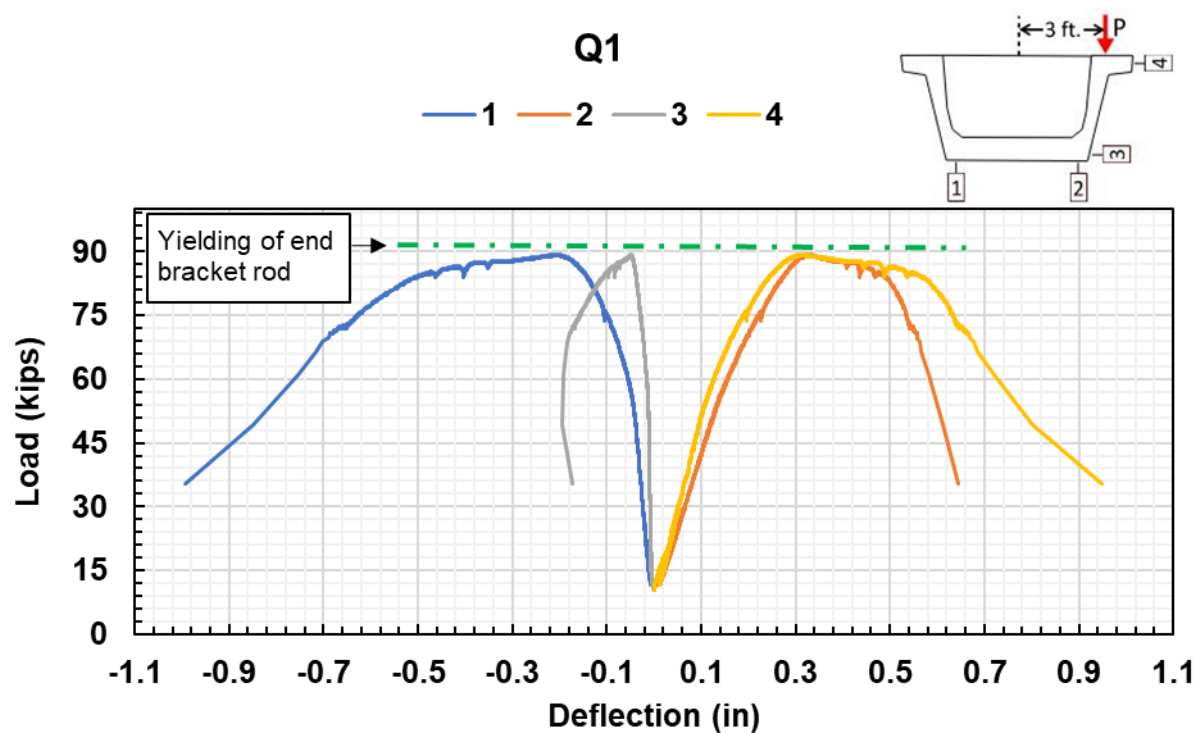


Figure 3.37. Tx-U40 combined bending and torsion test after cracking graph of load-deflection at first quarter point (Q1)

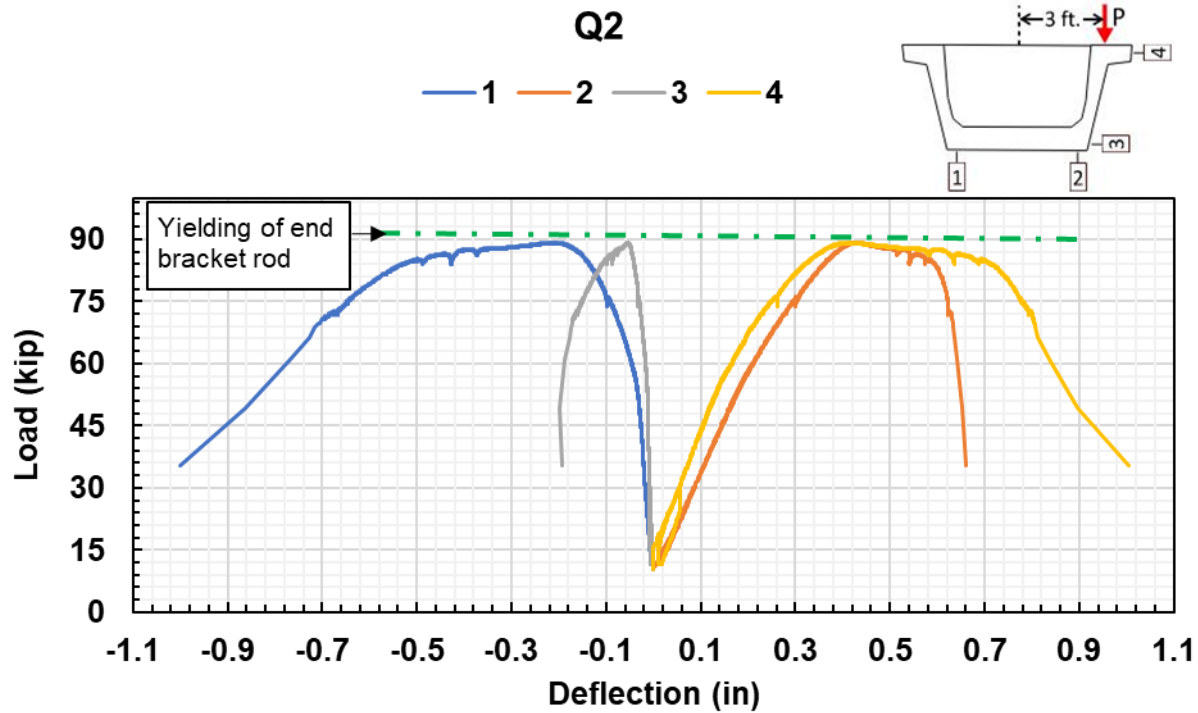


Figure 3.38. Tx-U40 combined bending and torsion test after cracking graph of load-deflection at midspan (Q2)

The VWG strain readings for these cycles are shown in Figure 3.39 through Figure 3.41. The VWGs measured strains in the cross-sections at Q1, Q2 (midspan), and Q3. Considering the measured strain behavior at the quarter points in Figure 3.39 and Figure 3.41, prior to the yielding at the supports, the strains were relatively linear at all points. After the yielding, there was some nonlinear behavior in a few gauges. However, this was likely due to the rigid body rotation of the section. The data of interest, which are the measurements prior to the end bracket yielding, were used to validate the FEA model for the parametric studies. There was some nonlinearity observed in the strain readings at midspan, as shown in Figure 3.40. That data represents some cracking and potential yielding in the reinforcing steel and was compared with predictions from the FEA model.

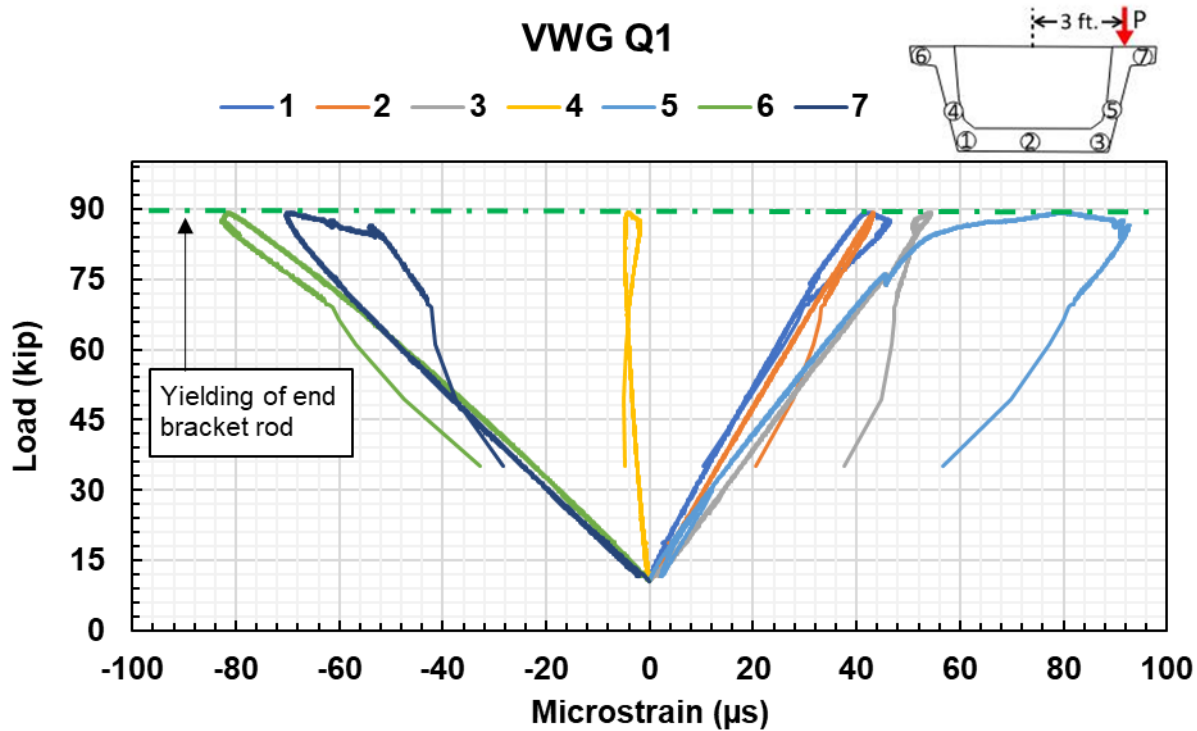


Figure 3.39. Tx-U40 combined bending and torsion test after cracking graph of load-strain at first quarter point (Q1)

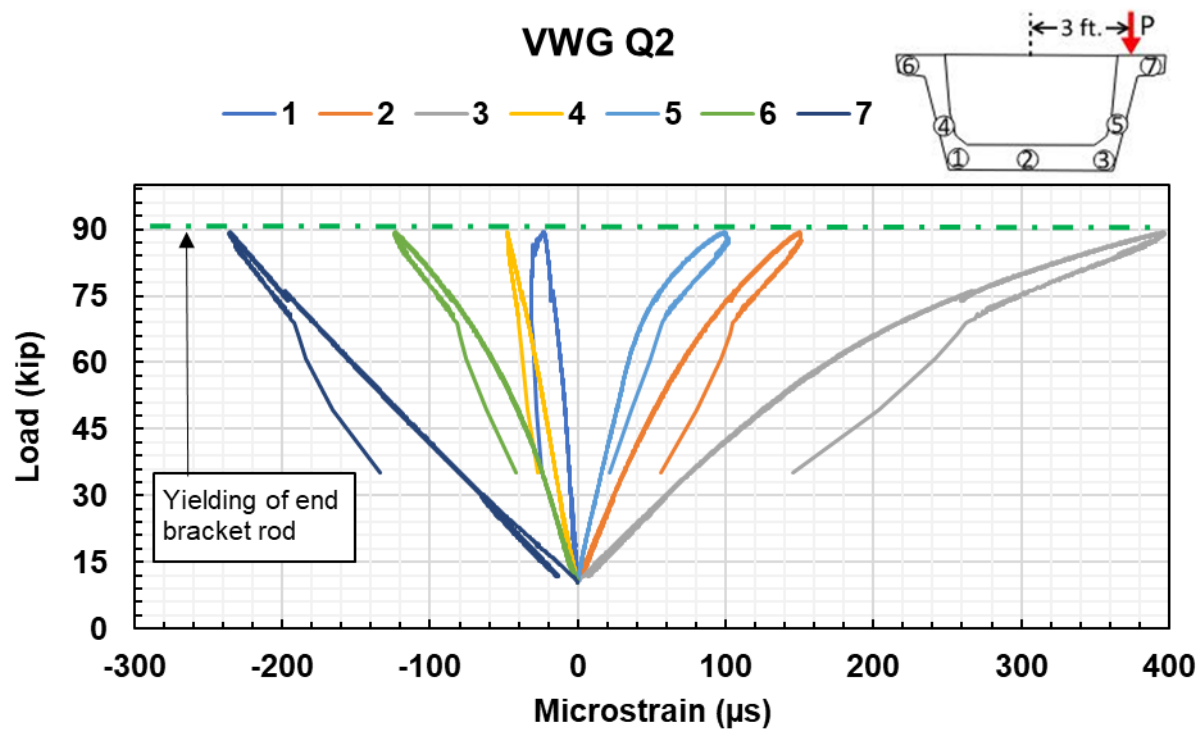


Figure 3.40. Tx-U40 combined bending and torsion test after cracking graph of load-strain at midspan (Q2)

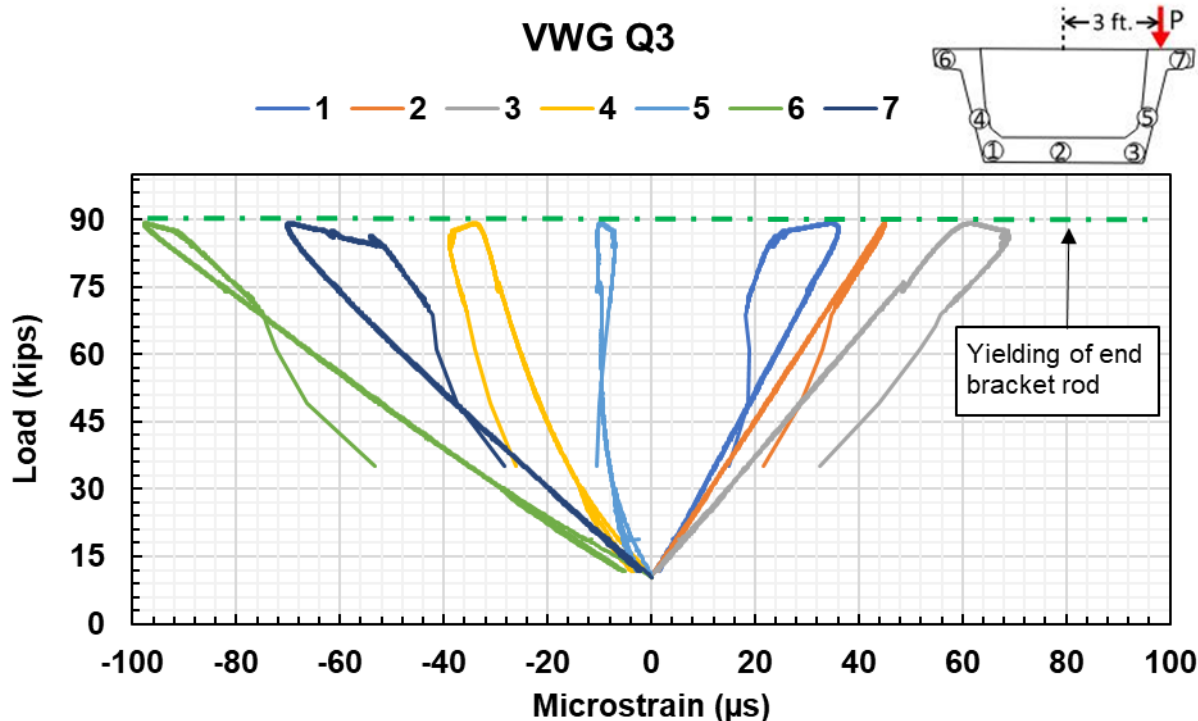
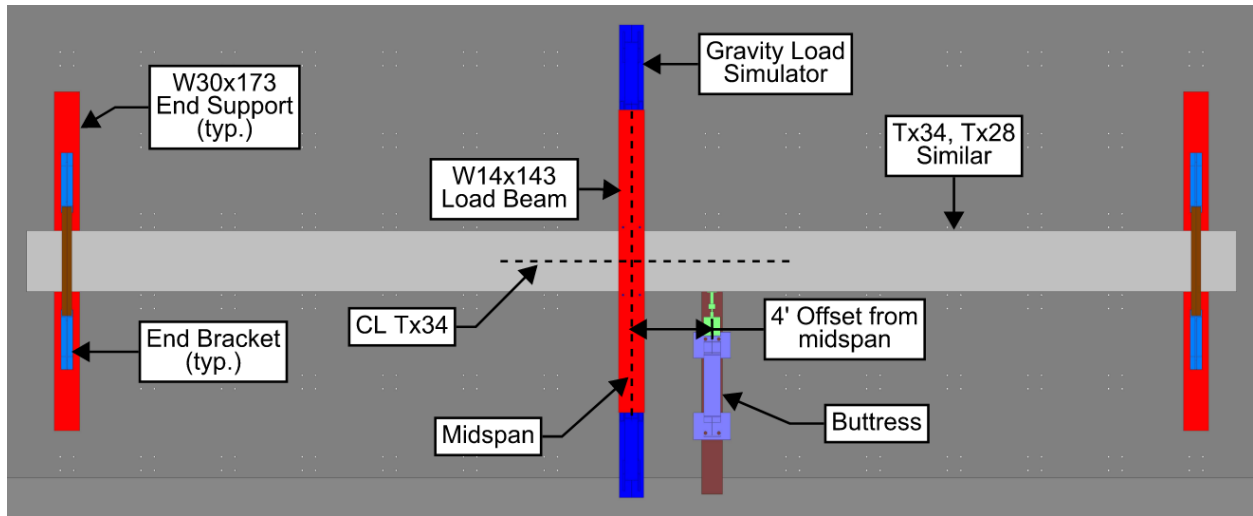


Figure 3.41. Tx-U40 combined bending and torsion test after cracking graph of load-strain at second quarter point (Q3)

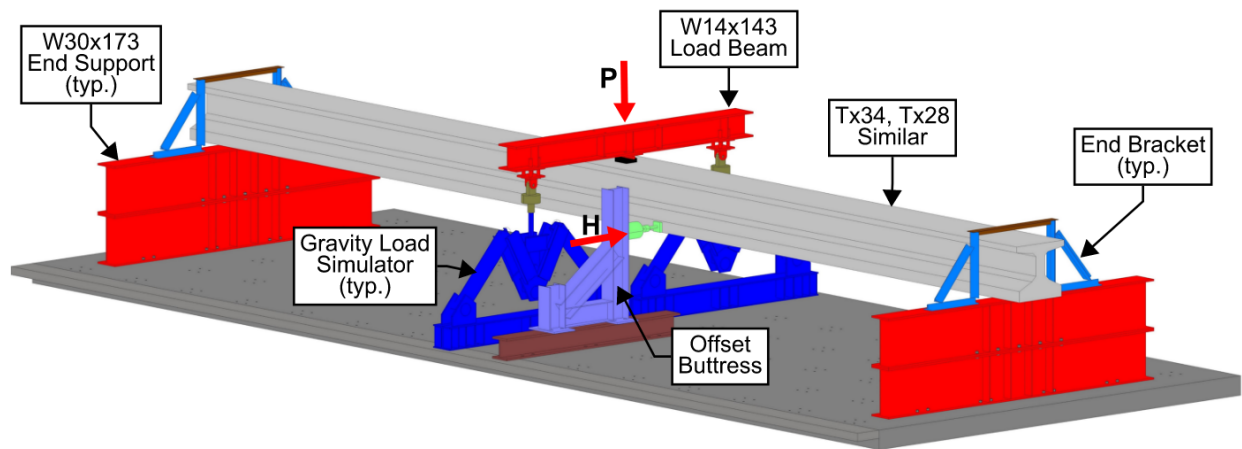
The data obtained from the various load cases conducted on the U40 girder provide insight into the section properties of the girders in the uncracked and cracked state. That data was used to validate modeling decisions, as discussed in Chapter 4.

3.3. Tx28 and Tx34 I-Shaped Girder Laboratory Experiments

The experiments on the I-shaped girders followed closely those performed on the Tx-U40 girder. However, due to the differences in geometry, the experimental setup was modified significantly. As noted earlier, the total length of the Tx28 and T34 girders were 60 ft. The span between the centerline of the supports was 56 ft to accommodate the 4 ft spacing between anchors on the strong floor and to provide suitable room for the bearings. Because the girder geometries are more narrow than the U-girders, the gravity load simulators did not require as much clearance from interfering with the girders. Therefore, the elevation of the setup only required two W30 sections, as depicted in Figure 3.42, which shows the key components of the setup. Many of the testing protocols were similar to lateral loading, vertical loading, and torsional loading discussed for the U-girder. One planned difference between the I-girders and the U-shaped girders was conducting lateral load tests under a constant applied vertical load. A picture of the Tx34 girder setup is shown in Figure 3.43. Figure 3.44 shows an overhead view of the Tx28 specimen.



(a)



(b)

Figure 3.42. (a) Plan view of test setup (b) Overview of the experimental test setup for the Tx34 specimen



Figure 3.43. Tx34 setup in the lab (Tx28 similar)

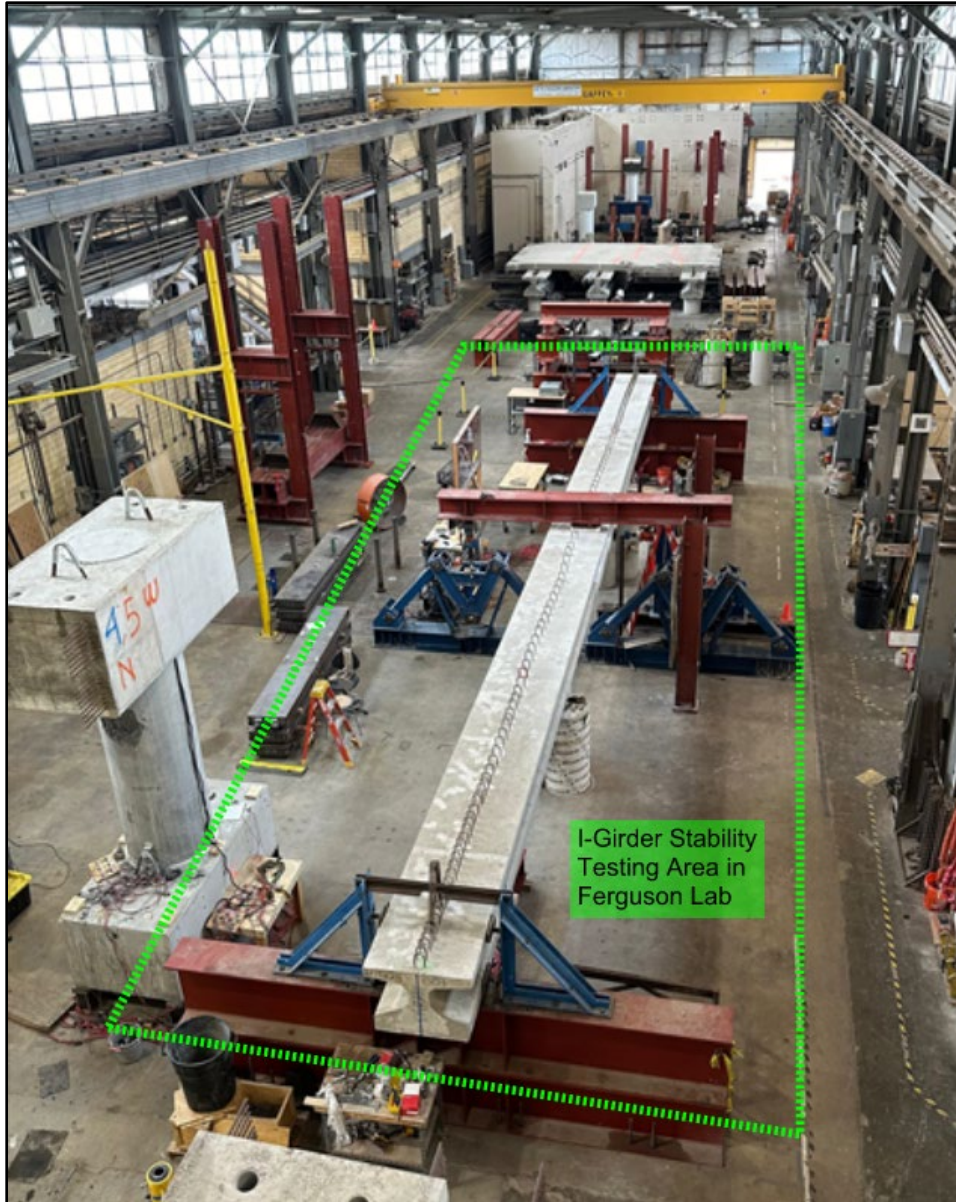


Figure 3.44. Overhead view of I-girder test setup

Vibrating wire strain gauges were located at the two quarter points and midspan. Linear potentiometers for measuring vertical and lateral displacements were placed near the end, first-quarter point, and the midspan of the girders. The location on the cross-section of the linear potentiometers (LP) is shown in Figure 3.45.

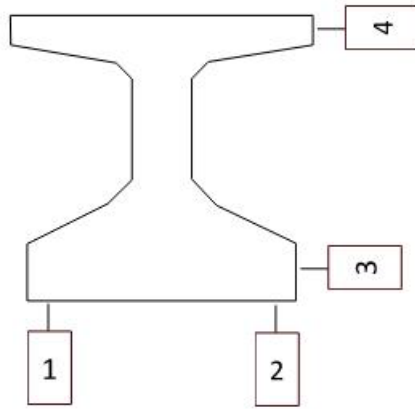


Figure 3.45. Location of displacement sensor locations on Tx34 cross-section (Tx28 Similar)

The following tests were performed on both Tx34 and Tx28 specimens:

1. Lateral Bending (uncracked)
2. Bending (uncracked)
3. Combined Bending and Torsion with 6-ft eccentricity (uncracked)
4. Bending into the cracked region
5. Combined Bending and Torsion with 6-in. eccentricity into the cracked region

The order of tests was selected to capture the behavior in the uncracked condition for both lateral and vertical bending, as well as combined bending and torsion. After the sections were cracked in concentric tests, the combined bending and torsion test was repeated to capture the behavior of the girder in the cracked state.

Since the Tx34 and Tx28 only vary in web height by six inches, many of the experimental results are presented for both girders in the same figure. This allows for easy comparisons regarding the structural response of the specimens. For brevity in this report, not all results are presented since many of the curves are similar to those presented for the U-girders, and can be found in Appendix A.

3.3.1. Lateral Load Test of I-Girders (Uncracked)

Lateral bending tests were conducted to evaluate the bending stiffness of the test specimens about the weak axis. Figure 3.46 shows the lateral loading setup consisting of the piston (ram), load cell, and clevis. The position of the lateral load was offset from the midspan of the girders so as to avoid contact with the gravity load simulators. The offset was 4 ft, which was based on the spacing between floor anchors. The loads were intentionally kept low to prevent cracking. The loads were applied at mid-thickness of the bottom flange, as depicted in Figure 3.46. These tests were not pure “flexural” tests, as the loads were applied below the shear center of the cross sections. Although

the eccentricity was relatively small, a minor twist did occur as shown in the load-deflection graph in Figure 3.47. The red lines in the graph are for the Tx28, while the black lines represent the behavior of the Tx34. The lateral displacements at the top of the sections are represented by the dotted lines, while the lateral displacements of the bottom of the sections are represented by the solid lines.



Figure 3.46. Lateral load application for I-girders 4 ft offset from midspan

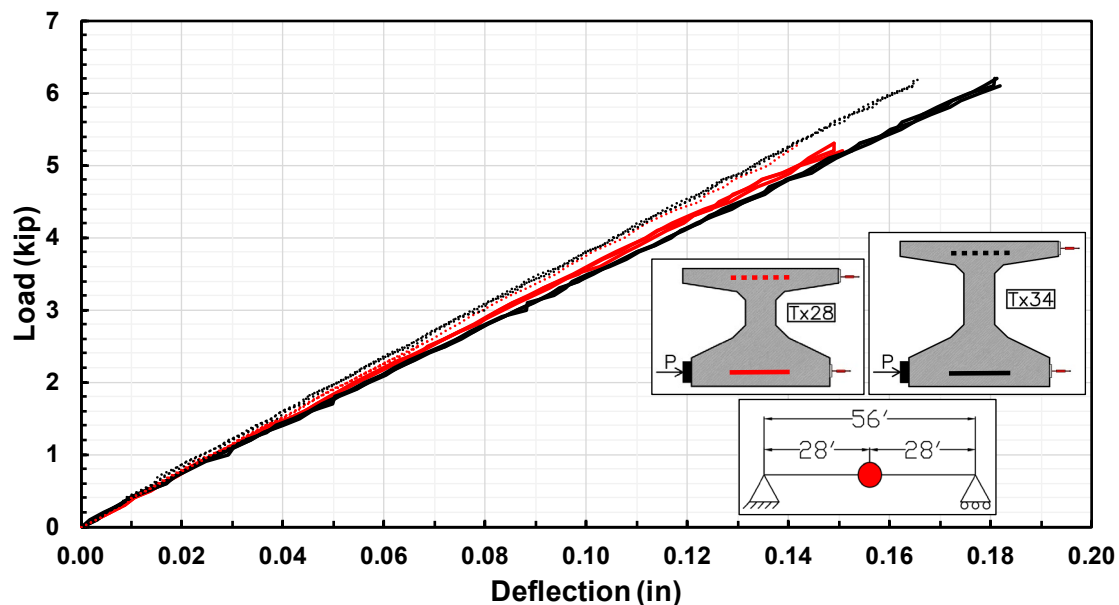


Figure 3.47. Load-deflection of I-Girders at midspan

Because the top and bottom flanges of the two girders are essentially identical, the weak axis moment of inertia is only different by the contribution of the web, which is relatively small compared to the contribution of the flanges. As a result, the lateral stiffness of the Tx34 is only slightly larger than the Tx28, which is indicated in the graph. Figure 3.48 shows the moment-curvature response plotted to compare the curvature across the flanges in the horizontal plane as measured by the strain gauges.

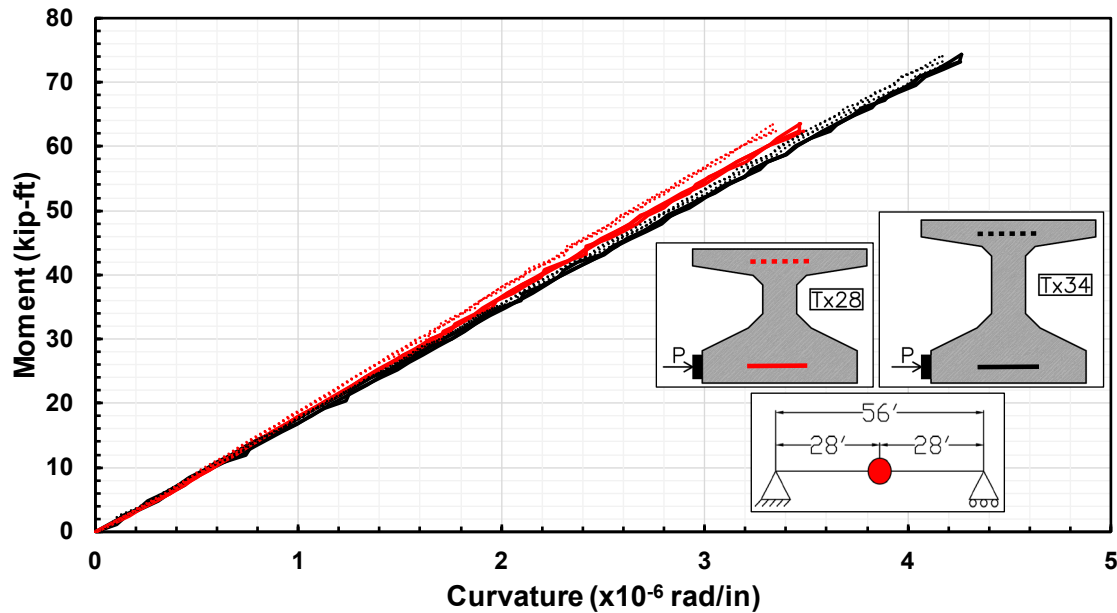


Figure 3.48. I-girder moment-curvature response to lateral loading

3.3.2. Description of Vertically Applied Loads with Gravity Load Simulators

Similar to the U-girder experiments, the gravity load simulators were positioned to apply load six feet on either side of the center line of girders at the midspan. This concentric loading setup is shown in Figure 3.49. The load was applied through the same fixture used for the U40 experiments as shown in Figure 3.50.

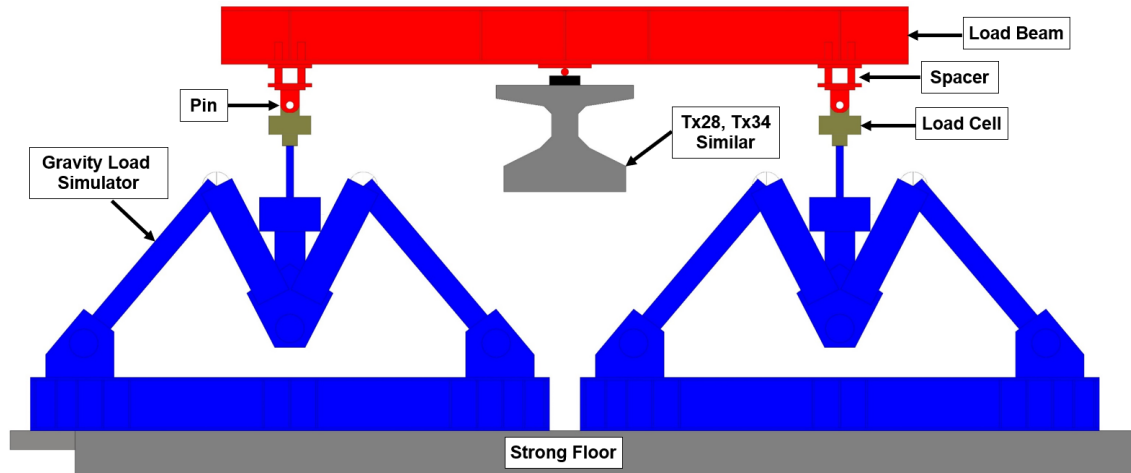


Figure 3.49. View near midspan of the I-girder concentric loading setup



Figure 3.50. (A) Concentric loading setup at midspan (B) Load application via round bar on steel plate embedded in hydrostone

Unlike the U-girder, the top flanges of the I-girders can only accommodate a small amount of eccentricity due to the smaller width. Therefore, a clamping frame was fabricated so that the load from only one gravity load simulator could be applied at an eccentricity of six feet. This results in a much more torsional-dominated combined bending and torsion load onto the specimen and hence provides a larger range for the torsional behavior to be observed. The six feet eccentric loading setup is shown in Figure 3.51, and the load applications on the top and bottom flanges are presented in Figure 3.52 and Figure 3.53. Another setup was also used where the load was offset on the flange for combined bending and torsion that was “bending dominant” and is discussed later in this chapter.

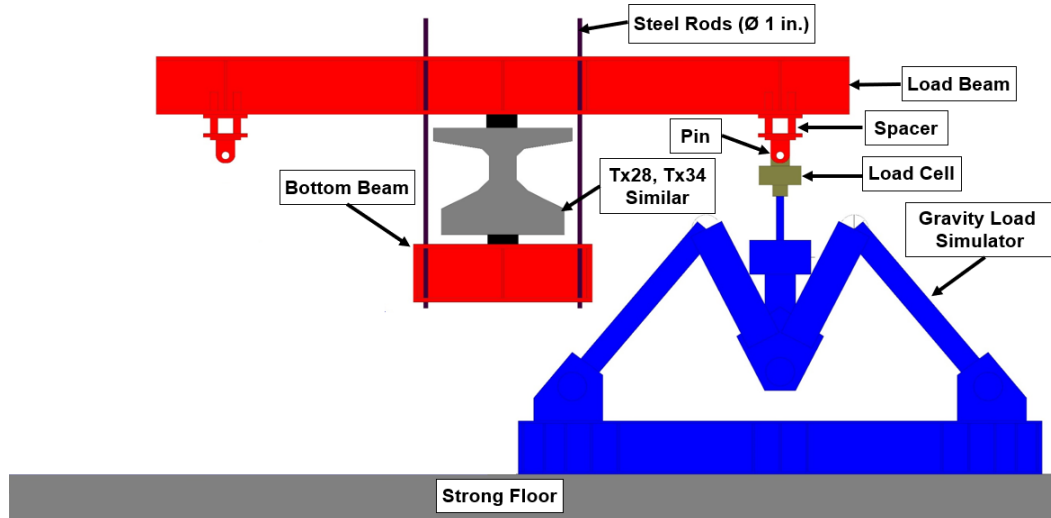


Figure 3.51. View near midspan of the I-girder with clamping frame for 6-foot eccentricity loading setup

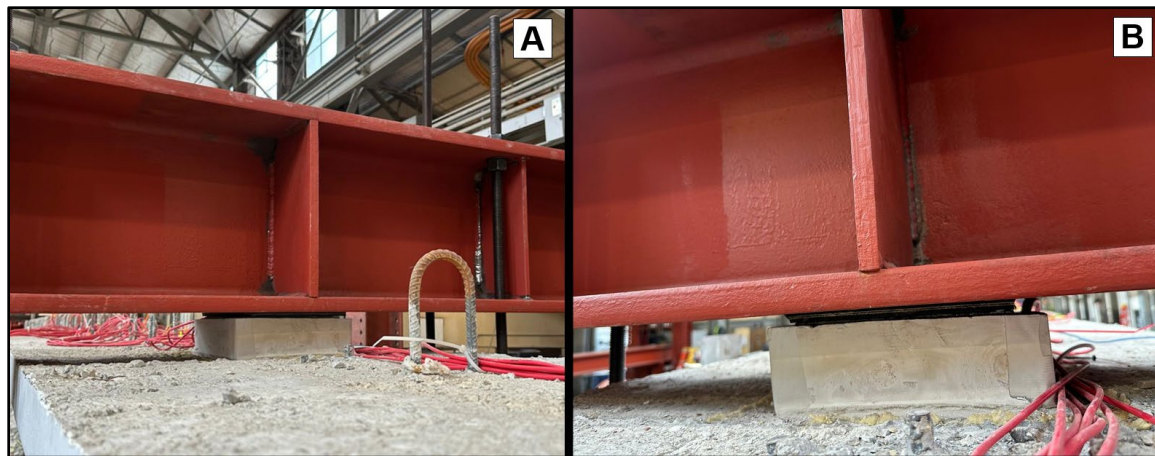


Figure 3.52. (A) View showing rods and bearing plate on top of I-girders (B) View showing high-density rubber bedding with hydrostone-embedded steel plate on top of I-girders

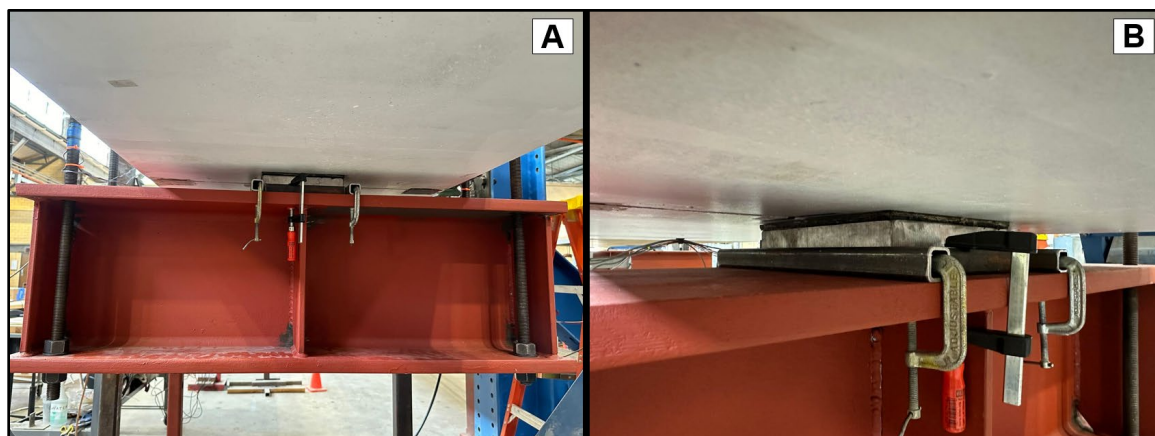


Figure 3.53. (A) View showing load beam, rods, and bearing plate on the bottom of I-girders (B) View showing high-density rubber bedding with a steel plate on the bottom of I-girders

The final load setup was designed to have enough bending into the cracked region of the I-girders and also apply torque to the cracked section. The loading fixture was modified to have an eccentricity of six inches on the top flange. This eccentricity was selected to ensure there was not a local failure of the girder flange in combined bending and shear, and was confirmed by the FEA models not to have a local failure. The six-inch, eccentric loading setup is shown in Figure 3.54, and the load applications on the top flange are presented in Figure 3.55. As noted earlier, this load setup has combined bending and torsion but is bending-dominant compared to the loading with a 6 ft offset that was torsion-dominant.

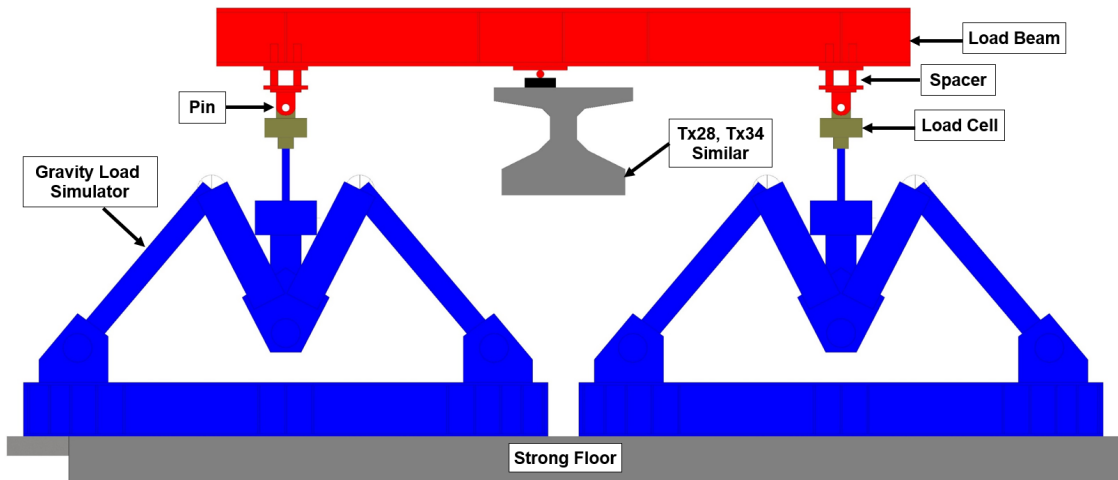


Figure 3.54. View near midspan of the I-girder with 6-inch eccentricity on the top flange loading setup

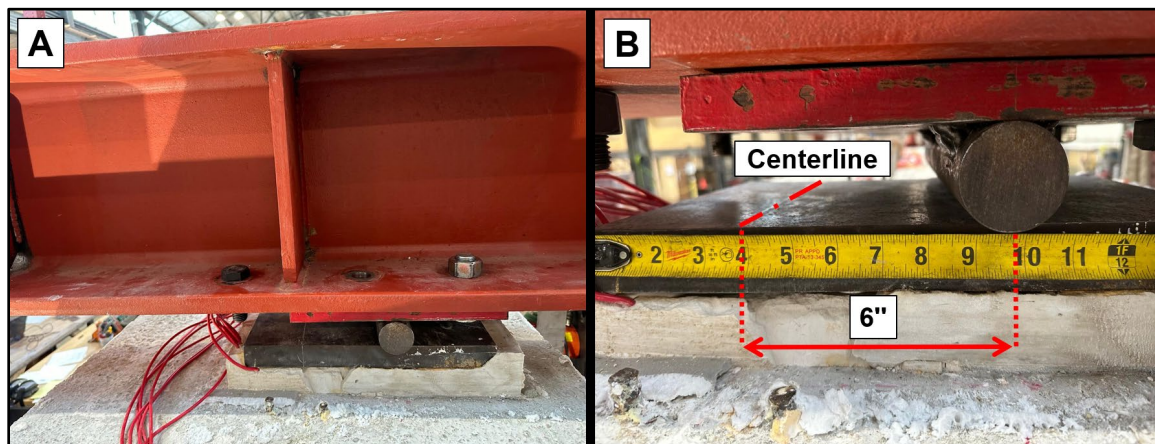


Figure 3.55. (A) View of loading bar and bearing plate (B) View of 6-inch eccentricity from the centerline of I-girder

3.3.3. Vertical Bending Tests

The bending tests were conducted to analyze the bending stiffness of the specimens in the strong axis for both uncracked and cracked states. This section includes measurements of curvature and

deflection, which enable the estimation of the vertical bending stiffness. Additionally, the combined bending and torsion test, where the load was applied with a 6-inch eccentricity, is presented to discuss the impact of eccentric loading. The Tx34 girder was the first I-section tested.

The moment-curvature behavior of the Tx34 girder at midspan is shown in Figure 3.56. Three load cycles were carried out for the bending test. For the case with concentric loading, the black line represents the first cycle, and the red lines depict the second and third cycles. Minor visible cracks began to emerge during the first cycle when the load and corresponding moment reached approximately 75 kips and 1050 kip-feet, respectively. The load essentially remained within the linear-elastic range up to this point. As the load increased, the stiffness degraded due to crack propagation. It is important to note that the graph shows the curvature as determined near the midspan region where cracking existed. The curvature was determined from the VWGs in this vicinity. The overall stiffness of the girders was still significant, as is shown in Figure 2.57. However, in the localized region around midspan, significant reductions in the effective stiffness occurred as a result of cracking. The second and third cycles for the concentric loading are denoted by the red line and exhibited consistent behavior that had very similar stiffness behavior with the initial stiffness as the first cycle until a load and corresponding moment of nearly 60 kip and 840 kip-feet, respectively. The stiffness gradually reduced as the load increased due to the opening of existing cracks from the first cycle and the potential formation of new cracks.

The combined bending and torsion test for the 6-inch load eccentricity is indicated with the blue line in Figure 3.56, and shows that the small offset in load did not alter the curvature behavior significantly from the case with concentric loading. While slightly different than the previous cycles, the curve showing the behavior exhibited the same rounded bi-linear response with similar values of the initial and cracked stiffness.

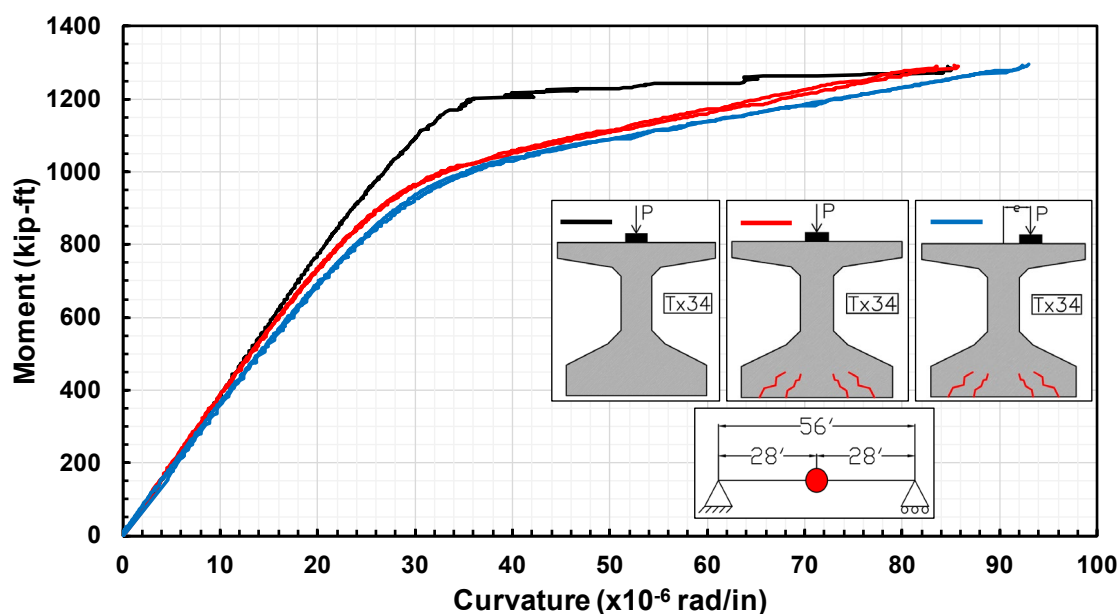


Figure 3.56. Bending moment-curvature of Tx34 at midspan

As noted above, the curvature response from the cracked region showed a dramatic reduction in stiffness, which is not as pronounced when the entire girder is considered. The heavily cracked region only impacts a relatively small portion of the girder length and therefore the overall girder stiffness is not significantly impacted. This is evident in the load-deflection response of the Tx34 girder measured at midspan is shown in Figure 3.57. The black line depicts the first cycle of the vertical bending test, while the second and third cycles are displayed with red lines. There was very little change in the vertical deflection among the tests, as evidenced by the nearly coincidental lines. In the first cycle, the load was essentially within the linear-elastic range until reaching approximately 70 kips. Beyond this load, the curve became non-linear due to the onset of cracking. The second and third cycles exhibited similar behavior. The combined bending and torsion test is displayed with blue and green lines, which represent the individual LP readings near the flange edges. The slight difference between the two results is most likely from the minor twisting of the girder in response to the torsion. Two loading cycles were carried out, which showed consistent values of the initial and cracked stiffness.

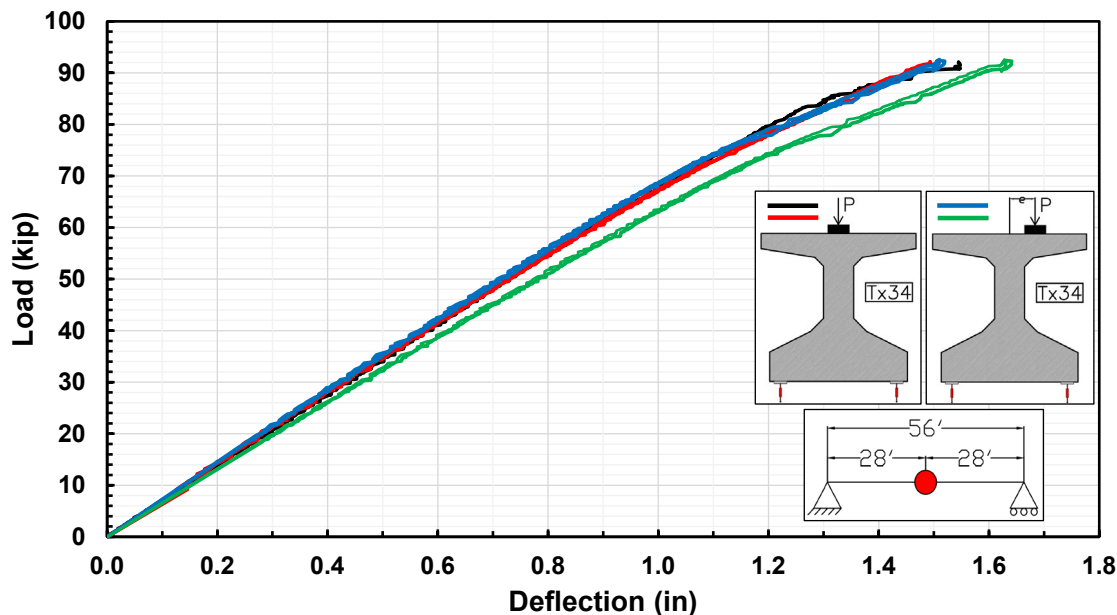


Figure 3.57. Load-deflection of Tx34 under bending at midspan

A summary of the effective EI_x determined from the curvature measurements in the uncracked and cracked states is provided in Table 3.2. The initial cracking cycle recorded a drastic reduction in stiffness of the cracked region, with a sectional stiffness ratio of only approximately 2%. The sharp reduction is likely a result of local cracking damage across the vibrating wire gauges that causes excessive measured strain. After the initial cracking cycle, the sectional stiffness ratio had approximately 14% stiffness for both the concentric and eccentric load cases. While the sharp reductions in stiffness are very significant, the impact on stability behavior will not likely be as severe since this damage is highly localized. The researchers have considered the impact of

localized reductions on the sectional properties on the overall behavior through computational modelling methods as discussed subsequently in this report.

Table 3.1 Tx34 uncracked and cracked bending stiffness

Test \ Parameter	$EI_{x, \text{ uncracked}}$ ($\times 10^6$ kip-in. ²)	$EI_{x, \text{ cracked}}$ ($\times 10^6$ kip-in. ²)	$\frac{EI_{x, \text{ cracked}}}{EI_{x, \text{ uncracked}}}$
Vertical Bending Test (1 st cycle)	465 (<60 kips)	10	2.2%
Vertical Bending Test (2 nd and 3 rd cycles)	465 (<60 kips)	67	14.4%
Combined Bending and Torsion Test	422 (<60 kips)	58	13.8%

The Tx28 girder was tested with the same protocols as the Tx34 girder. Figure 3.58 depicts the moment-curvature behavior of the Tx28 girder at midspan. Three loading cycles were conducted for the bending test. For the case with concentric loading, the black line shows the first cycle, while the red lines represent the second and third cycles. During the first cycle, cracks began to appear when the load and corresponding moment reached approximately 50 kips and 700 kip-feet, respectively. Up to this point, the load essentially remained within the linear-elastic range. As the load increased, the stiffness reduced as the cracks propagated. The second and third cycles displayed similar behavior. The behavior was within the linear-elastic range, maintaining the same initial stiffness as the first cycle, up to a load and moment near 40 kips and 560 kip-feet, respectively. Blue lines indicate three cycles of the combined bending and torsion test, all of which showed consistent behavior.

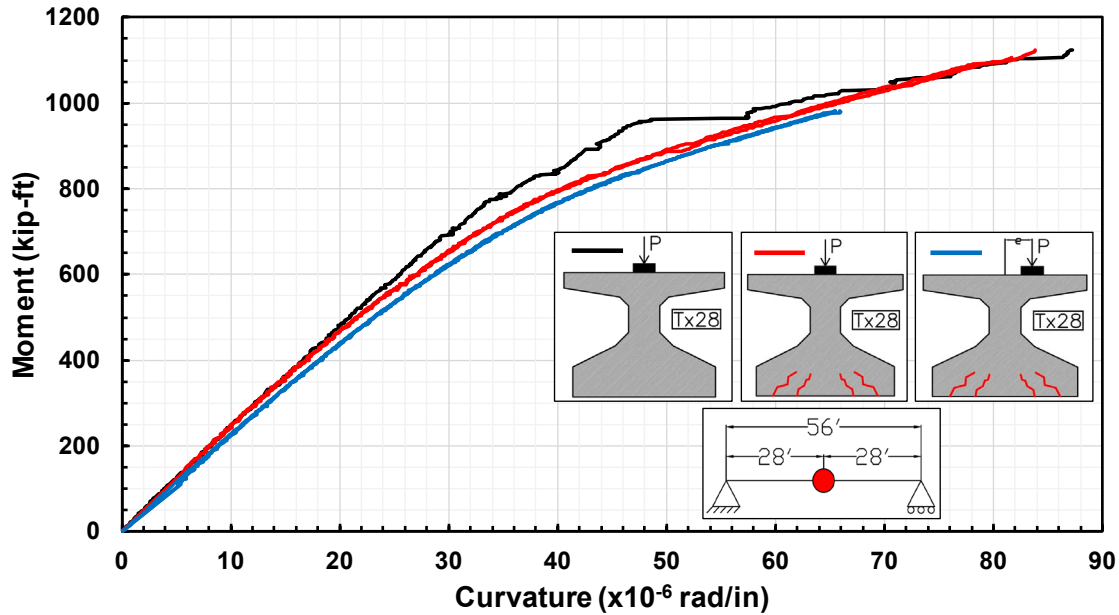


Figure 3.58. Moment-curvature of Tx28 under bending at midspan

The load-deflection relationship for the Tx28 girder is shown in Figure 3.59, measured at midspan. The black line represents the first cycle of the vertical bending test, while the second and third cycles are depicted with red lines. In the first cycle, the load remained within the linear-elastic range until a level of approximately 50 kips. After this point, the stiffness reduced due to crack formation. The second and third cycles exhibited similar behavior, maintaining the same initial stiffness as the first cycle until the load reached approximately 40 kips. As the load increased, the stiffness gradually decreased as existing cracks opened and potentially new minor cracks formed. The combined bending and torsion test is shown with blue and green lines. These two measurements differ due to the eccentric loading and consequential twist. Three loading cycles were conducted, which showed similar behavior. The initial stiffness obtained from the sensor located on the opposite side of eccentricity, shown with the blue line, closely resembled the initial stiffness obtained from the vertical bending test. In contrast, the initial stiffness measured from the sensor positioned on the side of the eccentricity, presented with the green line, was slightly lower. Similar to the vertical bending test, both cycles were within the linear-elastic range until the applied load reached approximately 40 kips.

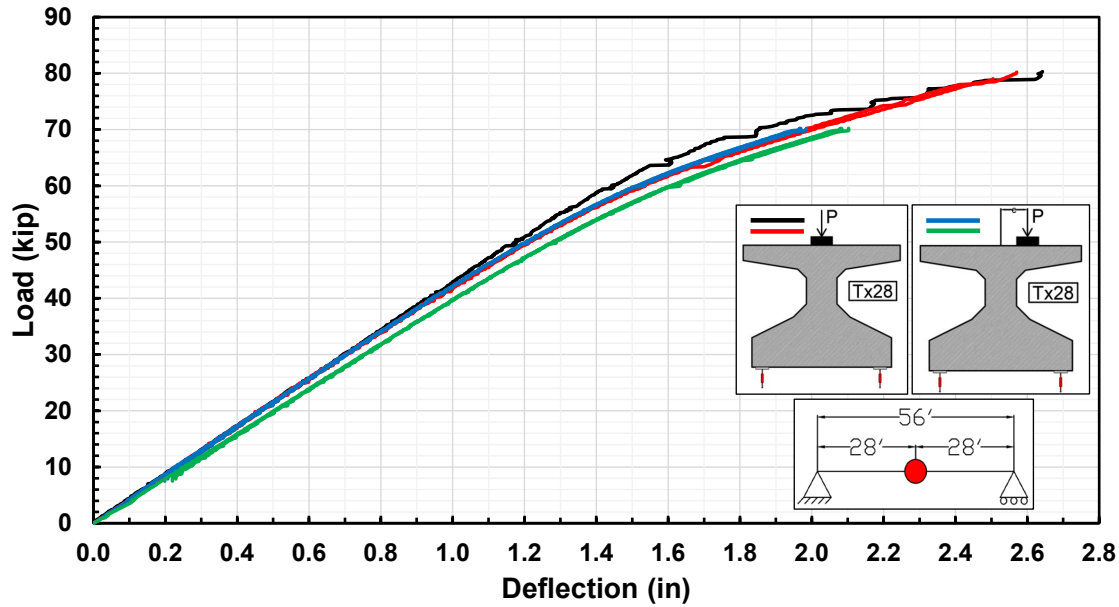


Figure 3.59. Load-deflection graph of Tx28 under bending at midspan

A summary of the uncracked and cracked stiffness values is presented in Table 3.2. Tx28 uncracked and cracked bending stiffness. Notably, the reduction in stiffness is near 70% comparing cracked to uncracked sections. As noted for the Tx34 girder, the reductions in stiffness represent only the portions of the girders that are cracked and not the entire span of the girder. The Tx28 exhibited similar behavior in the cracked region as the Tx34. Namely, a high reduction in stiffness from the VWG moment-curvature and a lower reduction in the global stiffness demonstrated in the load-deflection plots.

Table 3.2. Tx28 uncracked and cracked bending stiffness

Parameter Test	$EI_{x, \text{uncracked}}$ ($\times 10^6$ kip-in. ²)	$EI_{x, \text{cracked}}$ ($\times 10^6$ kip-in. ²)	$\frac{EI_{x, \text{cracked}}}{EI_{x, \text{uncracked}}}$
Bending Test (1 st cycle)	285 (<50 kips)	76	26.7%
Bending Test (2 nd and 3 rd cycles)	285 (<40 kips)	87	30.5%
Combined Bending and Torsion Test	265 (<40 kips)	90	34.0%

A final comparison is provided between the Tx34 and Tx28 moment-curvature responses. Both specimens have the same number and position of strands, concrete mix design, and only vary in the height of the web by six inches. This comparison is presented in Figure 3.60. Black lines are for all three cycles of the Tx34 and the red lines are for all three cycles of the Tx28.

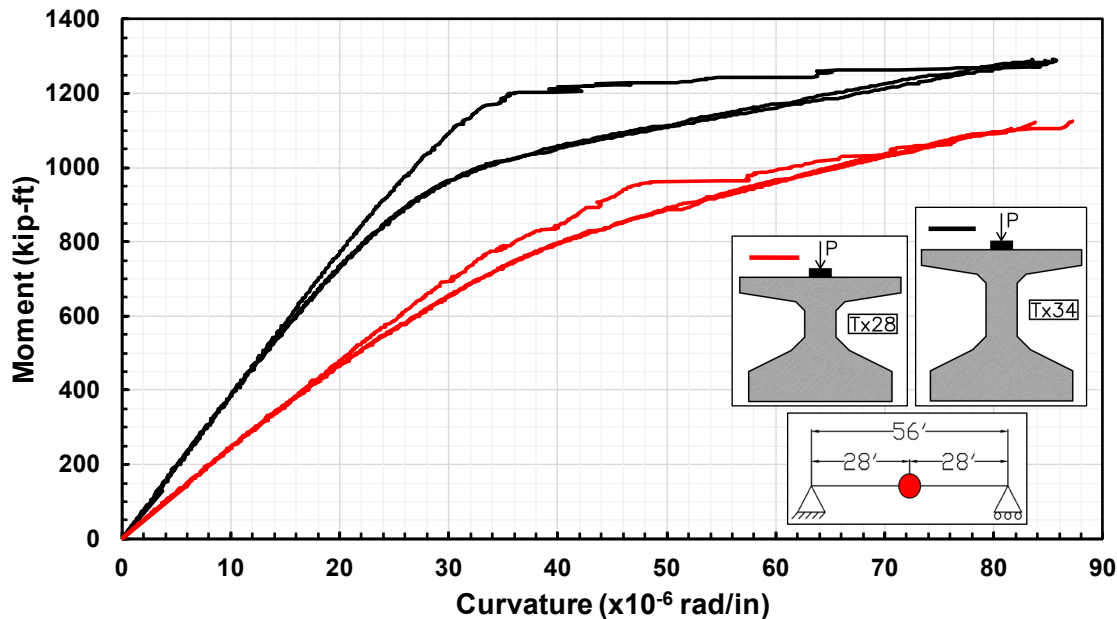


Figure 3.60. Moment-curvature comparison of Tx34 and Tx28 under bending at midspan

As expected, the taller girder (Tx34) had a higher cracking moment and initial stiffness. However, the cracked stiffness were nearly parallel in the post-cracking branches of the curve, which indicates the values of the stiffness were approximately the same post-cracking.

The strain distributions for the cross-sections at midspan during the bending tests at the maximum load for the first cycle are presented in Figure 3.61. Figure 3.61(A) illustrates the distribution for the Tx28 girder subjected to an applied load level of approximately 80 kips, and Figure 3.61(B) depicts the Tx34 girder subjected to an applied load level of 92 kips. In both cases, the applied load was concentric. The strain measurements from the VWGs are marked on the sections with straight line extensions drawn laterally and vertically. The strains at the edges of the flanges were assumed to be equal to the closest strain gauge reading, and the strains at the extreme fibers of the webs were calculated using linear interpolation. All strains are presented in absolute values, and compression and tension zones are labeled accordingly. The extrapolated neutral axis locations are shown as 2-7/16 inch and 7-3/4 inch from the web gauge for the Tx28 and Tx34 girder, respectively. The web gauge was intentionally placed at the centroid of the sections. The compression flange strains match what is expected from beam theory. The tension flange strains show spikes from local cracking effects. Additional strain profiles are available at the quarter points of the girders from all load cases. The strain data as well as the girder deflections provide important data for validating the FEA models, which is discussed later in the report.

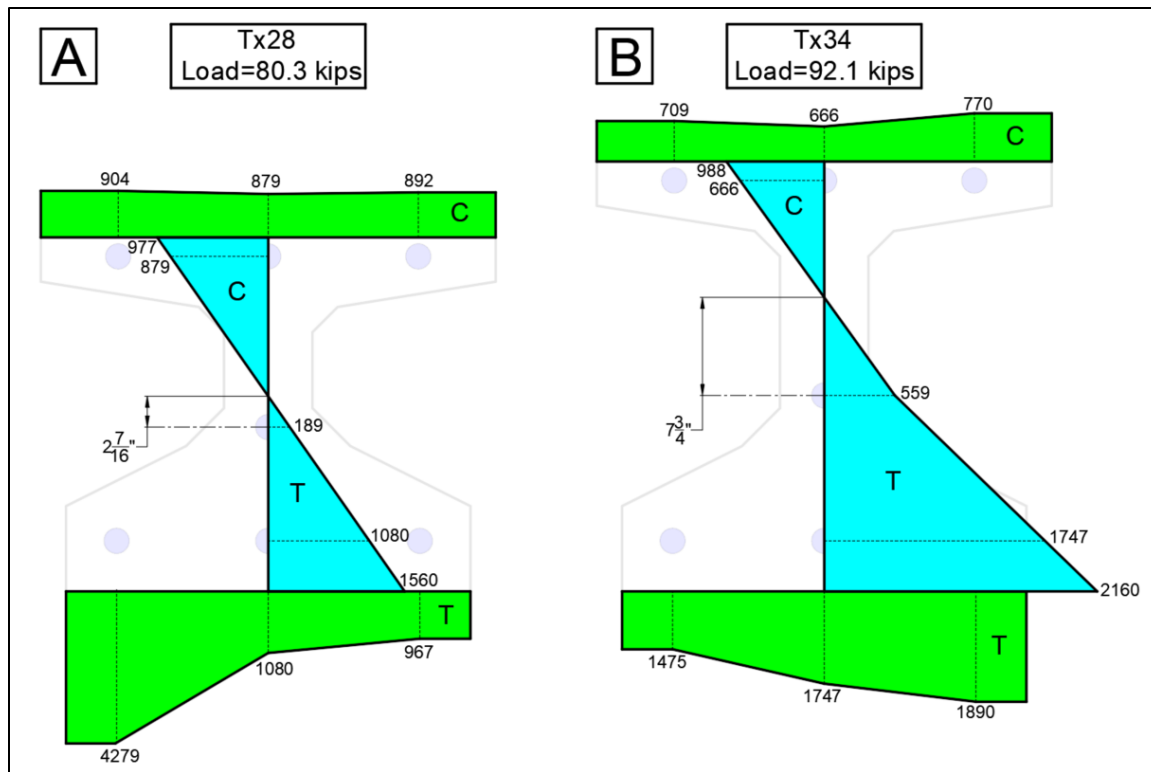


Figure 3.61. Strain distribution for max bending test at midspan for (A) Tx28 and (B) Tx34

3.3.4. Combined Bending and Torsion Tests

Although the last section included load cases with 6-inch offsets in the point of load application that caused a small amount of torque, the tests were dominated by bending behavior. As noted earlier, additional load cases were done with much higher eccentricities by making use of the load bracket. These tests with the large eccentricity were only conducted with the girders in the uncracked state. The tests were conducted to investigate the torsional response of the specimens in both the uncracked and cracked states. This section includes measurements of twist at the quarter point and midspan of the girders. It is important to note that the twist measurements at the quarter point and midspan are relative to the twist at the end supports. Small displacements occur at the end brackets as forces are induced and the bracket members reacted against the applied torque. As noted earlier, the torsion tests were conducted with one gravity load simulator and a clamping frame to give a load eccentricity of 6-feet. After the girders had cracked, the torsion tests were loaded on the top flange with a 6-inch eccentricity.

The torque-twist response for the Tx34 girder is displayed in Figure 3.62. The dashed lines represent the uncracked specimen, and the solid lines represent the cracked state. Furthermore, measurements obtained from the quarter point are indicated with red lines, and black lines

represent measurements taken from midspan. The twist measurements reflecting the uncracked state of the specimen displayed linear behavior. However, during tests examining the cracked behavior, slight nonlinear behavior was observed at the midspan of the specimen.

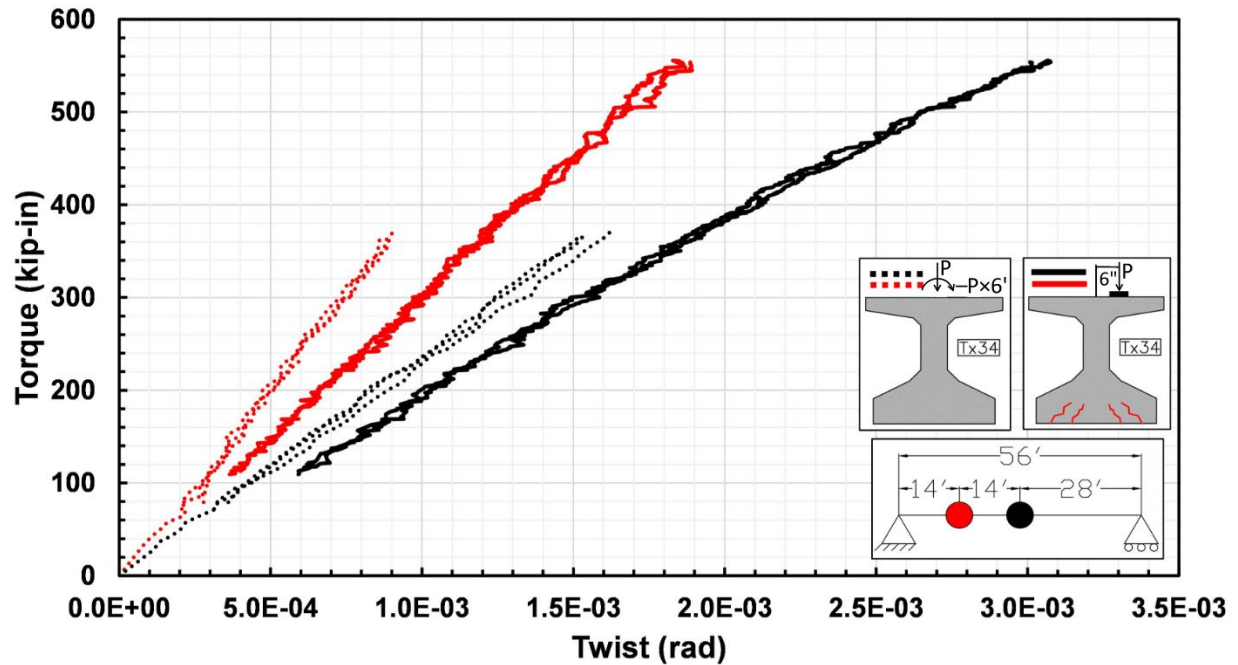


Figure 3.62. Torque-twist response of the Tx34 specimen at first quarter point and midspan for both uncracked and cracked states

The torque-twist response for the Tx28 girder is displayed in Figure 3.63 following the same format outlined for the Tx34 girder. Similar to the Tx34 girder, nonlinearity is observed at midspan for the cracked state. With a torque of 400 kip-in. the Tx28 specimen twist was 210% higher than that of the Tx34 at the same load. This is a combination of the extent of cracking at this load and section properties.

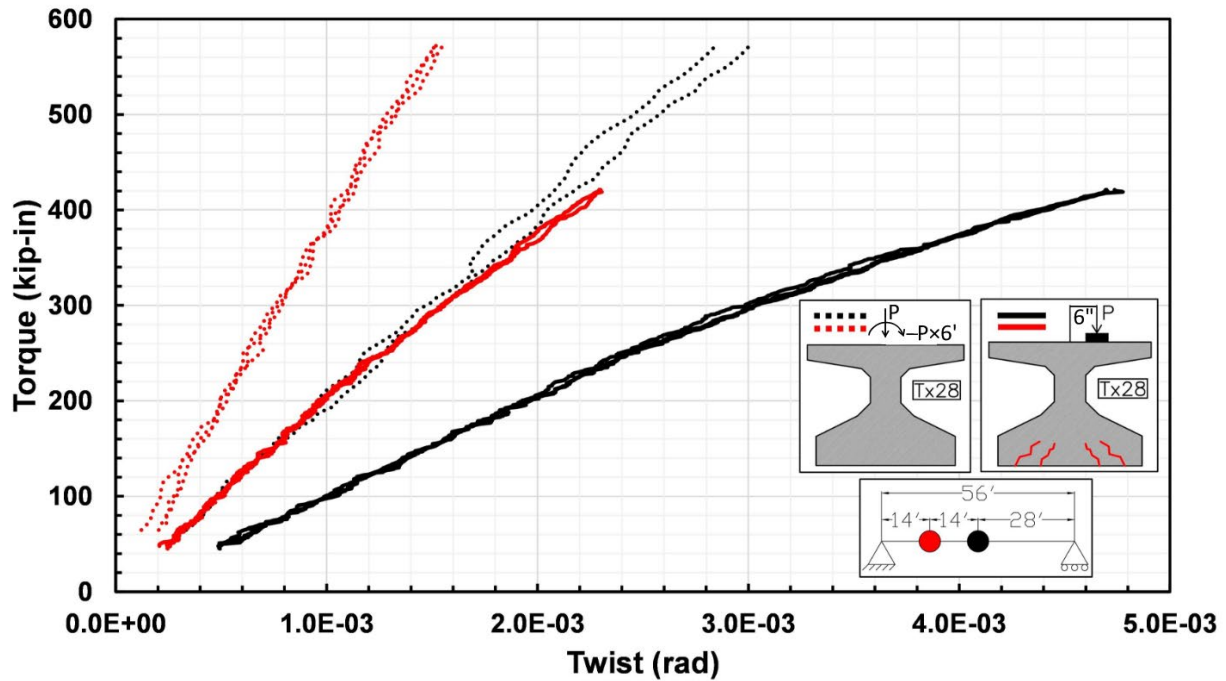


Figure 3.63. Torque-twist response of the Tx28 specimen at first quarter point and midspan for both uncracked and cracked states

The strain distribution of the cross-sections at midspan during the combined bending and torsion tests evaluating the uncracked behavior of the I-girder specimens are shown in Figure 3.64. These figures illustrate the strain profiles for the maximum load for each test. All values are positive with tension and compression annotations. The load was applied at a 6-foot eccentricity at the right side of the cross-sections shown. Due to the significant eccentricity, the torsional effects (longitudinal warping strain) are evident in the strain profiles, while the bending effects are less pronounced.

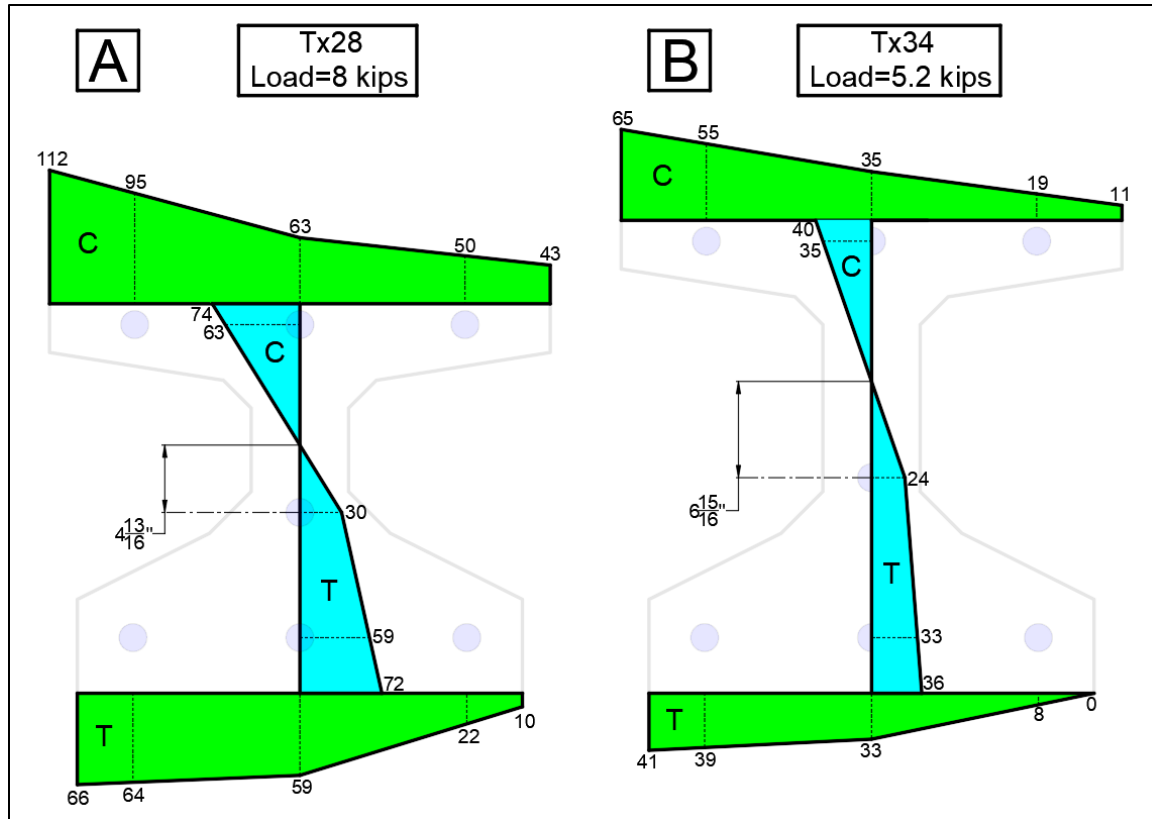


Figure 3.64. Strain distributions for the I-girders at midspan for the maximum applied loads during the combined bending and torsion tests at a 6-foot eccentricity (uncracked state)

For the cracked condition of combined bending and torsion Figure 3.65 shows the strain profiles at midspan for a maximum load applied at a 6-inch eccentricity. In contrast to the uncracked combined bending and torsion tests, the strain profiles do not reflect the torsional effects as clearly. The reason is that the eccentricity was chosen so that the existing flexural cracks could be opened without causing local damage to the top flanges. Therefore, the bending effects are more prominent, and strain spikes can be seen on the tension flange from cracking.

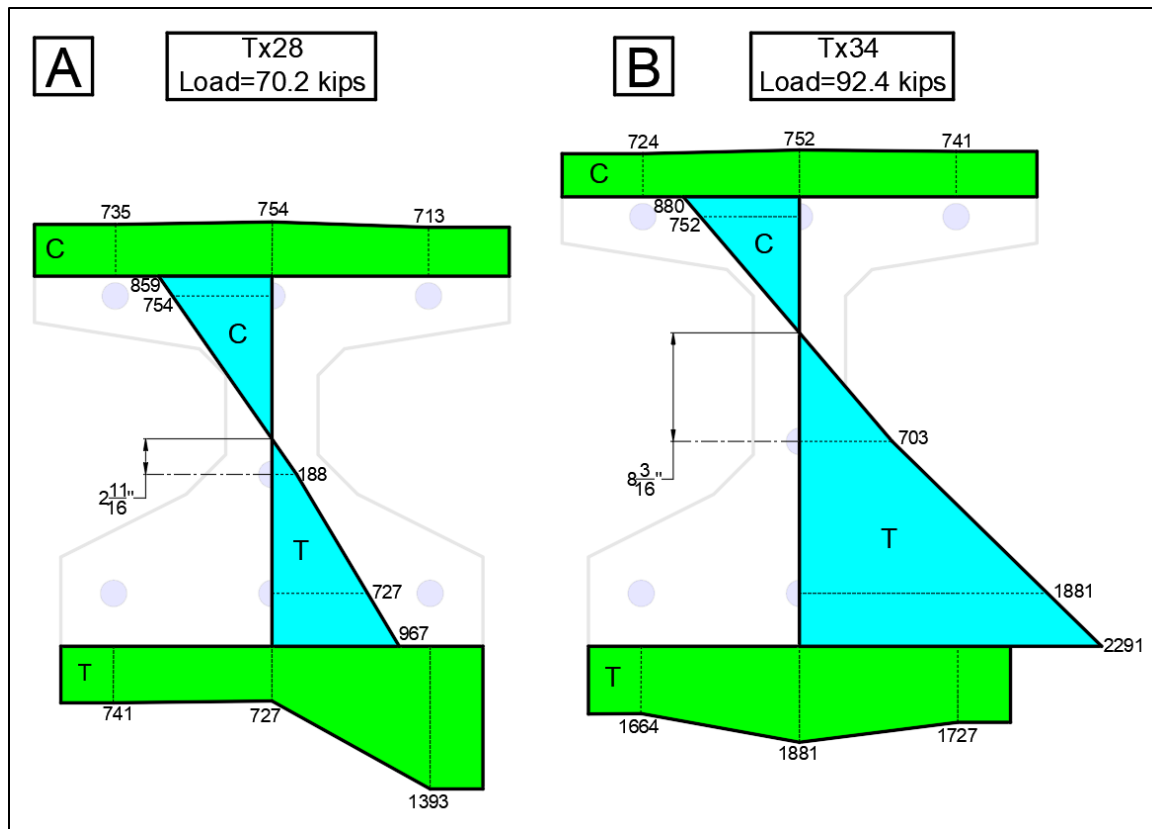


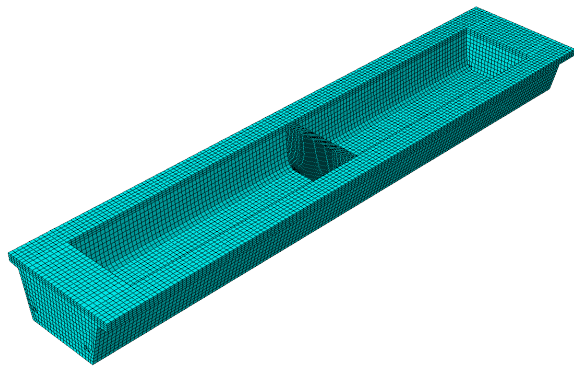
Figure 3.65. Strain distributions at midspan for the maximum applied loads during the combined bending and torsion tests (cracked state)

Chapter 4. FEA Validation Studies of Full-Scale Laboratory Testing Related to Stability and Live Load Distribution

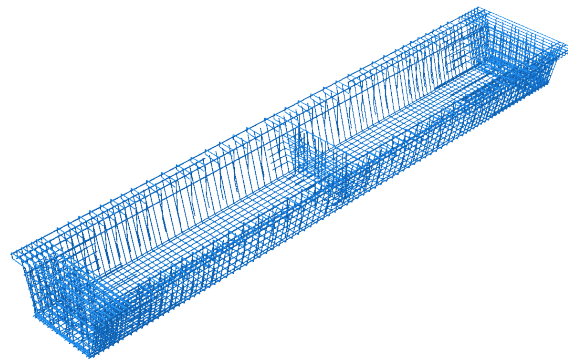
This chapter discusses the development of FE models in Abaqus that were validated with the laboratory experiments. The work provided modeling methodologies that were then applied to the stability and live load parametric studies. Developing confidence in the modeling strategies based upon comparisons measurements from the experimental tests was a crucial step prior to conducting parametric studies related to the girder stability.

4.1. Description of U and I-Girder Models

The numerical model presented in this study employs a continuum-based, three-dimensional finite element approach using Abaqus (2023). The concrete material was modeled with eight-node, hexahedral, solid elements. Longitudinal reinforcement, prestressing strands, and shear reinforcement were modeled using truss elements (line members), as illustrated in Figure 4.1 and Figure 4.2. The geometry of the models align with the TxDOT standard shape dimensions provided in earlier sections. The construction process of the I-girders involved the use of debonded strands at certain locations to control stresses at release. The modeling approach for both girders accounted for this by deleting elements representing the debonded regions. The only difference in the section geometries of the I-girders is that web of the Tx34 girder is six inches taller than the Tx28 girder.

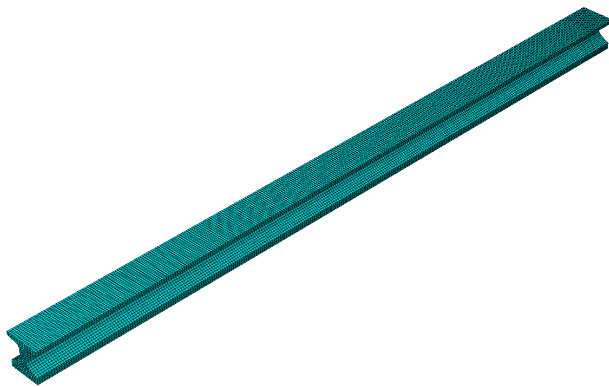


(a) Isometric view of the model

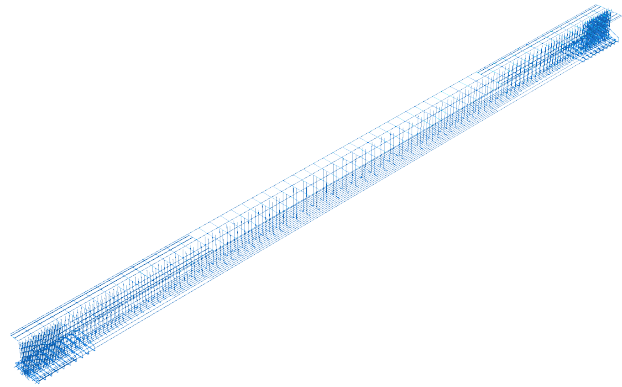


(b) Reinforcement layout

Figure 4.1. Overview of the finite element model for the Tx-U40 specimen



(a) Isometric view of the model



(b) Reinforcement layout

Figure 4.2. Overview of finite element model for Tx28 specimen (Tx34 similar)

4.1.1. Material Models

The constitutive model adopted for concrete and steel is based on the previous pertinent studies in the literature (Popovics 1973; Thorenfeldt 1987; Chang and Mander 1994; Aliasghar-Mamaghani and Khaloo 2021). The reinforcing bars and prestressing strands are assigned a uniaxial constitutive law with elastoplastic behavior (Chang and Mander 1994; Aliasghar-Mamaghani and Khaloo 2021). The elastic modulus for the former was set to 29,000 ksi, while the corresponding value for the latter was set to 27,000 ksi to account for the effective modulus of a spiral-wrapped,

seven-wire strand. The constitutive law for the concrete material is triaxial and relies on the Concrete Damaged plasticity (CDP) model. The uniaxial, nonlinear compressive response involves hardening and softening branches, which have been modeled in accordance with pertinent studies in the literature (Popovics 1973; Thorenfeldt 1987; Aliasghar-Mamaghani and Khaloo 2021), as shown in Figure 4.3(a). The peak compressive strength of concrete for both specimens was set to 8,000 psi. The concrete elastic modulus for the Tx34 specimen – similar to the U-40 girder – was set to 4,150 ksi, while the corresponding value for the Tx28 girder was 5,098 ksi. The latter was determined based on the knowledge gained from the calibration study. The constitutive response in tension is assigned a linearly-elastic law up to the maximum tensile strength, $f_t = 4.5\sqrt{f'_c}$, followed by an exponential decay function (Stevens et al. 1991), as depicted in Figure 4.3(b).

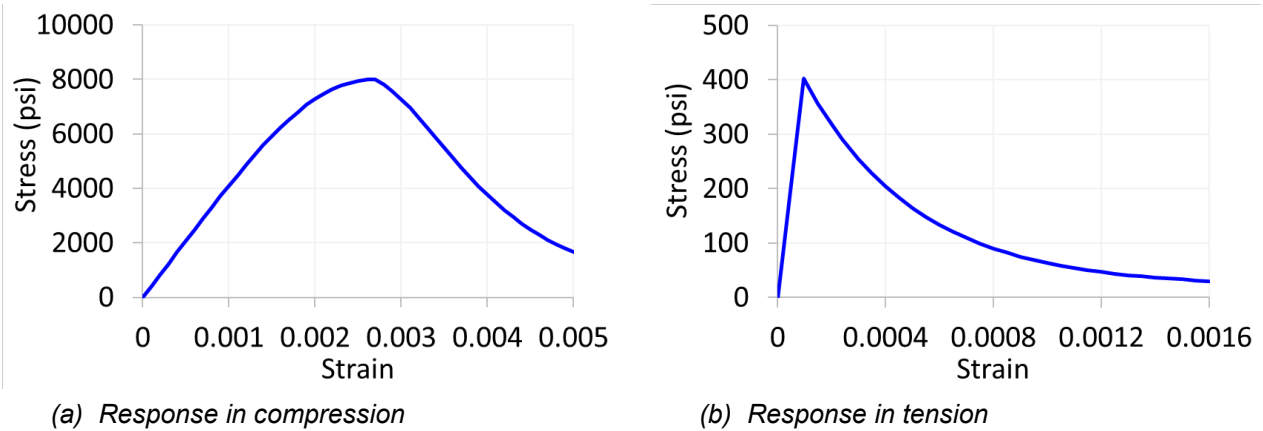


Figure 4.3. Uniaxial behavior of concrete

The triaxial nature of the constitutive model for concrete necessitates the definition of additional model parameters. Specifically, a Poisson's ratio of 0.18 was assigned to the concrete in model. The ratio of equiaxial compressive yield strength to uniaxial compressive yield strength was set to 1.16. The introduction of plastic flow in the CDP model requires the definition of the model parameter K , which specifies the shape of the yield surface in the deviatoric plane. This parameter was assigned a value of $2/3$ (Lubliner et al. 1989). The dilation angle and eccentricity values, which specify the direction of plastic flow, were assigned values of 30° and 0.1, respectively.

4.1.2. Tx-U40 Loading Schemes, Boundary Conditions, and Analysis Methodology

As previously discussed, a variety of loading protocols were pursued in the experimental tests to obtain the torsional and bending properties of the girders. The analysis of the Tx-U40 girder involved two loading schemes, consistent with the experimental testing. The loading configuration set-up for the first test involved positioning a point load at the edge of the section at midspan, as

depicted in Figure 4.4(a). This configuration subjected the specimen to a combination of forces, i.e., shear, bending, and torsion. The second loading configuration setup involved concentric loading by positioning a point load in the middle of the diaphragm at the midspan, as shown in Figure 4.4(b). The latter subjected the specimen to shear and bending.

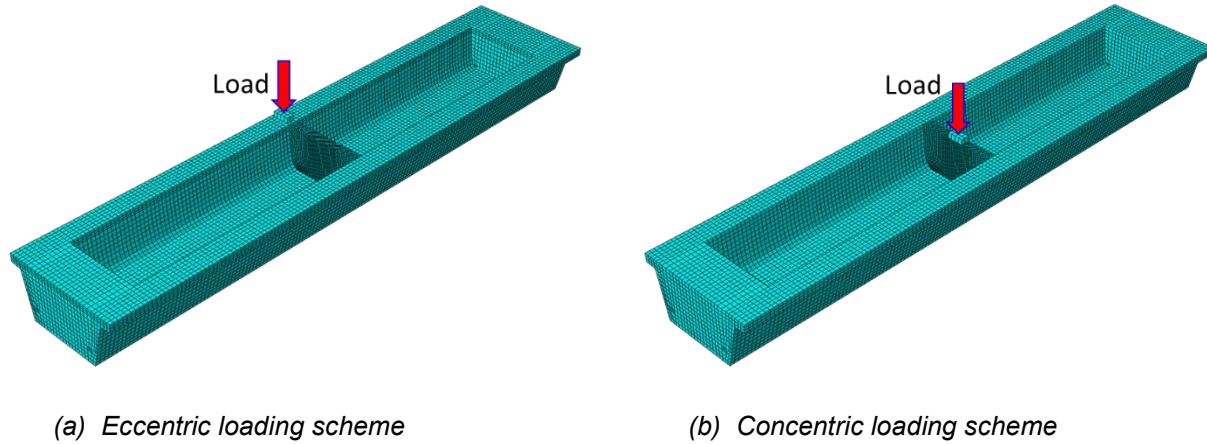


Figure 4.4. Loading schemes for the finite element model of the Tx-U40 specimen

In order to obtain good correlation between the experimental data and the FEA results, the boundary conditions and loading conditions from the experiments were simulated in the analysis. The boundary conditions in the test set-up were designed to ensure the stability of the structure under various loading conditions. Two support types were adopted in the experiments to restrain the out-of-plane (lateral) movement and both the vertical and longitudinal movements of the specimens. The analysis of the girders adopted a similar approach. Ideally, the boundary conditions are assumed to be rigid with no flexibility. However, depending on the problem under consideration, this assumption may impact the results. The most accurate modeling approach involves accounting for the flexibility of the bearings at the supports by introducing spring elements.

The analysis of the Tx-U40 specimen is approached in two ways. The initial set of analyses assumed rigid (ideal) boundary conditions, neglecting the flexibility of the supports. The schematic view of these boundary conditions is shown in Figure 4.5. The analysis of the specimen was repeated for a second time, accounting for the flexibility of the supports. Figure 4.6 provides the details of the boundary conditions for the specimen, taking into account flexible supports.

The boundary conditions for the analysis of the Tx-U40 specimen, as shown in Figure 4.6(a) and Figure 4.6(b), fall into two categories: twist-resisting supports, which resist motions normal to the longitudinal axis of the girder, and roller and pin supports. The modeling of twist-resisting supports involved the use of compression-only springs, while roller supports were represented by elastomeric bearing pads. Models of the pin support incorporated both elastomeric bearing pads and an additional spring with high stiffness. The elastomeric bearing pads were assumed to exhibit linearly-elastic behavior. As the name suggests, the constitutive model for the compression-only

springs, representing the out-of-plane supports (twist-resisting supports), is defined differently in compression and tension and is assumed to be active only in compression.

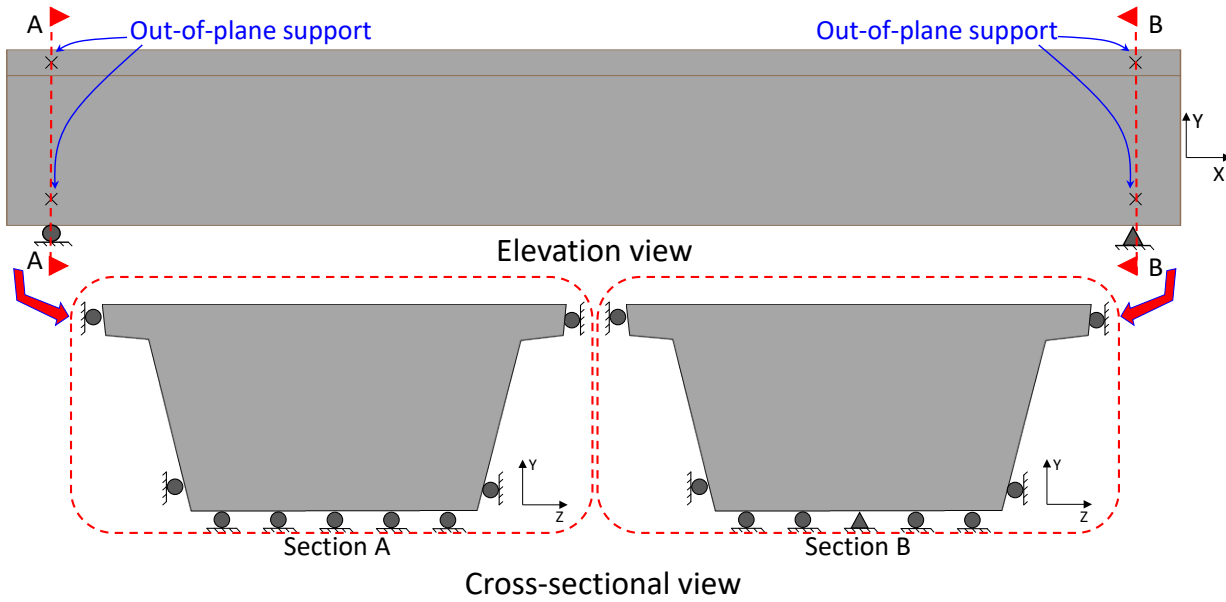


Figure 4.5. Rigid (ideal) boundary conditions for the Tx-U40 model

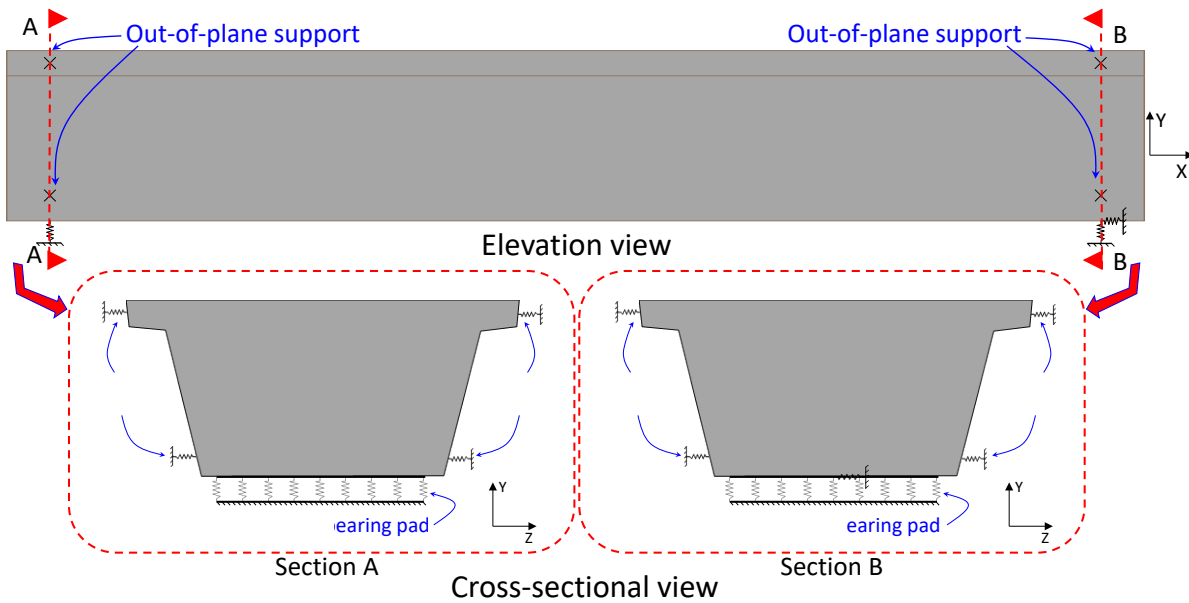


Figure 4.6. Flexible boundary conditions for the Tx-U40 model

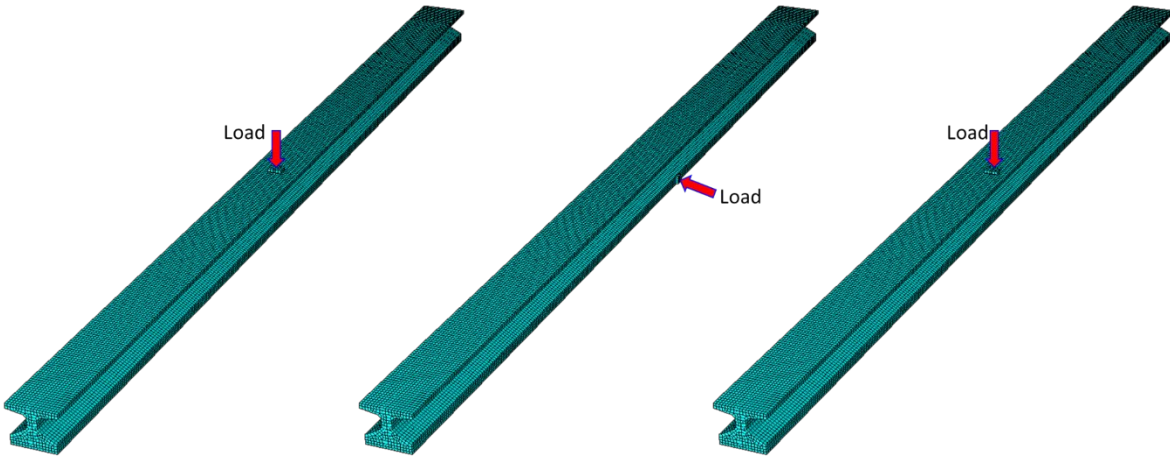
The analysis adopted in this study employed an implicit static approach. The analysis needs to consider both nonlinear geometry as well as nonlinear material behavior. Equilibrium is solved at

each time step using a full Newton-Raphson iterative scheme. An additional consideration in the analysis of the girders pertains to the modeling of the fabrication phase. The fabrication of the specimen involved prestressing of the strands. The analysis of specimens followed a similar approach to that of experimental testing by assigning an initial pre-strain (self-induced strain) value to strands corresponding to the effective prestressing level.

4.1.3. I-Girder Loading Schemes, Boundary Conditions, and Analysis Methodology

A variety of loading protocols were pursued for analysis considerations – consistent with the experimental testing procedures. The analyses of the Tx34 and Tx28 specimens involved three loading configurations to investigate the performance of the girders in the cracked and uncracked states. The planned loading configuration for both specimens involved concentric tests, eccentric tests, and lateral tests. The former involved positioning a point load, at the midspan centerline, as indicated in Figure 4.7(a) – thereby subjecting the specimen to pure bending. The eccentric test involved positioning a point load, 6 feet away from the midspan center line. The particular configuration subjected the specimen to combined bending and torsion about the strong axis. The lateral test setup involved positioning a point load four feet away from the midspan and perpendicular to the longitudinal axis of the girder in the lateral direction, as detailed in Figure 4.7(b).

The configurations were given a load level, smaller than what would introduce mechanical damage in the concrete. The loading was intended to only capture the uncracked properties of the specimens. The second phase of testing involved the deliberate introduction of cracking, with a particular emphasis on cracked sectional properties of both specimens, obtained through concentric and eccentric testing. The former involved positioning a point load at the midspan centerline but for a load causing material cracking. The eccentric test involved positioning a point load 6 inches away from the midspan centerline, as shown in Figure 4.7(c). In a similar vein, the load levels were selected as such to introduce cracking into the specimen.



(a) Concentric loading scheme

(b) Lateral loading scheme

(c) Eccentric loading scheme

Figure 4.7. Loading configurations for the Tx34 and Tx28 girders

The boundary conditions in the test set-up were designed to ensure the stability of the structure under various loading conditions. The analyses of the Tx34 and Tx28 accounted for twist-resisting supports, rollers and pins – similar to experimental set-up – to restrain out-of-plane (lateral) motion normal to the longitudinal axis, as well as both vertical and longitudinal movements of the girders. Given the primary objective of the computation to investigate the performance of the girder specimens, irrespective of the potential deformability at supports, the boundary conditions are treated ideally with rigid configurations, as schematically shown in Figure 4.8.

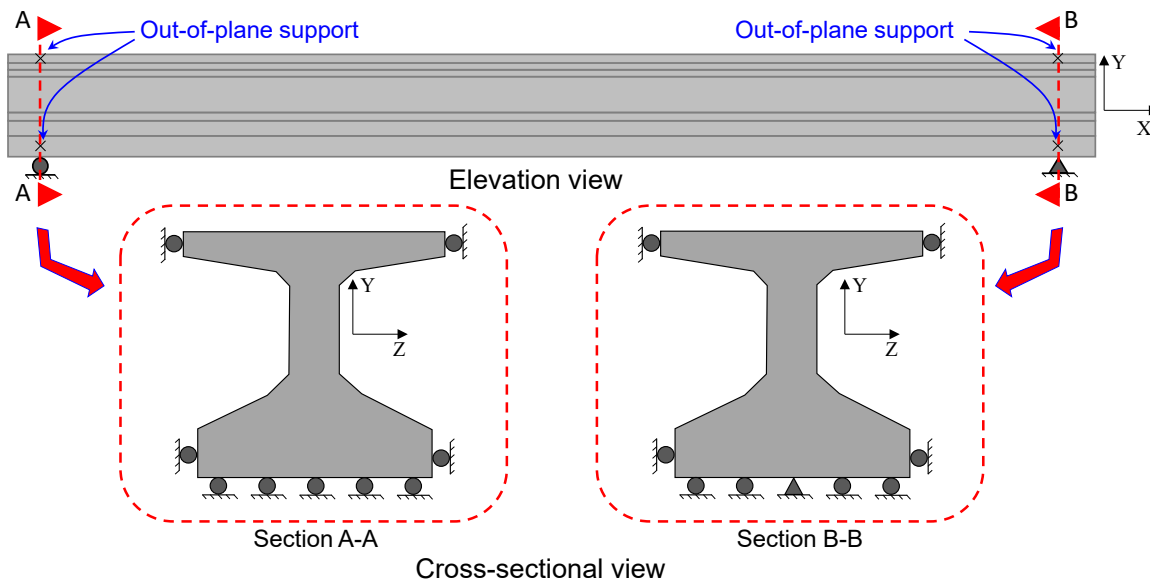


Figure 4.8. Schematic of boundary conditions for Tx34 and Tx28 models

4.2. Analysis Results

4.2.1. Tx-U40 Girder

The analytical load-displacement response for the concentric test is compared with the experimental data, employing rigid boundary conditions in Figure 4.9(a). At this stage, the analytical response excluded the deformability of the supports. To pursue a fair comparison, the experimental data (shown in Figure 4.9 (a)) were obtained by subtracting the vertical displacement in the middle of the span from that near the support. Although the experimental test was conducted for multiple cycles, the analysis focused only on the monotonic response. As shown in the figure, the analysis effectively captured the overall experimental response of the girder. The initial stiffness, cracking load, post-cracking response, and ultimate capacity of the specimen were well reproduced by the analytical model. An important observation pertains to the effective flexural stiffness of the system (EI_{eff}) – i.e., the flexural stiffness of the system after cracking – which can significantly impact the stability and deformability of the system. As deduced from the Figure 4.9(a), the effective flexural stiffness of the system was remarkably reduced after cracking, reaching 20.3% ($EI_{\text{eff}}=0.203EI_g$) of the initial, uncracked (gross) stiffness (EI_g) value.

The analysis of the Tx-U40 specimen was repeated to account for the flexibility of the supports. The analytical load-displacement response for the concentric test is compared with its experimental counterpart in Figure 4.9(b). The experimental data shown in the figure were obtained from the linear potentiometers installed in the midspan. As seen, the analysis was found capable of accurately reproducing the overall experimental response, including initial stiffness, cracking load, post-cracking response, and ultimate capacity. Accounting for the flexibility of the supports may not be of significant importance in the analysis of the concentric test, given that the specimen may not experience significant rigid body motion. However, it can play a crucial role in the analysis of the eccentric test, which is demonstrated later in this chapter.

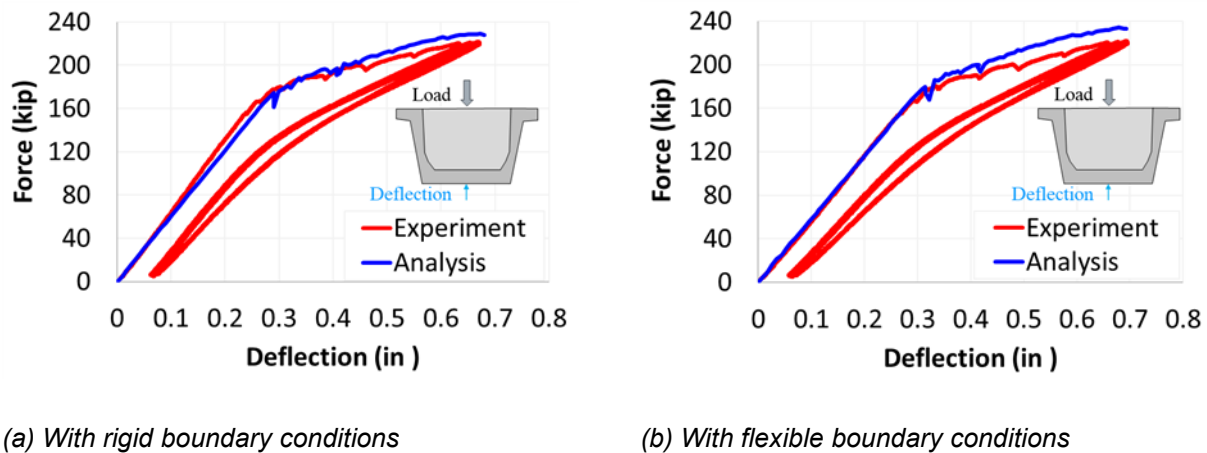


Figure 4.9. Comparison of experimental and analysis results for Tx-U40 specimen subjected to concentric force

An interesting part of the analysis results pertains to the prediction capabilities of the damage by the finite element analytical (FEA) model. Figure 4.10 demonstrates the analytically-obtained crack initiation and propagation for the Tx-U40 specimen, along with the experimentally-observed behavior at the end of the test. Cracks initiated on the bottom face of the girder and subsequently propagated to the web. The FEA crack patterns are similar to the experimental results.

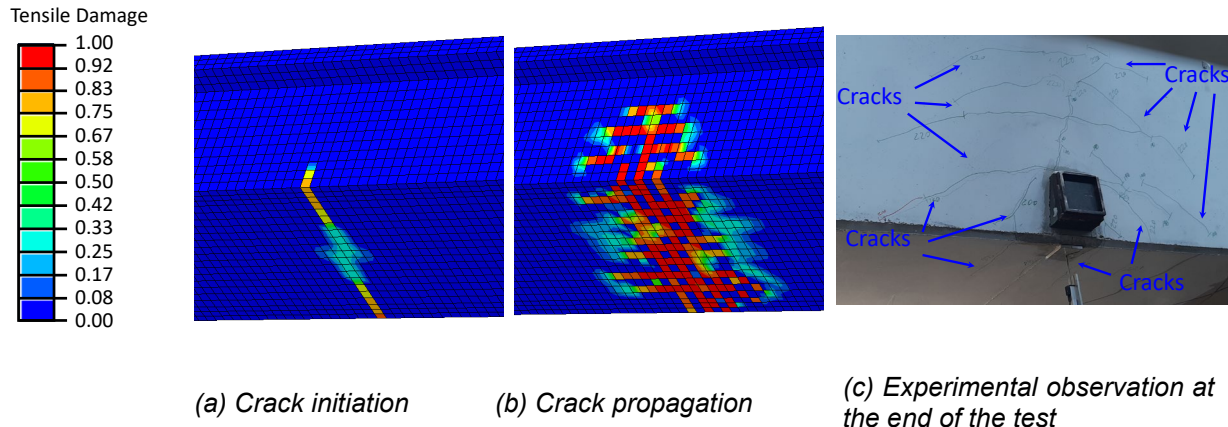


Figure 4.10. Experimental and analytical tensile damage observed in the concentric test for the Tx-U40 girder

The load-displacement response obtained from the analysis of the Tx-U40 girder, subjected to the eccentric loading, is compared with the experimental results in Figure 4.11. The specific analysis provided in Figure 4.11(a) excludes the flexibility of the supports assuming rigid boundary conditions. The experimental results in Figure 4.11(a) were obtained by subtracting the vertical deformation of the girder near the support from the midspan deflection. The latter is pursued to eliminate the rigid-body motion from the experimental result, ensuring a fair comparison with the analysis results for the case with rigid boundary conditions. As shown, the analysis was capable of capturing the initial stiffness of the specimen, providing a good agreement with experimental results. As noted earlier, the experimental response exhibited a failure of the support bracket at a displacement of 0.27 in. However, this behavior is due to the inelastic deformation and yielding of the out-of-plane boundary conditions that represent the twist-resisting supports and therefore not of interest in the girder response.

The analysis for the eccentric test was repeated for a second time, accounting for the flexibility of the supports. The comparison between the experimental response and analytical results for this case is shown in Figure 4.11 (b). The experimental result in this particular figure was derived from vertical measurements of the linear potentiometer installed in the middle of the span (without subtracting the vertical displacement of the girder near the support). Hence, the experimental data depicted in Figure 4.11 (b) differs from that in Figure 4.11(a) due to the presence of rigid body motion. The analysis result depicted in Figure 4.11(b) was capable of accurately capturing the experimentally-measured initial stiffness. However, at a displacement of 0.3 in., the numerical response deviated from the experimental response due to the problem with the support bracket.

Subsequently, as opposed to the response predicted by the model, the experimental response presented a plateau followed by a drop in the force. Because the failure of the setup component is not of interest in the parametric studies, the data of interest for model validation is adequately captured in the analysis.

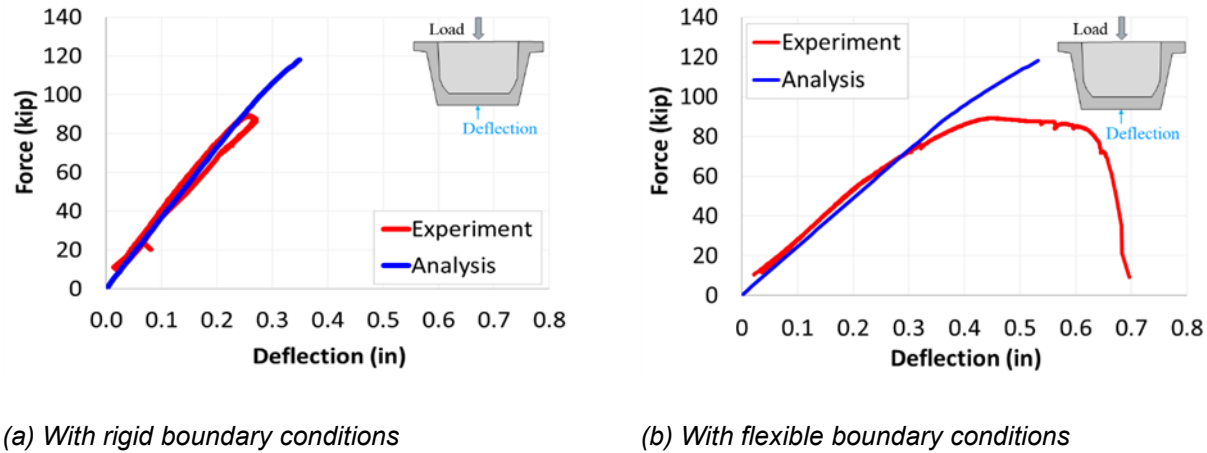


Figure 4.11. Comparison of experimental and analysis results for Tx-U40 specimen subjected to eccentric force

The vibrating wire strain gauges that were embedded in the girder cross-section provide another set of validation parameters that can be used to give additional appraisal to the FE model. The strains measured during the eccentric test have both flexural and torsional (warping) components. The experimental data obtained from strain gauges are compared with the computational results at the quarter points and midspan locations along the length of the girder. The naming convention of gauges for the first quarter, second (midspan) and third quarter points begin with “A”, “B”, and “C” followed by the gauge number, respectively. Figure 4.12 compares the analytically obtained strain values with the experimentally obtained values from vibrating wire gauges installed at different locations at the first quarter point. The experimental measurements were predominantly linear since cracking did not occur at the given load levels at this location. The strain values represent strains along the girder length, i.e., ϵ_{xx} per Figure 4.5. The analysis effectively captured the strain values observed in the experiment for all fibers on the top and bottom flange portions, thereby validating the numerical approach used herein. The gauge readings are positive on the bottom flange, as opposed to those on the top flange. This is due to the state of stress developed in the particular regions, which was tension in the former, and compression in the latter.

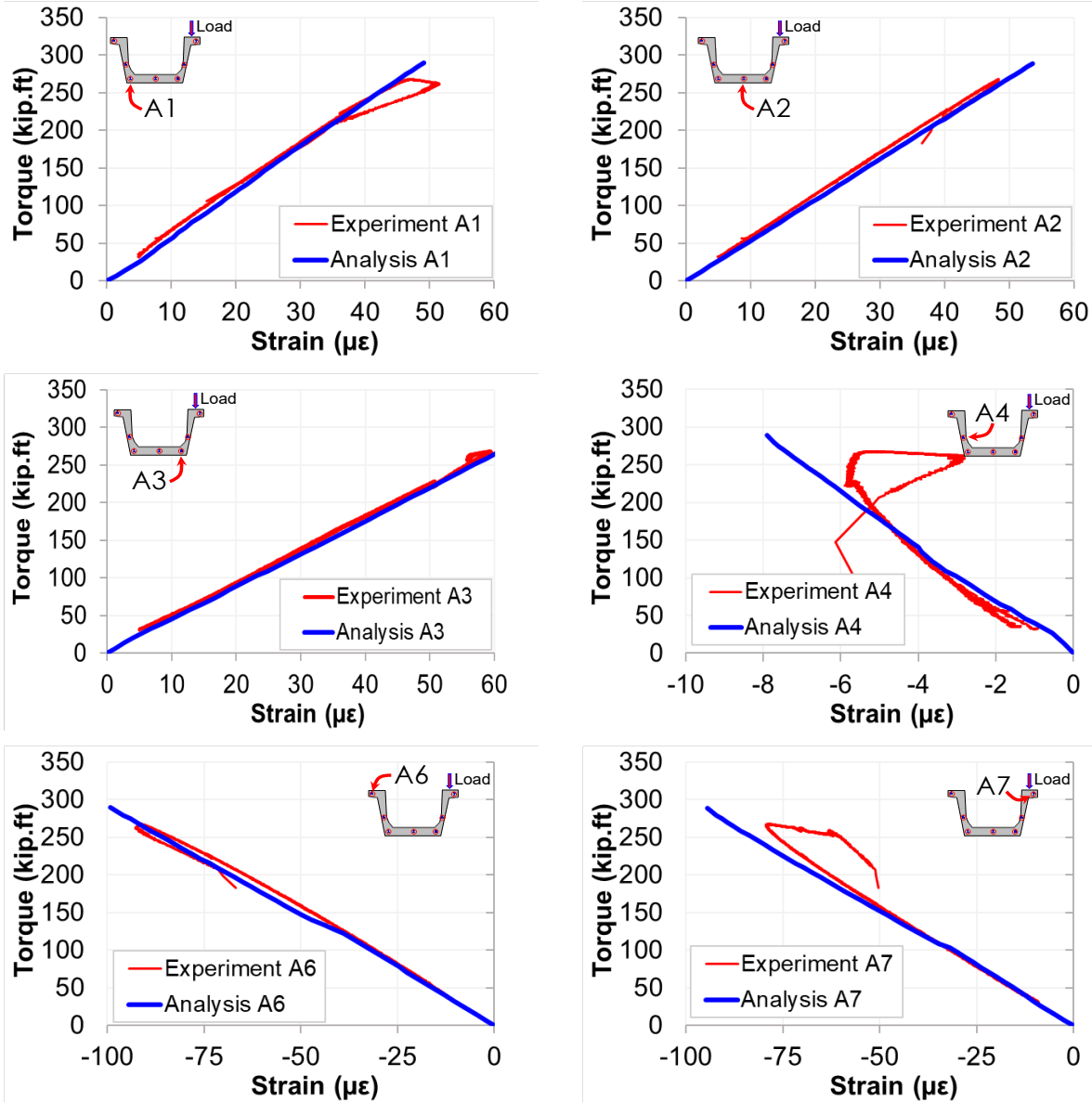


Figure 4.12. Numerical and experimental strain values at different locations in the cross-section at the first quarter point of the Tx-U40 specimen

The strain values obtained from the vibrating wire gauges installed at the midspan (second quarter point) are compared with the corresponding numerical results in Figure 4.13, while the values at the third quarter point are provided in Appendix A. Similar to the results for the first quarter point, the strain values are associated with the ϵ_{xx} component per Figure 4.5. Given the nonlinear response of the specimen in the vicinity of the midspan region, the girder had incurred significant cracking, as deduced from Figure 4.13. Overall, the analysis effectively captured the strain values for the top and bottom fibers. The state of stress for the top fibers was compression, resulting in negative strain readings. In contrast, the corresponding strain values for the bottom fibers were positive – except for gauge B1 – which indicated the development of tensile stress in that particular region.

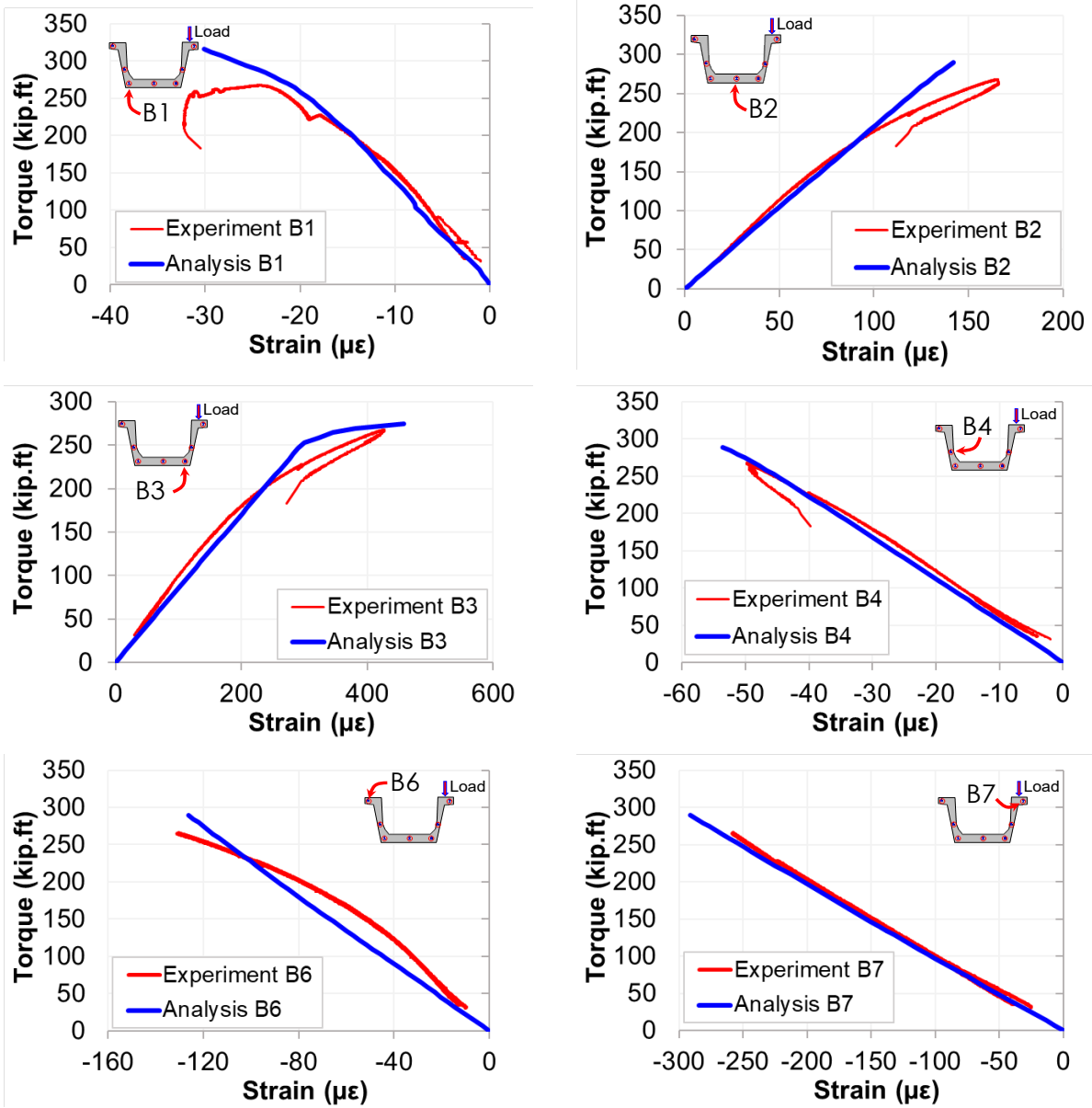


Figure 4.13. Numerical and experimental strain values at different locations in the cross-section at the midspan (second quarter point) of the Tx-U40 specimen

The propensity of damage for the eccentric test, corresponding to 4.11 (b), is illustrated in Figure 4.14. The analysis results predicted cracking in the midspan, at the intersection of the diaphragm, and near the end regions. The intensity of the damage was verified by the experimental observations.

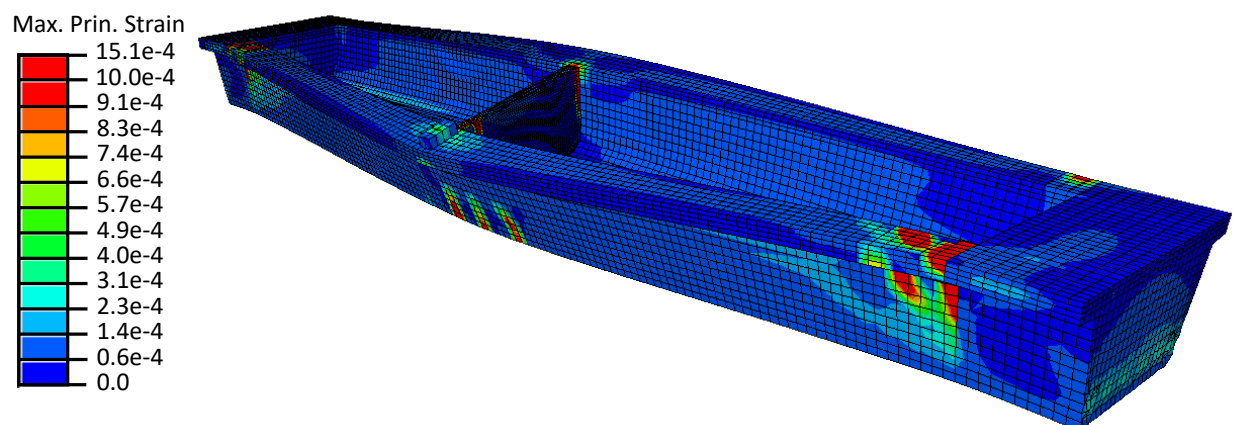


Figure 4.14. Contour plot for maximum principal strain observed in the analysis of Tx-U40 specimen subjected to an eccentric force (scale factor=50).

4.2.2. Tx34 Girder

The calibration and validation procedure of the Tx34 girder – as part of numerical study – is discussed in this subsection. For the sake of brevity, the current document only focuses on the final – calibrated – results. The analytically-obtained force-deflection response for the Tx34 girder, subjected to concentric loading, is compared with the experimental counterpart in Figure 4.15. The objective of the test was to capture the uncracked flexural properties of the specimen. The analytical result was found capable of capturing the experimental response with an acceptable accuracy.

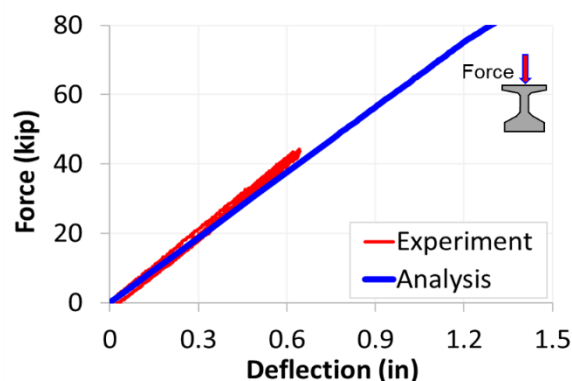


Figure 4.15. Force-deflection response for Tx34 specimen subjected to concentric loading in uncracked state

The lateral load test, aimed at capturing the flexural-torsional response about the weak axis, involved two steps. The specimen was subjected to a concentric vertical load at the midspan first, followed by a lateral load positioned four feet away from the midspan. The experimental and

analytical results pertaining to different load steps are shown in Figure 4.16. The analysis provided a good estimate of the experimental response under different loading conditions.

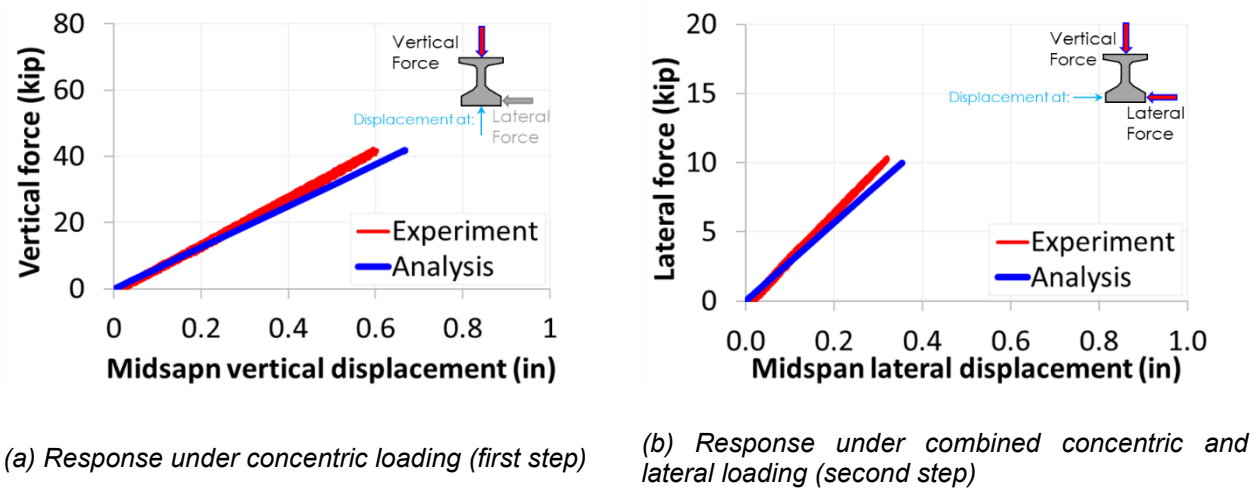
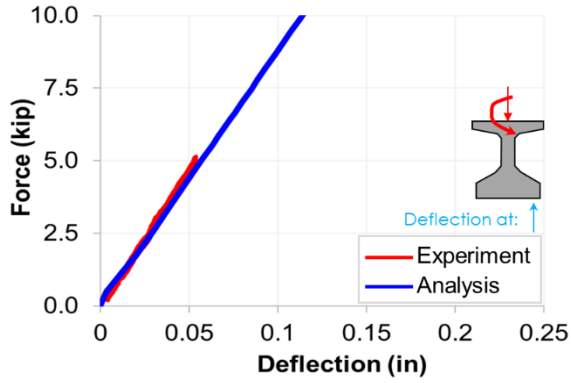
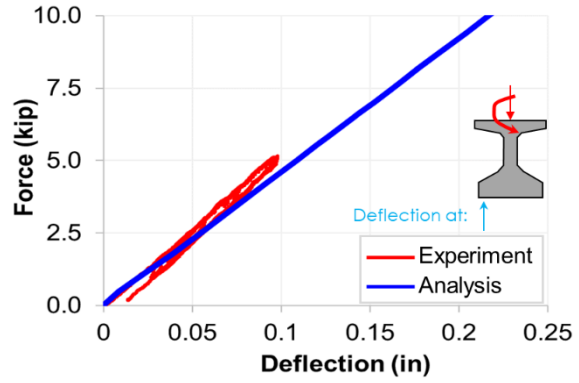


Figure 4.16. Force-displacement response of Tx34 specimen subjected to lateral loading in uncracked state

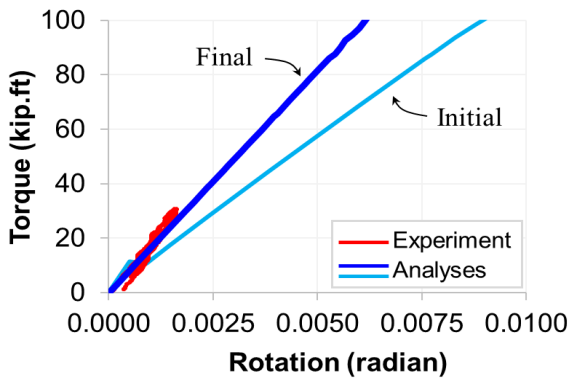
The experimental and analytical response of the Tx34 specimen subjected to eccentric loading are compared in Figure 4.17. The specific configuration subjected the specimen to combined bending and torsion. The force-deflection response, obtained from different locations at the midspan, is provided in Figure 4.17(a) and Figure 4.17(b), followed by the torque-rotation response in Figure 4.17(c). The analyses captured both the experimentally-obtained displacement and rotation with good accuracy. Figure 4.17(c) also includes the initial analysis results based on the first engineering guess, which obviously underestimated the experimental response.



(a) Force-deflection response (deflection at east)



(b) Force-deflection response (deflection at west)



(c) Torque-rotation response

Figure 4.17. Torsional response of Tx34 specimen subjected to eccentric loading in uncracked state

The analytical force-displacement response for the concentric test is compared with the experimental counterpart in the uncracked state in Figure 4.18. This specific test is of significant importance as it provides insight into the flexural response of the specimen in the cracked state. The analysis effectively captured the experimental behavior, including the initial stiffness, cracking load and post cracking response. In addition, the effective stiffness of the system was evaluated. As deduced from the figure, the effective flexural stiffness of the system was significantly reduced after cracking, reaching 41% ($EI_{\text{eff}}=0.41EI_g$) of the initial uncracked (gross) stiffness (EI_g), highlighting the significance of cracking effects on the system stability and deformability response.

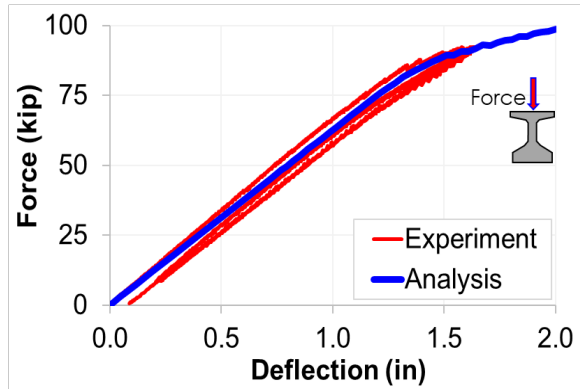
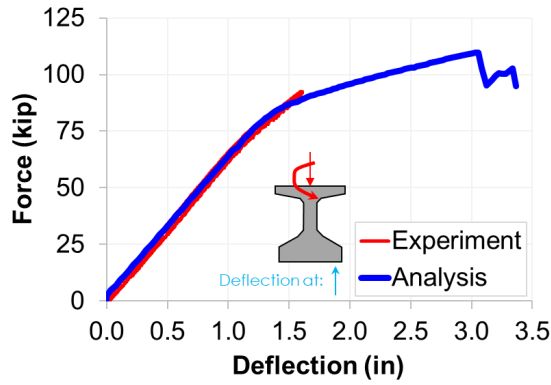
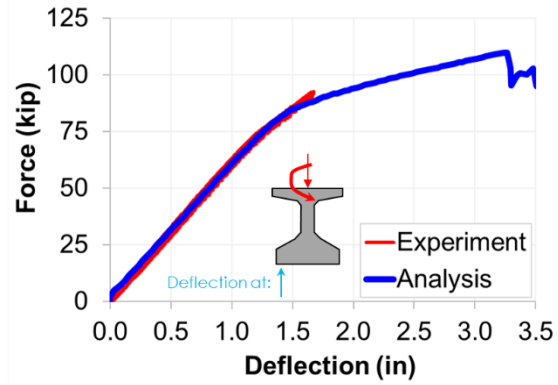


Figure 4.18. Force-deflection response of Tx34 specimen subjected to concentric loading in cracked state

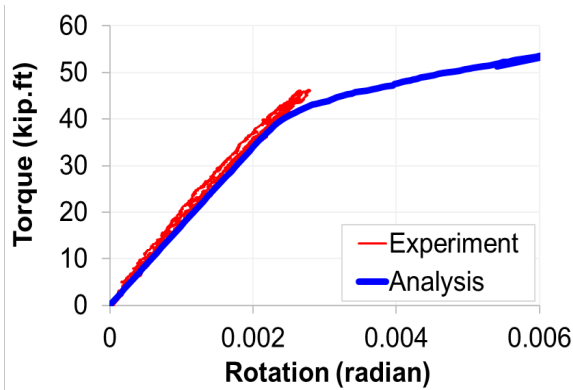
The experimentally observed torsional response for the Tx34 specimen in a cracked state is compared with the analytical counterpart in Figure 4.19. The analysis was capable of reproducing the experimental force-displacement response for two locations across the midspan, as depicted in Figure 4.19(a) and Figure 4.19(b). The initial stiffness and post-cracking stiffness were captured with acceptable accuracy. In addition, the analysis provided a good estimate of the girder rotation at the midspan, consistent with the experimental observations.



(a) Force-deflection response (deflection at east)



(b) Force-deflection response (deflection at west)



(c) Torque-rotation response

Figure 4.19. Rotational response of Tx34 specimen subjected to eccentric loading in cracked state

Similar to the validation of the Tx-U40 specimen, the strain values obtained from vibrating wire strain gauges are compared with the analysis results for the Tx34 girder at the three-quarter points along the girder length to further elaborate on the validation process. The experimental results were obtained from the eccentric test which included both flexural and torsional (warping) components. The naming convention of the gauges for the Tx28 and Tx34 follows that used for the Tx-U40 specimen. Figure 4.20 compares the analytically-obtained strain values with the experimental results obtained from the gauges installed at different locations across the section height at the first quarter point. The strain values corresponded to those along the girder length, i.e., ϵ_{xx} per Figure 4.8. Since cracks had not initiated in the section at the first quarter point, the experimental measurements exhibited predominantly linear behavior. The analysis effectively captured the strain values observed in the experiment for all fibers on the top and bottom flanges, thereby validating the numerical approach. The gauge readings are positive (tension) on the bottom flange, in contrast to those on the top flange. This difference stems from the developed states of stress in these regions, with tension in the former and compression in the latter.

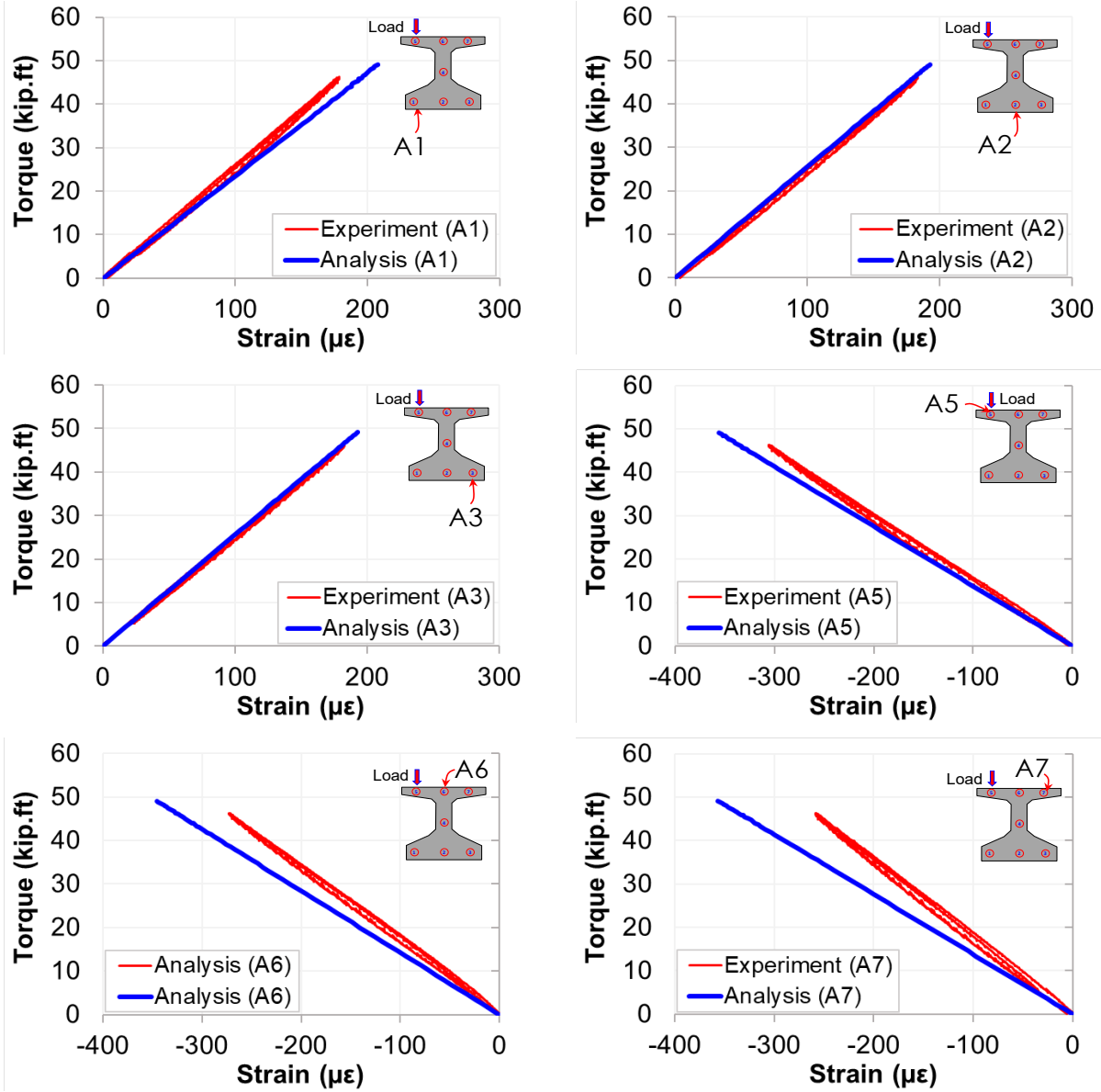


Figure 4.20. Numerical and experimental strain values at different locations in the cross-section at the first quarter point of the Tx34 specimen

The experimentally-measured strain values obtained from the vibrating wire gauges installed at the midspan (second quarter point) are compared with the corresponding numerical values in Figure 4.21, while the values at the third quarter point are compared in Appendix A. Similar to the results for the first quarter point, the strain values correspond to the ϵ_{xx} component per Figure 4.8. As deduced from the results shown in Figure 4.29, cracks developed near the midspan region due to the nonlinear response of the specimen. The analysis effectively captured the strain values for the top and bottom fibers. The states of stress for the top and bottom fibers were tension and compression, resulting in positive and negative strain readings, respectively.

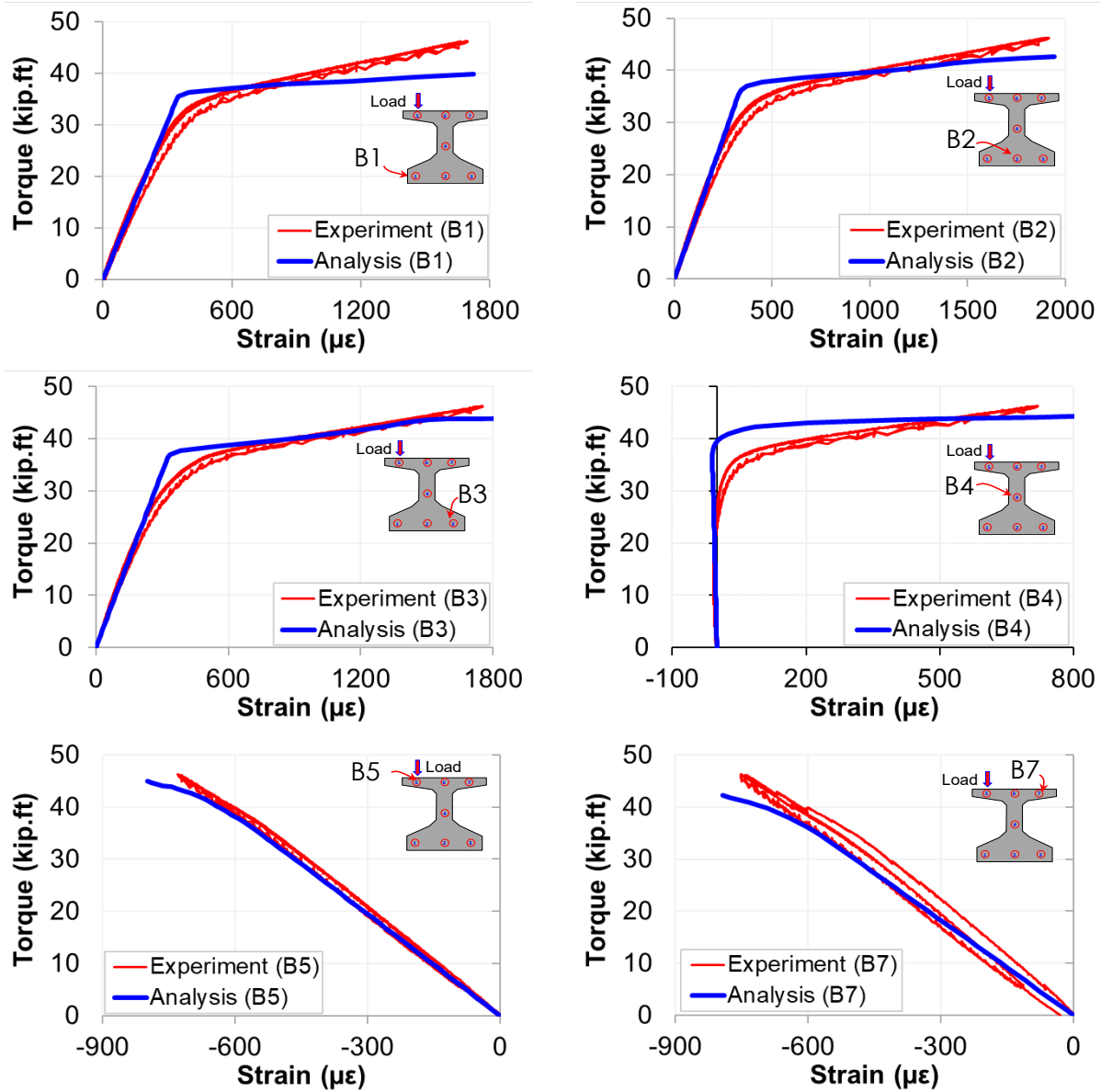


Figure 4.21. Numerical and experimental strain values at different locations in the cross-section at the midspan (second quarter point) of the Tx34 specimen

The propensity of mechanical damage observed in the experiment is compared with the analytical results in Figure 4.22. Cracks initiated in both the experiment and the analysis from the bottom flange of the girder and subsequently propagated to the web region. As per experimental observations, cracks in the web region appeared in the bottom portion only, which was consistent with the numerical results. The analysis also captured the orientation of the cracks with acceptable accuracy. The crack orientations were straight in the bottom flange and bottom face of the girder. According to experimental observations, cracks remained straight in the immediate vicinity of the midspan in the web region but became inclined in the region away from the midspan. The

orientation of the straight cracks was captured by the analytical results, however, due to the mesh size, the inclined cracks in the web were partially captured.

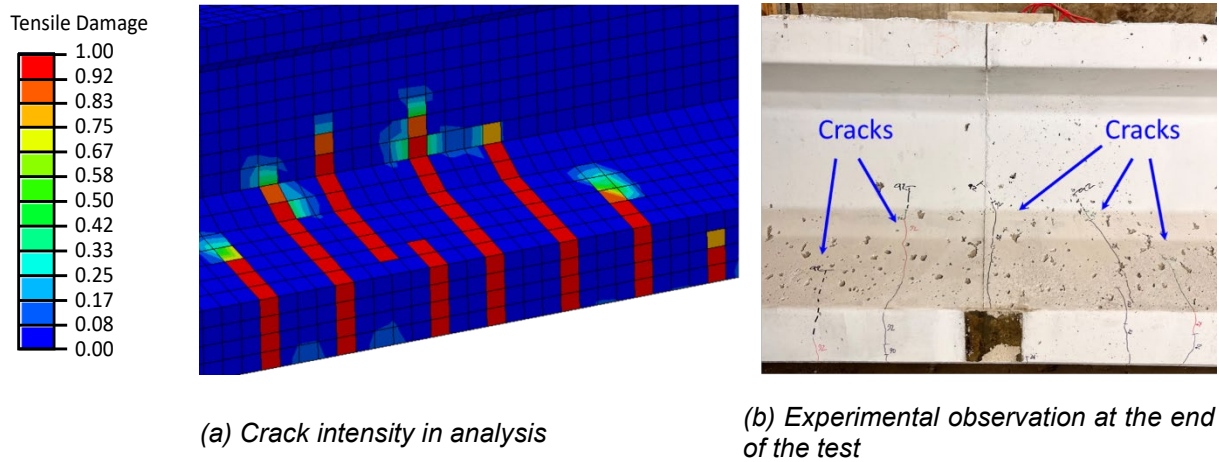


Figure 4.22. Experimental and analytical tensile damage observed in the eccentric test for the Tx34 girder

4.2.3. Tx28 Girder

The flexural response of the Tx28 specimen observed in the experiment is compared with its analytical counterpart in Figure 4.23. This test provided insight into the uncracked flexural properties of the specimen, and the analysis gave a near-perfect match reproduction of experimental response.

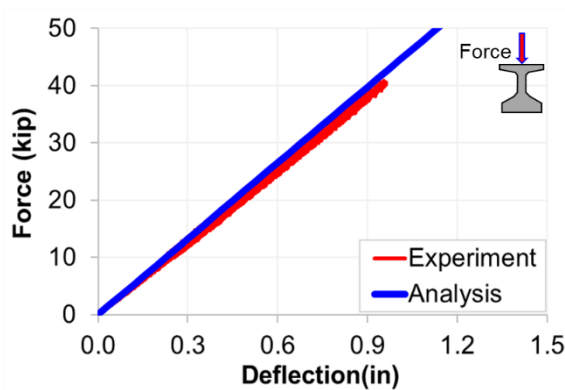
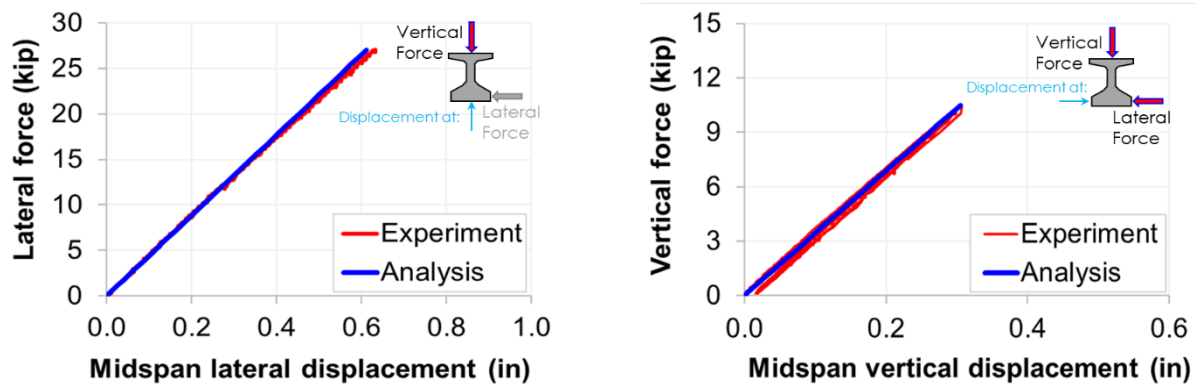


Figure 4.23. Force-deflection response of Tx28 specimen subjected to concentric loading in uncracked state

The specimen was subsequently subjected to a lateral test to quantify the torsional properties of the girder about the weak axis. The testing protocol for the lateral test is similar to that of the Tx34

girder and involved two steps. The girder was initially subjected to a downward point load at the midspan, followed by a lateral load positioned 4 feet from the midspan. Figure 4.24(a) compares the experimentally- and numerically-obtained force-deflection responses for the first step. The analysis effectively captured the experimental response. The experimental and numerical results are subsequently compared in Figure 4.24(b) for the second step and the analysis gave a good reproduction of the experimental response.

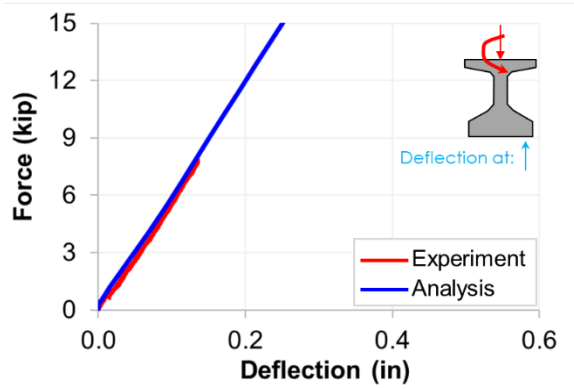


(a) Response under concentric loading (first step)

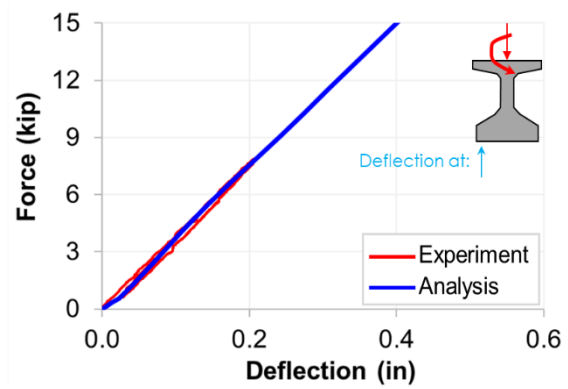
(a) Response under combined concentric and lateral loading (second step)

Figure 4.24. Force-displacement response of Tx28 specimen subjected to lateral loading in uncracked state

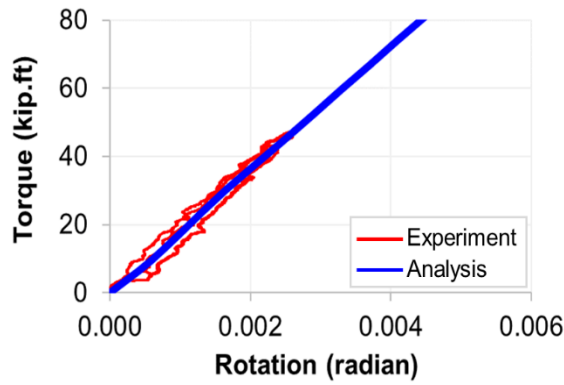
The torsional response of the Tx28 girder was then investigated as the final test in the uncracked state. Figure 4.25(a) and Figure 4.25(b) compare the experimental and numerical force-deflection response of the Tx28 girder, measured at different locations across the midspan. The torsional properties of the specimen, obtained from analysis and experiment, are illustrated by the torque-rotation curves in Figure 4.25(c). Overall, the analyses gave a good estimate of the experimental response regarding the displacement and rotation measurements.



(a) Force-deflection response (deflection at east)



(b) Force-deflection response (deflection at west)



(c) Torque-rotation response

Figure 4.25. Torsional response of Tx28 girder subjected to eccentric loading in uncracked state

In the subsequent tests, the cracking response of the specimen was investigated. Figure 4.26 compares the analytically obtained load-deflection response of the Tx28 bridge girder, subjected to concentric loading in the cracked state, with the experimental counterpart. The analysis captured the experimental response with a remarkable level of accuracy, including the initial stiffness, cracking load, and post-cracking behavior. As inferred from Figure 4.26, the effective flexural stiffness of the system incurred a substantial reduction after cracking, reaching 32% ($EI_{\text{eff}}=0.32EI_g$) of the initial uncracked (gross) stiffness (EI_g). This specific observation underpins the significance of cracking effects on the stability and deformability of the system.

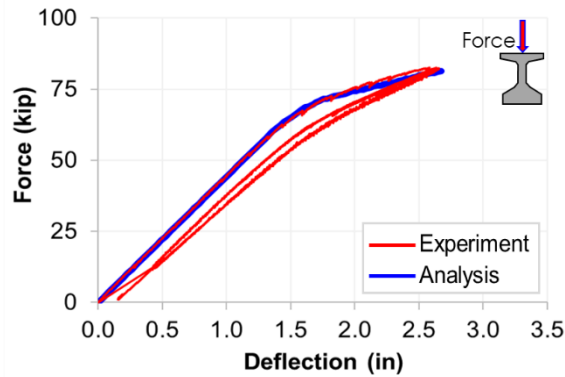
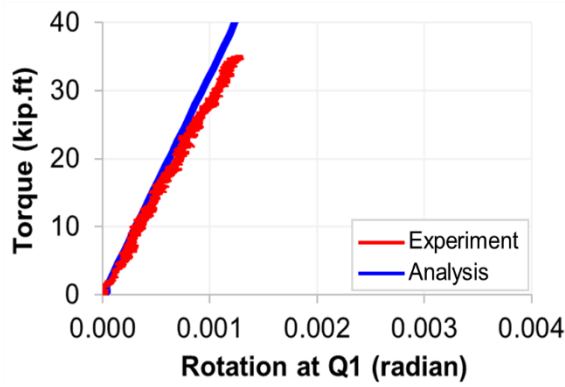
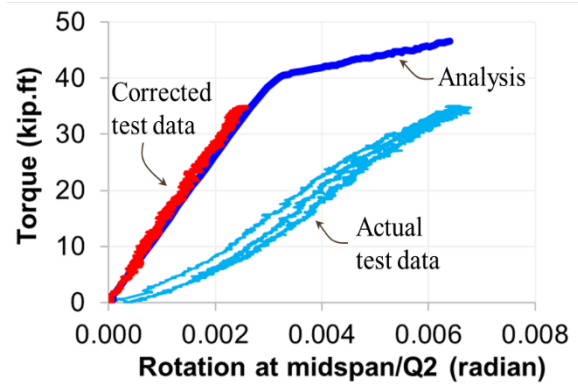


Figure 4.26. Force-deflection response of Tx28 specimen subjected to concentric loading in cracked state

The torsional properties of the specimen in the cracked state were subsequently investigated. Figure 4.27(a) compares the experimental and numerical torque-rotation response for the girder at the first quarter point. The analysis was capable of satisfactorily capturing the torsional response of the structure – incurred at first quarter point. The corresponding results at the midspan (second quarter point) is provided in Figure 4.27. As deduced from the figure, the experimentally-observed rotation, obtained from the linear potentiometer installed on the girder bottom flange encountered issues and entailed errors. As such, a synthetic data set was generated for the specific test, based on measurements from the first quarter point. Specifically, the rotation observed at the first quarter point was multiplied by the length ratio to the midspan location to that of first quarter point (i.e., $0.5L/0.25L = 2.0$). The analysis effectively captured the torsional response of the synthetic data. Because the girder was not cracked at the quarter points, the synthetic (corrected) test data can only provide an indication of the uncracked section; however, the FEA data predicts cracking at a torque of approximately 40 k-ft.



(a) Response at the first quarter point (Q1)



(b) Response at the midspan/second quarter point (Q2)

Figure 4.27. Force-displacement response of Tx28 specimen subjected to eccentric loading in cracked state

The validation analyses were further expanded by comparing the strain values obtained from vibrating wire strain gauges with the numerical counterparts. The strain values were obtained from the eccentric test, which included both flexure and torsional (warping) effects. The experimental data obtained from strain gauges are compared with the computational results for the three-quarter points along the length of the girder. Figure 4.28 compares the analytically-obtained strain values with the experimentally-obtained values from vibrating wire gauges installed at different locations at the first quarter point. The experimental measurements were predominantly linear since cracking had not been initiated at this location. The strain values represent strains along the girder length, i.e., ϵ_{xx} per Figure 4.8. The analysis effectively captured the strain values observed in the experiment for all fibers on the top and bottom flange portions, thereby validating the numerical approach used herein. The gauge readings are positive on the bottom flange as opposed to those on the top flange. This is due to the state of stress developed in the particular regions, which was tension in the former and compression in the latter.

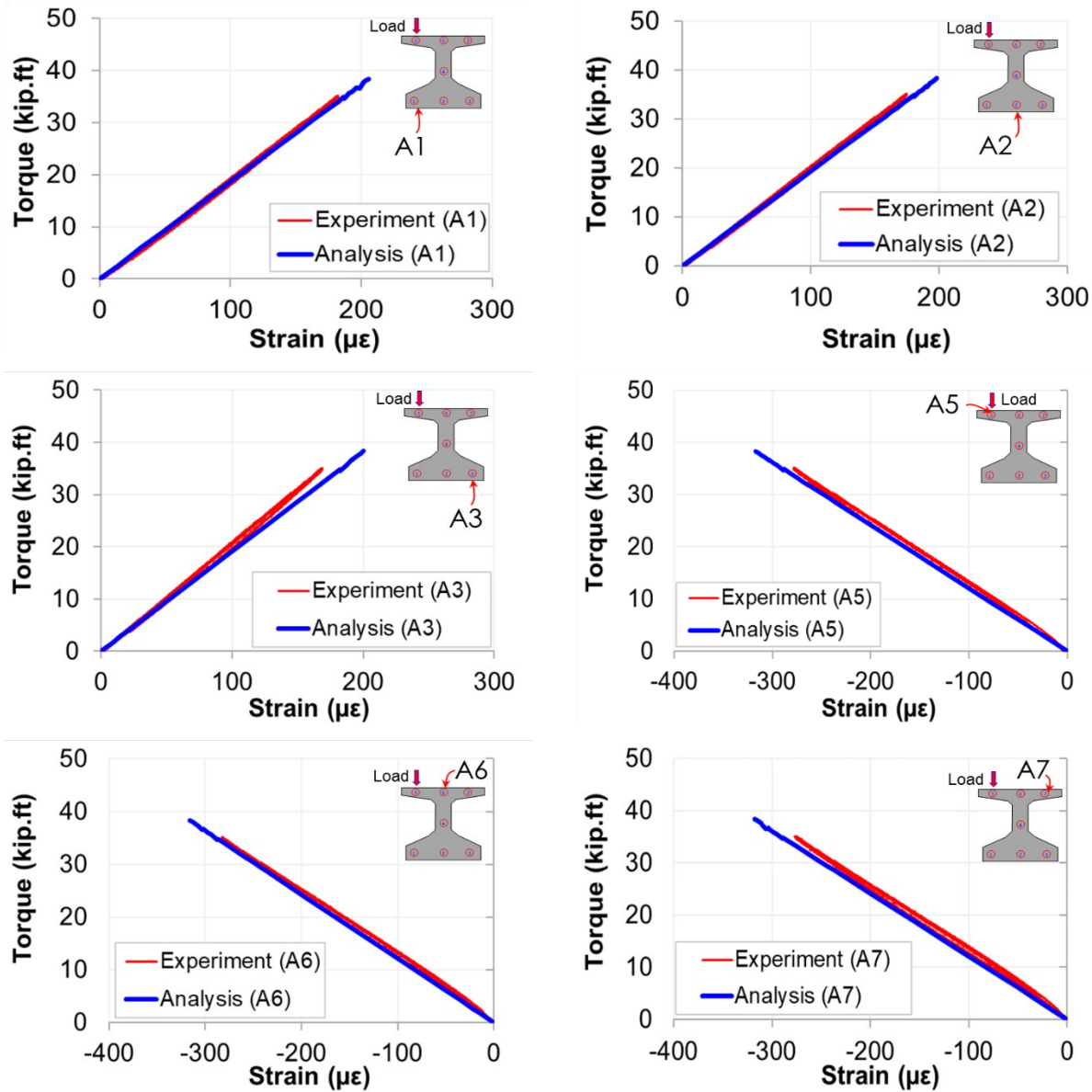


Figure 4.28. Numerical and experimental strain values at different locations in the cross-section at the first quarter point of the Tx28 specimen

The strain values obtained from the vibrating wire gauges installed at midspan (second quarter point) are compared with the corresponding numerical results in Figure 4.29, while the data for the third quarter point is presented in Appendix A. Similar to the first quarter point, the strain values correspond to ϵ_{xx} , per Figure 4.8. Due to the nonlinear response of the specimen in the midspan region, the girder experienced significant cracking, as deduced from Figure 4.29. Overall, the analysis effectively captured the strain values for the top and bottom fibers. The state of stress

for the top and bottom fibers was tension and compression, resulting in positive and negative strain readings, respectively. The analyses provided a better estimate for the top fibers strain, compared to the bottom fibers, considering the significant crack intensity and local crack effects that occurred in the latter.

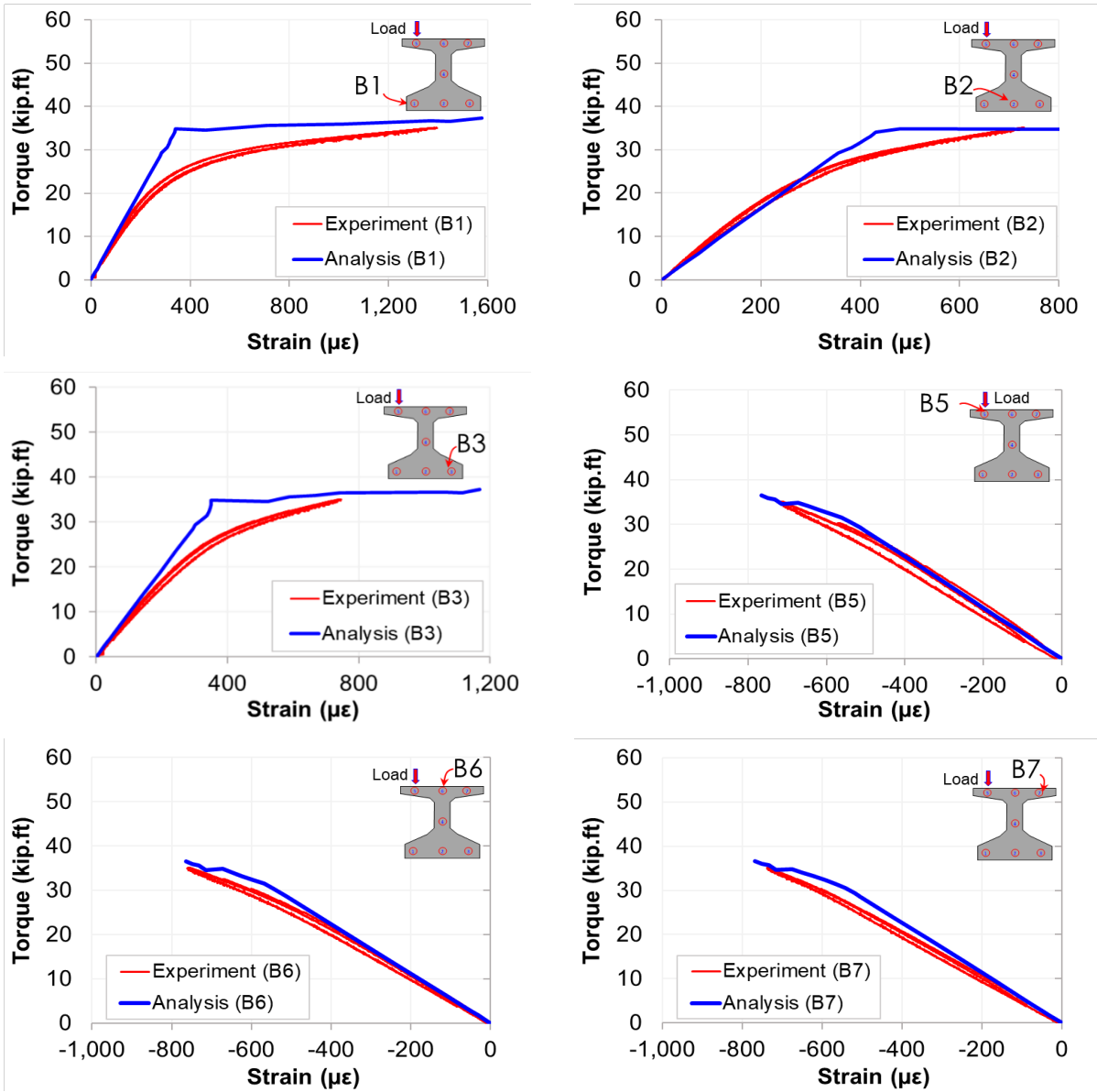


Figure 4.29. Numerical and experimental strain values at different locations in the cross-section at the midspan (second quarter point) of the Tx28 specimen

The analytical response also gives rise to the propensity of cracking, as observed in the contour plots depicting the material damage. Figure 4.30 compares the cracking pattern observed in the analysis and the experiment. The analytical response effectively captured the initiation, propagation, and orientation of the cracking observed in the experiment. The only slight discrepancy in the analytical response, compared to the experiment, pertains to the evolution of

the inclined cracks in the web region, which appears more smooth in the experiment. The latter was partially captured by the analysis due to the size of the mesh.

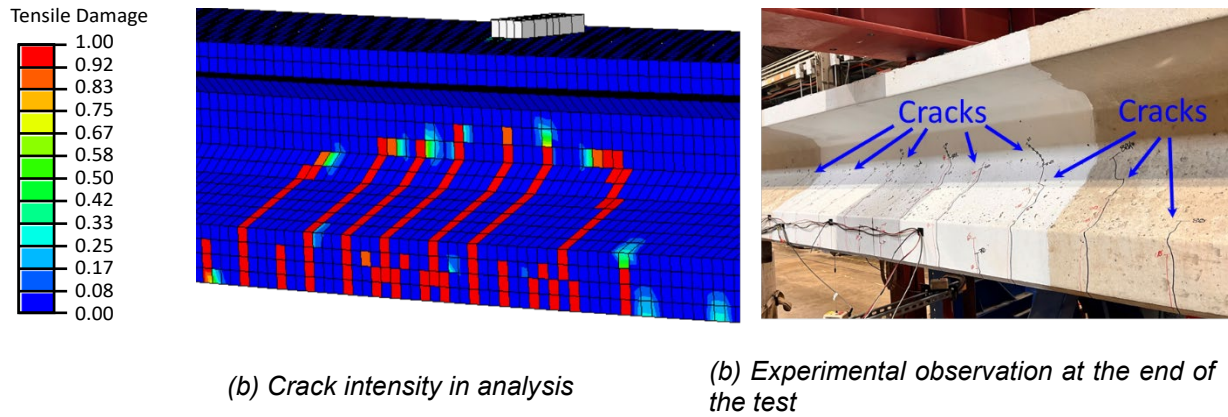


Figure 4.30. Experimental and analytical tensile damage observed in the eccentric test for the Tx28 specimen

Chapter 5. Parametric FEA Studies on Stability

In order to understand the stability behavior of prestressed concrete I and U-girders, four stability studies evaluating stiffness and strength were conducted:

- Validation of in-plane girder stiffness and brace stiffness behavior
- Evaluation of stability behavior during erection
- Evaluation of stability behavior during deck placement
- Evaluation of the impact of geometric imperfections and cracked sections

Extensive studies have previously been conducted related to bracing in steel I-girder systems. Based upon previous studies, the stability brace stiffness behavior is often dictated by springs in series, with the least stiff spring controlling the system stiffness behavior. The studies first focused on quantifying the stiffness components for prestressed girder systems. The investigation was accomplished by idealizing the girder systems and subsequently isolating the stiffness components. Steel cross-frames were used initially to quantify the torsional brace stiffness. Past investigations have provided closed-form solutions for the torsional brace stiffness of cross-frame systems. The solutions idealize the cross-frame members as truss elements dominated by the axial stiffness of the individual members. The results were compared to current details commonly utilized in Texas, which generally consist of cast-in-place concrete diaphragms.

The stability evaluation in the study was focused on the stability behavior during erection and deck placement studies since these are the most likely stages of concern. Once the concrete deck has cured, the girders are continuously restrained both lateral and torsionally. Overall, individual girders of typical dimensions were analyzed first to determine the necessity of relying on system behavior. The stability of the system is generally improved as girders are added to the system.

The final phase of the investigation considered the impact of geometric imperfections on the buckling capacities and brace stiffness requirements of the girder system. Geometric imperfections result in larger deformations with a nonlinear load displacement response. Individual girders were analyzed to evaluate the stages that might be critical to the behavior. The analyses also examined the stress and strain distributions of the girders and bracing at various stages of erection and construction.

As part of the study, a survey was conducted of spliced-girder bridges constructed in Texas. Key bridge geometries and parameters are summarized in Table 5-1. It is recognized that these bridge types, sizes, and locations often have unique project requirements and constraints that do not necessarily overlap with previous or future projects. However, the data provides an indication of ranges of geometric parameters that are likely to occur in Texas bridges. Table 5-2 presents the

parameters for constant girder depth bridges and Table 5-3 corresponds to variable (haunched) girder depth bridges.

Table 5-1. Survey of spliced girder bridges in Texas

Bridge Location and Feature	Owner	Girder Depth	Girder Type	f'ci, f'c (ksi)	Span	Span Ratio	Span/Depth	Haunch Ratio	Drop-in	Splice Loc. %L	# Girders	Girder Spa.	Overhang	Skew	Plans
Clear Fork Bridge, Fort Worth, Tx	Fort Worth	Variable	mT-VI(72") - 120"		165-220-165	0.75	32.8	1.7	124	21.8%	4			0	N
Dallas Horseshoe, Dallas, Tx Over Trinity River	Dallas	Variable	mTx82"-130"		240-260-260-240	0.92	34.5	1.6	160	19.2%	9			0	N
					250-250-250-250	1.0	33.1		150	20.0%					
					240-270-270-240	0.89	35.8		170	18.5%					
Trinity River, Dallas, Tx DART	DART	Variable	mTx82"-130"		145-260-145	0.56	34.5	1.6				10		0	N
IH10 Access Road over UPRR, Seguin, Tx	TxDOT	Constant	mTx82	6, 8.5	123-200-126	0.63	26.5	1.0	153.4	11.7%	4	9	3	38	Y
Sylvan Avenue Bridge, Dallas, Tx	TxDOT	Constant	mTx82"	6.5,	171-200-171	0.86	26.5	1	113.5	27.3%	12	9.55	3.5	20	Y
		Variable	mTx82"-130"	8.5	201-250-201	0.8	33.1	1.58			11	10.0	3.5		
I-35 N UPRR Overpass, Round Rock, Tx	TxDOT	Variable	mTx82"-130"	6, 8.5	180-250-180	0.72	33.1	1.6	149.8	14.0%	4	6.67	3	0	Y
Bridge at Sargent Beach, Sargent Beach, Tx	TxDOT	Variable	STx8-12.5	6, 8.5	195-300-195	0.65	34.4	1.6	186	19.0%	5	10.25	4.5	0	Y
FM1495 at Old Brazos River, Freeport, Tx	TxDOT	Constant	STx7	6, 10	160.1-215-160.1	0.74	27.9	1.0	122	21.6%	7	9	4.5	0	Y
					132.7-192-124.5	0.65	24.9		85	27.8%	6	10.8	4.5		
SH 249 NB to SH99WB DC (Ramp C), Houston, Tx Toll Road	Harris Co. Toll	Constant Rad 920'	U96"	6, 8.5	212-212 146-239-239-180	1.0 0.75	24.34 27.4	1.0	83 94-94	30.2% 30.3%	2	16	3.46	0	Y
PennDOT Std. Splice Girder Bridges 3,4,5 & 6 Span Configurations	PennDOT	Variable	I Gird 87.5"-120"	?, 8.0	215-270-215	0.83	33.8	1.4	148	22.6%	5 & 6	10 11	5	0	Y

Note: Dimensions are in feet unless noted

STx = Spliced Tx Girder

m = modified (standard shapes with thickened flanges and/or web)

Span Ratio is calculated by taking an adjacent span and dividing it by the main (longest) span

Span-to-Depth Ratio is calculated with girder depth + 8.5" slab thickness

Per TxDOT Extended Spans: Tx84 max single span length is 170 ft. STx & HSTx Girders spans can vary from 170 to 325 ft.

Cast-in-place concrete diaphragms at piers, splice locations, and midspan.

Temporary diaphragms resembling the TxDOT standard are generally detailed to be between 20-60 feet spacing.

Table 5-2. Parametric study variables for constant girder depth bridges

Parameter	Variables		
Skew (deg.)	0	30	45
Girder Spa. (ft)	8	10	12
Number of Girders	4		
Overhang (ft)	4.5		
Diaphragms	Supports	Supports + Splice Loc.	Supports +Splice +Midspans
Deck thickness (in)	8.5		
Girder Depths	STx7	STx9	
Span / Depth Ratio (including deck)	25	30	
Bridge Span Arrangement	0.7L-L-0.7L		

Table 5-3. Parametric study variables for variable (haunched) girder depth bridges

Parameter	Variables		
Skew (deg)	0	30	45
Girder Spa. (ft.)	8	10	12
Number of Girders	4		
Overhang (ft.)	4.5		
Diaphragms	Supports	Supports + Splice Loc.	Supports +Splice +Midspans
Deck thickness (in.)	8.5		
Girder Depths	STx7	STx9	
Haunched Girder Depth	1.6 x Constant Depth		
Splice Loc.	20%L		
Span / Depth Ratio (including deck)	30	35	
Bridge Span Arrangement	0.7L-L-0.7L		

5.1. Bracing Requirements for Stability

As mentioned in the foregoing sections, effective stability bracing must satisfy both strength and stiffness requirements. Timoshenko's (1961) derivation of the classic expression for lateral-torsional buckling of doubly-symmetric girders demonstrated that effective bracing of beams control twist of the girder. Because buckling is a compression-driven phenomenon, effective bracing can therefore be achieved by either preventing lateral movement of the compression flange

or twist of the girder cross-section. During construction, most bridge systems make use of cross-frames or diaphragms to establish girder stability. However, at various stages of erection, lateral restraint can be provided from abutments, piers, or temporary shore towers. In general, torsional bracing restrains girder twist along the longitudinal (roll axis) of the girder – as depicted in Figure 5.1. The brace is idealized in the figure as an “analog” coiled watch spring, but in reality, that brace is a structural member or system that develops a restoring torque to resist girder twist.

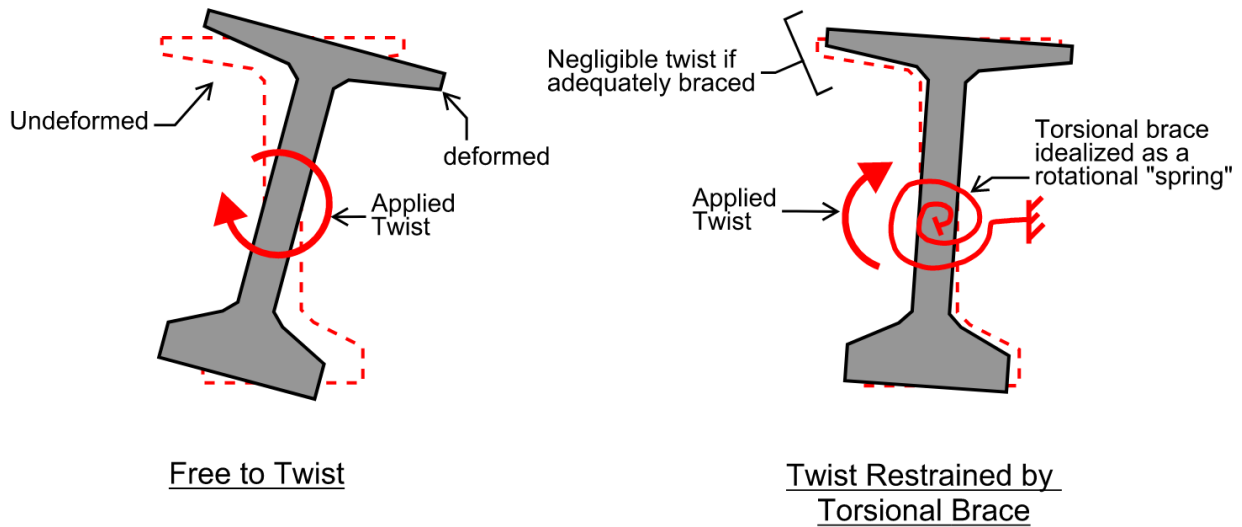


Figure 5.1. Effects of torsional bracing between support points

Winter (1960) was the first to demonstrate that effective bracing must satisfy both strength and stiffness criteria and further demonstrated that stability brace forces are a function of the magnitude of the initial imperfections. To control brace forces, a stiffness larger than the ideal brace stiffness is generally utilized. The ideal stiffness is the minimum stiffness required to force a perfectly straight member to buckle between the brace points. The ideal stiffness can be identified computationally by conducting an eigenvalue buckling analysis. Past studies on torsional bracing of steel girder systems (Yura 2001) have identified three primary brace stiffness components consisting of the stiffness of the brace (β_{br}), the in-plane stiffness of the girder (β_g), and the cross-section distortional stiffness (β_{sec}). The total system stiffness ($\beta_{T,ideal}$) is given by the expression for springs in series:

$$\frac{1}{\beta_{T,ideal}} = \frac{1}{\beta_{br}} + \frac{1}{\beta_g} + \frac{1}{\beta_{sec}} \quad (5.1)$$

Although a wide range of studies have been conducted on the stability of steel girder systems, very little work has been devoted to the stability of concrete girders. As part of the study, the applicability of closed-form buckling solutions for concrete girder systems was evaluated utilizing FEA results. In addition, the applicability of past stiffness formulations developed for concrete girder systems were also conducted to assess the ideal brace stiffness requirements as compared

to the formulations for $\beta_{T,ideal}$, β_{br} , and β_g . Because nearly full depth braces were used along with the stocky webs of concrete girders, β_{sec} was deemed infinite for the studies.

Abaqus (2023) was used to conduct the parametric FEA studies. For the purposes of evaluating closed-form buckling solutions, the constitutive model utilized linear-elastic material properties for all elements. The boundary conditions consisted of idealized pin and roller conditions with lateral restraint at the top and bottom of the girder at support locations. Solid hexahedron elements (C3D8R) were utilized. The mesh size was between two to three inches in the cross-section of the girder and up to six inches along the length of the girder. Uniform moment loading at the ends of girders was applied. The uniform moment loading was applied using continuum coupling to avoid localized effects in the models from point loads. The application of continuum coupling to achieve uniform moment across the entire girder length and other modeling properties are shown in Figure 5.2.

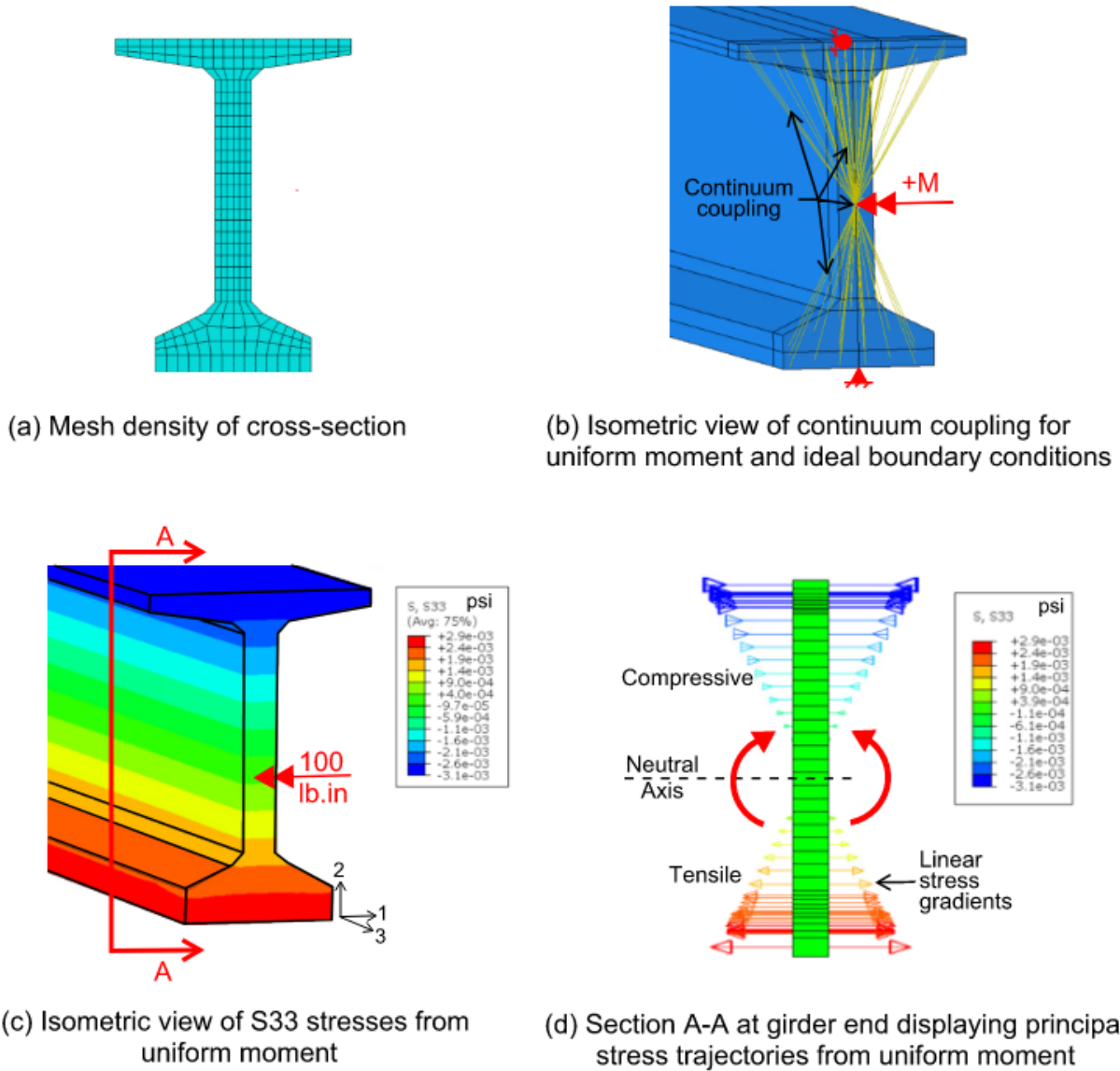


Figure 5.2 FEA modeling and uniform moment application on STx7 girder

An eigenvalue buckling analysis was performed employing the subspace iteration approach to calculate the critical buckling moment (M_{cr}). An initial reference load of 100 lb.-in was used for the eigenvalue solution. The first mode buckling solution of a 175 ft, STx7 simply supported girder with an $f'_c = 8,000$ psi and modulus of elasticity (E_c) of 5,260 ksi is shown in Figure 5.3. The theoretical calculation value shown in Figure 5.3 is from Galambos' (1968) singly symmetric girder solution provided in Equation 2.1. Similarly, the second buckling mode is presented in Figure 5.4.

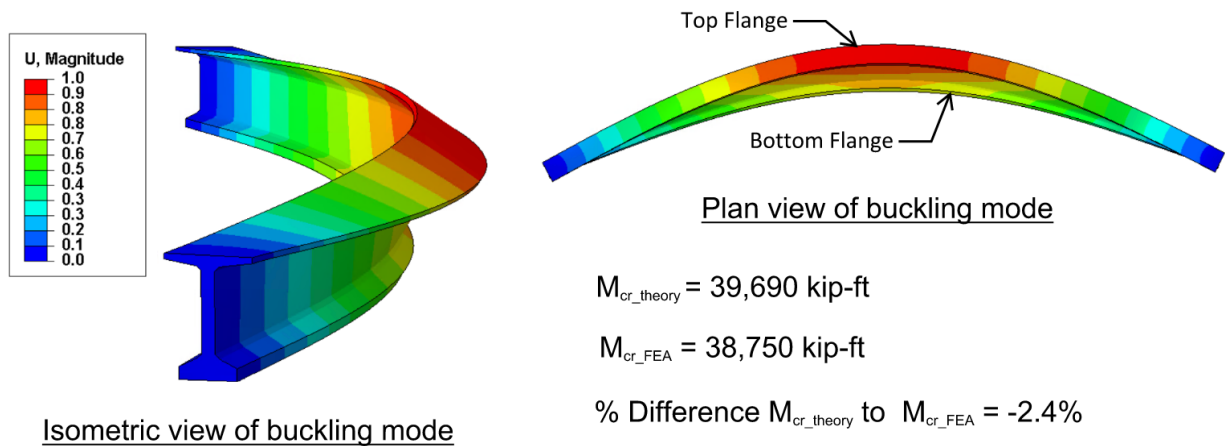


Figure 5.3. Buckling mode 1 of a 175 ft long, simply supported STx7 girder under uniform moment

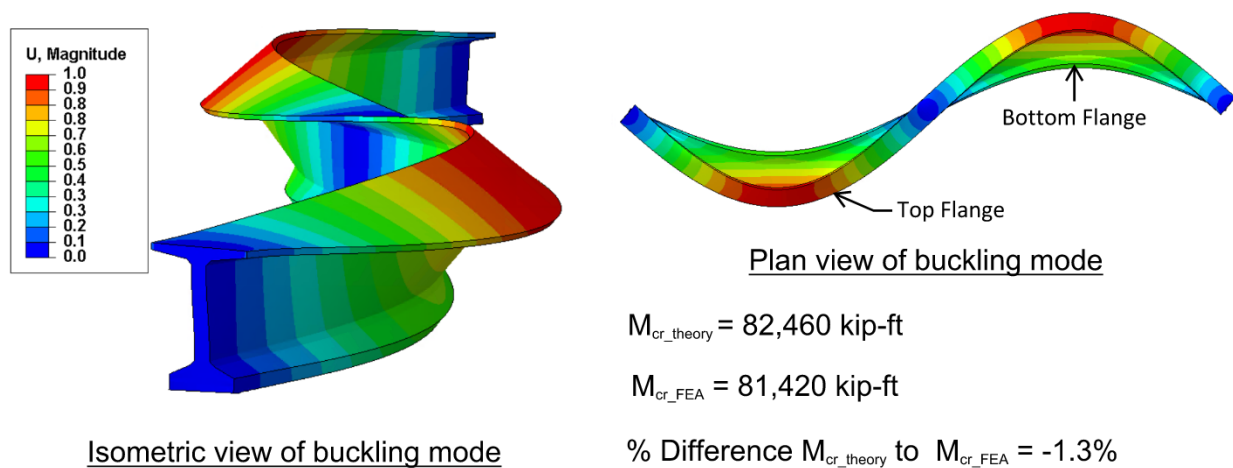


Figure 5.4. Buckling mode 2 of a 175 ft long, simply supported STx7 girder under uniform moment

As presented in Figure 5.3 and Figure 5.4, good estimates of the buckling capacity were achieved with the closed-form solutions. These figures also demonstrate an increase in buckling capacity between mode one and mode two of 110%. While higher modes provide an indication of the buckling behavior, in practice the higher modes can only be reached if braces are provided to control twist of the girder at intermediate locations. The most common form of bracing in bridge girders are torsional braces as outlined above. Although detailed parametric FEA studies were conducted on typical prestressed concrete girders used in Texas, preliminary assessments were conducted to evaluate the basic stiffness parameters outlined above.

5.2. Evaluating and Isolating Brace Stiffness Components

As noted earlier, extensive studies have been conducted on stability bracing in steel bridge systems. These past studies have provided solutions for the various stiffness components (Wang 2013; Battistini et al. 2016; Reichenbach et al. 2021 and Park et al. 2023). The studies included full-scale experimental tests on single angles and WT sections with gusset plates, as well as on full cross-frames systems. Solutions were provided to account for the impact of eccentric connections on the cross-frame stiffness, that have now been included in the AASHTO BDS (2024). The primary objective of cross-frames is to provide a restoring moment in cases where girders tend to buckle or resist an applied torsional load. The moment in the cross-frame can be idealized as a force couple at the location where the cross-frames are connected to the girders. The force couple magnitude is $F \cdot h_b$, where F is the force and h_b is the height of the cross-frame. The amount of rotation, θ , of the cross-frame can then be calculated and measured. If a beam or diaphragm-type brace is used and is near full girder height, the beam restrains the twist and deforms in double curvature. Both types of braces are schematically shown in Figure 5.5.

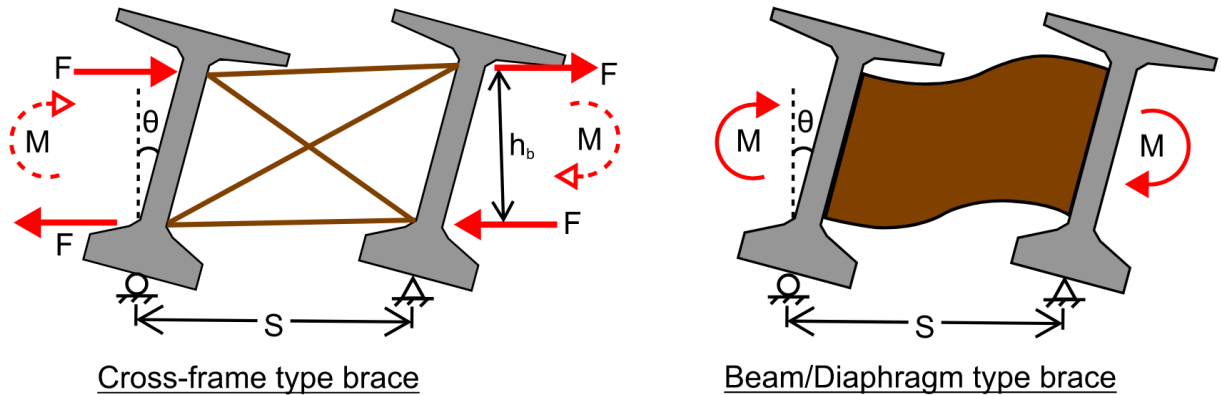


Figure 5.5. Rotational stiffness for cross-frames and beam/diaphragm type braces

Given the quantity of the force and rotation, the rotational stiffness can be calculated through Equation (5.2). Battistini presented the analytical stiffness equations for concentric cross-frames of type “X,” “K,” and “Z” as outlined in previous studies (Yura 2001; Helwig and Yura, 2012). Most cross-frames and diaphragms in steel bridge applications make use of single angles or WT sections that usually have eccentric connections. The experiments documented in Wang 2013 and Battistini et al. (2016), demonstrated that connection eccentricity can significantly reduce the stiffness of the cross-frames relative to the stiffness equations that are based upon a truss analogy. Reichenbach et al. 2021 and Park et al. 2023 extended this work to a wider range of bridges and also included the effects of WT sections. For the purposes of this study, analytical stiffness expressions were used, and the FE models used concentric detailing to remove variables that were not of interest. The “Z” type cross-frame was used in the FE models due to its simplicity and ability to vary the size of the members to develop moment stiffness curves. The brace rotational stiffness formula for a “Z” cross-frame is given in Equation (5.3). Similarly, the expression for diaphragm braces is given in Equation (5.4).

$$\beta_{br} = \frac{M}{\theta} = \frac{Fh_b}{\theta} \quad (5.2)$$

$$\beta_{br_z} = \frac{\beta_{br}}{\frac{2L_c^3}{A_c} + \frac{S^3}{A_h}} \quad (5.3)$$

Where: L_c = Length of diagonal member
 S = Girder spacing
 A_c = Area of diagonal member
 A_h = Area of horizontal members

$$\beta_{br_dia} = \frac{6EI}{S^3} \quad (5.4)$$

Spliced-girder bridges have been built in Texas using both types of braces. The girder heights require either cross-frames or cast-in-place concrete diaphragms. No instances have been found of any steel bent-plate diaphragms that have been utilized for spliced girder systems in the state of Texas. In cases with cross-frame applications for PS girders, formed holes are typically placed in the concrete girders to allow bolting of steel cross-frames and are also used in concrete diaphragms for the reinforcing steel to be continuous. Both bracing types are depicted in Figure 5.6.

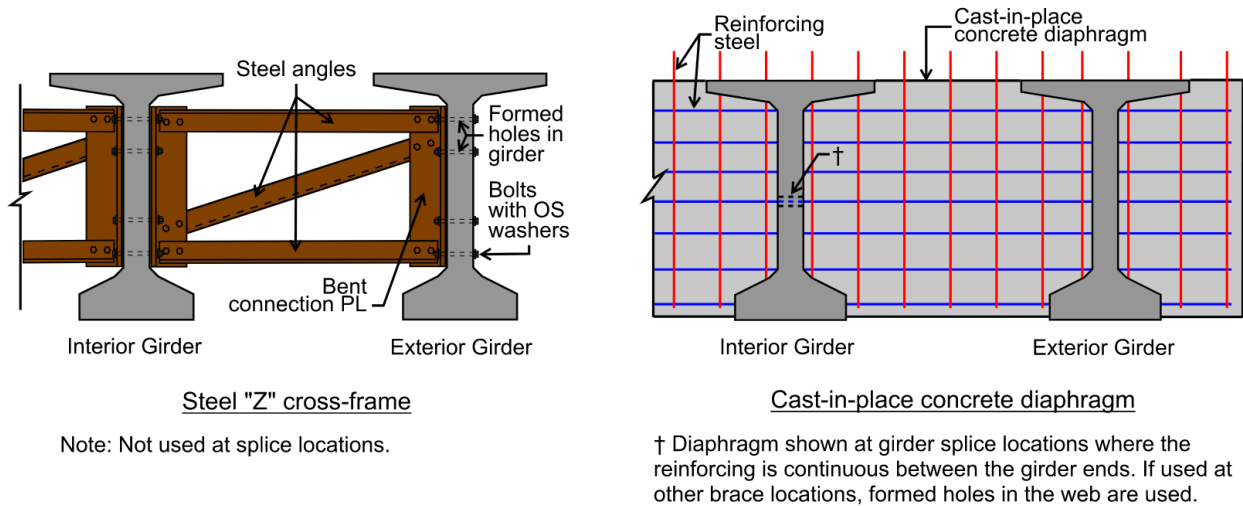


Figure 5.6. Steel and concrete brace types used in Texas

As part of the FEA studies, the methods of isolating the brace stiffness and in-plane stiffness that was utilized in Bjelland et al. (2025) and Fish et al. (2025) was employed. Rigid, vertical boundary conditions were used in both the testing and the development of the analytical equations. These

are necessary to react against the vertical shear that is developed by the brace moment. The effect of the in-plane stiffness of the girder (β_{br}) was first recognized in Helwig et al. (1993) with a simple design solution that was further modified in Yura (2001), which is the current solution utilized in the AASHTO BDS (2024). Significant improvements in the behavior are presented in Fish et al. (2025) with the development of an improved design expression. The in-plane stiffness effects are caused by shears that develop at the ends of torsional braces and result in upward and downward shears on adjacent girders that impact the stiffness behavior of the bracing system. This concept and the resulting rigid body rotation are depicted in Figure 5.7. The process outlined in Bjelland et al. (2025) and Fish et al. (2025) recognized that the vertical shears and subsequent deflections in the girders decreases with larger girder spacing; it is convenient to isolate β_{br} in the FE modeling by having a very large girder spacing such that the in-plane girder deflection is negligible. Figure 5.8 depicts a very large girder spacing and that $\beta_{T,ideal}$ is now equal to β_{br} .

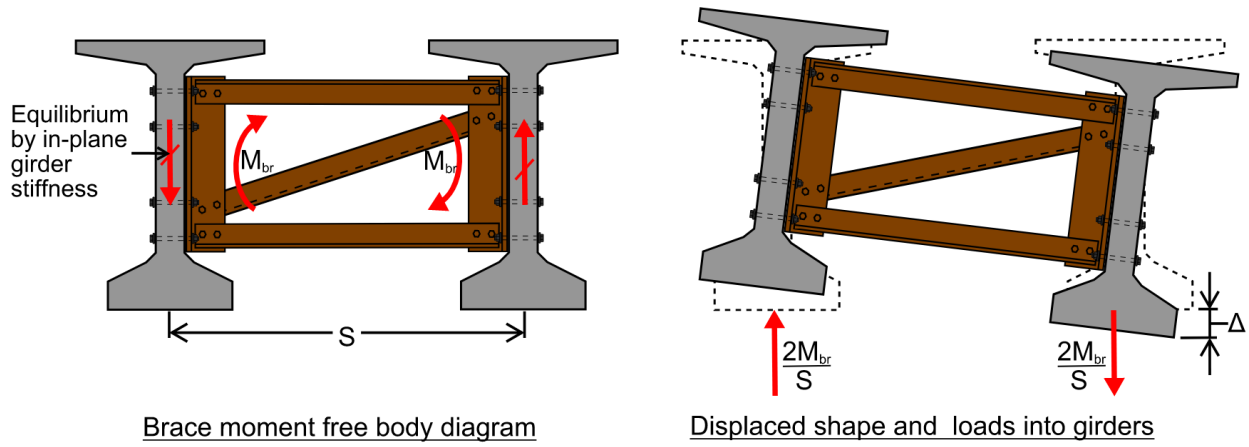


Figure 5.7. In-plane girder stiffness effects and deformations

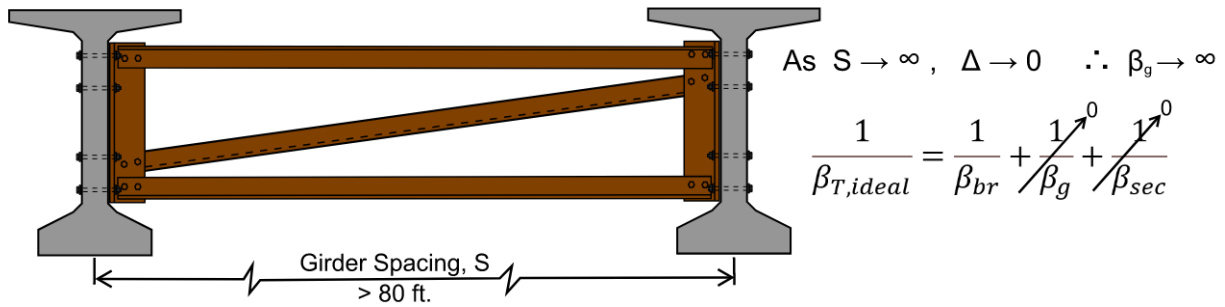


Figure 5.8. Isolating β_{br} by minimizing the effects of in-plane girder stiffness

The ideal torsional brace stiffness, $\beta_{T,ideal}$ represents the minimum stiffness required so that the girders buckle between the brace points. The moment that causes buckling between brace points is referred to as M_o . An iterative approach was used in the FE solution by varying the areas of the Z-frame truss elements, therefore varying β_{br} . The results are graphically presented by plotting the ratio of M to M_o against the ratio of β_{br} to $\beta_{T,ideal}$. These plots are referred to as a moment-stiffness curves. AASHTO BDS (2024) §6.7.4.2 specifies the required torsional stiffness of the

brace systems for steel I-girders, $\beta_{T,req}$. This specification is listed in Equation 5.5. for near full depth cross-frames. Equation 5.5 is twice the theoretical, ideal stiffness, $\beta_{T,ideal}$. The concrete I-girders of this study generally fit this criterion, though it is not currently applied in AASHTO. Moment-stiffness curves for STx7 and STx9 are shown in Figure 5.9 and Figure 5.10. The corresponding value of the AASHTO equation is noted in both curves.

$$\beta_{T,req} = \frac{2.4L}{\phi_{sb}nEI_{yeff}} \left(\frac{M_u}{C_b} \right)^2 \quad \text{AASHTO Eqn. 6.7.4.2.2-2} \quad (5.5)$$

- Where:
- L = Length of span under consideration
 - M_u = Factored major-axis bending moment (taken as M_o in this example)
 - ϕ_{sb} = Stability bracing resistance factor, 0.8
 - n = number of braced points within the span
 - E = modulus of elasticity
 - I_{yeff} = effective out-of-plane moment of inertia
 - C_b = moment gradient modifier (taken as 1.0 for uniform moment)

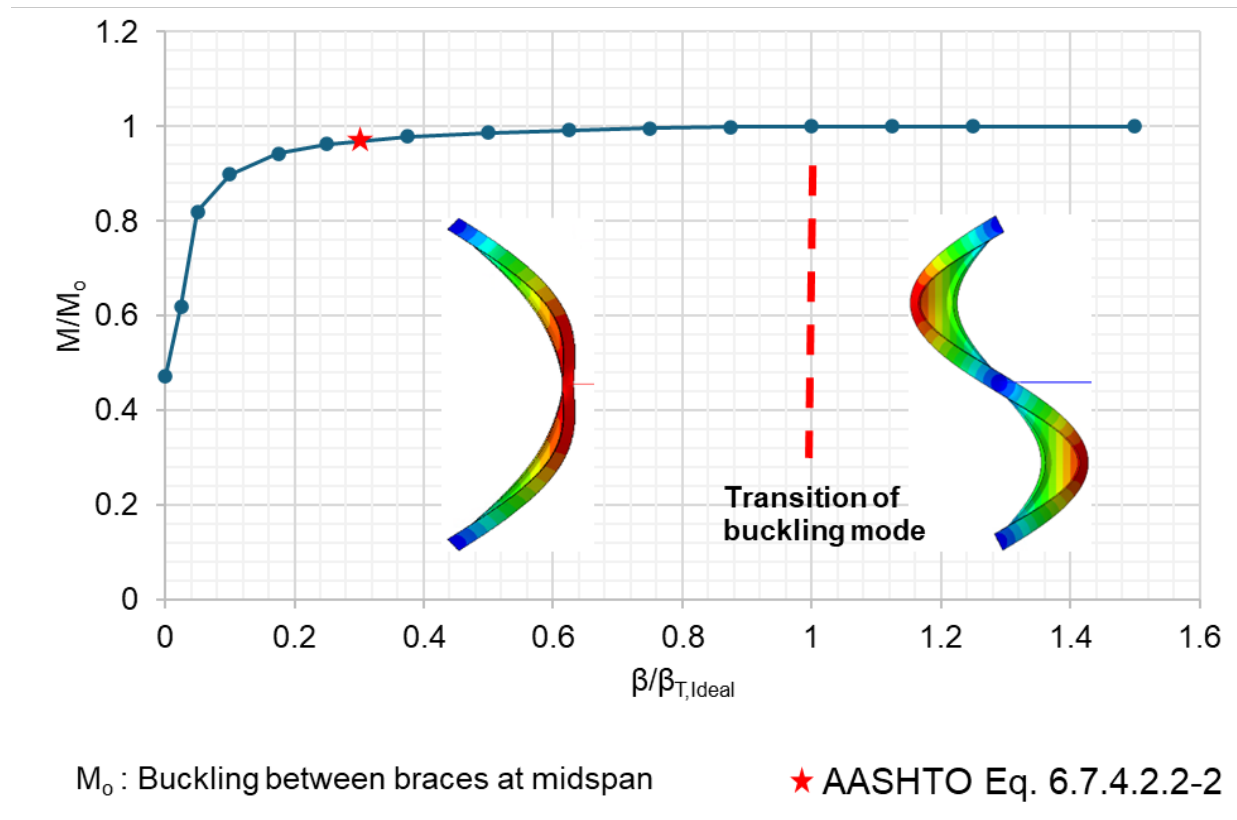


Figure 5.9. Moment-stiffness curve for a 175 ft long STx7 girder

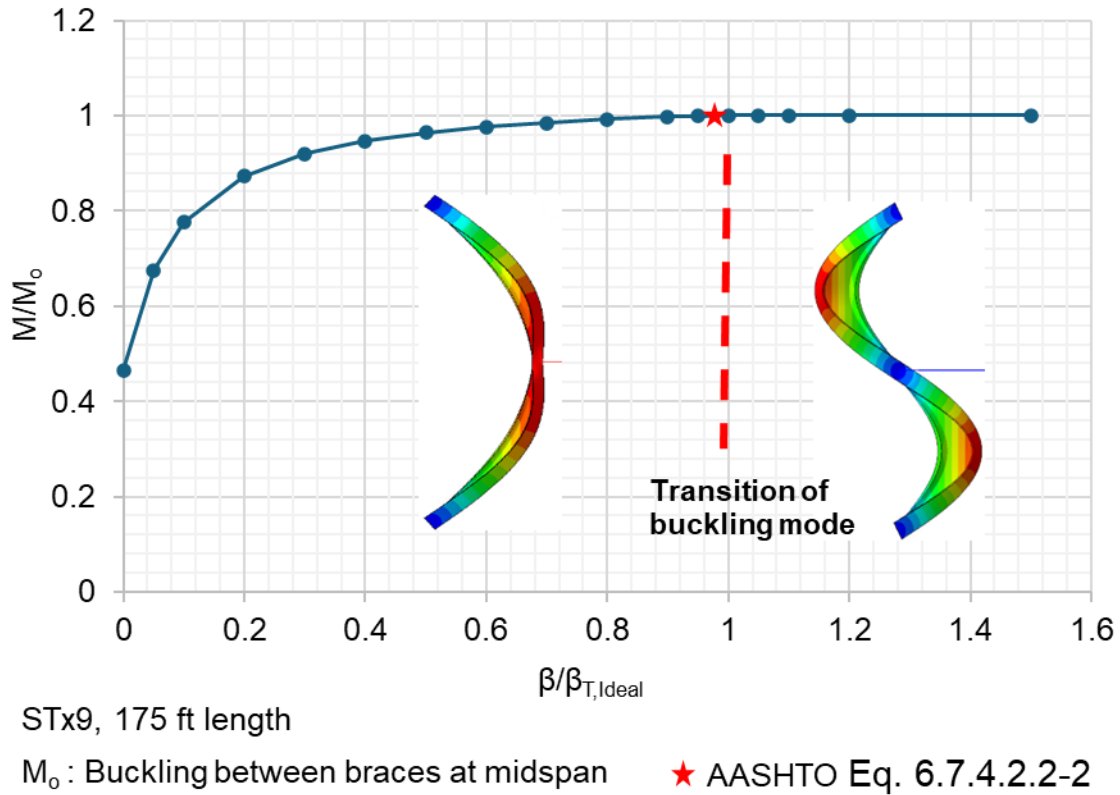


Figure 5.10. Moment-stiffness curve for a 175 ft long STx9 girder

Asymptotic behavior is evident in Figure 5.9 and Figure 5.10. This illustrates an increase in buckling capacity with a brace stiffness much lower than the ideal stiffness. It is evident that adding brace stiffness, even if it is less than the ideal stiffness, increases the buckling capacity. AASHTO Eqn. 6.7.4.2.2-2 yielded a brace stiffness less than the ideal stiffness, but still increased the buckling capacity significantly.

With the required system stiffness known, the combined effects of β_{br} and β_g can be evaluated in the reciprocal spring equation presented in Equation 5.1. Five differing combinations of β_{br} and β_g were analyzed for both STx7 and STx9 girder systems. This was accomplished by analyzing a two-girder system of 175 ft girder lengths. The brace stiffness was varied by changing the area of a Z cross-frame, and the in-plane girder stiffness was varied by changing the girder spacing. These configurations are not meant to be practical bridge geometries, rather, they provide insight into the interaction between β_{br} , β_g and the overall system stiffness. Five configurations differing β_{br} and β_g components are presented in Figure 5.11. These combinations are plotted along the reciprocal curve in Figure 5.12.

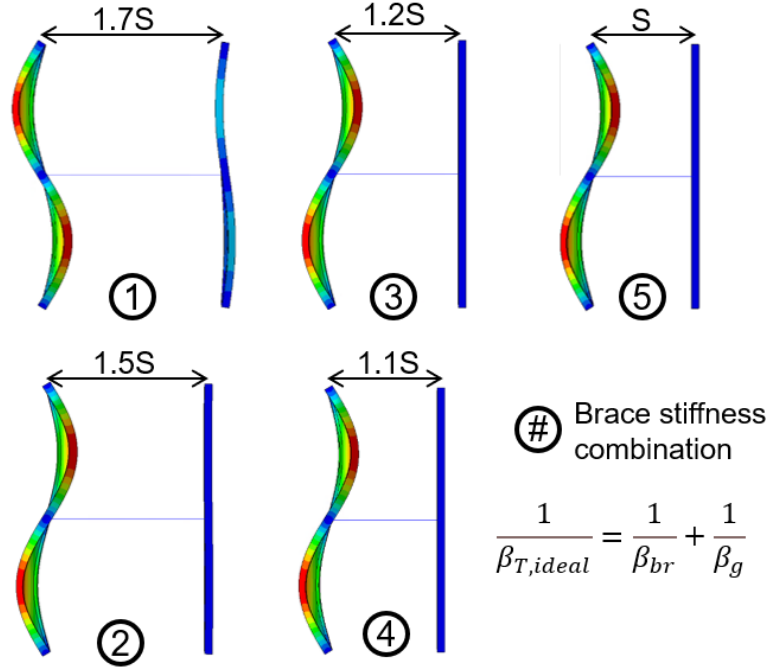


Figure 5.11. Plan view of STx9 stiffness combinations varying β_{br} and β_g

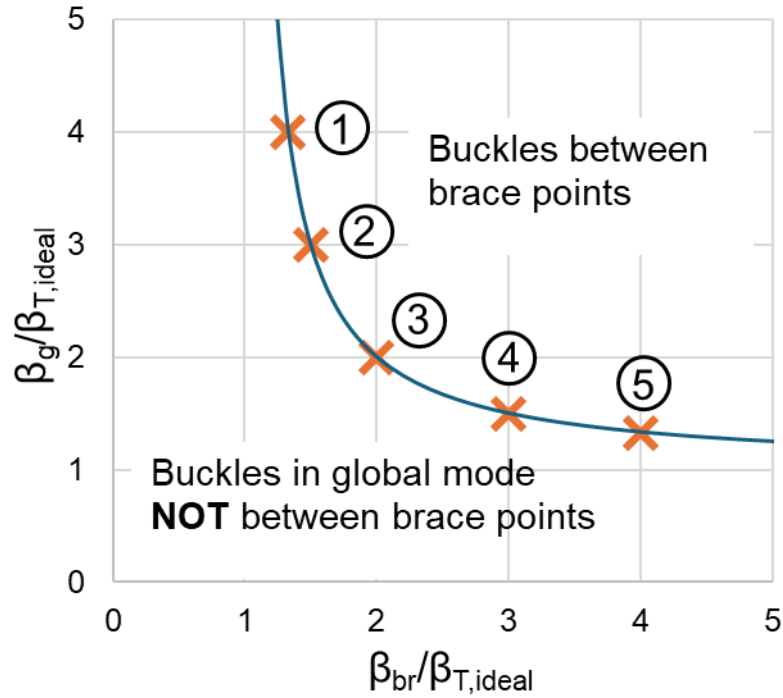


Figure 5.12. Plot of combinations of β_{br} and β_g

All five combinations of β_{br} and β_g for both STx7 and STx9 cases buckled between the brace points when the total system stiffness was greater than $\beta_{T,ideal}$. Demonstrating that the

methodology developed for steel I-girders can be applied to spliced-concrete I-girders. Further sections in this report will assess the potential need of intermediate braces on these systems.

5.3. Evaluation of Stability Behavior during Erection

The erection scenarios for spliced-girder bridges are highly variable and often dictated by the means and methods chosen by the contractor and erection engineer. While the splices and diaphragms are designed by the engineer of record, the girders are often erected onto either falsework or make use of strong backs for temporary support. The approach in the analysis has assumed that the erector/contractor will restrain twist at the ends of a given segment during the erection scenario. Once fully erected with the splices completed (continuous girder), the critical loading will generally be during the deck placement. Although the loads are substantially less at the erection stage, stability is still a concern since the girder continuity has not generally been established. As a result, the restraint to warping is substantially less.

The objective of this section is to evaluate the stability of the single girder systems throughout various stages of construction. The analyses provided in this section are based on the presumption that the temporary supports possess adequate strength and stiffness to brace the girders at the specific locations. Additionally, these analyses do not include any potential locked-in force effects from the erection procedures, nor do they include potential secondary post-tensioning effects. These force effects are very dependent on the means and methods of construction.

The analyses throughout the parametric studies included both linear-elastic properties as well as cracked section properties. The elastic section properties provide a good indication of the upper limit on the stability behavior. Although care is often taken to promote uncracked conditions throughout fabrication, shipping, and certain stages of construction, in all likelihood there is some cracking that occurs. However, as discussed later in the report – the cracked nature of a prestressed girder is often “invisible” since the cracks close from the prestressing. However, the impact on the stiffness of the girder from a stability perspective can be significant.

A measure of the girder stability was assessed using an eigenvalue buckling analysis (Subspace Iteration Method) to establish the critical buckling capacity. The self-weight of the girders were considered with boundary conditions believed to be representative of the field conditions.

In an eigenvalue buckling analysis, the girders are subjected to a “reference” load that is representative of the load that is applied to the structure. The critical load that leads to an instability on the structure is calculated as the product of the magnitude of the reference load and the eigenvalue. For example, if a girder is subjected to self-weight and the eigenvalue from the analysis is 1.0 – the analysis indicates that the critical moment of the girder is essentially equal to the maximum moment caused by the self-weight reference load. Establishing a “safe” level of the eigenvalue in such a case of expected loads, such as self-weight, is somewhat subjective. Clearly an eigenvalue less than 1.0 is unsatisfactory since this indicates that the girder cannot support its self-weight. However, the eigenvalue analysis is relatively insensitive to factors such as imperfections that generally have an adverse impact on the girder stability. Therefore, in most cases, eigenvalues significantly higher than 1.0 for self-weight conditions are preferred.

Because it is not readily apparent which load case will control, staged buckling analyses were utilized with the service loads preloading the structure in one stage and then the eigenvalue buckling load being evaluated in a later step. There is also a “Base” load case considered, where all of the service loads are placed on the structures, and the eigenvalue analysis is applied to all of the loads on the structures until buckling occurs. The values for the parameters considered for the analyses are given in Table 5-4. An overview of the cases to be analyzed for stability during erection is shown in Figure 5.13. As noted in the figure, it is assumed that twist is restrained at all points of vertical support, which includes piers, abutments, and strong backs.

Table 5-4 Parameters for Evaluating Stability during Erection

<i>Parameter</i>	<i>Values</i>
Girder Types	STx7, STx9, U96-9
Span-to-Depth Ratio (L/D)	25, 30 (constant-depth bridges) 30, 35 (variable-depth I-girder bridges)
Haunched girder depth	$1.6 \times$ Depth of prismatic girder
Span Arrangement	$0.7L + L + 0.7L$
Splice Locations	$0.2L$ from interior supports
Concrete Strength	8,000 psi
Concrete Modulus of Elasticity	5,260,000 psi
Concrete Girder Unit Weight	150 lbs./ft ³

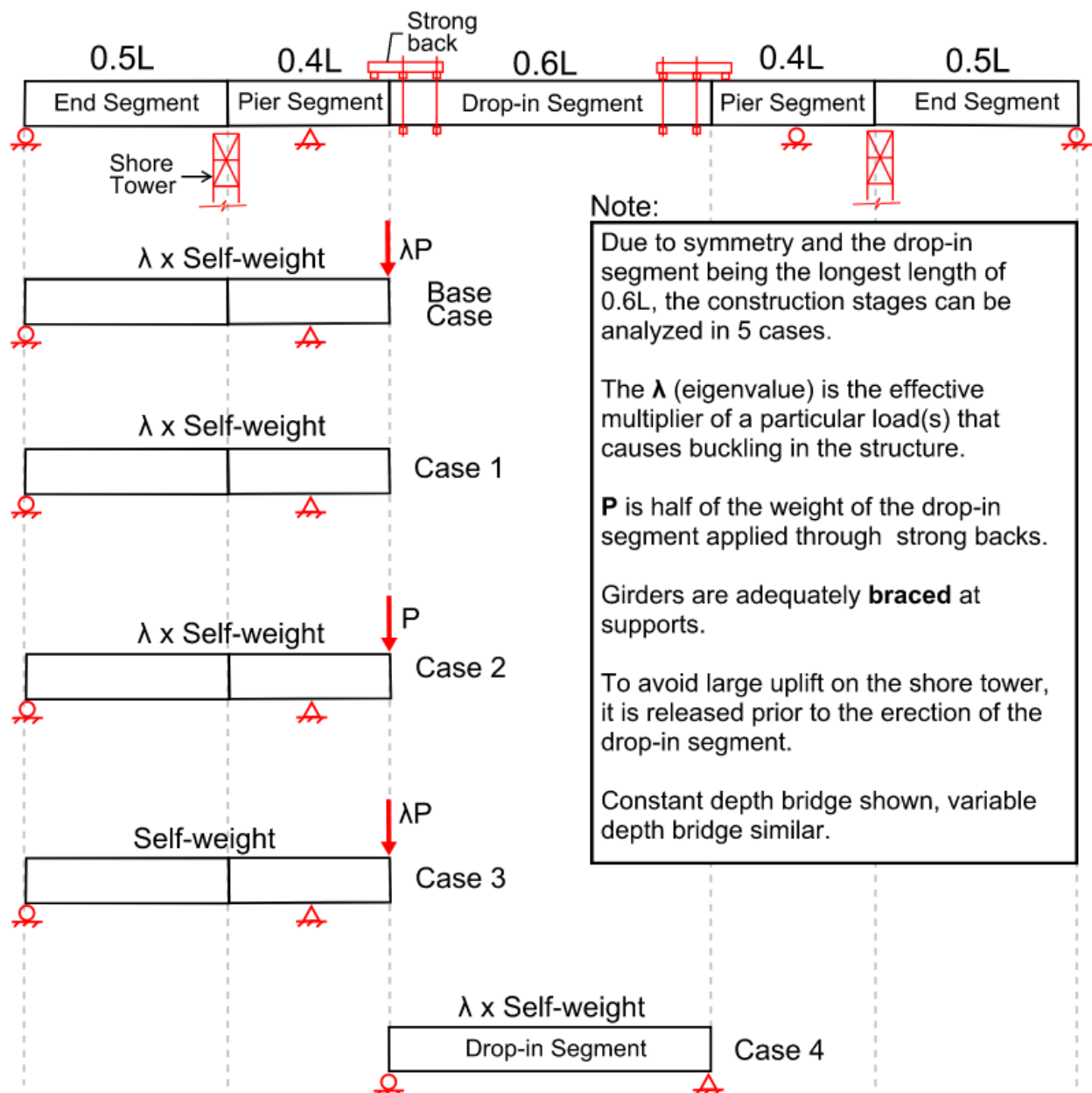


Figure 5.13 Stability cases analyzed during erection

There are other methods of temporarily supporting the drop-in segment prior to splicing the segments e.g., embed plates or corbel and ledge connections. The most stable option is generally providing temporary support using a shore tower at the main span splice. However, site constraints often prevent shore towers due to right-of-way issues below the splice location. A strong back configuration is a commonly used method of temporary support since right-of-way is not an issue and was used for much of the analysis documented in the erection analyses. A summary of the eigenvalue buckling analysis results is provided in Table 5-5. It should be noted that if spans (L) are smaller than the cases in Table 5-5, the eigenvalue will generally increase.

Table 5-5 Eigenvalue buckling analysis during erection

Girder Type	L/D	L (ft)	Load Case	Eigenvalue λ	Girder Type	L/D	L (ft)	Load Case	Eigenvalue λ
STx7	25	193	Base	40.4	Tx-U96-9	25	217	Base	25.6
			1	20.8				1	13.5
			2	21.4				2	14.0
			3	27.2				3	30.1
			4	26.3				4	15.6
STx7	30	231	Base	23.5	Tx-U96-9	30	261	Base	16.4
			1	11.9				1	8.6
			2	12.5				2	9.2
			3	16.1				3	20.6
			4	15.2				4	10.1
HSTx7	35	270	Base	15.0	Note: L/D is the length of the span divided by the depth of the structure considering an 8.5 in. slab.				
			1	7.5					
			2	8.1					
			3	11.2					
			4	9.2					
STx9	25	243	Base	18.5					
			1	9.5					
			2	10.2					
			3	13.0					
			4	12.3					
STx9	30	291	Base	10.8					
			1	5.5					
			2	6.1					
			3	7.9					
			4	7.0					
HSTx9	35	340	Base	7.0					
			1	3.5					
			2	4.1					
			3	5.7					
			4	4.4					

The eigenvalues were considerable for nearly all cases considered. The smallest values occurred for the HSTx9 section, which had a span-to-depth ratio (L/D) of 35 with a maximum span length (L) of 340 ft. The smallest eigenvalue for that system was the Case 1 loading condition, which had a value of 3.5. Load Case 1 occurs when the end segment and pier segment are spliced, and the shoring tower is removed. The self-weight of the girder is the only active load, and the eigenvalue represents the factor multiplied by the self-weight that causes the structure to buckle. Select buckling mode shapes and graphical results for the HSTx9, $L/D = 35$ are presented in Figure 5.14 through Figure 5.18. The lowest eigenvalue is generally the critical case. In some instances, the 2nd eigenvalue may be relatively close to the first and may be of interest if there is greater uncertainty in the load cases or boundary conditions associated with that mode.

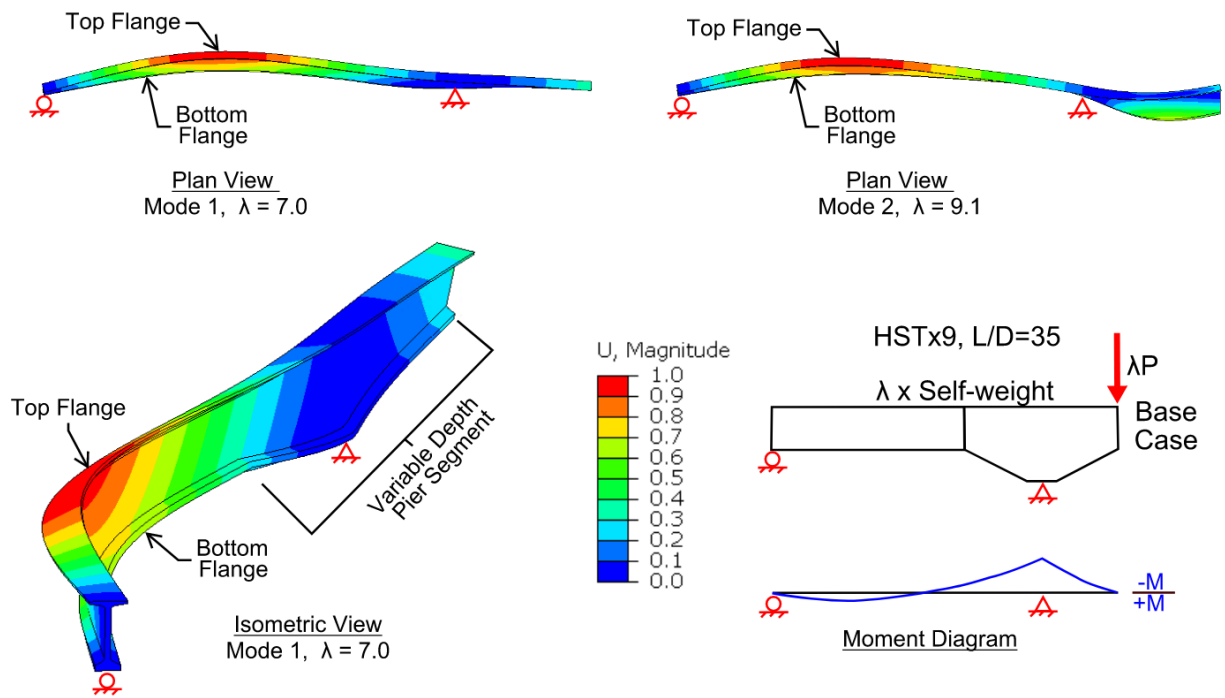


Figure 5.14. HSTx9 Base Load Case buckling analysis

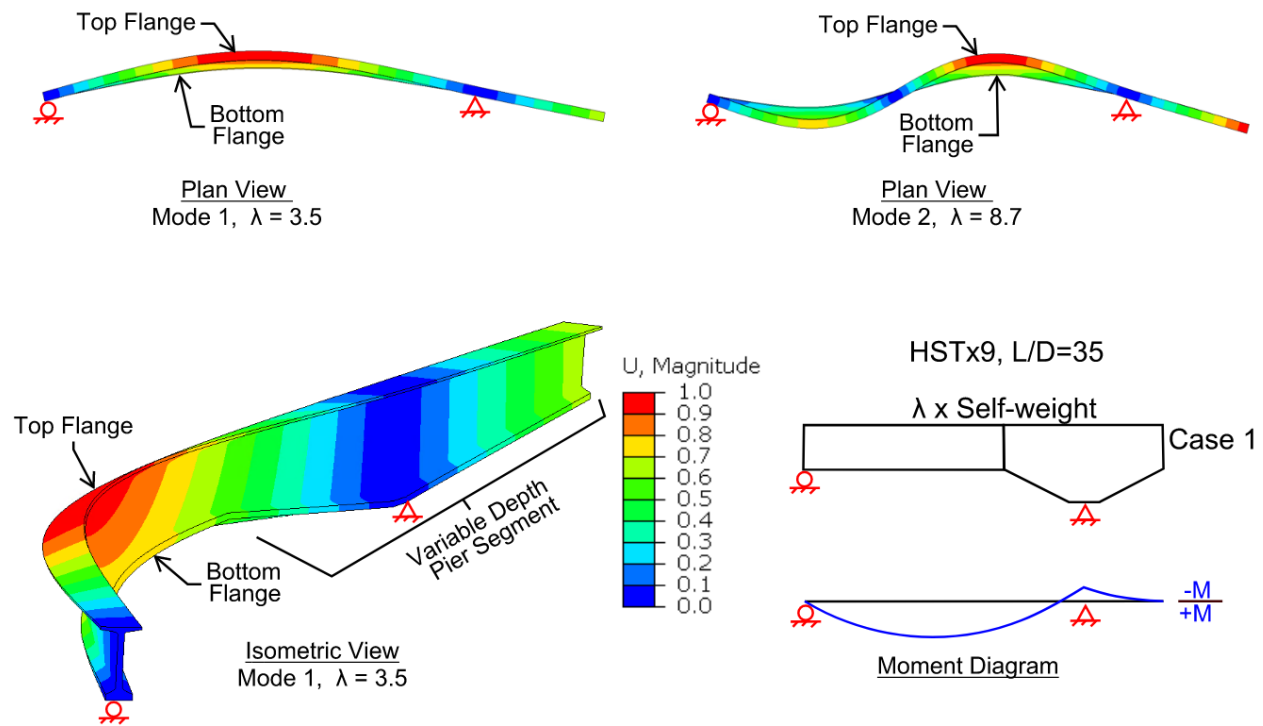


Figure 5.15. HSTx9 Load Case 1 buckling analysis

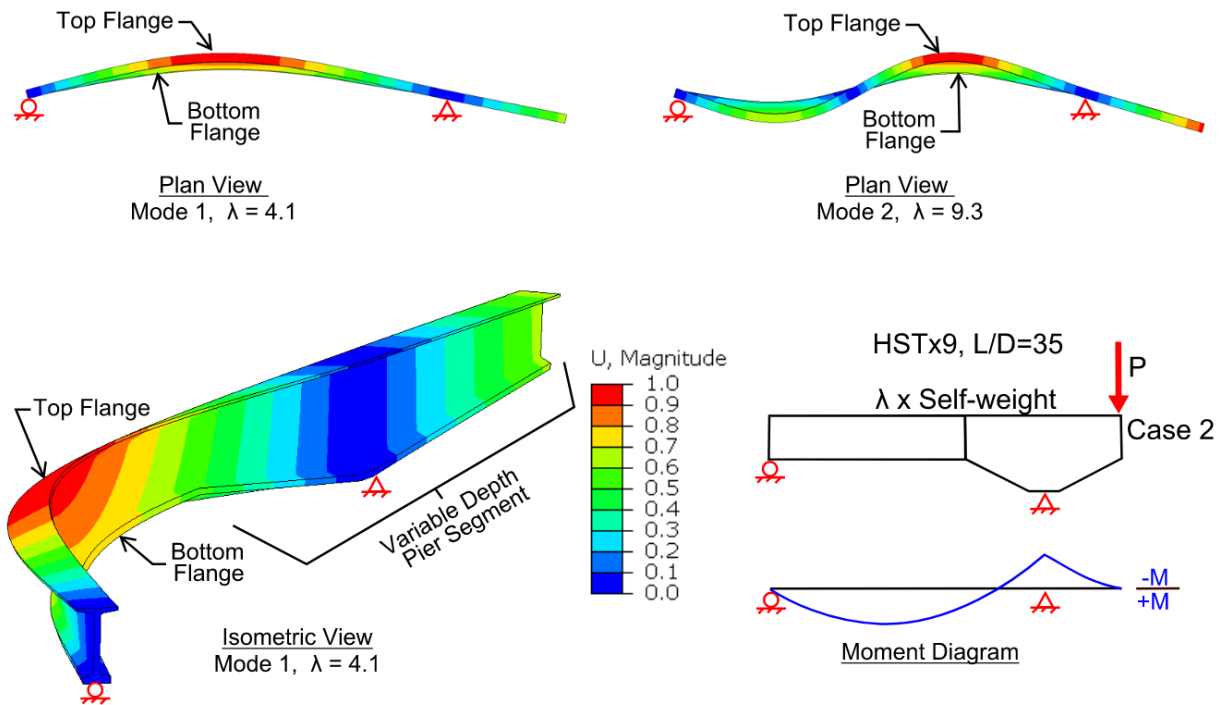


Figure 5.16. HSTx9 Load Case 2 buckling analysis

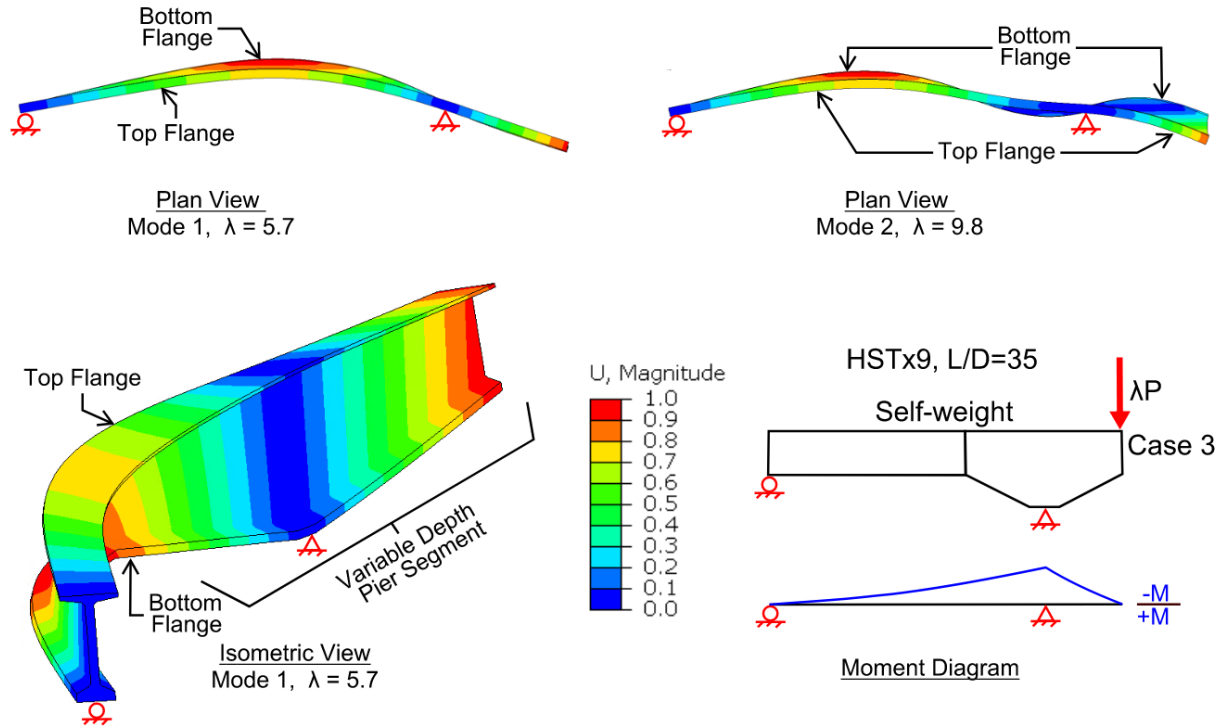


Figure 5.17. HSTx9 Load Case 3 buckling analysis

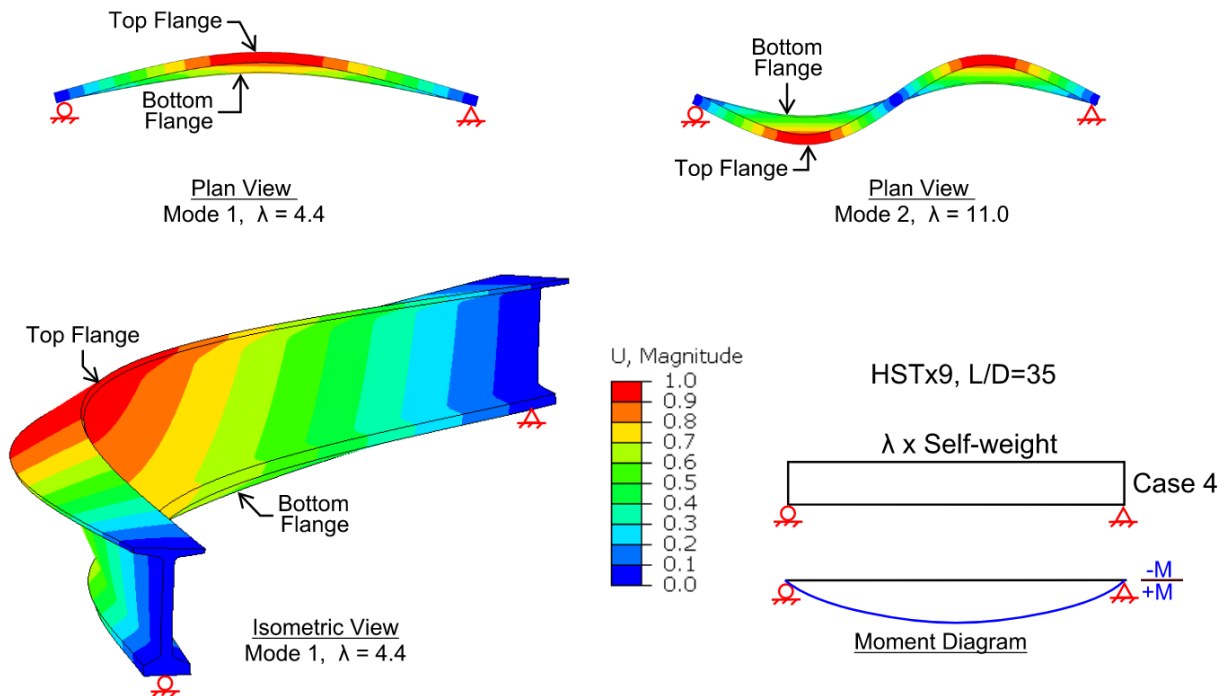


Figure 5.18. HSTx9 Load Case 4 buckling analysis

While the critical case of the HSTx9, $L/D = 35$ is shown above, the general trend can be observed in the data in Table 5-5. The moment diagrams are also very similar for all girder types and L/D ratios due to the selection of consistent span ratios of $0.7L$ - L - $0.7L$. The results may seem counterintuitive that the controlling load case is Load Case 1 for all configurations, but when comparing the moment diagrams it becomes clear that the cantilever over the pier reduces the positive moment in the end span. None of the configurations had a controlling buckling mode of negative moment in the cantilevered portion. However, in Load Case 3, where the drop-in load is the eigenvalue load, the end span buckles in an upward fashion, where the bottom flange becomes the compression flange. The second buckling mode of Load Case 3 presents the cantilever tip distorting laterally and torsionally. It is important to note that the buckling mode shapes (eigenvectors) only characterize the buckled shape and not the actual deflections experienced during buckling. It is common to normalize the buckled mode shapes to unity. Where 1.0 is the largest relative displacement and 0.0 represents no displacement. This is represented in the contour legend in the figures, where U is the displacement variable, and the absolute magnitude is shown based on the movement in all three coordinate directions.

The U-girders behave similarly to the I girders as presented in Table 5-5. However, one key distinction is observed in how the compression flanges behave. The cross-section is open, and there is no direct continuity between the flanges. More simply put, the U-girder cross-section does not behave as a rigid cross-section. Meaning the flanges displace differently relative to each other. This is presented in Figure 5.19.

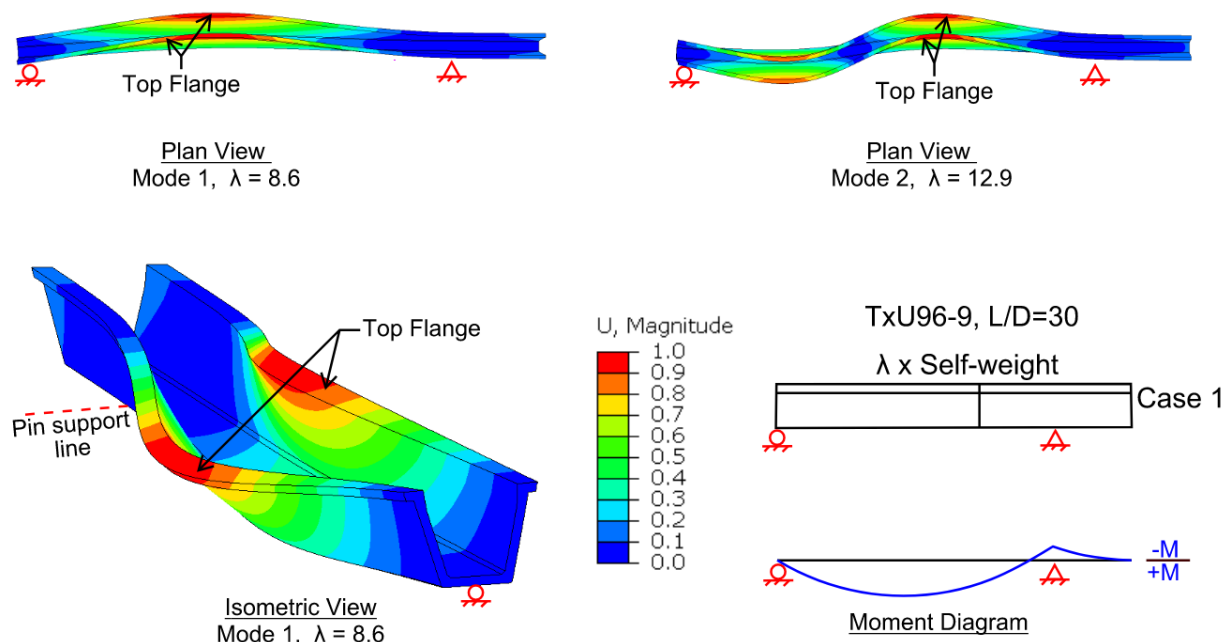


Figure 5.19. TxU96-9 Load Case 1 buckling analysis

5.3.1. Effects of Initial Imperfections and Cracked Sections During Erection

Eigenvalue buckling solutions provide an indication of the critical buckling loads on the system behavior. However, these solutions are relatively insensitive of initial imperfections that can lead to significant reductions in the stability of the system. As noted in the last subsection, the critical load case for the systems evaluated was Case 1 (self-weight of girder with no shoring tower) and this section analyzes that controlling case considering geometric imperfections. With different length ratios, it is possible that other load cases might become critical; however, Case 1 will nearly always be relatively close to another critical case with reasonable changes in the span ratios. Therefore, focusing on Case 1 provides good indicators of performance considering geometric and material nonlinearity. The following girder types in Table 5-6 were considered.

Table 5-6. Cases considering initial imperfections during erection

Girder Type	L/D	L (ft)
STx7*	30	231
HSTx7	35	270
STx9*	30	291
HSTx9	35	340
TxU96-9*	30	261

*Effects of cracked sections also considered

The imperfection magnitudes in these analyses followed *TxDOT Specification Item 424* and *PCI* sweep tolerances of an 1/8 inch per 10 ft of girder length. This specification equates to an imperfection of $L/960$. Two initial imperfection magnitudes (Δ_0) were examined: $L/500$ and $L/1000$, where L represents the respective span length. The reason for the larger $L/500$ imperfection is twofold: 1) the 2nd imperfection demonstrates the sensitivity of the structure to the magnitude of the imperfection; 2) owners may allow tolerances to be exceeded with the approval of the engineer of record. The imperfections in this section were applied in the FE model using a scaled eigenvector (critical buckled shape) of the controlling buckling mode by the imperfection magnitude. This method was used because the girders being erected are generally simply supported with a cantilever end, and in some cases, include haunched sections. Therefore, the distribution that causes the most destabilizing effect was not readily apparent. The previously presented buckling modes show the general shape, which are predominantly characterized by a twist of the longitudinal cross-section along the length of the girder. The imperfection analysis was carried out in Abaqus using the Riks' Arc-Length method for the nonlinear, geometric solver. The solver incrementally solves for increasing loads considering the geometric stiffness. Equilibrium is solved for in each step on the deformed geometry, and the deformed geometry and stiffness are updated in subsequent increments. To demonstrate the impact of material nonlinearity, analyses were conducted with both elastic materials and cracked-section properties. The results for the analyses with elastic materials are presented first followed by the results using cracked section properties.

Figure 5.20 through Figure 5.24 present the cases in Table 5-6. These load-displacement curves have between 50 and 250 data points showing the girder performance. Data point markers are not shown for clarity. The load-displacement figures normalize the bending moment in the vertical axis to that occurring under the self-weight only during erection, listed as M_{const} . For curves in the (a) sublabel, the horizontal axis plots the absolute lateral displacement (total sway) of the top flange during incremental loading – listed as Δ_{tf} . For curves in the (b) sublabel, the horizontal axis is the relative lateral deflection normalized with the initial imperfection value of $L/1000$. Two additional reference lines are included in the plots. The first is M_{crit} , which is the critical buckling moment from eigenvalue buckling analysis. The second is $1.4M_{const}$, this is used to help assess the impact of imperfections on the stability of the girders. The AASHTO BDS (2024) §3.4.2.1 prescribes a dead load component (DC) load factor of 1.4 for fully erected steel superstructures and is used for cases critical to structural stability. The amplifier of 1.4 is not due to uncertainty in the magnitude of the actual loads, but instead to account for uncertainties in how loads from wet concrete, finishing machines, etc. are applied to the girder system that might lead to an instability. Although the load case is prescribed for steel superstructures, applying the same load factor to the prestressed girders in this case is appropriate since instabilities are being evaluated. This helps account for construction tolerances and initial imperfections.

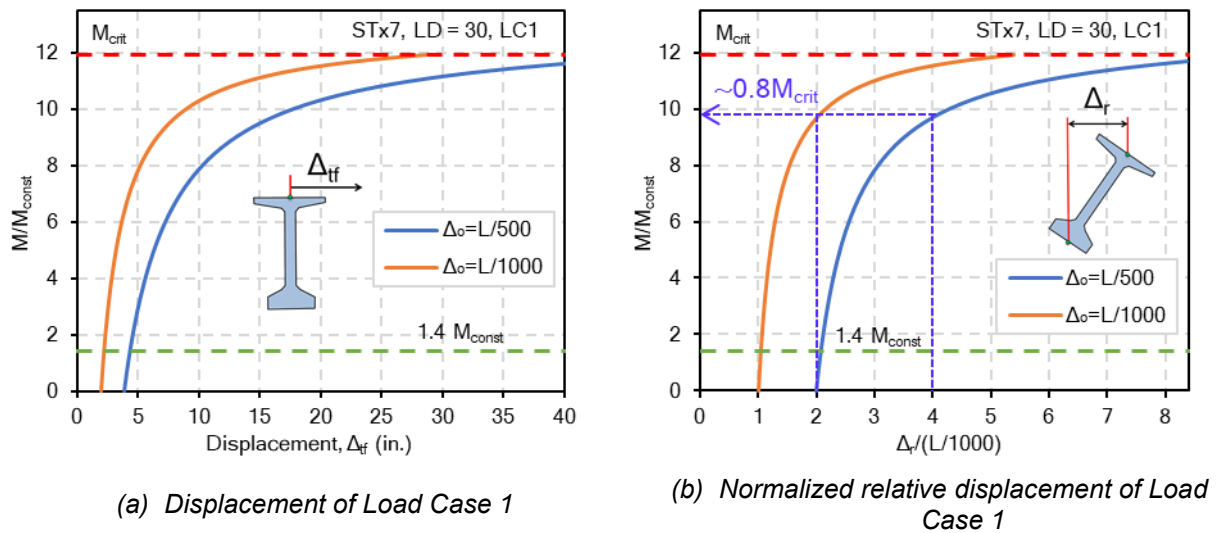


Figure 5.20. Load-displacement curves for STx7, LD=30 during erection

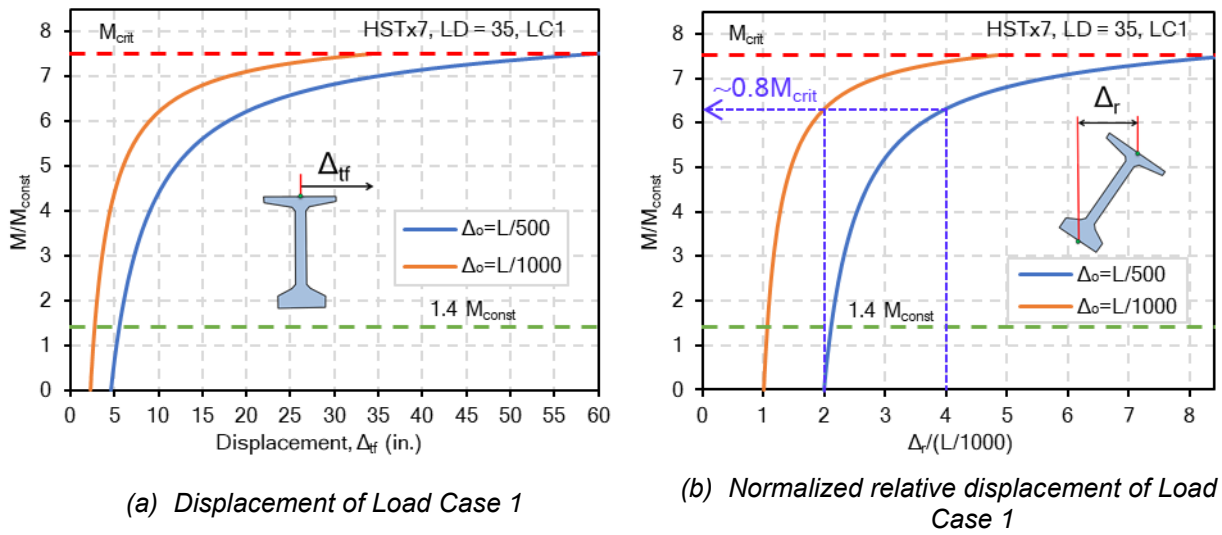


Figure 5.21. Load-displacement curves for HSTx7, LD=35 during erection

The STx7 and HSTx7 girders presented relatively stable behavior as witnessed in Figure 5.20 and Figure 5.21. The curves did not begin to show significant nonlinearity until load levels consistent with $>5 \times$ the expected construction moments. With M_{crit} values above 11 and 7, respectively. These cases also did not have appreciable lateral deflections. The behavior between the STx7, $L/D=30$, and the HSTx7, $L/D=35$, was quite similar, considering the change in span length and the HSTx7 having haunched sections. This is ideal for this stage in construction as it helps control the geometries of future construction stages.

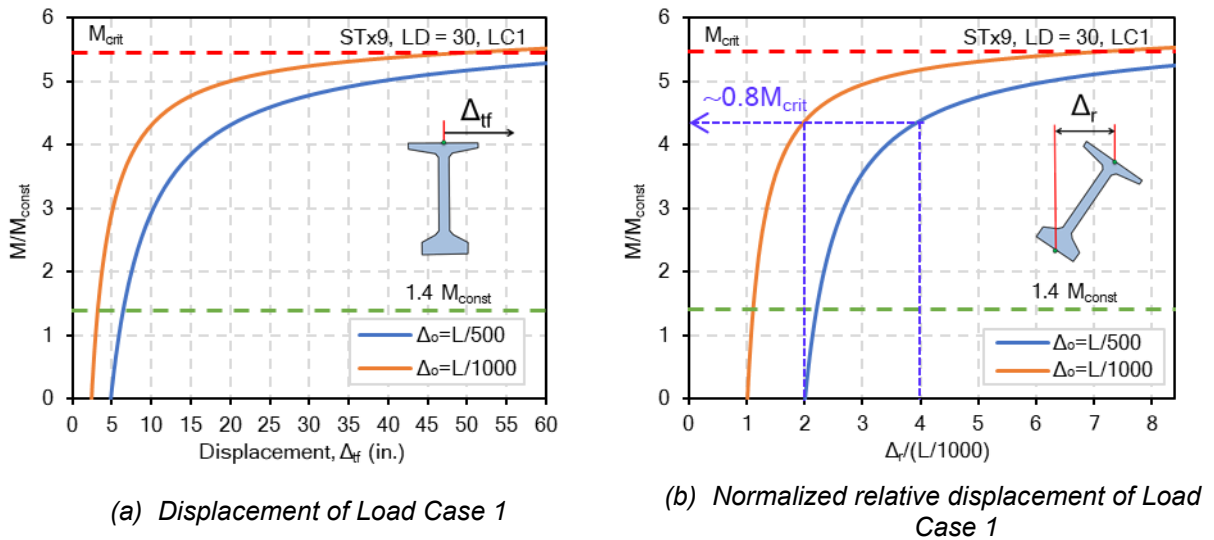


Figure 5.22. Load-displacement curves for STx9, LD=30 during erection

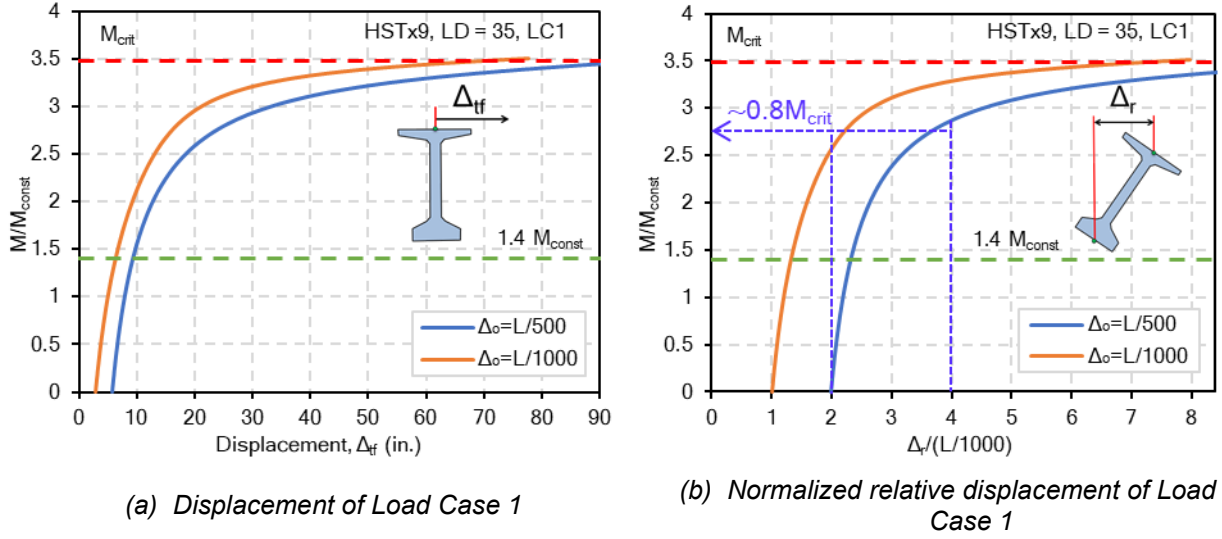


Figure 5.23. Load-displacement curves for HSTx9, LD=35 during erection

The cases for STx9 and HSTx9 also presented relatively stable behavior. However, the girders exhibited a lower M_{crit} ratio than that of the STx7 cases. This is due to the increase in span-to-depth ratio with the flange sizes being unchanged. The HSTx9 cases had a larger amount of lateral displacement but both STx9 and HSTx9 cases reached $\sim 0.8M_{crit}$ when the lateral deflection approached twice the initial imperfection. The lateral movements of the STx9 cases exhibit reasonable construction tolerances, especially considering that the girders in the system are unlikely to have the same location, direction, and magnitude of initial imperfections.

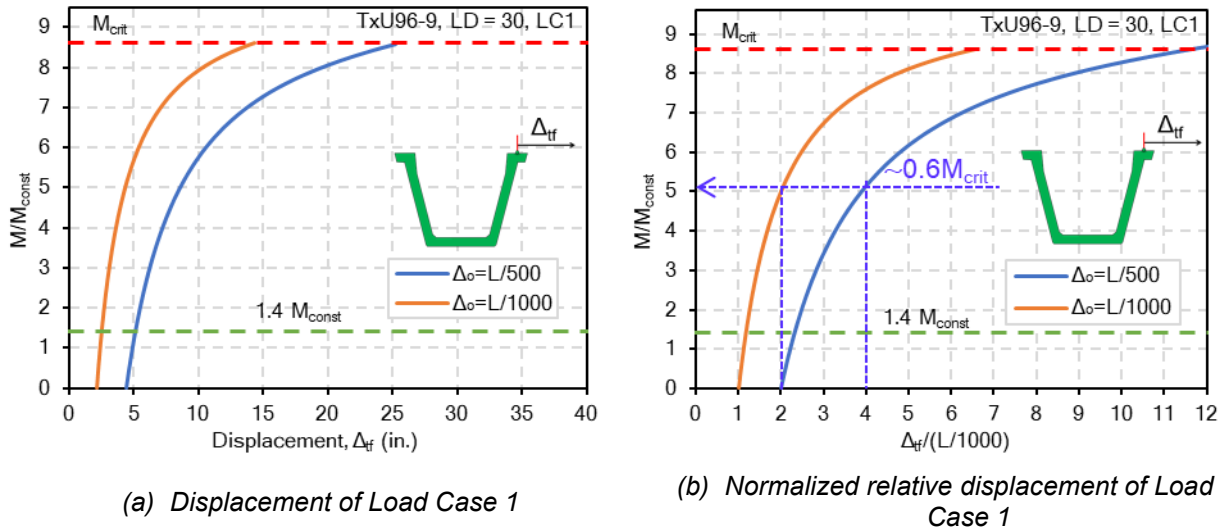


Figure 5.24. Load-displacement curves for TxU96-9, L/D=30 during erection

The U96 cases differed from the I-girder cases in that they approached twice the initial imperfection at a ratio of $\sim 0.6M_{crit}$. The U96 cases still reached M_{crit} for both imperfection cases and did not incur significant lateral displacements at the construction load.

Examining the load-displacement curves also demonstrated the effects of the girder slenderness. This is well seen in the difference between the HSTx7 and HSTx9 curves. With the same L/D ratio, there is a significant difference in the capacities and the displacement behavior. The HSTx9 is more slender at the same L/D ratio because the top and bottom flanges are unchanged, and only the height of the web is increased. Meaning there is only a small difference in the lateral bending stiffness between the two sections, and the span length is increased by ~26%. Overall, the erected girders, braced laterally at supports only, demonstrated stable behavior and reasonable lateral displacements when analyzed for the effects of nonlinear geometry.

The analyses demonstrating the combined effects of nonlinear geometry and reduced stiffness from cracked concrete sections are presented below. The FEA methodology only changes in one parameter to accomplish this. Namely, select regions of the concrete girder are assigned a negligible modulus of elasticity that emulates the reduction in stiffness witnessed in the laboratory experiments and classical moment-curvature analysis of prestressed concrete sections. The goal of the analyses was to understand how the girder system stability is impacted by the inclusion of reduced stiffness regions due to cracking. The locations of and specifics of the cracked regions are detailed in Figure 5.25. At first glance, the level of cracking may appear extreme; however, accounting for the impact of cracked sections on the behavior is of paramount importance when considering the issue with bridge safety due to the ramifications of a potential stability problem. The AASHTO BDS (2024) and design engineers in general limit loads to below cracking levels during many stages of fabrication, transportation, and construction. However, assessing the actual cracking condition of the girders in the field can be very difficult since the cracks are often “invisible” to the naked eye which is shown later in Figure 5.50. Significant cracks that occurred in the laboratory tests and were clearly visible under high load levels essentially disappeared when loads were removed. Although the researchers chose relatively conservative modelling conditions with significant crack distributions in critical regions of the span – these decisions were made to provide confidence in the stability of the girders systems throughout the critical stages of construction.

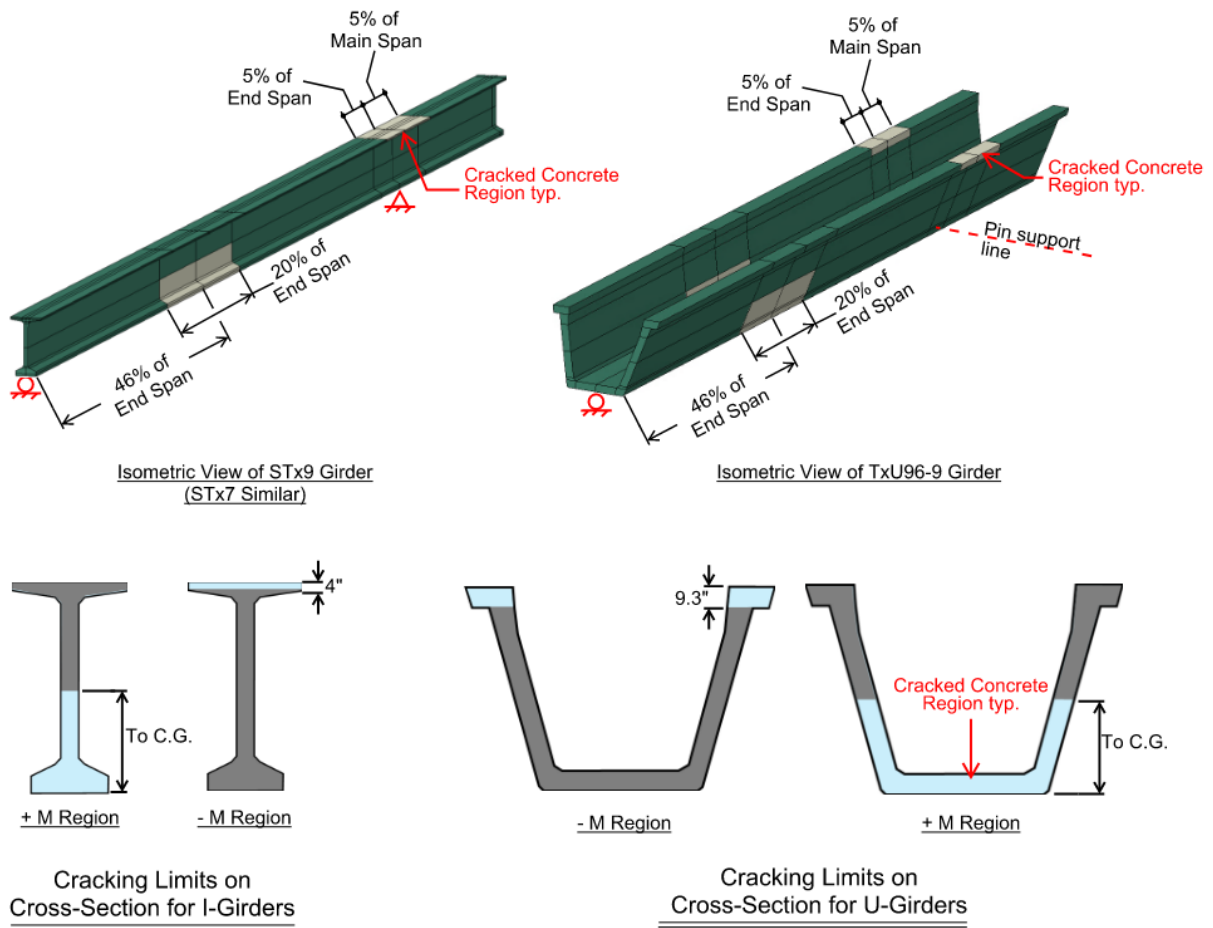


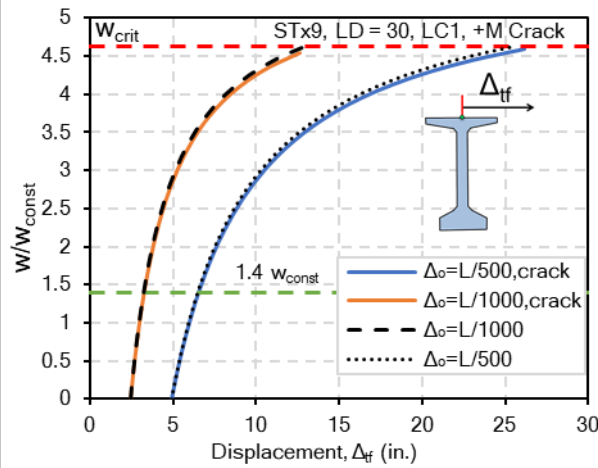
Figure 5.25. Details of cracked section regions for analysis during erection

The crack locations were chosen to be the maximum positive and negative moment areas and limits set by observations from the lab experiments and the moment gradient in those regions. Eigenvalue buckling analysis was performed prior to the nonlinear analysis, and results for the cracked sections are provided in Table 5-7. The L in the table is the length of the main span, not the end spans.

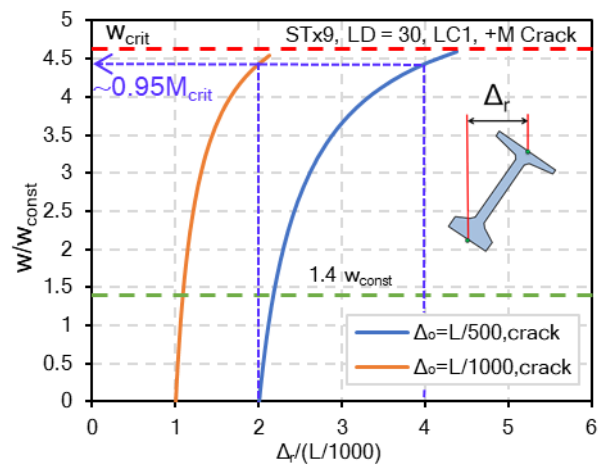
Table 5-7. Eigenvalue buckling analysis of cracked sections during erection

Girder Type	L/D	L (ft)	Load Case	Eigenvalue λ
STx7	30	231	1 (uncracked)	11.9
			1 with positive cracking	9.9
			1 with positive & negative cracking	9.8
STx9	30	291	1 (uncracked)	5.5
			1 with positive cracking	4.6
			1 with positive & negative cracking	4.6
TxU96-9	30	261	1 (uncracked)	8.6
			1 with positive cracking	5.3
			1 with positive & negative cracking	5.3

The eigenvalue buckling analysis with cracking considered decreased the critical buckling capacity by roughly 20% for the I-girders and by nearly 40% for the U-girders. The load-displacement results are shown in Figure 5.26 and Figure 5.27. The figures are scaled in terms of applied construction load, w/w_{const} , rather than moment. Where w is the self-weight of the girder. This is consistent with what is done when considering continuous girder cracking in later sections, where there is moment redistribution when cracking occurs in the system. Results from the STx7 cases are not shown because the sections were dominated by vertical bending and did not have appreciable later deflections. The cause being the uncracked section in the positive moment cracked region is essentially a *T-shaped section* with a web depth of 42 in. and a flange width of 59 in. Making the horizontal and vertical inertias very similar. Moreover, the erection condition is a simple span condition that is statically determinate (the effective vertical stiffness at midspan is much less than the continuous span case). Another outcome of the analysis is that there was a negligible impact on the load-displacement curves (tenths of inches), considering cracking in the positive region only and cracking in both positive and negative regions. Therefore, the positive and negative figures are not presented. This was consistent with the eigenvalue buckling analyses performed.

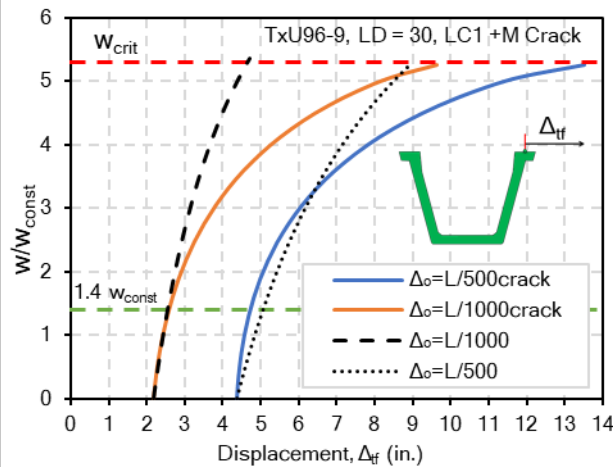


(a) Displacement of Load Case 1 with cracked and uncracked cases shown

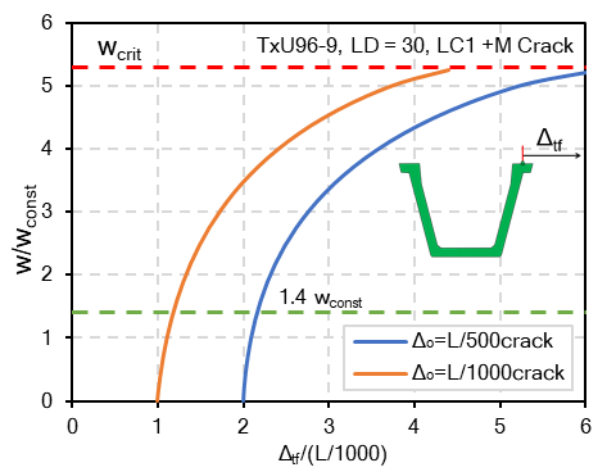


(b) Normalized relative displacement of Load Case 1

Figure 5.26. Load-displacement curves for STx9, $L/D=30$ during erection with +M cracking



(a) Displacement of Load Case 1 with cracked and uncracked cases shown



(b) Normalized relative displacement of Load Case 1

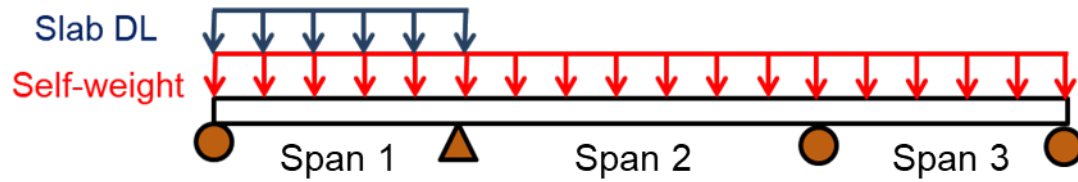
Figure 5.27. Load-displacement curves for TxU96-9, $L/D=30$ during erection with +M cracking

The results of the geometric nonlinear analyses considering cracked sections do approach/reach the eigenvalue buckling critical moment. The STx9 cracked case did not significantly alter the lateral displacement of the girder; only the buckling capacity was reduced. Conversely, the TxU96 cracked case produced ~ 4.5 in. more deflection at the cracked critical load level for both $L/1000$ and $L/500$. Regardless of what level of cracking may or may not be allowed during construction for long-term serviceability concerns, if structural cracking is present during the erection of spliced girder bridges, the nonlinear material response results in decreases in the stability of the system. Prudent caution and understanding of the root cause and severity of the cracking is important for these systems. Although the researchers believe the assumed level of cracking are conservative

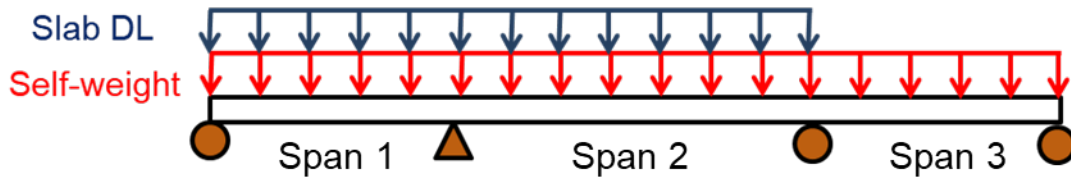
compared to what might be expected to occur in practice, the conservatism is believed to be warranted considering stability limit states. Section 5.4.3 of this report examines the large displacement behavior of the erected bridge under deck placement loads and includes a parametric study with a variety of cracked section scenarios.

5.4. Evaluation of Stability Behavior during Deck Placement

The primary objective of this section is to investigate the stability of bridge systems under construction loads during the concrete bridge deck placement. The adopted computational framework includes a linear-elastic eigenvalue buckling analysis of the bridge during deck placement using two primary construction load sources: 1) girder self-weight, and 2) slab dead load. Two load cases are considered to represent the critical conditions for the maximum possible positive and negative moments, as shown in Figure 5.28. The implementation of loading has been treated differently for each component. The girder self-weight is applied as a gravity load, while the slab dead load is applied as a uniform pressure over the top flange of the girder, as depicted in Figure 5.29. Load location on the cross-section can have a significant impact on the stability of the girder system with gravity loads applied higher on the cross-section resulting in destabilizing effects. As an initial step, the constitutive modeling of the material properties assumes a linear-elastic response for ease of computation. Subsequently, the analyses account for the nonlinear response from both geometric and the potential role of cracking on system stability during the deck placement. The deck placement sequence was assumed to be a continuous placement with the freshly placed deck concrete remaining plastic/fluid across the entire bridge length. A staged or “skip” placement sequence may be used or dictated by the design engineer based on the bridge span lengths, widths, and ready-mix concrete volume availability. The RT has observed from the spliced-girder bridge survey that both continuous and staged deck placement is common practice. The staged placement practice consists of placing the end span positive moment regions (separately or concurrently) followed by the positive moment region of the main span, then placement of the negative moment region above the interior supports. This is consistent with TxDOT’s current Standard Specification item 422 *Concrete Superstructures*. The RT chose to analyze the continuous placement sequence because TxDOT specifications require the previously placed concrete stage to achieve a compressive strength of 3,000 psi before the next stage can be placed. The cured concrete stage dramatically decreases the unbraced lengths of the girders and provides lateral and warping restraint to subsequent placements. Therefore, the critical limit state the RT investigated was the continuous placement option along the length of the bridge.



(a) Load Case 1 (Maximum positive moment)



(b) Load Case 2 (Maximum negative moment)

Figure 5.28. Load case under construction load during deck pour

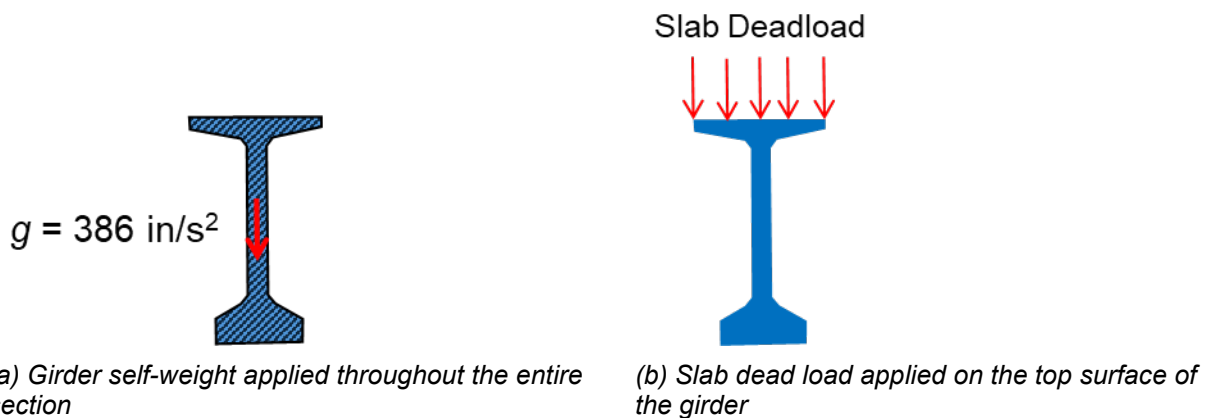


Figure 5.29. Load application in eigenvalue buckling analysis

5.4.1. Stability Behavior of PSC I-girder

This section describes the stability behavior of PSC I-girders initially considering the behavior of a single girder line with the system is idealized as an individual girder. Subsequently, analysis results considering a multi-girder system are presented, and the results are compared to those of the single-girder analysis in terms of modeling effort and accuracy. Additionally, the effects of diaphragm layout and skew are examined. Finally, the influences of geometric and material nonlinearity are investigated by incorporating geometric imperfections and cracking into the FE models. A large displacement analysis was conducted to evaluate girder behavior during deck placement.

5.4.1.1. Stability Behavior of Single Girder Line

The analysis started with a system comprising a single girder line, without diaphragms or lateral bracing. The girder was modeled as a continuous beam with an exterior-to-interior span length ratio of 70%. Solid elements with reduced integration (C3D8R) are adopted for the girder modeling. The eigenvalue buckling analyses were performed with the boundary conditions shown in Figure 5.30. These boundary conditions were applied to the bottom flange of the girder at the center of the support. To model a continuous beam, vertical (U_y) and transverse (U_x) translation restraints were applied to all nodes at the supports. The longitudinal translation restraint (U_z) was applied only to one of the interior supports. Additionally, transverse translation restraints were applied to the top of the web at the support locations to simulate temporary support during construction.

An eigenvalue buckling analysis was conducted to study the stability during the deck pour, and both Load Case 1 and Load Case 2 from Fig. 5.28 were considered. Two girder types, namely, STx7 and STx9, with span-to-depth ratios of 25 and 30, were studied. The effect of skew on stability behavior was also taken into account. The maximum spacing of the girders considered were 12 ft and the corresponding tributary wet concrete load was factored into the uniform pressure applied to the top of each girder. The adopted model parameters are summarized in

Table 5-8.

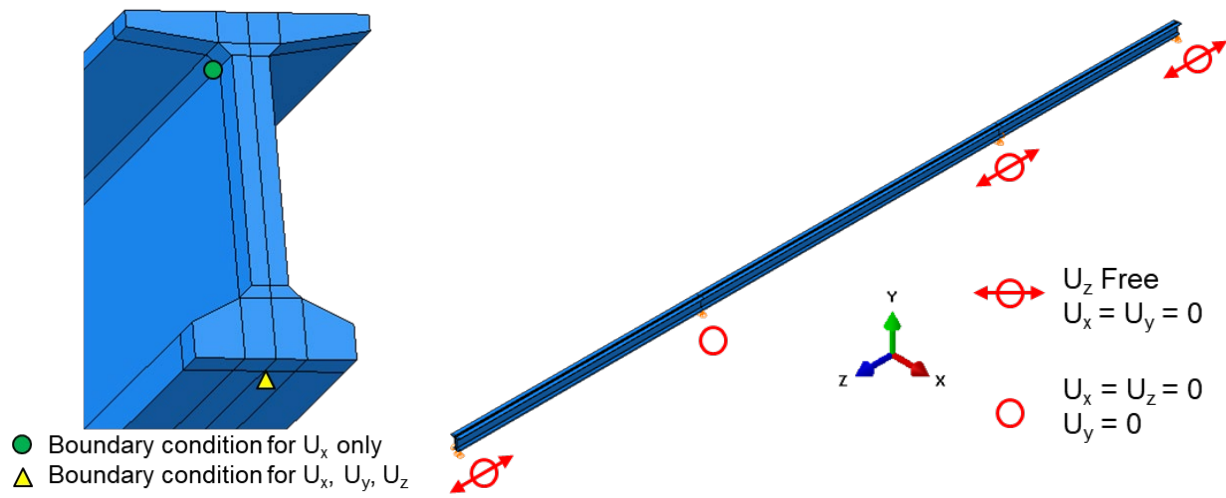
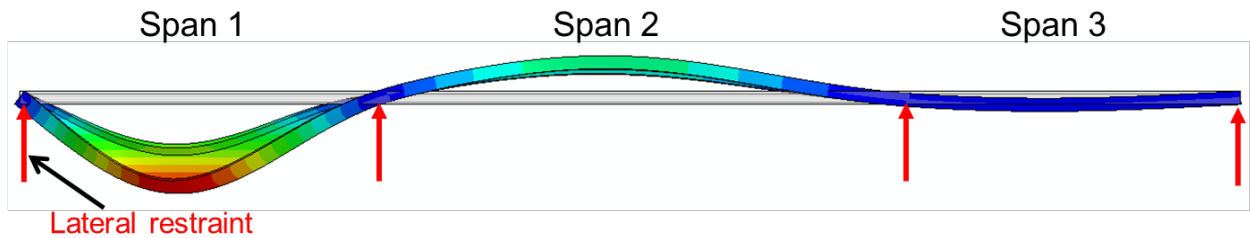


Figure 5.30. Boundary conditions of a single girder line analysis

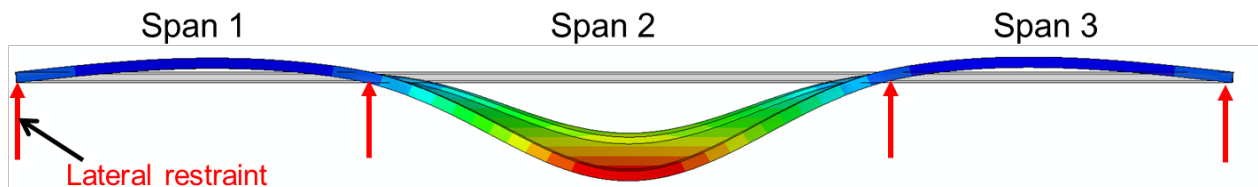
Table 5-8 Summary of the parameters for the analysis of a single girder line

Parameter	Range
Girder type	STx7 and STx9
Span-to-depth ratio (L/D)	25 and 30 for constant-depth girder 30 and 35 for variable-depth girder
Span arrangement	$0.7L + L + 0.7L$
Girder spacing (S)	12 ft
Overhang width (w_{oh})	4.5 ft
Slab thickness (t_s)	8.5 in
Skew angle (θ)	0° , 30° , and 45°

Figure 5.31 shows the first buckling mode (eigenvector) for Load Case 1, with the maximum positive bending moment occurring in span 1. For Load Case 2, the maximum positive moment is in Span 2, which corresponds to the buckling location. The location of the maximum moment can be observed from the moment diagram during deck placement in Figure 5.32. Although Figure 5.32 represents the moment diagram for the STx9 girder with an L/D ratio of 30, the other cases had similar moment diagrams with different magnitudes. The selected results are therefore representative of the behavior of all of the analyses that were conducted.



(a) Load case 1



(b) Load case 2

Figure 5.31. Plan view of buckling mode 1 of single-girder line analysis

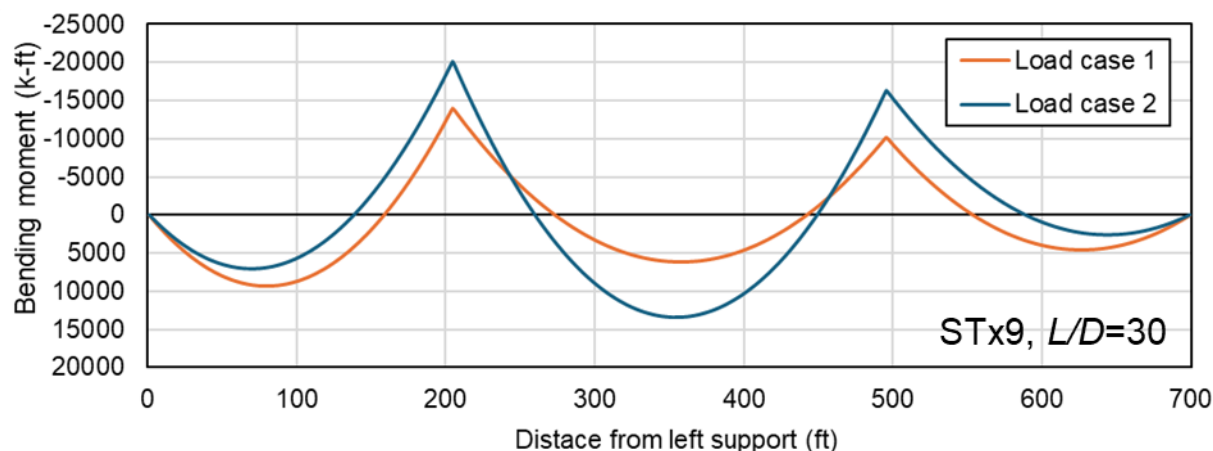


Figure 5.32. Moment diagram during deck placement for Load cases 1 and 2

Since the applied loads represent the actual expected loads (in service) during deck placement, the critical buckling load can be determined by the product of the eigenvalue – obtained from the eigenvalue buckling analyses – and the applied load. In this study, the critical buckling moment ($M_{cr,FEA}$) was obtained by multiplying the eigenvalue with the maximum service moment of the buckling span –where the specimen incurs the maximum possible deformation prior to buckling. For example, the critical buckling moment for Load Case 1 was calculated using the eigenvalue and the maximum positive service moment of Span 1. Based on the results in Table 5-9, the eigenvalues are relatively large. The lowest eigenvalue is 2.1 for the STx9 with all others significantly larger, indicating that the evaluated girders are reasonably stable during deck placement. This is also true with a load factor of 1.4 considered for girder self-weight and slab dead load for a perfect system with no imperfections. Because the eigenvalue of 2.1 is greater than the load factor of 1.4. As noted in the discussion of the analyses on girder erection, the 1.4 factor in the AASHTO BDS (2024) is generally reserved for steel superstructures. However, that factor is utilized to assess stability limit states and is therefore deemed appropriate for assessing the stability of PS girder systems. The most critical case for stability during deck placement is the girder with an L/D of 35 and a variable depth section. The unbraced length of the girders in these analyses are considerably larger than what is currently used in Texas where most applications make use of diaphragms at the splice locations and potentially additional locations – which is a good practice in the minds of the researchers even though the results show larger magnitudes of the eigenvalues. The beneficial impact of diaphragms provided at the splice locations and at midspan are covered later in the chapter.

Additionally, the critical buckling moments from FEA were evaluated by comparing the computer solutions with the theoretical critical moment when applying a uniform moment ($M_{cr,UM}$). The critical buckling moment from the FEA is generally larger than the theoretical critical moment by 10–30% for the constant-depth girder, as presented in Table 2.2. However, the differences are greater for the variable-depth girder, since the theoretical critical moment was evaluated using the section at mid-span and neglected the effect of the tapered sections at the support regions. Therefore, the theoretical critical moment for the variable-depth girder is more conservative than

that of the constant-depth girder. Furthermore, the results from the FEA solutions were compared with the theoretical solution, including the moment-gradient factor (C_b). The results from FEA are lower than the theoretical critical moment with the C_b factor. The reduction in the buckling capacity is most likely due to the top flange loading (slab dead load) and aligns with the study documented by Helwig et al. (1997), which found that load height has a significant effect on the buckling capacity of a singly symmetric I-beam. As such, the researchers propose using a C_b factor of 1.0 to account for the load-height effects. Note that C_b factor was evaluated based on the equation given in the AASHTO BDS (2024) and AISC (2022).

Table 5-9 Summary of the eigenvalue buckling analysis of a single girder line

Load case	Section	Girder type	L/D	Eigenvalue	$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$	$\frac{M_{cr, FEA}}{C_b M_{cr, UM}}$
1	STx7	Constant	25	15.0	58,137	1.16	0.68
		depth	30	8.8	48,745	1.17	0.68
		Variable	30	10.3	50,674	1.22	0.57
		depth	35	6.4	43,406	1.23	0.57
	STx9	Constant	25	7.6	49,158	1.13	0.63
		depth	30	4.4	41,049	1.14	0.63
		Variable	30	5.3	43,713	1.21	0.55
		depth	35	3.2	36,561	1.19	0.54
2	STx7	Constant	25	8.5	47,160	1.36	0.76
		depth	30	5.0	39,477	1.37	0.76
		Variable	30	6.7	40,348	1.40	0.54
		depth	35	4.2	34,448	1.40	0.54
	STx9	Constant	25	4.2	40,034	1.33	0.73
		depth	30	2.5	33,377	1.33	0.73
		Variable	30	3.4	34,750	1.39	0.53
		depth	35	2.1	28,999	1.36	0.52

5.4.1.2. Stability Behavior of Multi-girder Systems

The computation of the stability response also included multi-girder systems. The adopted Finite Element (FE) models represent a three-span continuous bridge with four girders and 1.5-ft thick diaphragms at the support locations, as shown in Figure 5.33. Solid elements (C3D8R) were used for both girders and diaphragms, and tie constraints were applied to connect the girders and diaphragms. Unlike the single girder line analysis, boundary conditions are applied only at the bottom flange of the girders, as the diaphragms provide lateral restraint and alternate load paths for the system.

Figure 5.34 presents the boundary conditions of the multi-girder systems. Typically, two types of supports were adopted for PSC I-girder systems: a pinned (fixed) support and a roller (expansion)

support. A fixed support is assigned to one of the interior supports, where the translation is restrained in all directions. Expansion supports were assigned to the other supports, allowing movement in the longitudinal direction. Note there were certain cases where more than one longitudinal support was fixed or made integral to address extreme event limit states, such as earthquake or vessel collisions. From a stability standpoint, evaluating only one longitudinally pinned support results in less restraint than other configurations. Eigenvalue buckling analyses, using the Subspace Iteration Method, were performed to study the stability behavior of the system for both Load Case 1 and Load Case 2. The parameters and the associated ranges are similar to those in the single girder system study, as shown in Table 5-10. Additionally, various girder spacings were investigated, as the spacing of the girders impacts the stiffness of the multi-girder systems.

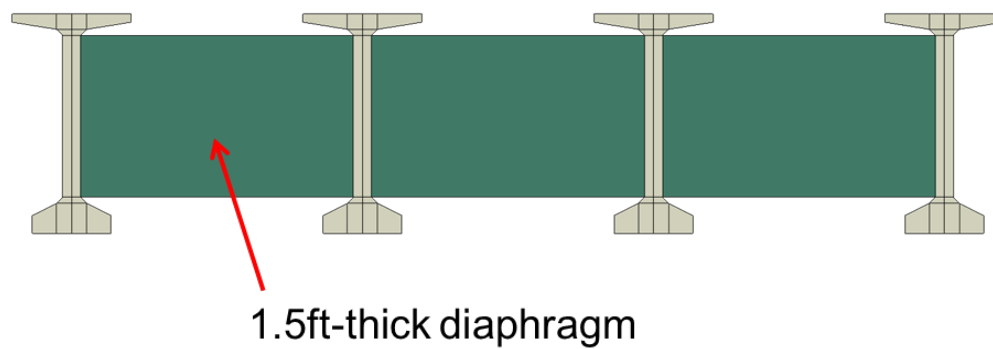


Figure 5.33. Cross-section of multi-girder systems

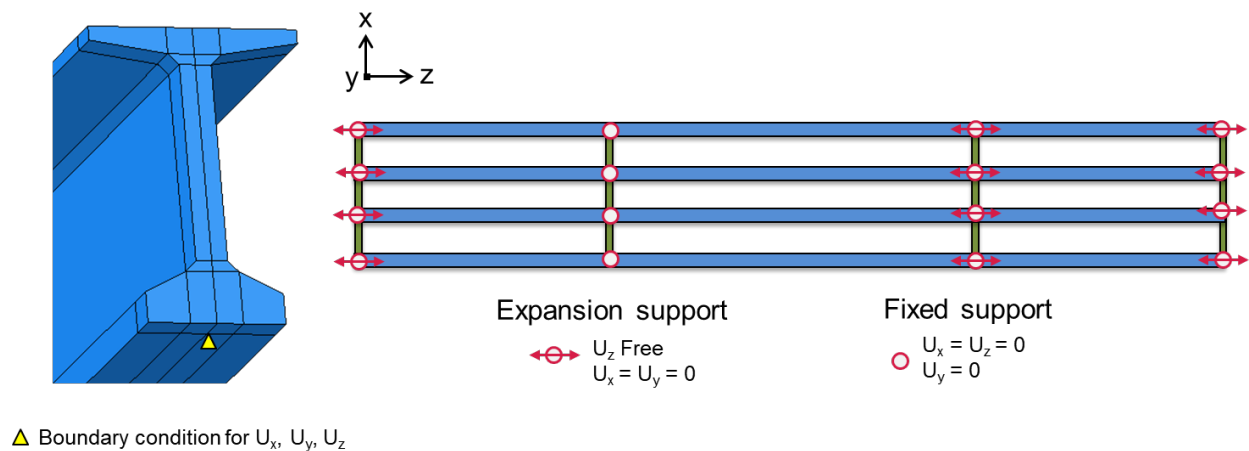


Figure 5.34. Isometric and plan view of boundary conditions for multi-girder systems

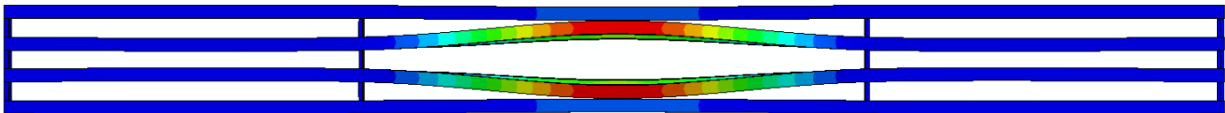
Table 5-10 Summary of the parameters for analysis of multi-girder systems

Parameter	Range
Girder type	STx7 and STx9
Span-to-depth ratio (L/D)	25 and 30 for constant-depth girder 30 and 35 for variable-depth girder
Span arrangement	$0.7L + L + 0.7L$
Girder spacing (S)	8, 10, and 12 ft
Overhang width (w_{oh})	4.5 ft
Slab thickness (t_s)	8.5 in
Skew angle (θ)	0° , 30° , and 45°
Number of girder (N_b)	4

The eigenvalue buckling analyses of multi-girder systems revealed stability behavior similar to that of a single-girder system, as shown in Figure 5.35. However, for Load Case 1, the buckling locations were somewhat inconsistent. In some cases, buckling occurs in Span 2, even though Span 1 typically experiences a larger maximum positive moment. This is because Span 2 had a greater unbraced length, resulting in a lower buckling capacity, which in turn made the span more susceptible to instability.

Additionally, the specific girder that experienced buckling (exterior or interior) did not always align with the expected load distribution based on slab tributary width. In some cases, buckling occurred in the exterior girder, despite that girder carrying less load than the interior girder. This suggests that diaphragms provide different levels of warping restraint to the exterior and interior girders.

In contrast, for Load Case 2, the stability behavior is consistent, with buckling always occurring in Span 2. In this case, the buckling girder was determined by the girder carrying the greater load, which corresponds to the wider slab tributary width. Further details on the buckling behavior of each system are provided in Appendix B.

*(a) Load Case 1**(b) Load Case 2***Figure 5.35. Buckling mode 1 of multi-girder analysis (Top view)**

After obtaining eigenvalues from the parametric study, the buckling moments were calculated by multiplying the eigenvalues with the maximum positive moment in the critical buckling span and girder.

Table 5-11 presents the eigenvalues, corresponding buckling moments, and a comparison between the computed and theoretical buckling moments. The results indicate that all evaluated girders remain stable during deck placement, as all eigenvalues exceed the 1.4 demand load factor.

Table 5-11 only includes cases with a 12-ft girder spacing, as this represents the most critical scenario among all tested spacings. Results for the 8-ft and 10-ft girder spacings are presented in Appendix B.

Table 5-11 Results of the eigenvalue buckling analysis of multi-girder systems (12-ft girder spacing)

Load case	Section	Girder type	L/D	Eigenvalue	$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$	$\frac{M_{cr, FEA}}{C_b M_{cr, UM}}$
1	STx7	Constant depth	25	18.5	71,498	1.42	0.83
			30	11.0	60,581	1.46	0.85
		Variable depth	30	12.7	62,919	1.51	0.70
			35	8.1	54,343	1.54	0.71
	STx9	Constant depth	25	9.5	61,936	1.42	0.79
			30	5.6	35,070	1.40	0.57
		Variable depth	30	6.7	51,369	1.42	0.63
			35	4.1	46,949	1.53	0.69
2	STx7	Constant depth	25	9.1	50,515	1.46	0.81
			30	5.3	42,333	1.47	0.81
		Variable depth	30	7.0	42,390	1.47	0.57
			35	4.4	36,161	1.47	0.57
	STx9	Constant depth	25	4.6	43,071	1.43	0.79
			30	2.7	35,963	1.44	0.79
		Variable depth	30	3.5	36,638	1.46	0.56
			35	2.2	30,527	1.43	0.55

5.4.1.3. Comparison between Single-girder Line and Multi-girder system

This section focuses on the effect of the modeling techniques on the eigenvalue buckling analysis results. The results from the single-girder line analysis are compared with those from the multi-girder systems. The key difference between these two approaches is that support diaphragms are explicitly modeled in the multi-girder systems, providing lateral support. In contrast, for the single-girder line analysis, rigid lateral support is applied at the top of the girder to simulate temporary bracing, as shown in Figure 5.30. With the single girder models, warping restraints that would normally be applied were conservatively neglected.

As seen in Table 5-12, eigenvalues from multi-girder systems are higher than those from the single-girder line analysis, particularly for Load Case 1, where an approximately 25% increase is observed. This suggests that diaphragms play a significant role in providing warping restraint to the attached girders. Figure 5.35 further illustrates that the lateral displacement of the span adjacent to the buckling span is relatively small in multi-girder systems. Conversely, Figure 5.31 shows that lateral displacement in other spans is more pronounced when diaphragms are absent. However, for Load Case 2, which is more critical than Load Case 1, the increase in eigenvalues due to diaphragms is only 5–10%. This indicates that single-girder line analysis is a reasonable approach and is not overly conservative, even when diaphragms are not explicitly considered.

Table 5-12 Summary of the eigenvalue buckling analysis of multi-girder systems

Load case	Section	Girder type	L/D	$\lambda_{\text{single-girder}}$	$\lambda_{\text{multi-girder}}$	$\frac{\lambda_{\text{multi-girder}}}{\lambda_{\text{single-girder}}}$
1	STx7	Constant depth	25	15.05	20.2	1.23
			30	8.82	11.8	1.25
		Variable depth	30	10.27	12.7	1.24
			35	6.44	8.1	1.26
	STx9	Constant depth	25	7.55	10.2	1.26
			30	4.40	5.9	1.27
		Variable depth	30	5.26	6.7	1.27
			35	3.23	4.1	1.27
2	STx7	Constant depth	25	8.50	9.7	1.07
			30	4.98	5.7	1.06
		Variable depth	30	6.66	7.0	1.05
			35	4.16	4.4	1.06
	STx9	Constant depth	25	4.24	4.9	1.08
			30	2.47	2.9	1.09
		Variable depth	30	3.37	3.5	1.04
			35	2.06	2.2	1.07

5.4.1.4. Effect of Skew

Skewed supports may influence the stability behavior due to changes in the diaphragm configuration. Therefore, the effect of skew on girder stability was examined. In this study, the modeling assumptions and load applications for skewed bridges were similar to those for normal-support bridges, as shown in Figure 5.36. The range of parameters considered for skewed bridges is presented in Table 5-10.

In order to evaluate the impact of skew, the eigenvalues of skewed bridges were normalized against those of normal-support bridges. Figure 5.37 presents the normalized eigenvalues, illustrating the

comparison. The box-and-whisker plot shows that the difference in eigenvalues between the two bridge types are within 5%, which is reasonable given the unchanged unbraced length in skewed bridges. Based on these findings, the effect of skew on the buckling moment can reasonably be ignored in most cases.



Figure 5.36. Plan view of FE model of a multi-girder system with diaphragms parallel to the skew at supports

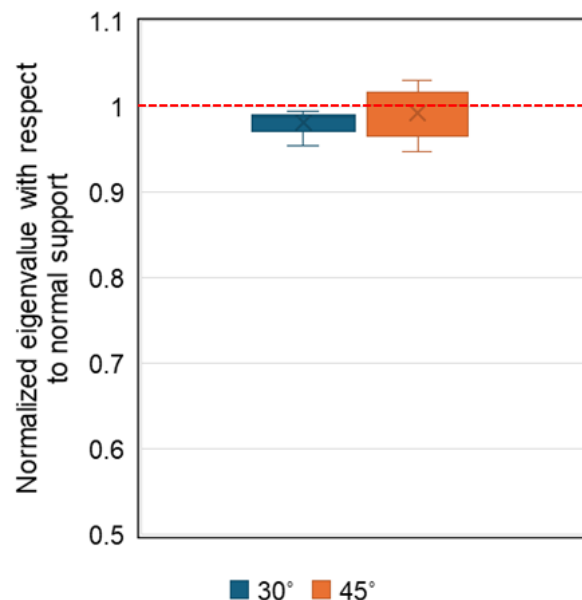


Figure 5.37. Statistical distribution of eigenvalue ratio between skew bridges and normal support bridges

5.4.1.5. Effect of Diaphragm Layout

Diaphragms play a significant role in providing stability during deck placement. The buckling analysis of multi-girder systems with diaphragms only at support locations was discussed in Section 5.4.1.3, where results showed improved stability compared to systems without diaphragms (i.e., single-girder line analysis with rigid lateral support). Although the analyses indicated the stable nature of the systems without diaphragms, there was particular interest to further examine their stability behavior when diaphragms were placed at the spliced and mid-span locations.

In this investigation, the buckling analyses were performed for different diaphragm layouts, as shown in Figure 5.38. Layout-0 represents the system without diaphragms, assuming rigid lateral support (single-girder line analysis). Layout-0 assumes girders have adequate temporary bracing at supports to brace the system for construction and environmental loads. The braces in Layout-0 provide no warping restraint in contrast to permanent concrete diaphragms. Layout-1 includes

diaphragms only at the supports, while Layout-2 and Layout-3 incorporate additional diaphragms at spliced and mid-span locations. The analyses focused on systems with the STx9 girders and larger L/D ratios, which represent the most critical cases. The range of parameters considered is summarized in Table 5-13.

Table 5-14 Summary of the parameters for analysis of multi-girder systems with differing diaphragm layouts

Parameter	Range
Girder type	STx9
Span-to-depth ratio (L/D)	30 for constant-depth girder 35 for variable-depth girder
Span arrangement	$0.7L + L + 0.7L$
Girder spacing (S)	12 ft
Overhang width (w_{oh})	4.5 ft
Slab thickness (t_s)	8.5 in
Skew angle (θ)	0°
Diaphragm layout	Layout-0 = No diaphragm* Layout-1 = Diaphragms at support locations Layout-2 = Diaphragms at support and spliced locations Layout-3 = Diaphragms at support, mid-span and spliced locations

* = Single-girder system with rigid lateral support

Table 5-15 presents the eigenvalues of selected bridge systems with different diaphragm layouts. The comparison between Layout-0 and Layout-1 has been discussed in Section 2.4.2.2. As shown in Table 5-15, Layout-2 and Layout-3 exhibit significant stability improvements—approximately 50%—due to the reduction in unbraced lengths. Among all layouts, Layout-3 provides the highest buckling capacity, as that model has the shortest unbraced length. Based on these findings, the RT recommends incorporating diaphragms at the support locations a necessity that should not ever be changed for these long-span PS girder systems. The diaphragms at the splices and midspan also provide significant enhancements to the girder stability. While the diaphragms at midspan are likely extra effort by the contractor, the diaphragms at the splice locations seem to be “relatively” simple since the splices must be cast. The researchers believe including the splice diaphragms lead to improved stability that provide significant improvements in the behavior and should be included. The diaphragms at midspan also significantly improve the behavior of the girders. However, if these diaphragms were eliminated, the results show that the girder stability was not likely impacted significantly based upon the expected construction loads.

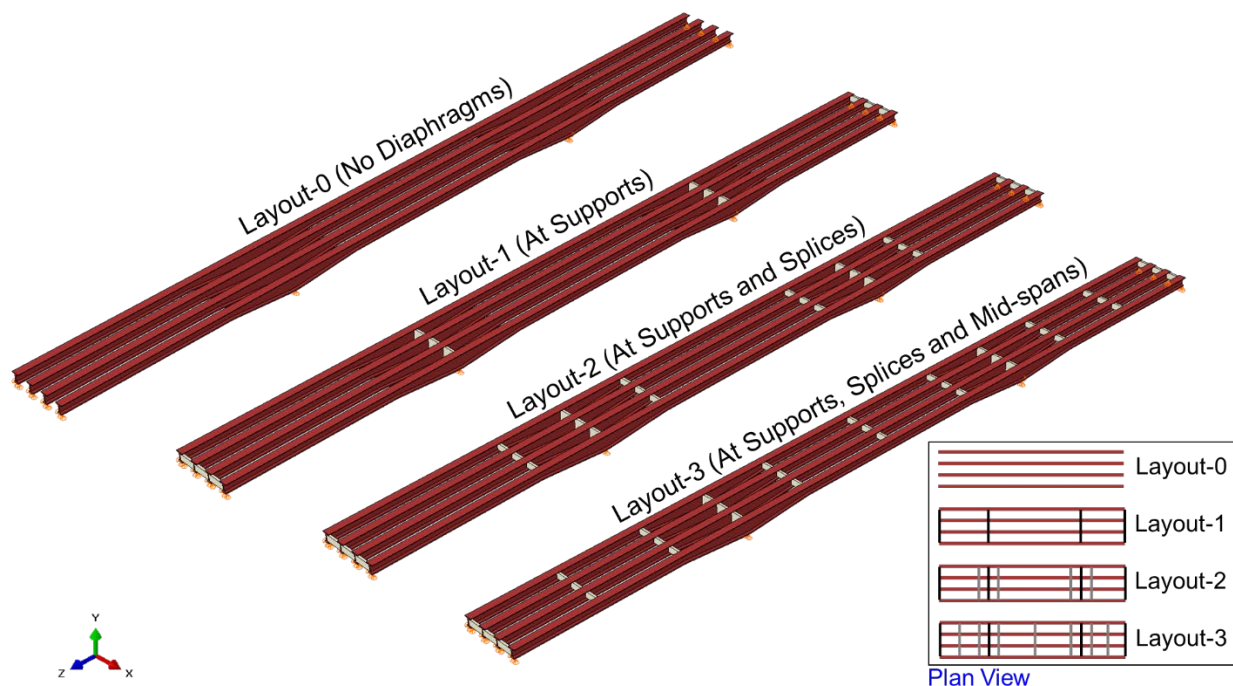


Figure 5.38. FE model of a multi-girder system with different diaphragm layout

Table 5-15 Results of the eigenvalue buckling analysis of multi-girder systems with different diaphragm layouts (12-ft girder spacing)

Load case	Section	Girder type	L/D	Eigenvalue (λ)			
				Layout 0	Layout 1	Layout 2	Layout 3
1	STx9	Constant depth	30	4.8	5.9	8.6	15.3
		Variable depth	35	3.2	4.1	6.1	10.4
2	STx9	Constant depth	30	2.6	2.9	5.0	8.8
		Variable depth	35	2.1	2.2	4.1	6.7

Additionally, the RT investigated the buckling behavior of girders with different diaphragm layouts. It was observed that, for both Load Cases, the girders in Layouts 1 and 2 buckled between diaphragm locations, as shown in Figure 5.39 and Figure 5.40. The theoretical buckling moment, calculated using the unbraced length between diaphragms, was compared with results from the FEA. As discussed previously in Section 5.4.1.2, the FEA results for Layout 1 showed that the buckling moment was 40-50% higher than the theoretical prediction. Table 5-16 presents the FEA results and their comparison with the theoretical solutions for Layouts 2 and 3. Similar to Layout

1, the buckling moment for Layout 2 was approximately 50-60% higher than the theoretical value. Based on these findings, the RT concludes that the theoretical solution is applicable to Layout 2 as well.

In contrast, the results for Layout 3 indicate that global buckling of the entire girder system governs the behavior, with all girders buckling together, unlike the individual girder buckling observed in Layouts 1 and 2. Although global buckling governs in this case, the theoretical solution is still conservative by 20-30% compared to the FEA results. It should be noted that the RT recommends using a C_b factor of 1.0 due to capacity reduction from the load-height effect.

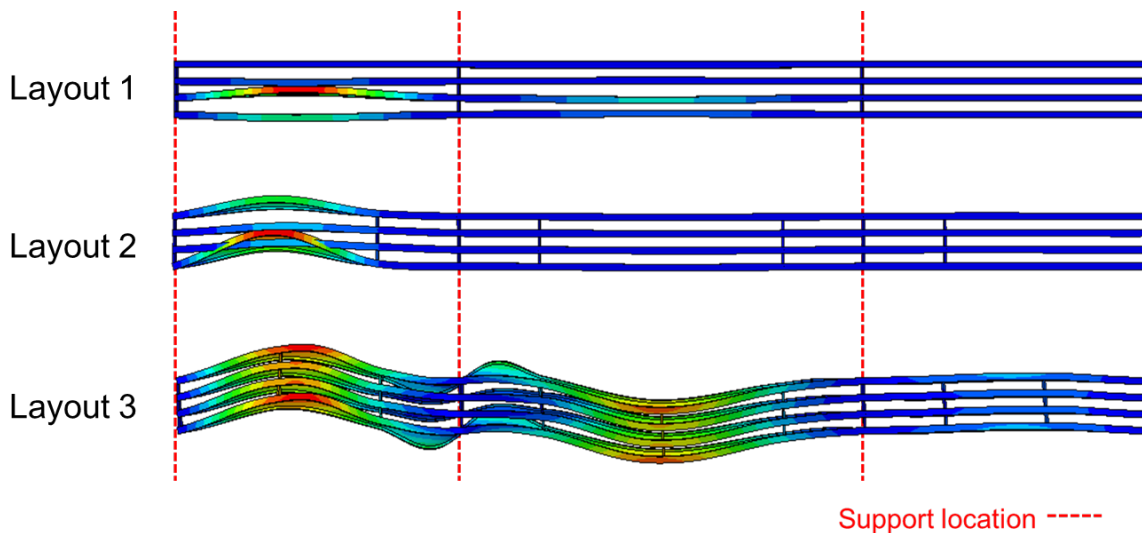


Figure 5.39. Comparison of buckling shapes with different diaphragm layout for Load Case 1

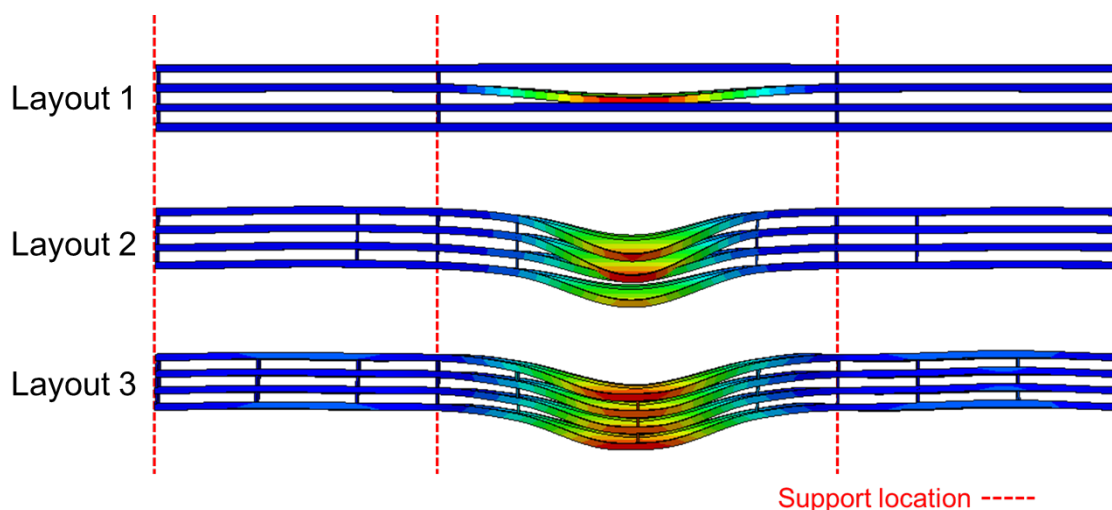


Figure 5.40. Comparison of buckling shapes with different diaphragm layout for Load Case 2

Table 5-16 Comparison of buckling moment between the results from FEA and theoretical solution for Layout 2 and 3

Load case	Section	Girder type	L/D	Layout 2		Layout 3	
				$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$	$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$
1	STx9	Constant depth	30	76,686	1.49	104,347	1.16
		Variable depth	35	66,011	1.51	118,606	1.28
2	STx9	Constant depth	30	67,510	1.59	119,605	1.33
		Variable depth	35	57,173	1.58	93,953	1.25

Based on these findings, the RT recommends TxDOT continue incorporating diaphragms at the supports and splice points. These diaphragms significantly enhance the stability of the system, approximately doubling the critical buckling capacity when compared to the single girder case. These results also show, due to the significant overhead of capacity compared to the construction loading, that including diaphragms at the midspan is not a necessity for stability for the design range tested. Nevertheless, the benefits of these diaphragms will be further cemented in the next section, which will demonstrate an improvement to constructability when using at minimum Layout 2.

5.4.1.6. Evaluation of the Impact of Geometric Imperfections

While the last section focused on the critical buckling loads, additional parametric studies were conducted to assess the effects of geometric imperfections on system stability. The study focused on critical single-girder systems identified in previous analyses. The primary objective was to evaluate stability behavior by generating load-displacement (translation or rotation) curves and assessing changes in the system stiffness before the theoretical eigenvalue buckling capacity is reached.

In this study, the evaluated systems consist of the STx9 girders with the longest span length and the largest girder spacing (higher slab dead load). The modeling parameters are summarized in Table 5-17. *TxDOT Specification Item 424* and *PCI* list the sweep tolerance as an 1/8 inch per 10 ft of girder length. This equates to an imperfection of $L/960$. Two initial imperfection magnitude (Δ_0) were examined: $L/500$ and $L/1000$, where L represents the respective span length. The material was initially assumed to be linearly elastic and cracked section properties were later incorporated to examine the effects of material nonlinearity on the stability behavior.

The assumed cross-sectional imperfection shape pivots about the midline of the bottom flange while the top flange exhibits a lateral sweep of Δ_0 , as illustrated in Figure 5.41. This creates both a twist and sweep of the cross-section along the length of the girder. This imperfection shape has

been identified as critical for I-shaped steel girders (Han and Helwig, 2020). Additionally, the imperfection follows a sine wave distribution along the girder, with the initial imperfections applied based on each span length, as shown in Figure 5.41.

The analysis was conducted using the Riks Arc-Length Method, which incorporates iterative loading, changes in geometric stiffness (due to incremental load in the system), and equilibrium being taken on the deformed geometry in each step. The modeling assumptions and load applications followed the same approach as discussed in the single-girder line analysis. Loads were incrementally applied to the girder until buckling occurred, and lateral displacement was recorded throughout the loading process.

Table 5-17 Parameter ranges for evaluation of geometric imperfections

Parameter	Range
Girder type	STx9
Span-to-depth ratio (L/D)	30 for constant-depth girder 35 for variable-depth girder
Span arrangement	$0.7L + L + 0.7L$
Girder spacing (S)	12 ft
Overhang width (w_{oh})	4.5 ft
Slab thickness (t_s)	8.5 in
Skew angle (θ)	0°
imperfection	$L/500$ and $L/1000$

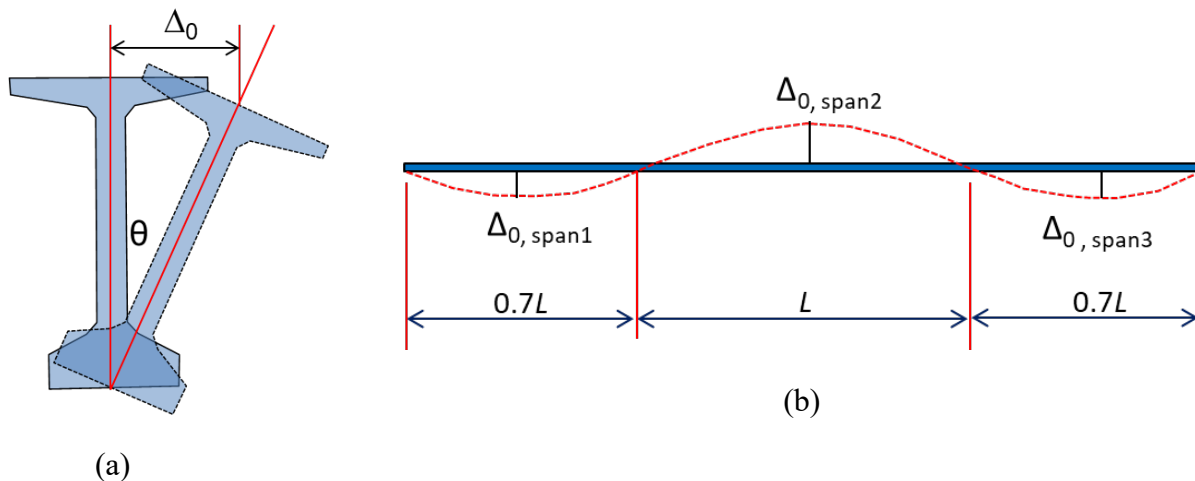


Figure 5.41. (a) Cross-sectional imperfection shape and (b) plan view of imperfection distribution along the length of the girder

Figure 5.43 illustrates the relationship between bending moment (M) and lateral displacement (Δ) at midspan. The moment is normalized by the expected service moment during deck placement (M_{const}). The figure includes two reference lines: the critical buckling moment (M_{crit}) and the

expected moment during deck placement with a load factor of 1.4 ($1.4M_{const}$). The normalized moment value at M_{crit} corresponds to the eigenvalue from the eigenvalue buckling analysis. The displacement shown represents the total lateral displacement of the top flange (Δ_{tf}), as illustrated in Figure 5.42. However, when comparing displacement to the initial imperfection (i.e., normalized displacement), the relative displacement—defined as the difference between the displacements of the top and bottom flanges—is used. This is because relative displacement better reflects the girder's twisting behavior and is more directly related to lateral-torsional buckling.

Lateral displacement was observed in mid-span of the buckling span, as previous studies identified these spans as the primary buckling locations. The results indicate that at the factored design moment ($1.4M_{const}$), the additional relative displacement is approximately 20% of the initial imperfection for Load Case 1 and 50% for Load Case 2. Furthermore, the level of initial imperfections ($L/500$ and $L/1000$) do not significantly affect the relative displacement when the applied moment is well below the critical moment (i.e., in cases with high eigenvalues). Although the STx7 girder was not investigated in this study, the research team concluded that relative displacement should not be a concern, as its eigenvalue is significantly higher than that of STx9, and the only change between the cross-sections is a web that is 2 feet taller for the STx.

Additionally, the analyses found that relative displacement reaches approximately twice the initial imperfection at 70% of the critical moment. Therefore, if relative displacement is limited to twice the initial imperfection, designers can restrict the applied moment to no more than 0.7 of the critical moment.

Although total lateral displacement may not directly affect girder buckling, the deformations can pose constructability issues due to excessive lateral movement. The results indicate that lateral displacement can be a concern, particularly in Load Case 2. In this case, a displacement of up to 15 inches may occur during deck placement under unfactored loads with an initial imperfection of $L/500$. To address this, the RT recommends limiting initial imperfections to less than $L/1000$ to remain within acceptable sweep tolerances. The reported lateral displacements assume diaphragms are placed only at the support locations. Providing diaphragms at splices or midspan locations is expected to reduce lateral displacements. As noted in the last section, the researchers recommend requiring diaphragms at the splice locations.

The results in Figure 5.43 apply only to constant-depth girders, however, the buckling behavior of variable-depth girders with geometric imperfections is similar to that of constant-depth girders. Further details on both girder types can be found in Appendix C.

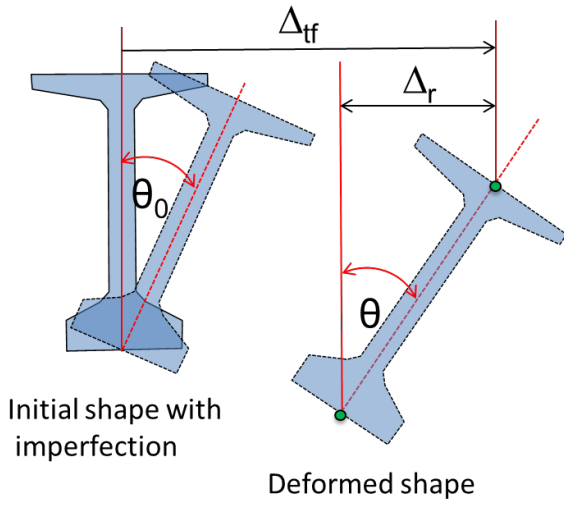
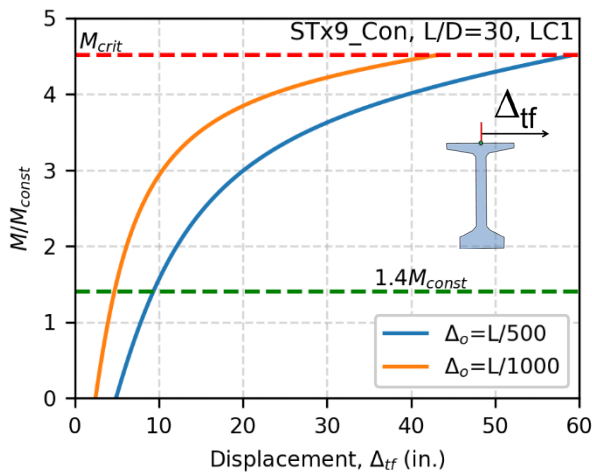
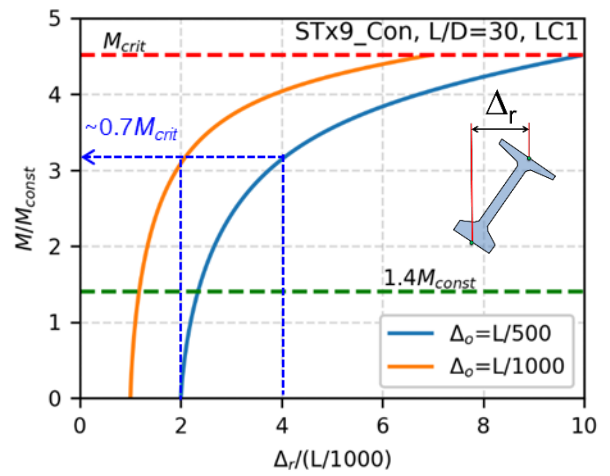


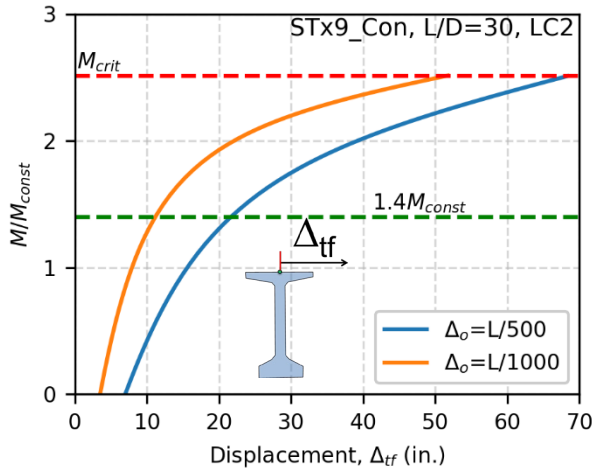
Figure 5.42. Lateral displacement and relative displacement of the girder



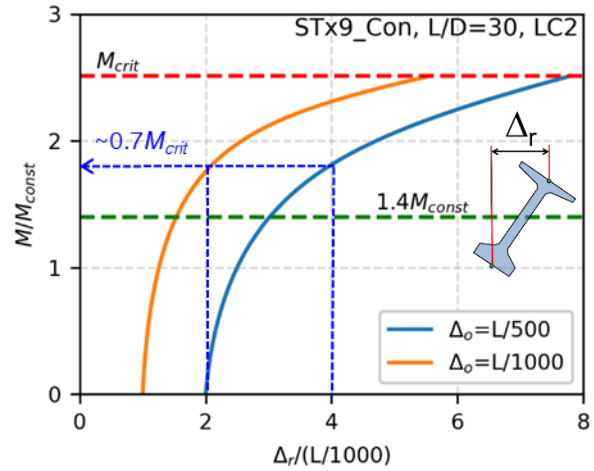
(a) Displacement of Load Case 1



(b) Normalized relative displacement of Load Case 1



(c) Displacement of Load Case 2



(d) Normalized relative displacement of Load Case 2

Figure 5.43. Load-displacement curve including the effect of initial imperfections on buckling behavior.

As discussed previously, the stability of the girder during deck placement is not a concern when diaphragms are provided only at the supports. However, the lateral displacement of the top flange may still be significant and can affect constructability. Therefore, the RT investigated the effect of diaphragm layout on girder behavior, including the influence of geometric imperfections.

In this study, the STx9 girder with constant depth and a span-to-depth ratio of 30 was analyzed. The RT considered an initial imperfection magnitude of $L/1000$. Figure 5.44 shows the relationship between the normalized moment and lateral displacement for Load Case 2. The results indicate that Layout 2 (diaphragms at supports and splice locations) significantly reduces lateral displacement compared to Layout 1 (diaphragms at supports only). The additional lateral displacement is less than 2 in. beyond the initial imperfection, and the relative displacement is less than 10% of the initial imperfection.

Although Layout 3 (diaphragms at supports, splice locations, and mid-span) provides a substantial increase in buckling moment capacity compared to Layout 2, its lateral displacement is only slightly smaller than that of Layout 2. Based on these findings, the RT recommends placing diaphragms at least at the supports and splice locations to ensure stability and constructability during deck placement.

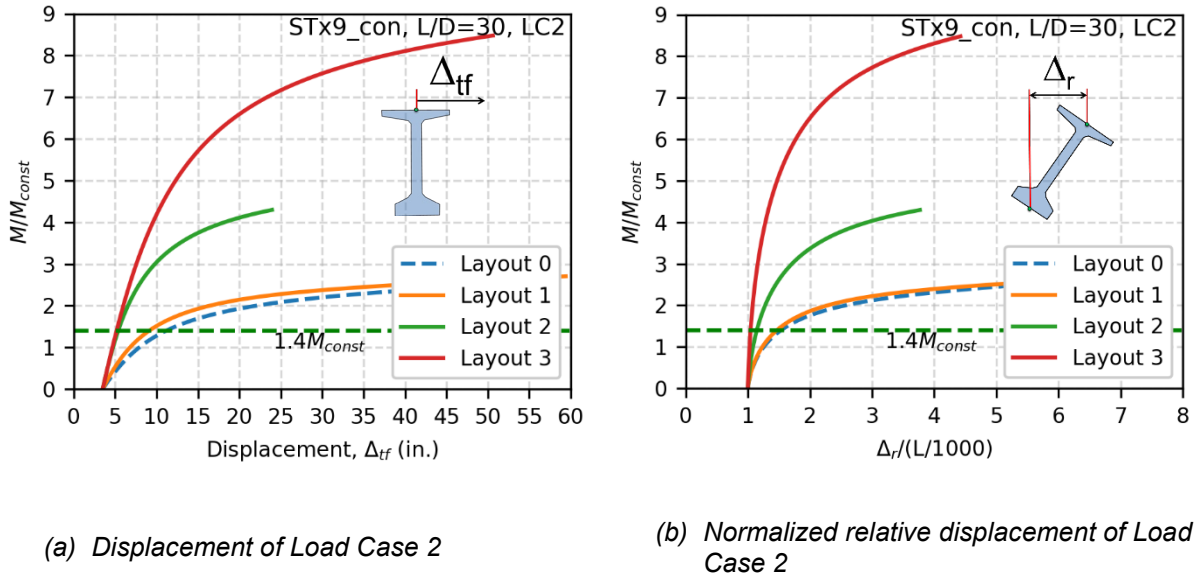


Figure 5.44. Load-displacement curve of the systems with different diaphragm layout including the effect of initial imperfections on buckling behavior.

5.4.2. Stability Behavior of PSC U-girder

5.4.2.1. Stability Behavior of Single Girder Line

Based on the analysis results of PSC I-girders, the single-girder line approach was observed to produce results that differed by approximately 5–10% from those of the multi-girder system under Load Case 2, which generally represents the critical loading scenario. This level of difference is considered reasonable by the research team for preliminary design and stability evaluations, especially given the significant reduction in modeling complexity and computational effort. Furthermore, the analysis showed that neglecting the effects of diaphragms led to conservative estimates of girder stability, as diaphragms typically enhance lateral-torsional stiffness. Given these findings, the RT elected to investigate the stability behavior of U-girders using the simplified single-girder line approach, aiming to balance accuracy with efficiency.

For the U-girder analysis, the TxU96-9 specimen was selected due to the relatively tall section and slender web of the girder, which increases the susceptibility to instability modes such as lateral-torsional buckling. The bridge cross-section geometry (girder spacing and overhang dimensions) used in the finite element models corresponded to the standard bridge drawings of the Pennsylvania Department of Transportation (PennDOT, 2016) as illustrated in Figure 5.45. TxDOT does not have standard plans/layouts for bridges of this type, only standard girder shapes. A span-to-depth ratio (L/D) of 30 was chosen to represent a typical constant-depth girder configuration used in long-span bridge applications. This study focuses mainly on constant-depth girders; variable-depth girders, which were found to have similar behavior, were excluded from

the scope of this investigation. A detailed summary of the bridge parameters, including material properties, section dimensions, and boundary conditions, is presented in Table 5-18.

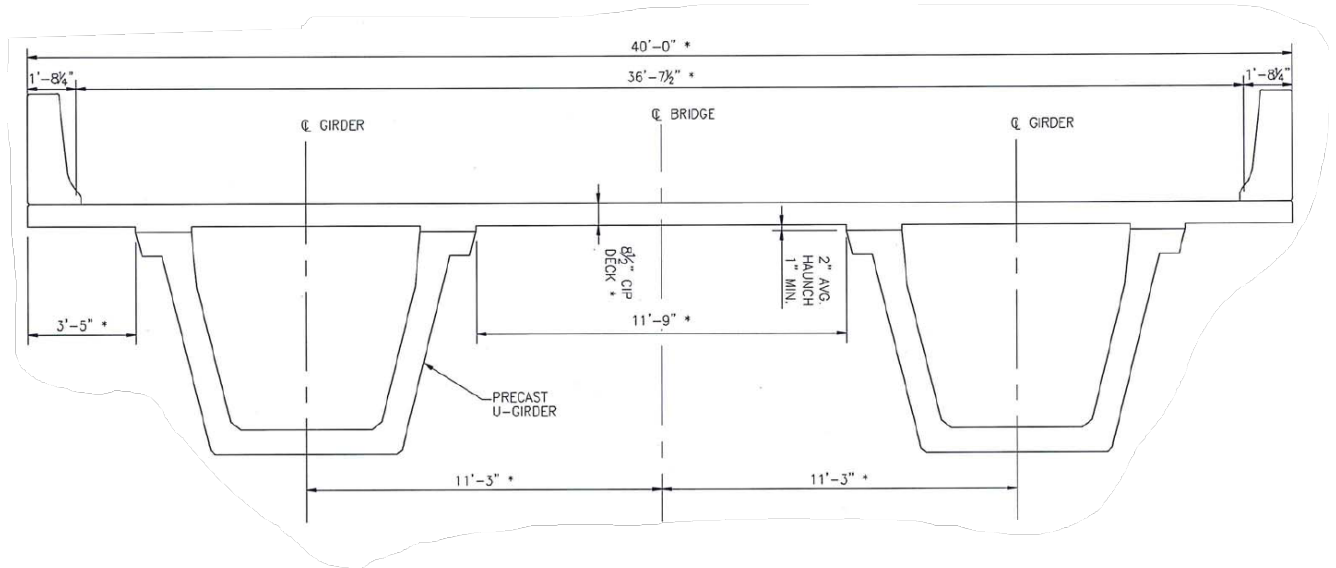


Figure 5.45. Typical bridge cross section of spliced concrete U-girder (PennDOT,2016)

Table 5-18 Summary of the parameters for analysis of U-girder

Parameter	Range
Girder type	U96-9
Span-to-depth ratio (L/D)	30 for constant-depth girder
Span arrangement	$0.7L + L + 0.7L$
Girder spacing (S)	22.5 ft
Overhang width (w_{oh})	3.42 ft from the edge of the top flange to the end of slab
Slab thickness (t_s)	8.5 in
Skew angle (θ)	0°

Eigenvalue buckling analyses were used to determine the critical buckling moment and assess whether the U-girder possesses sufficient capacity for the construction loads during the deck placement phase. Similar to the I-girder modeling approach, the U-girder was idealized as a continuous beam with an exterior-to-interior span length ratio of 70%, which reflects typical bridge geometry.

For FE modeling, 3D solid elements with reduced integration (C3D8R) were used to capture the structural behavior of the girder. Boundary conditions were applied to the bottom flange of the girder at the center of each support, following the same methodology used for the I-girder analysis, as illustrated in Figure 5.46. Specifically, vertical (U_y) and lateral (U_x) translation restraints were applied to all the supports, but only one interior support was restrained in the longitudinal direction.

Additionally, lateral translation restraints are applied to the top of the webs at the support locations to simulate temporary support during construction.

Two loading conditions were considered in the model: the self-weight of the girder and the slab dead load, as shown in Figure 5.47. The self-weight was accounted for by assigning the appropriate material density. The slab dead load was applied as a uniformly distributed load to the top flanges. For interior girders, it was assumed that the slab dead load is equally shared between the two top flanges, with each flange carrying half of the total load due to the symmetrical layout and construction staging.

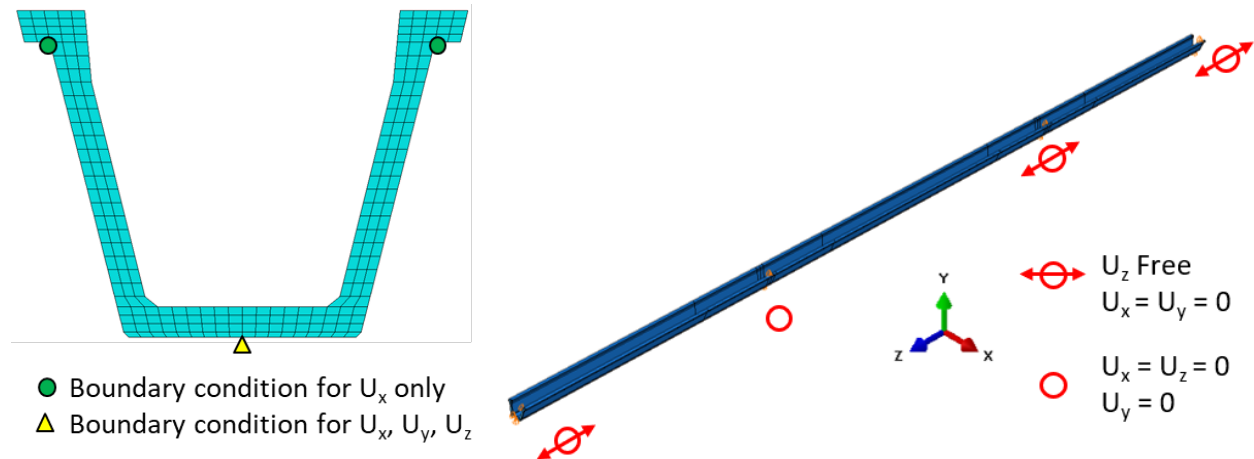


Figure 5.46. Boundary conditions of a single girder line analysis for U-girder

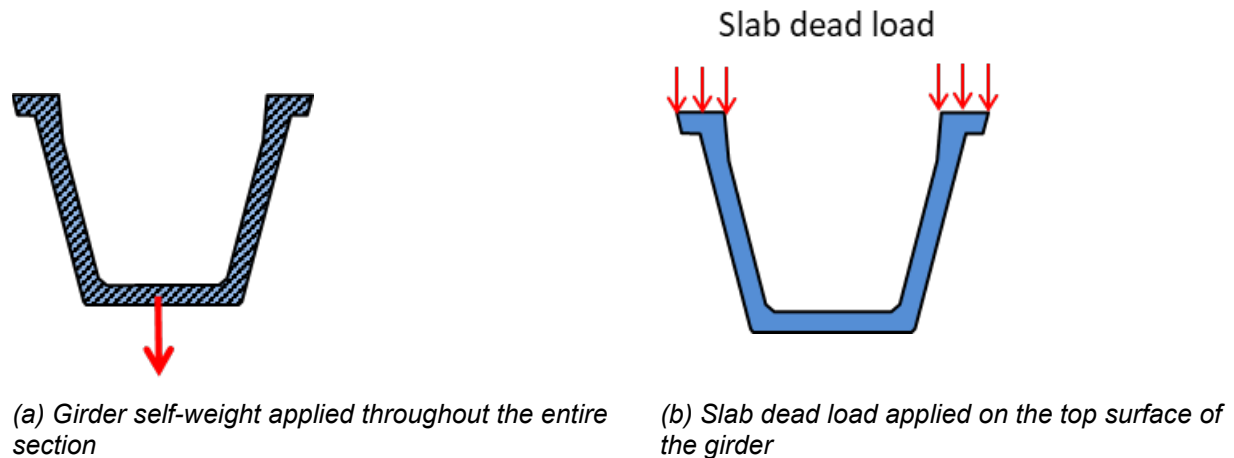


Figure 5.47. Load application of a single girder line analysis for U-girder

Once the eigenvalue buckling analysis was completed and the buckling moment determined, a static analysis using the Riks Arc-Length Method was performed to generate the load–displacement curve. Similar to the I-girder case, two initial imperfections of $L/500$ and $L/1000$ were examined. The imperfection was assumed to induce twisting at the centerline of the girder, with the pivot located at the bottom flange. A sine wave distribution was applied along the length

of each span, as depicted in Figure 5.48. Displacement was measured throughout the incremental loading steps until the buckling capacity was reached. The buckled shape revealed no significant distortion of the cross-section at the buckling moment. Moreover, the displacements of the two top flanges were nearly identical, with only minor differences due to section rotation. Therefore, only the displacement of the top flange deflecting in the direction of the initial imperfection is presented. The total displacement of the top flange, as illustrated in Figure 5.49 , was measured at each load increment.

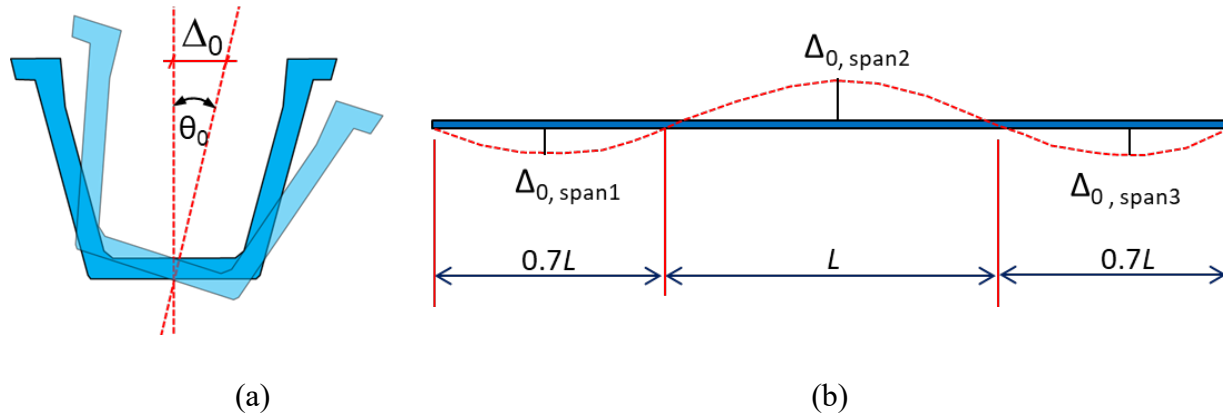


Figure 5.48. (a) Cross-sectional imperfection shape and (b) plan view of imperfection distribution along the length of the girder

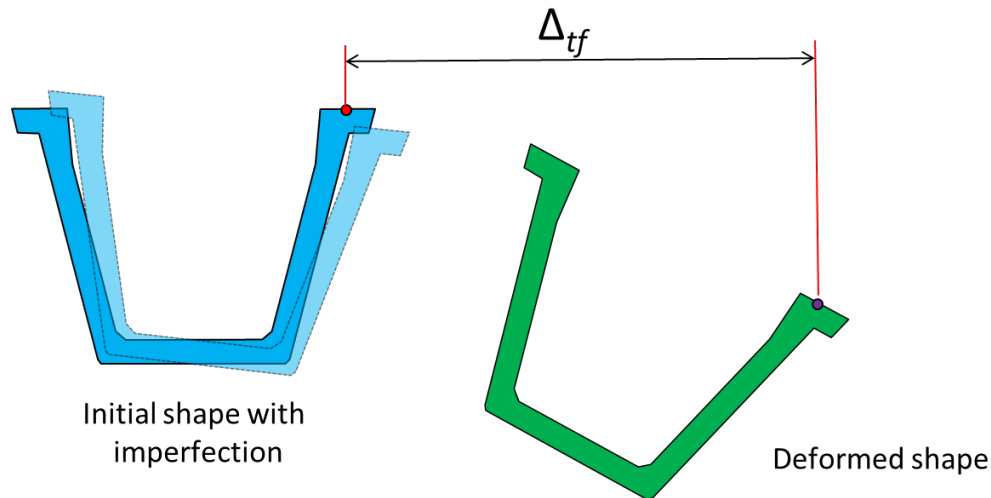
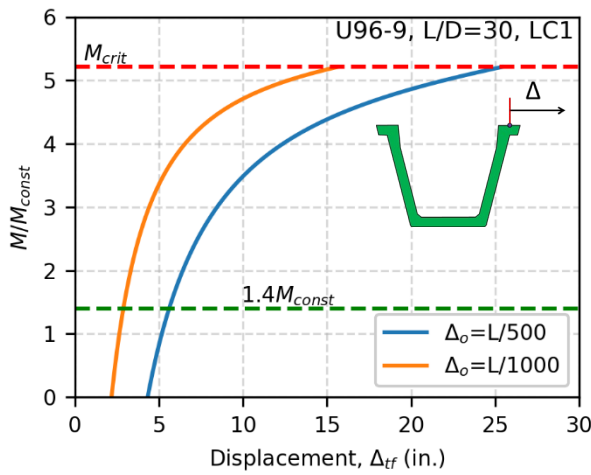


Figure 5.49. Displacement measurement of U-girder in the large displacement analysis.

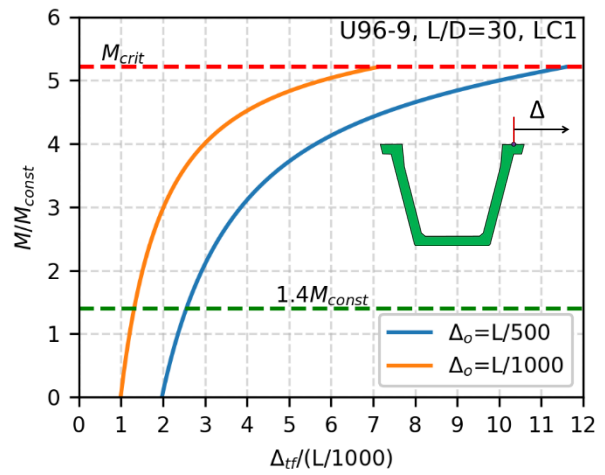
Figure 5.50 presents the load–displacement curves, where the bending moment is normalized by the expected construction-stage bending moment (M_{const}), specifically during deck placement. Displacement is shown as the actual movement of the top flange, enabling engineers to understand

the expected girder deflection during construction. Additionally, normalized displacement is provided, where displacement is scaled by the initial imperfection of $L/1000$.

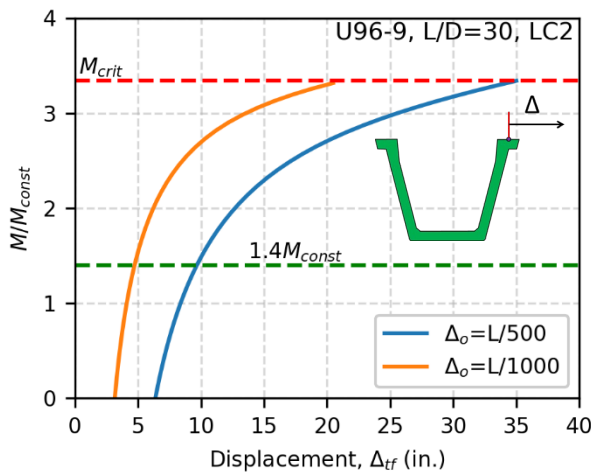
The results show that the critical buckling moment (M_{crit}) exceeds M_{const} in both load cases. Specifically, the M_{crit} is approximately a multiple of $5.2 M_{const}$ for Load Case 1, and a multiple of $3.3 M_{const}$ for Load Case 2. At the design moment ($1.4 M_{const}$), the additional lateral displacement is approximately 30% of the initial imperfection for Load Case 1 and 50% for Load Case 2. Based on these results, the RT concludes that the stability of the U-girder during deck placement is not a concern. The RT recommends, however, to maintain the PCI limit on the initial imperfection of approximately $L/1000$ (the actual value is $L/960$) to avoid constructability issues or excessive girder displacement during deck placement.



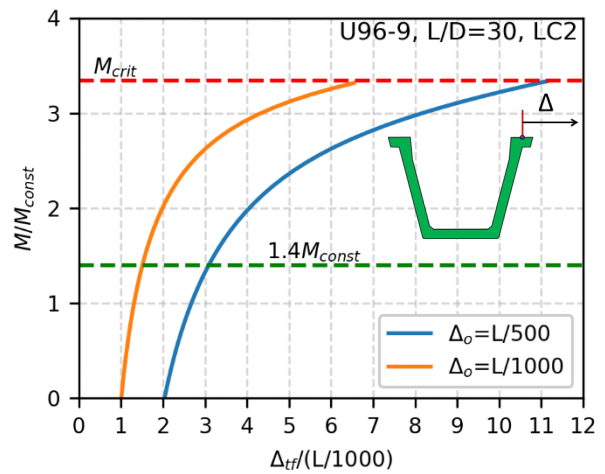
(e) Displacement of Load Case 1



(f) Displacement ratio of Load Case 1



(g) Displacement of Load Case 2



(h) Displacement ratio of Load Case 2

Figure 5.50. Load-displacement curve of U-girder with initial imperfections

5.4.2.2. Effect of Loading Condition

The analysis in the previous section was based on the assumption that the analyzed girder is an interior girder, with the slab dead load distributed equally between the two top flanges, as shown in Figure 5.51 (Girder A). However, unsymmetric loading on an exterior girder (Girder B) was investigated, since additional torsion is introduced under this condition. In this study, a single-girder line model was adopted, with modeling assumptions consistent with the previous analysis, including model details and boundary conditions. The only difference between Girder A and Girder B lies in the loading condition: Girder B experiences reduced loading on one of the top flanges due to a smaller tributary slab width. As in the previous analysis, two initial imperfections— $L/500$ and $L/1000$ —were examined. The initial imperfection was applied in a manner that increases torsion in the girder when both imperfection and unsymmetric loading are considered simultaneously. Note that only Load Case 2 is considered in this study as that is the critical load case.

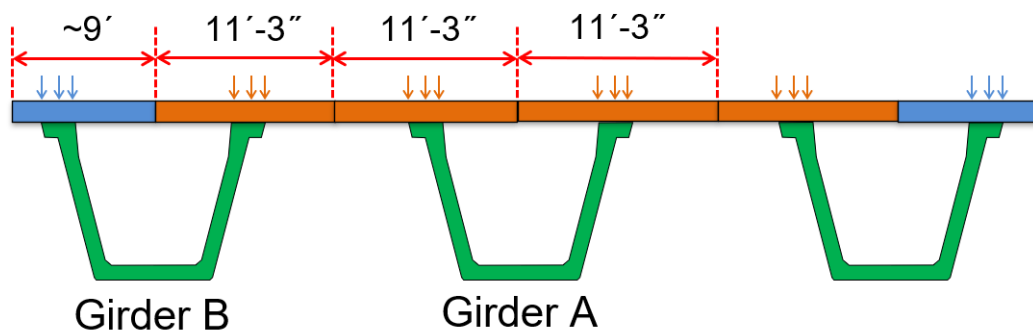


Figure 5.51. Load-displacement curve of U-girder with initial imperfections

Figure 5.52 compares the load–displacement curves for symmetric loading (Girder A) and unsymmetric loading (Girder B). The critical buckling moment of Girder B is slightly higher than that of Girder A, most likely due to the reduced load on one of the top flanges due to the overhang dimensions. However, the lateral displacement of Girder B (dashed line) is greater than that of Girder A due to the additional torsion caused by the unsymmetric loading. At the design moment ($1.4M_{const}$), Girder B exhibits approximately 20–30% greater lateral displacement than Girder A. Nevertheless, the additional displacement remains below twice the initial imperfection, indicating that the overall lateral deformation is still within acceptable limits. Therefore, the research team believes that stability is not a concern, even under unsymmetric loading. However, minimizing the initial imperfection is recommended to avoid excessive displacement of the girder during construction.

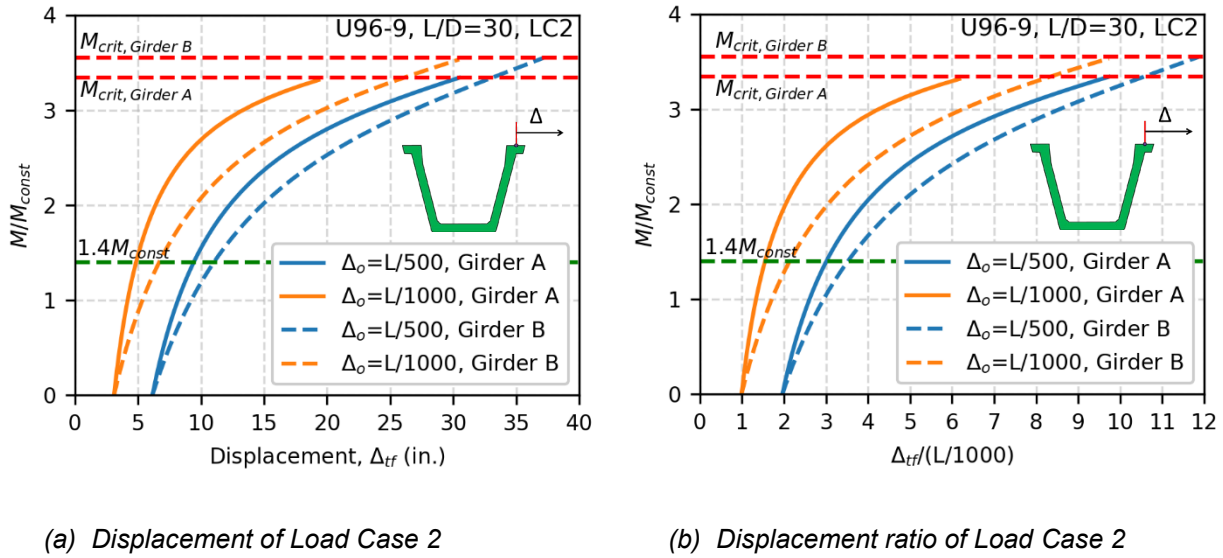
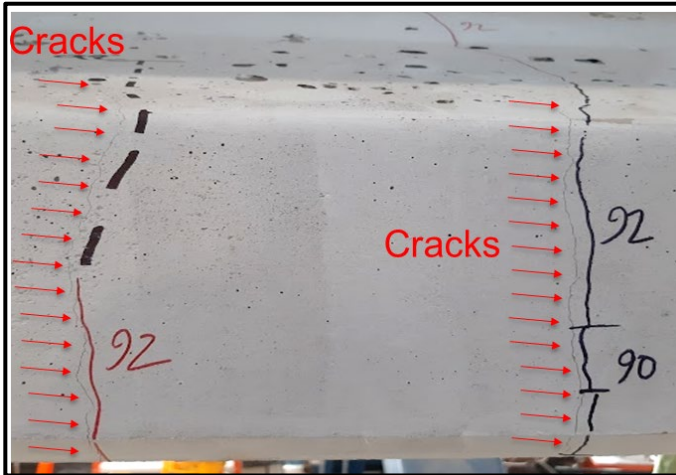


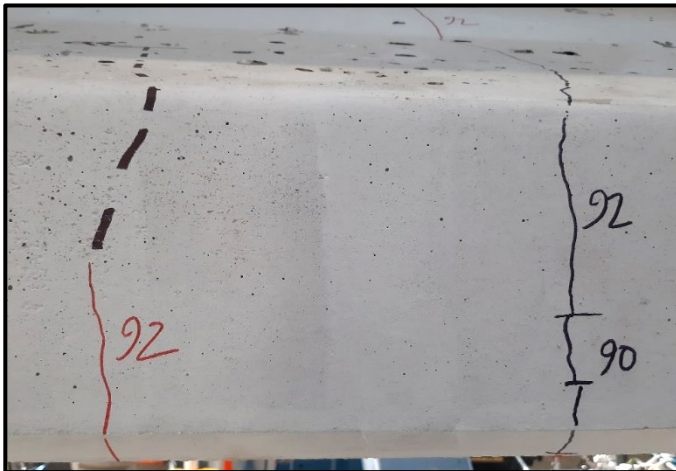
Figure 5.52. Comparison of Load-displacement curves between symmetric and unsymmetric loading

5.4.3. Impact of Cracking on Stability

In order to evaluate the impact of cracking on the stability performance of bridge systems, further investigations were conducted. While *TxDOT* and *AASHTO LRFD* specify allowable stress limits during fabrication and Service III load combinations, further investigations are essential given the potential role of cracking on system stability, load-bearing capacity, and deformability during construction. Cracking is an inherent characteristic of concrete and may occur in prestressed concrete bridge girders during lifting, transport, and erection. In most instances, cracks may not be visible to the naked eye due to the prestressing effect. For example, in Figure 5.53, the tests conducted on the Tx34 specimen, the experimental results on the load-deflection response, as well as strain gauge data, indicate the significant influence of cracking on the response of the girders. Along the same lines, experimental results on the hysteretic response of the girders (e.g., Figure 4.9) elucidate the potential role of cracking in damage accumulation, leading to a reduction in the load-carrying capacity of the girders in subsequent cycles, specifically a reduction in cracking moment, followed by significant stiffness degradation and increased deformability.



(a) Crack initiation and propagation in loaded state are clearly visible under load.



(b) Crack closure in unloaded state

Figure 5.53. Elevation view of the Tx34 girder bottom flange showing crack initiation, propagation, and closure during testing

The specific numerical methodologies pursued in this portion of the study rely on eigenvalue buckling and nonlinear incremental analyses. The primary objective of the eigenvalue buckling analysis is to determine the critical loads that lead to the elastic buckling of the system. The resulting eigenvalues, which characterize the system elastic buckling capacity, serve as multipliers of the reference load during construction (system self-weight and slab dead load). The objective of the incremental analysis, is to evaluate the progressive response of the system to the applied loading. While eigenvalue buckling analyses capture the system elastic response, incremental analysis can be customized to account for nonlinear material and geometric effects.

5.4.3.1. Stability Analysis Results for Sections with Cracking

Considering the results of the parametric study, the stability investigation with cracking focuses on the critical cases for each bridge girder type. The first girder system under investigation features the STx7 section. As shown in Table 5-9, the critical load scenario for this section corresponds to the second load case, in which the slab load is applied to both the exterior and interior spans. In addition, the stability investigation with cracking is based on a single-girder line system with an L/D ratio of 30, as the corresponding section in the multi-girder system demonstrates a higher eigenvalue and is therefore less prone to elastic buckling in that configuration, as shown in Table 5-11. An important consideration pertains to the implementation of cracking in the system. The numerical approach relies on experimental observations from the Tx28 experiment to account for cracking. In particular, as shown in Figure 4.30, crack propagation in the specimen extended approximately halfway through the section height – at the girder center of gravity – along a portion of the span length. While this level of cracking is significant and care is generally taken to prevent such cracking during construction, the numerical scheme adopts this representation to ensure a conservative assessment due to the impact of a stability failure.

The first step in implementing cracking in the model involved determining the critical location where the maximum (or minimum) bending moment occurs. As an initial approach, cracks were introduced in the positive bending moment regions of each individual span, by significantly reducing the material stiffness in the crack region to simulate the properties of a cracked section. The crack length was set to 20% of the span length – 10% on either side of the critical location – and cracks were uniformly distributed up to the section center of gravity, with the tension-stiffening effect disregarded, as shown in Figure 5.54.

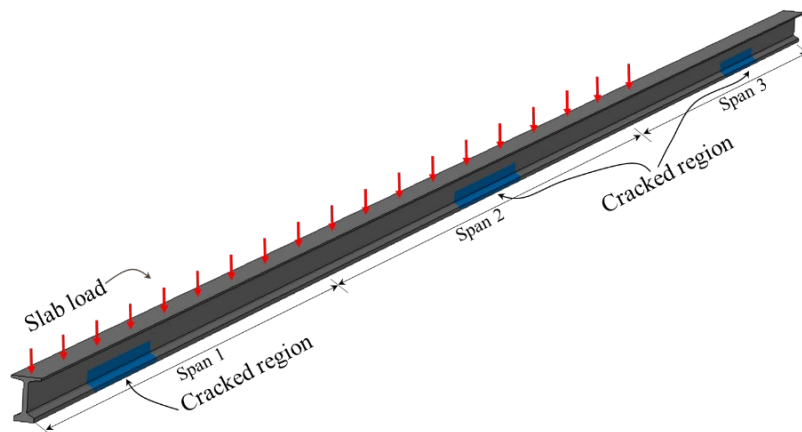


Figure 5.54. Crack Implementation plan in positive bending moment region of an STx7 single girder line

The implementation of cracking in the model revealed an important observation regarding the redistribution of bending moments. Specifically, the introduction of cracks altered the system

stiffness, resulting in a shift in the moment distribution, as shown in Figure 5.55. For models utilizing cracked section properties, the magnitude of the moment in the negative bending region at interior supports increased significantly and the moments in the positive bending region decreased. The consequence of this alteration in moment distribution is a change in the critical moment, which is discussed in detail later.

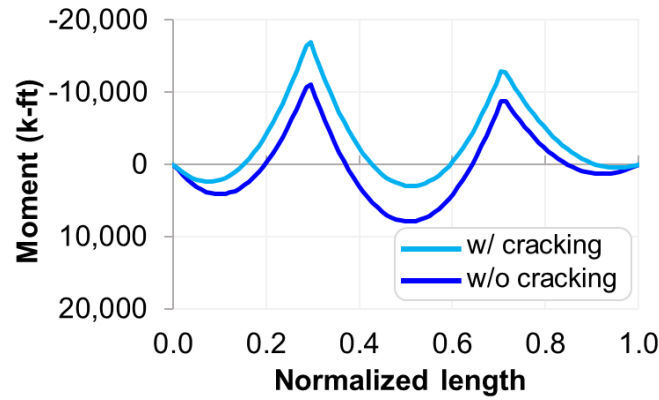


Figure 5.55. Impact of cracking in positive bending moment region of STx7 system on moment distribution

The results of the eigenvalue buckling analyses for the models with and without cracking are shown in Figure 5.56. Both cases incurred buckling in the interior (middle) span, with eigenvalues of 5.0 and 6.6 for models without and with cracking, respectively. The mode shapes of the models with and without cracking differ significantly. In the model without cracking, the majority of the deformation is restricted to the midspan, whereas the model with cracking buckles in both the first and second spans. In order to determine the critical moment for each case, the eigenvalues are multiplied by the moment at the maximum induced deformation, as specified by the eigenvector. Although the eigenvalue of the model with cracking was higher than that of the model without cracking, the redistribution of the moment due to cracking caused a reduction in the moment capacity in the positive moment regions. Therefore, the critical moment for the model without cracking was measured at 39,388 k.ft, while the corresponding value for the model with cracking was 19,731 k.ft. The observations made for this system can be extended to other section configurations.

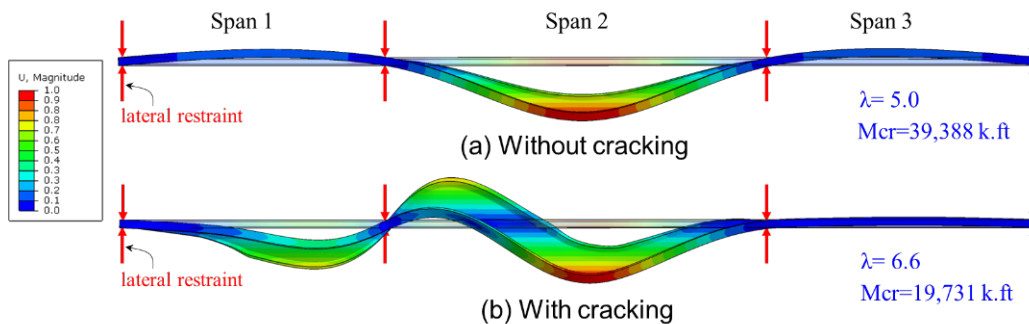


Figure 5.56 Contour plot for the first eigenvector (mode shape) of STx7 bridge system (plan view)

Although the STx7 model only made use of simulated cracks in the positive moment region, during lifting, transport, and erection, the concurrent occurrence of cracks in both regions is more likely, even if not visible to the naked eye. Since crack propagation causes moment redistribution along the span through non-uniform stiffness variations, quantifying redistribution effects in both positive and negative bending regions is essential. Therefore, the next phase of the study accounted for cracks in both positive and negative bending regions.

The cracks in the positive bending regions were assumed to occur within the middle 20% of the span length (i.e., 10% on either side of the critical section where the maximum moment occurs), while those in the negative bending moment regions were assumed within 10% of the span length (i.e., 5% on either side of the critical section). The crack depth in the positive bending regions was held constant, extending from the bottom flange to the section elastic center of gravity, whereas the crack depth in the negative bending moment regions varied across different cases, as shown in Figure 5.57. The basis of this presumption is the likelihood of crack propensity along the span, and also the potential role of simultaneous cracking on system stability. The crack depths in the negative moment were varied to illustrate the critical behavior of deep cracks in the negative moment region.

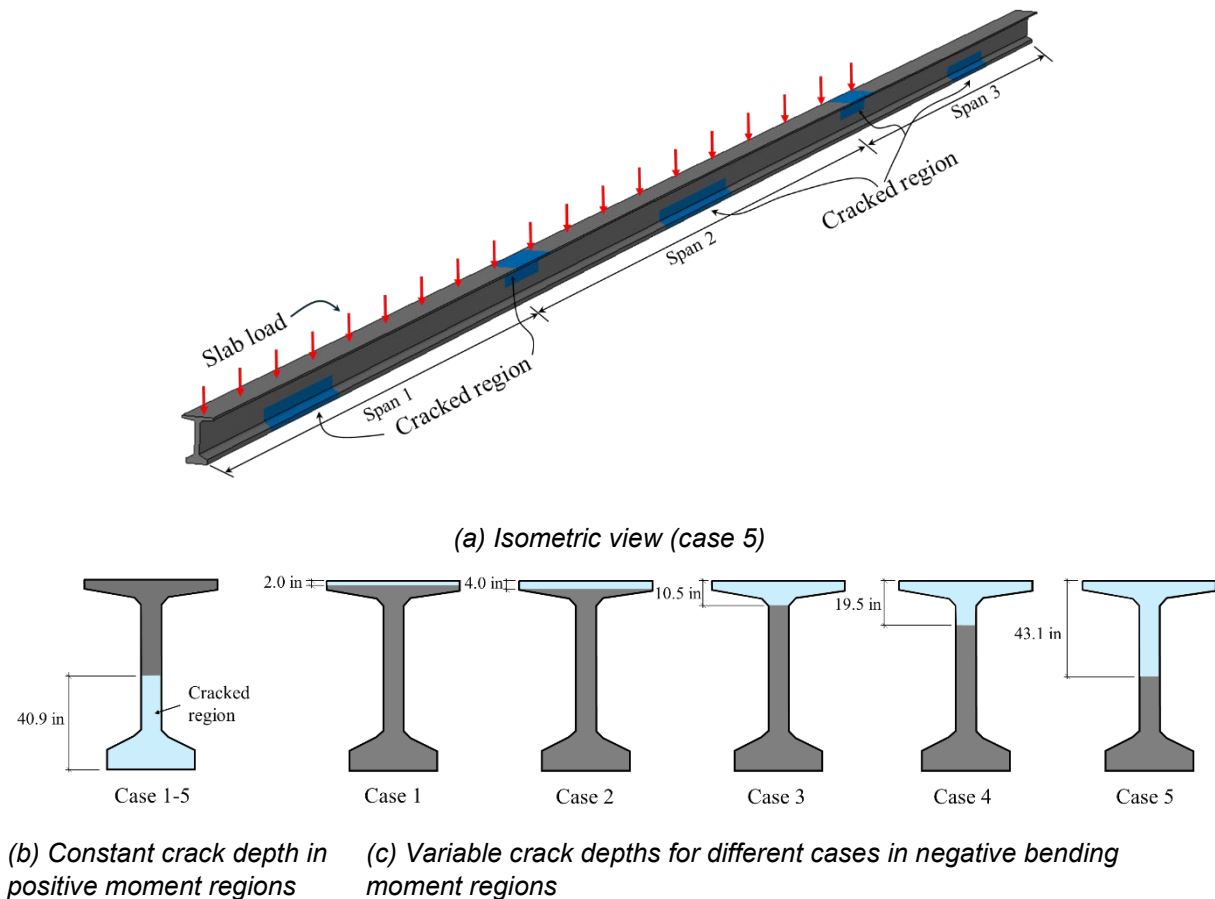


Figure 5.57. Crack implementation plan for STx7 bridge system (cracked regions shown in light blue)

Figure 5.58 shows the distribution of the moment along the span for the system featuring the STx7 section. As mentioned earlier, the crack depth in the positive moment region was maintained constant for all cases, while the corresponding crack depth in the negative bending moment region varied across five cases, extending through the top flange and web regions, as highlighted in the figure. The distribution of the moment for cases without cracking is also shown in the Figure 5.58 to provide a baseline for the data.

The introduction of cracking leads to stiffness variations along the length, which in turn cause a shift/redistribution in the bending moment. When the depth of the crack in the negative bending moment region is minimal, the maximum stiffness change is reached, resulting in a reduction of moment in the positive bending moment region and an increase in moment magnitude in the negative bending moment region – corresponding to the stiffness contribution. As the crack depth in the negative bending moment region increases, this shift in the moment diagram – relative to the initial distribution – gradually diminishes. Subsequently, when the crack depth reaches the center of gravity in both negative and positive bending moment regions, the stiffness alterations effectively cancel out, and the moment distribution nearly coincides with the initial moment distribution without cracks, as shown in the Figure 5.58.

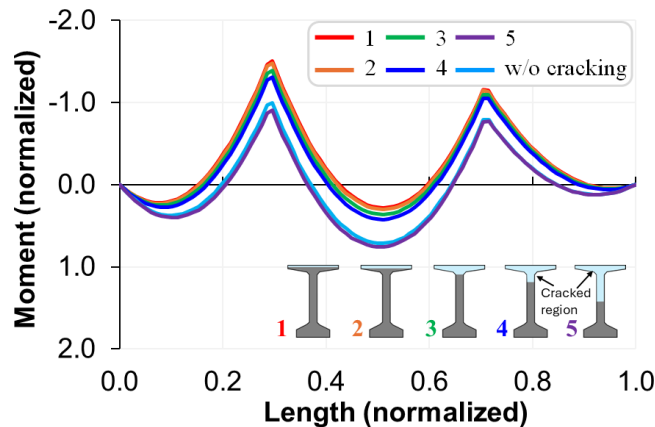


Figure 5.58. Distribution of moment with various crack depths in negative moment region for STx7 bridge system

The initial step in the stability appraisal of the system with cracking involves conducting eigenvalue buckling analyses, which provide an estimate of the elastic response. Figure 5.59 shows the results of the eigenvalue buckling analyses for systems featuring STx7 section with different levels of cracking in the negative bending moment region. As the crack depth in the negative bending moment region increases, the eigenvalue – serving as a multiplier on the reference load level – reduces, leading to a corresponding reduction in the system capacity.

A significant reduction in capacity occurs when cracks extend to the center of gravity in both the positive and negative bending moment regions, at which point the eigenvalue decreases to $\lambda=3.0$.

Still, for this configuration, the stability of the system is not compromised as the system shows the capacity to withstand construction loads by a factor of three ($\lambda=3.0$). On the other hand, considering the capacity of the system with crack depth penetrating halfway through the top flange portion ($\lambda=6.6$), the capacity of the system is reduced by 55%. This reduction is closely related to the moment redistribution along the length across different cases, as shown in Figure 5.59. In particular, as the depth of the crack in the negative bending moment region increases, the redistribution of the moment due to the cracking diminishes, and the magnitude of the moment in the positive moment region (demand), where the critical eigenvector occurs, increases.

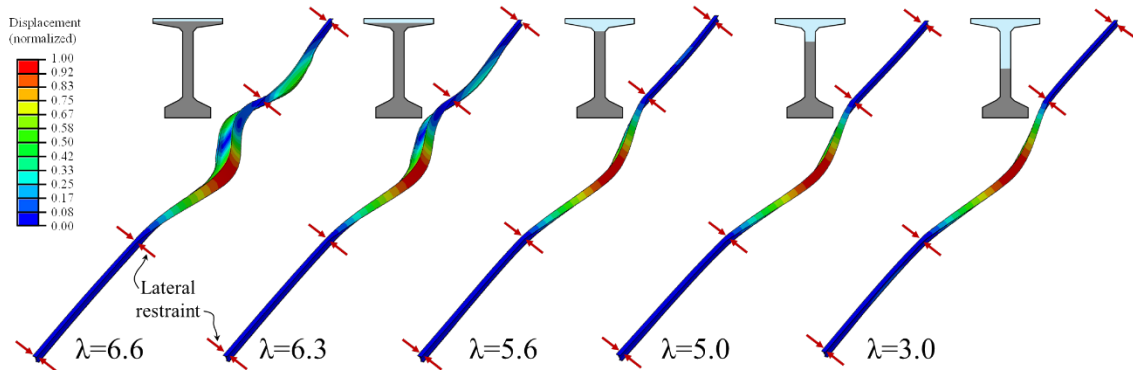
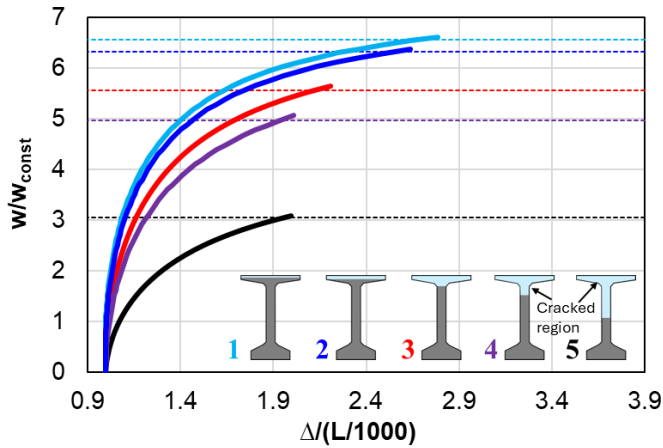


Figure 5.59. Impact of crack depth in negative bending moment region on eigenvalue buckling analysis results for STx7 bridge system

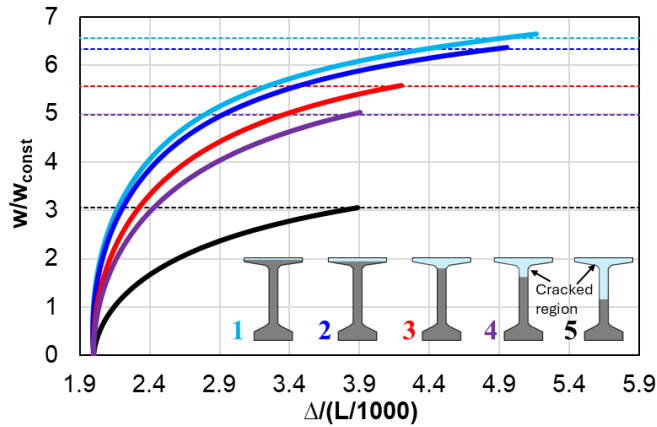
The stability assessment also included an evaluation of the system response through an incremental analysis under different imperfection levels – $L/1000$ and $L/500$ – while accounting for the nonlinear geometric effects associated with large deformations, a characteristic absent in the eigenvalue buckling analysis. Figure 5.60 shows the response of the STx7 system to varying crack depths in the negative bending moment region under different imperfection levels. The term Δ denotes the relative lateral displacement (sweep) between the top and bottom flanges, which is normalized with respect to the initial imperfection level of $L/1000$. The analyses are carried out up to the critical moments predicted from the eigenvalues (multiplier of the applied loads that causes buckling) obtained from the eigenvalue buckling analyses, as indicated by dashed lines.

The depth of the crack in the negative bending region had a significant impact on the system response. Specifically, crack penetration beyond the flange region led to a gradual increase in deformation for a given load level. Figure 5.60 also highlights the sensitivity of the system to different imperfection levels, with higher imperfections subjecting the system to greater deformation demands at a given load. The expected deformation at the construction load level ($w/w_{const} = 1.0$) was minimal for most cases, except for the final case, where it reached approximately 8% of the corresponding initial imperfection level. The deformation in the latter exponentially increases with an increase in the construction load level. This stipulates that a level of caution should be regarded when crack depth extends beyond the flange regions in both positive

and negative bending moment regions. The likelihood of cracks reaching well into the web during construction are obviously unlikely; however, the impact of the cracking on the behavior shows interesting behavior.



(a) Imperfection of $L/1000$



(b) Imperfection of $L/500$

Figure 5.60. Incremental response of STx7 bridge system with different imperfection levels and crack depths in negative bending moment region

The crack implementation configuration for the system featuring the STx9 section is similar to that of the STx7, with cracks introduced within the middle 20% of the span length in the positive bending moment regions, and within 10% of the span length in the negative bending moment regions. The depth of the cracks in the positive bending moment regions was kept constant, extending from the bottom flange to the section center of gravity, while the depth in the negative bending moment regions was varied across five different cases, as shown in Figure 5.61.

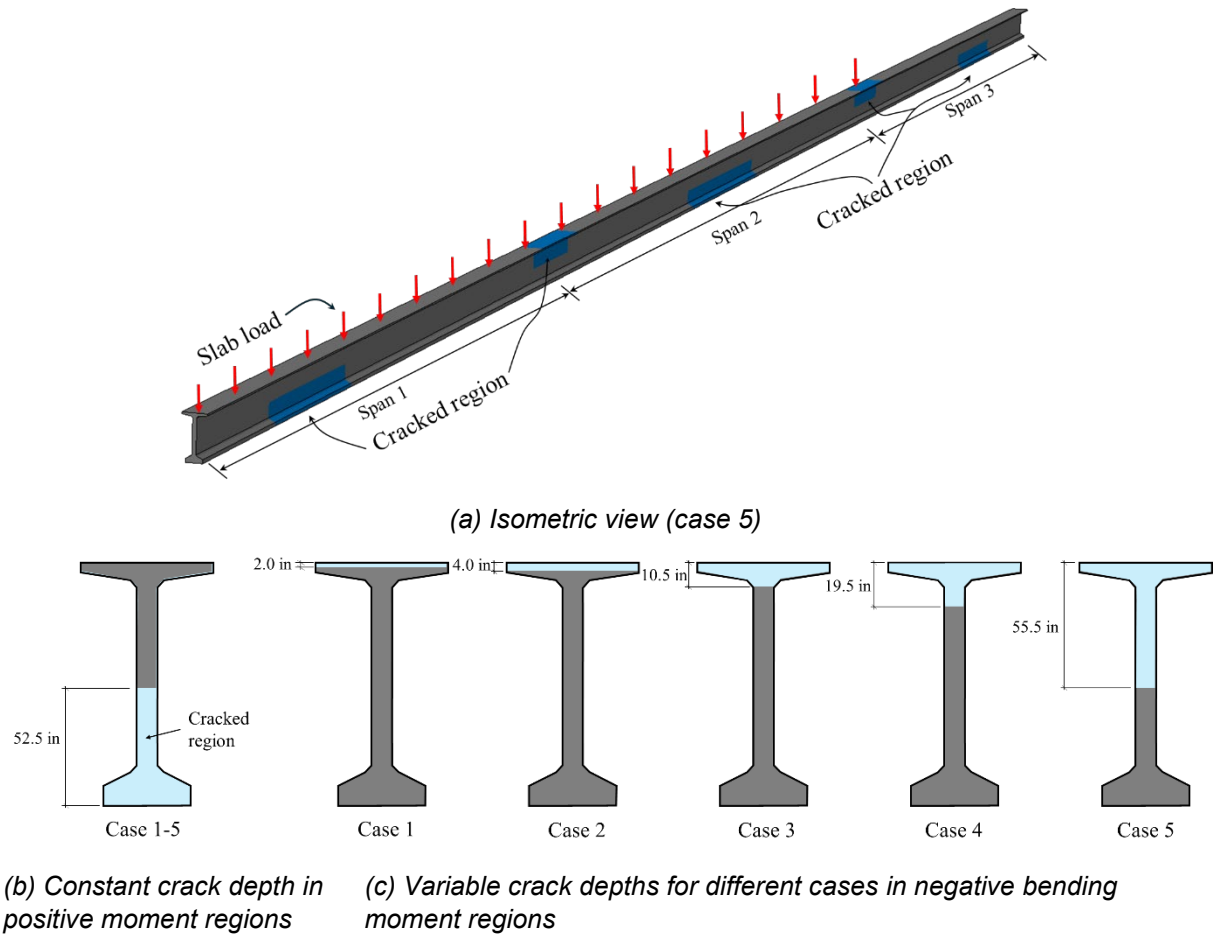


Figure 5.61. Crack implementation plan for STx9 bridge system (cracked regions shown in light blue)

The distribution of the moment for cases with varying crack depths in the negative moment regions is shown in Figure 5.62. Similar to the observations for the STx7 system, changes in the crack depth result in the redistribution of the moment along the system length. As the crack depth in the negative bending region increases, the moment magnitude in that region decreases, while the moment in the positive bending moment region increases. This observation stems from the crack implementation, which alters the system stiffness along the spans. The moment redistribution – relative to the initial uncracked condition – is effectively eliminated when the crack depth in both the negative and positive bending regions extends to the center of gravity. On the other hand, the shift in the moment diagram is expected to impact the system stability, given the altered demand in the critical span (interior) across various cases.

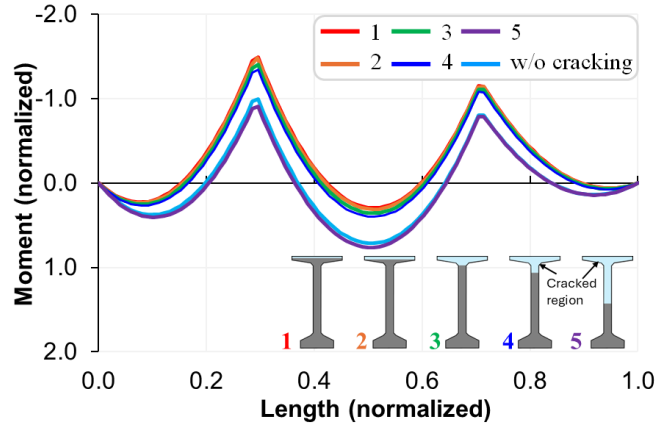


Figure 5.62. Distribution of moment with various crack depths in negative moment region for STx9 bridge system

The results of the eigenvalue buckling analysis for the STx9 system with variable crack depth in the negative bending moment region are shown in Figure 5.63. Consistent with previous observations, the stability of the system was influenced by the crack depth in the negative bending moment region. As the crack depth in the negative bending moment region increases, the eigenvalue decreases, indicating a reduction in system capacity. The most critical case corresponds to the final configuration, in which the crack depth in the negative bending moment region extends to the section center of gravity, resulting in an eigenvalue of $\lambda=1.7$. The system still retains the ability to withstand the construction load by a factor of 1.7 for this configuration.

Considering the cases under investigation, the capacity of the system is reduced by up to 50% for the initial and final configurations with eigenvalues of $\lambda=3.4$ and $\lambda=1.7$. The rate of capacity reduction is especially pronounced between the fourth and fifth configurations, corresponding to a notable shift in the moment diagram observed for the final case, as illustrated in Figure 5.62. The preliminary results from the eigenvalue buckling analysis underscore the significant role of cracking in reducing system capacity, highlighting the importance of limiting crack propagation during construction.

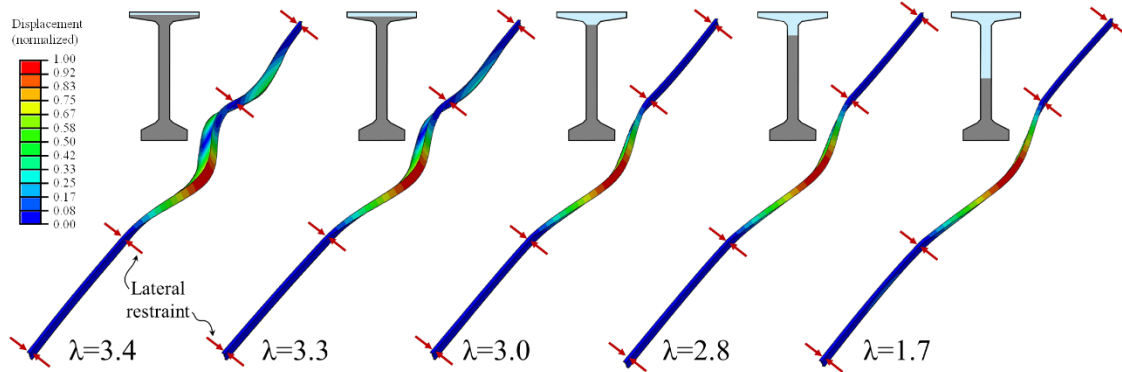
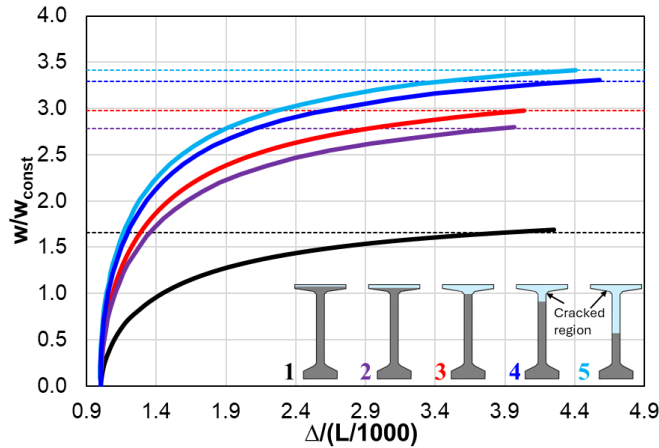


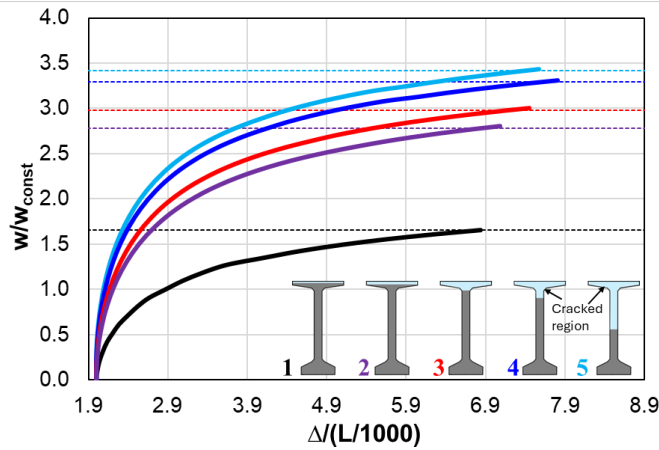
Figure 5.63. Impact of crack depth in negative bending moment region on eigenvalue buckling analysis results for STx9 bridge system

The incremental response of the STx9 bridge system was subsequently evaluated through large deformation computations that include nonlinear geometric effects. Figure 5.64 demonstrates the sensitivity of the system to varying crack depths in the negative bending moment region, considering two imperfections levels of $L/1000$ and $L/500$. The reported deformation represents the relative lateral translation between the top and bottom section flanges at the middle of the interior span, normalized with respect to $L/1000$. The analyses are carried out up to the critical moments predicted from the eigenvalues (load proportionality factors) obtained from the eigenvalue buckling analyses, as indicated by dashed lines.

The depth of cracks in the negative bending moment region was found to play a crucial role in the stability performance of the system, with deeper cracks resulting in greater deformations. Compared to the system with the STx7 section, the reduction in capacity of the STx9 systems was more pronounced, reflecting the geometric configuration of the latter. At the construction load level ($w/w_{const} = 1.0$), the deformation of the systems remained similar for the first four cases; however, it significantly increased for the final case, where crack depths in both positive and negative bending moment regions extended to the section center of gravity. In particular, the deformation at the construction load level for the final case reached approximately 45% of the initial imperfection. Although assuming cracks extending to the center of gravity in both regions is conservative, this finding emphasizes the importance of maintaining minimal crack depths – ideally limiting maximum penetration to the top flange – in the negative bending moment region. The overall system response was not significantly influenced by the two imperfection levels considered; however, at a given load level, higher imperfection levels resulted in greater deformation.



(a) Imperfection of $L/1000$



(b) Imperfection of $L/500$

Figure 5.64. Incremental response of STx9 bridge system with different imperfection levels and crack depths in negative bending moment region

The stability of the system featuring the U96-9 girder section – reflective of reduced-scale experimental tests on U-section girder – was also investigated. The implementation of the cracking for this specific system followed a methodology similar to that for the I-section systems. In particular, concurrent cracking in both the positive and negative bending regions was considered. Cracks in the positive moment region were assumed to occur within the middle 20% of the span (10% on either side of the critical section), while those in the negative moment region were confined to 10% of the span (5% on either side of the critical section). The crack depth in the positive bending region was maintained constant and assumed to extend from the bottom flange to the sections center of gravity, while the corresponding depth in the negative bending region was varied across four cases, ranging from the top flange down to the section center of gravity, as shown in Figure 5.65.

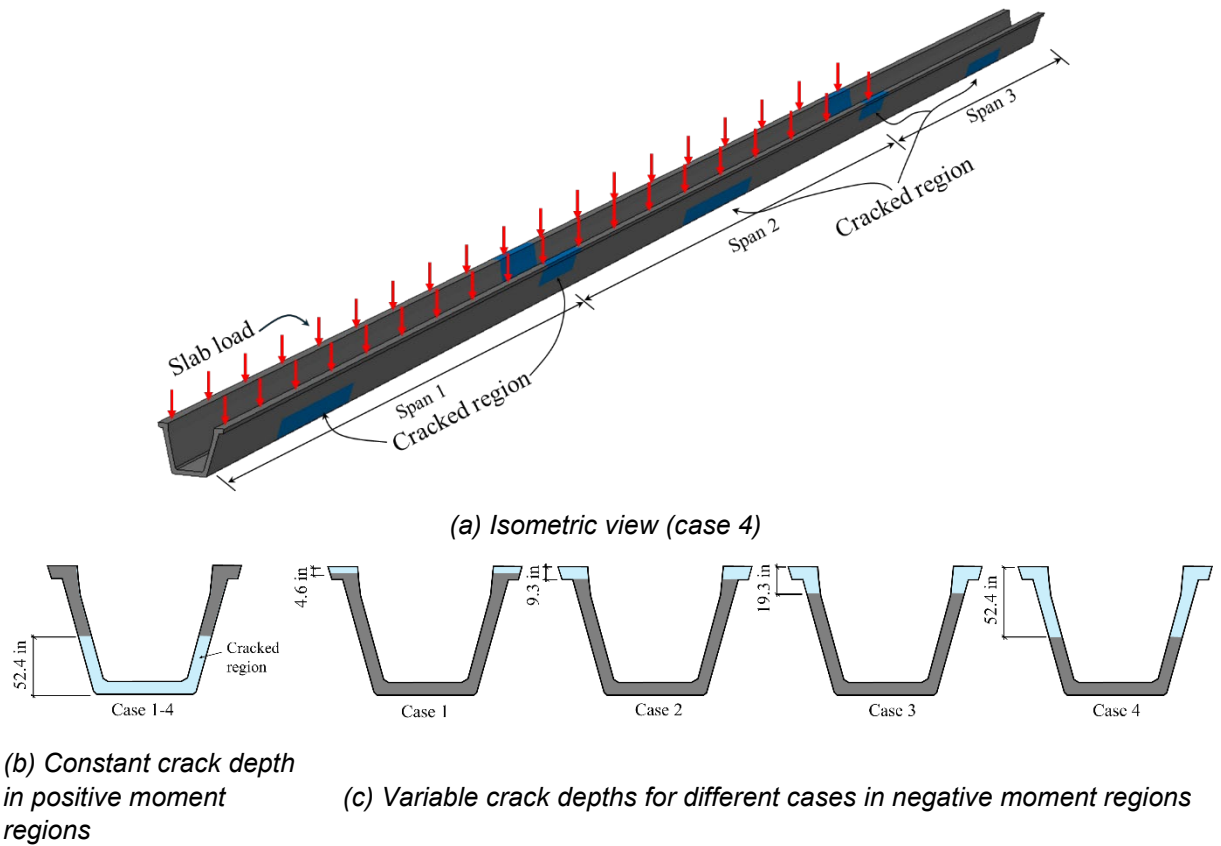


Figure 5.65. Crack implementation plan for U96-9 bridge system (cracked regions shown in light blue)

The distribution of the moment for systems featuring the U96-9 section with variable crack depths is depicted in Figure 5.66. The introduction of cracks in negative bending moment region affected the moment distribution along the span. As the depth of the crack increases in this region, the moment magnitude in the negative bending region decreases, while the moment in the positive bending region increases. The moment redistribution continues to evolve with increasing crack depth, and once the cracks extend to the center of gravity in both the positive and negative bending regions, the redistribution caused by cracking is effectively nullified. This results in moment distributions that are approximately similar for the fourth configuration and the uncracked case, as illustrated in Figure 5.66.

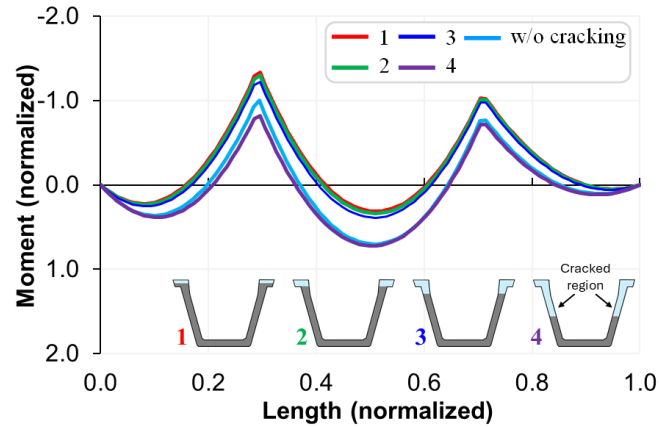


Figure 5.66. Distribution of moment with various crack depths in negative moment region for U96-9 bridge system

Figure 5.67 shows the results of the eigenvalue buckling analyses for the U96-9 bridge systems with varying crack depths in the negative bending region. The capacity of the specimen diminishes as the crack depth in the specific region increases. The maximum reduction in capacity occurs in the final configuration, where cracks extend to the center of gravity in both the positive and negative bending regions, resulting in an eigenvalue of $\lambda=1.9$. This indicates that the system was still capable of withstanding the construction load by a factor of 1.9. Considering the cases under investigation, the capacity of the system is compromised by more than 56%, based on first and final configurations with eigenvalues of $\lambda=4.4$ and $\lambda=1.9$. Compared to the I-shaped systems, the capacity reduction was more pronounced in the U96-9-section system, due to the open configuration of the system cross-section. On the other hand, the results highlight the role of cracking in reducing system capacity and emphasize the importance of maintaining minimal crack depth during construction.

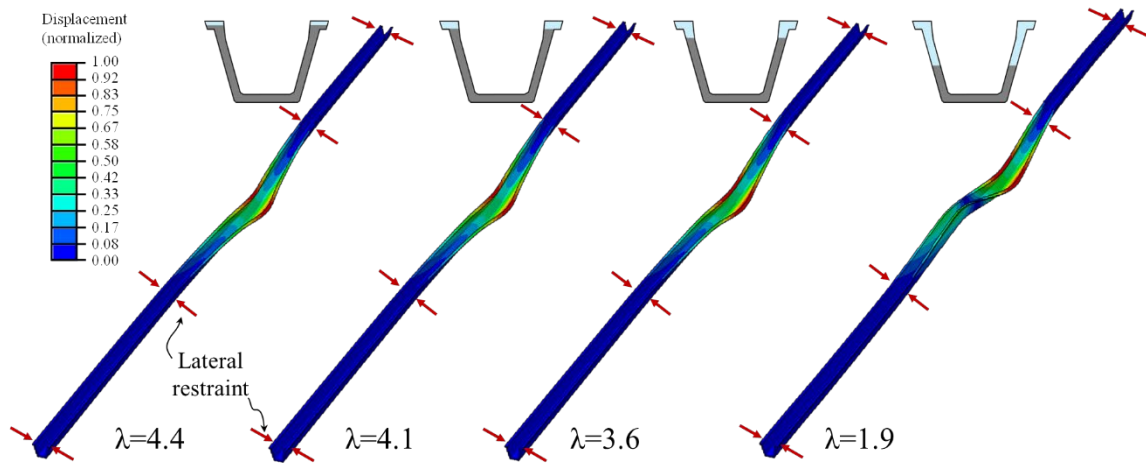


Figure 5.67. Impact of crack depth in negative bending moment region on eigenvalue buckling analysis results for U96-9 bridge system

Additional efforts devoted to assessing the stability of the U96-9 bridge system involved large-deformation incremental analyses accounting for nonlinear geometric effects. The incremental response of the U96-9 system differs from that of the I-shaped section in that, under the influence of cracking, the top flanges exhibit lateral deformation in opposing directions – a behavior primarily attributed to the open nature of the cross-sectional configuration. This phenomenon results in cross-sectional distortion of the system at the midspan of the interior span, particularly when cracks extend to the section center of gravity in both the positive and negative moment regions, as illustrated in Figure 5.68. The deformation illustrated in the figure represents the lateral displacement (sway) of the top flanges, normalized by $L/1000$. At the construction load level ($w/w_{const}=1.0$), the maximum deformation exceeds 75% and 89% of the initial imperfection value of $L/1000$, as deduced from the figure.

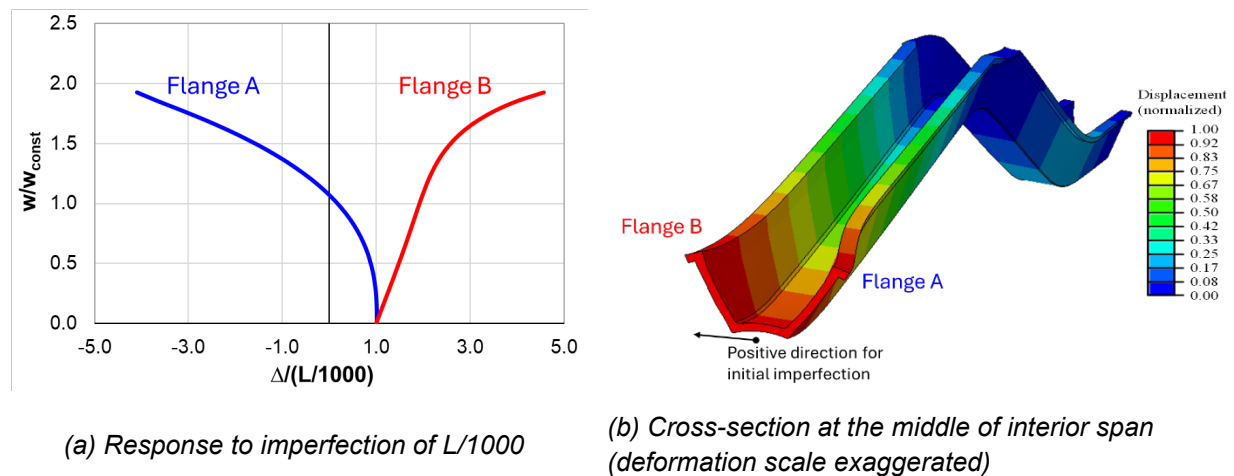
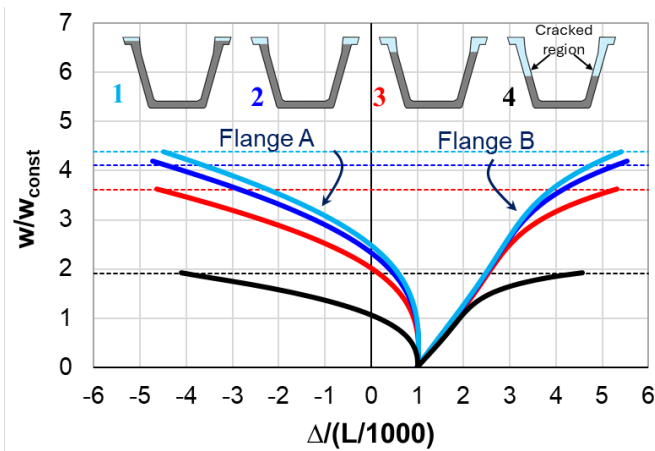
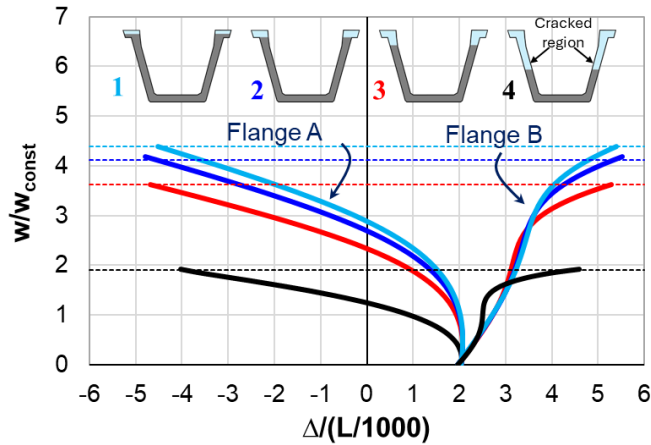


Figure 5.68. Incremental response of U96-9 bridge system with crack penetration depth extending to the center of gravity in positive and negative bending moment region

The incremental response of the U96-9 girder system to varying crack depths in the negative bending region is demonstrated in Figure 5.69. The response of both top flanges is presented for two imperfection levels, $L/1000$ and $L/500$. The analyses are carried out up to the load proportionality factors corresponding to the eigenvalues, as shown with dashed lines in the figures. Among the cases under investigation, the local distortion of the flange *B* remained limited up to the construction load level ($w/w_{const}=1.0$) for the first three configurations. In contrast, flange *A* exhibited more pronounced local distortion at the same load level. This behavior is attributed to the open cross-section configuration of the system, which renders it more susceptible to unequal deformation following crack initiation. In particular, once the bottom flange of the section is cracked, the top flanges tend to respond independently, particularly under higher load and deformation levels.



(a) Imperfection of $L/1000$



(b) Imperfection of $L/500$

Figure 5.69. Incremental response of U96-9 bridge system with different imperfection levels and crack depths in negative bending moment region

5.4.3.2. Implementation of Stiffness Reduction Factor to Account for Cracking

Thus far, the implementation of cracking has been directly (i.e., explicitly) accounted for in the modeling methodology – specifically in regions encompassing 10% and 20% of the span lengths subjected to negative and positive bending actions at given sectional depths. An interesting – yet necessary – area of research pertains to the capability of reproducing the cracked analysis results in an indirect (i.e., implicit) manner, without the need to directly accounting for cracking.

Hence, the final phase of this subsection focuses on an implicit cracking implementation methodology, with the primary objective of establishing reduction factors for cracked systems that can be adopted for designers and engineers without the need to explicitly account for cracking. In that regard, the initial elastic modulus of the material is calibrated and held constant along the entire length of the bridge system. Concrete is assumed to remain uncracked; however, the reduction in the system elastic modulus is intended to deliver results commensurate with those obtained from models that explicitly incorporate cracking.

The indirect cracking implementation plan for the bridge systems relies on the analysis results of systems with cracking in two primary regions. In particular, to offset the moment redistribution effects that may influence the load resistance of the systems, the analyses focused on models with similar cracking configurations in the positive and negative bending regions.

Figure 5.70 presents the eigenvalue buckling analysis results of the STx7 bridge systems employing both direct and indirect cracking implementation methodologies. As shown in Figure 5.70(b), the analysis of the indirect cracking implementation indicates that a 19% reduction in the elastic modulus of the concrete material is sufficient to reproduce the analysis results of the system with direct cracking implementation, when the crack depth is limited to the flanges. This corresponds to a reduction factor of 0.81 imposed on the initial intact model to replicate the cracked behavior. The indirect cracking implementation depicted in Figure 5.70(d) corresponds to the case where the crack penetration depth extends to the center of gravity. A reduction of 39% in the elastic modulus was deemed optimal for delivering results comparable to those of the direct cracked model. This corresponds to a reduction factor of 0.61 on the initial intact model to reproduce the cracked properties.

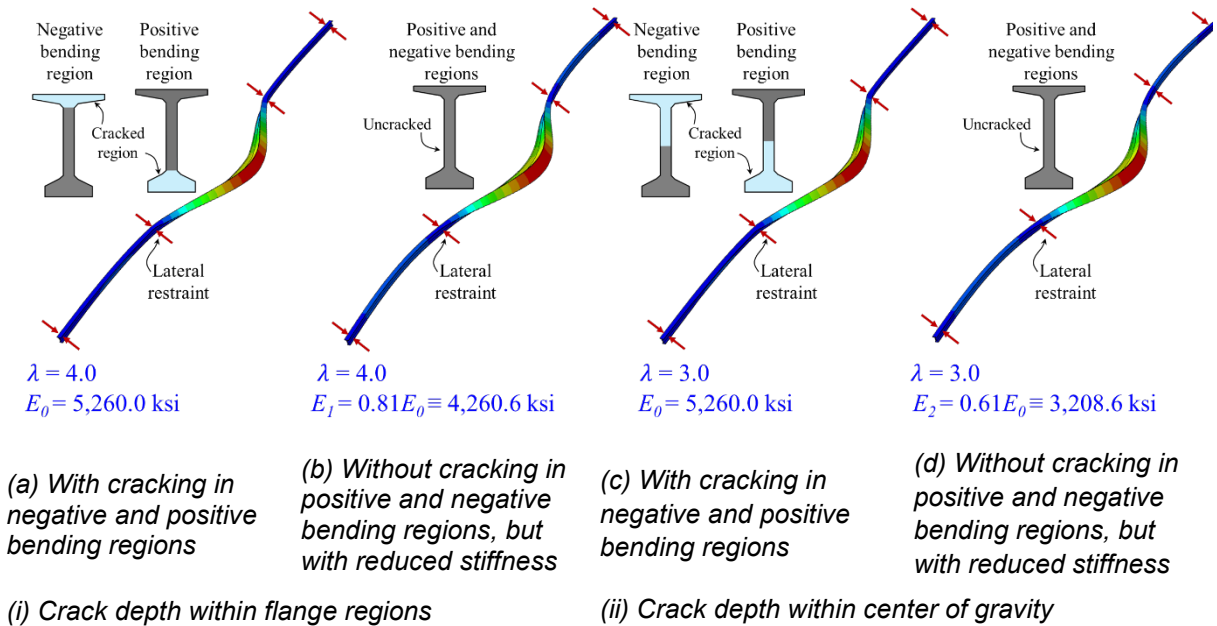


Figure 5.70. Eigenvalue buckling analysis results for models with direct (a, c) and indirect (b, d) cracking implementations for STx7 bridge system

The eigenvalue buckling analysis results of the STx9 bridge systems using direct and indirect cracking implementation approaches are presented in Figure 5.71. The analysis of the indirect cracking implementation, as shown in Figure 5.71(b), indicates that a 20% reduction in the elastic modulus of the concrete material is sufficient to reproduce the results of the system with direct cracking implementation, when the crack extends only within the flange zones. This corresponds to a reduction factor of 0.80 applied to the initial intact model to replicate the cracked properties. The indirect cracking implementation depicted in Figure 5.71(d) corresponds to the model with crack penetration depth extending to the center of gravity. A 37% reduction in the elastic modulus was deemed optimal for delivering results similar to those of the direct cracked model, corresponding to a reduction factor of 0.63 applied to the initial intact model.

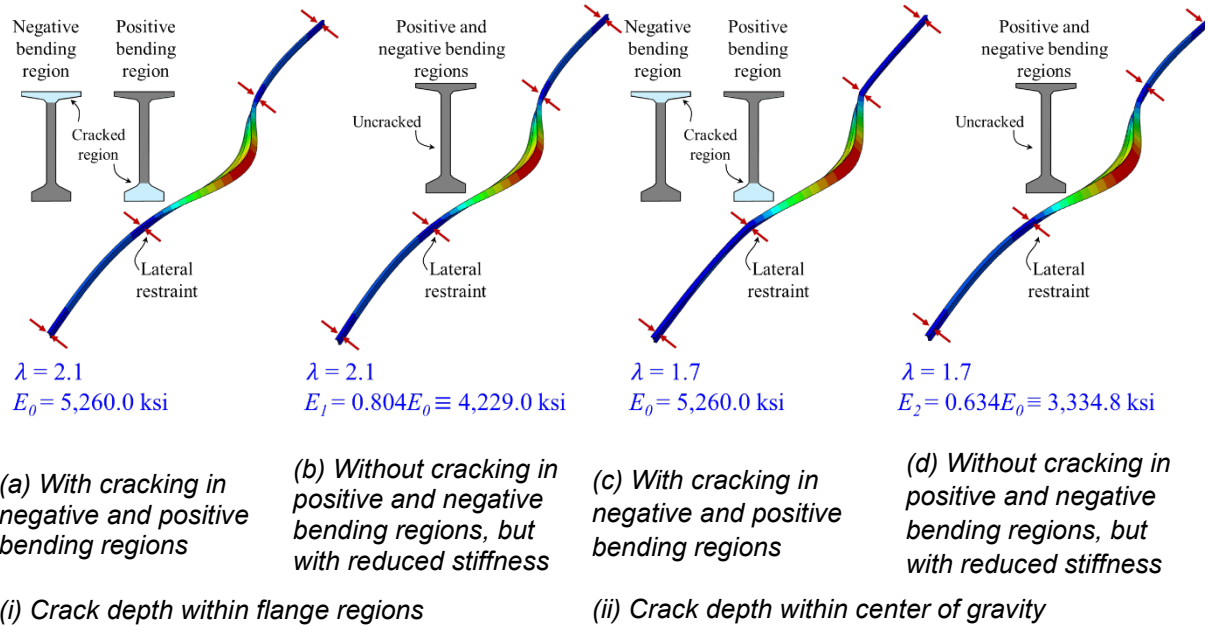


Figure 5.71. Eigenvalue buckling analysis results for models with direct (a) and indirect (b) cracking implementations for STx9 bridge system

Figure 5.72 presents the eigenvalue buckling analysis results of the U96-9 bridge systems incorporating both direct and indirect cracking implementation approaches. As shown in the results of the indirect cracking implementation plan in Figure 5.72(b), an 11% reduction in the elastic modulus of the concrete material proved sufficient to reproduce the analysis results of the system with direct cracking implementation, when crack depth restricted to the flanges. This reflects a reduction factor of 0.89 on the initial intact model with the objective of replicating the cracked behavior. Figure 5.72(d) demonstrates the results of the indirect cracking implementation, where the crack depth extends to the center of gravity. A reduction of 45% in the elastic modulus was found to effectively reproduce the response of the direct cracked model. This corresponds to a reduction factor of 0.55 applied to the initial intact model to simulate the cracked behavior. The only potential discrepancy between the models is related to the deformed shape of the model with crack depth extending to the center of gravity Figure 5.69(d), which was not found to affect the outcome of the results – eigenvalues.

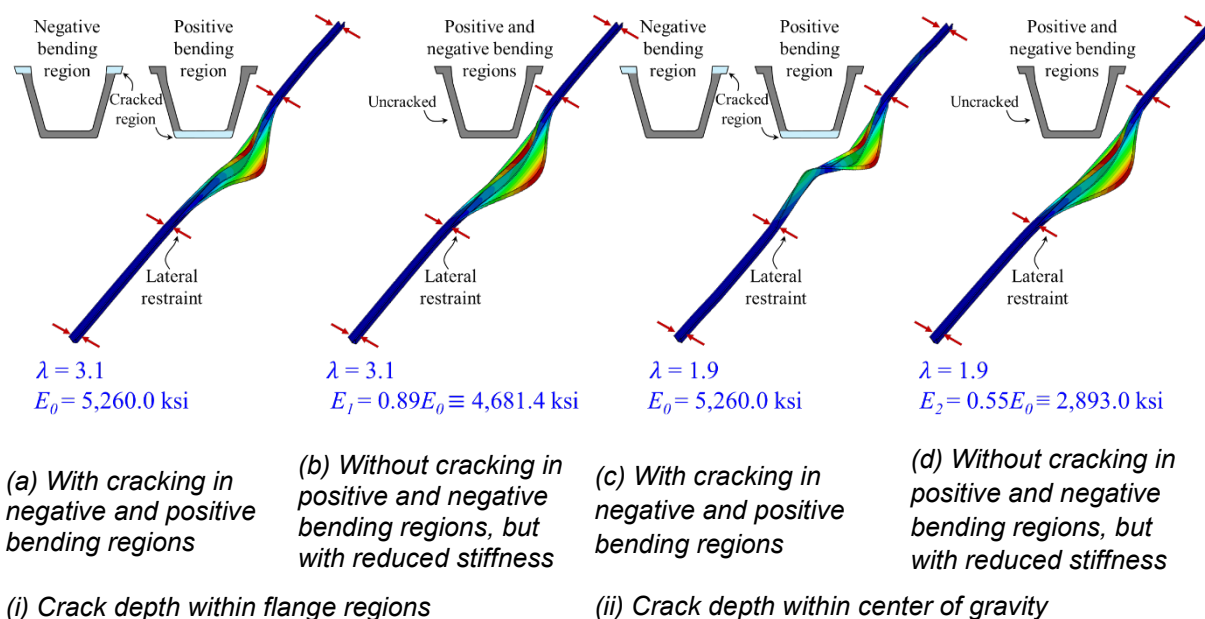


Figure 5.72. Eigenvalue buckling analysis results for models with direct (a) and indirect (b) cracking implementations for U96-9 bridge system

In summary, the simplified cracking implementation approach was found to satisfactorily reproduce the analysis results of the cracked sections. Based on the analysis of different bridge system configurations, the recommended capacity reduction factors for I-shaped systems are 0.80 and 0.60 when cracks are limited to the top flanges and the center of gravity, respectively. For U-shaped systems, the recommended capacity reduction factors are 0.80 and 0.50 for cracks limited to the top flanges and the center of gravity, respectively.

Chapter 6. Parametric FEA Studies on Live Load Distribution

The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2024) have long made use of live load distribution factors (LLDF) to convert line girder analysis results to the corresponding proportion of moments and shears the girder is subjected to. Zokaie et al. (1991) conducted a detailed parametric analysis on a variety of bridge geometries to develop distribution factors for moment and shear. The work generally focused on typical interior and exterior girders. The presence of design vehicles in multiple lanes has been incorporated into the distribution factors. The distribution factors in AASHTO (2024) are based upon a specified range of applicability that considers many factors such as bridge cross-section type, girder spacing, span length, number of girders, and the girder's longitudinal stiffness.

There are several aspects of spliced concrete girder bridges that are outside of the range of the original development of the live load distribution factors. Thus, there is interest to see if they are still applicable to spliced-concrete girder bridges outside of the parameters' range. Many of the geometric ranges that might exist in spliced girder bridges need to be evaluated with respect to the original studies upon which the factors were based. Spliced girder bridges generally fall outside the range of the original developed factors in the following ways: 1) Many of the spans exceed the maximum 240 ft. 2) The bridges often include non-prismatic (haunched/tapered) girders, and the original factors used constant sections. 3) The longitudinal stiffness of splice girder bridges exceeds the maximum stiffness used in the original factors. The original factors also did not consider the presence or effects of cross-frames or diaphragms in their development. The contribution of the diaphragms to live load distribution is also unclear in spliced girder bridges, since permanent diaphragms are only currently provided at splices and support regions. The impact of the different diaphragm layouts on the transverse stiffness of the bridge is evaluated through parametric finite element analyses.

6.1. Finite Element Modeling

The finite element (FE) models in the parametric study were developed in Abaqus based on the validation studies conducted in the previous chapters. The bridge girders act as a system along with the deck slab and the diaphragm during service. The girders and diaphragms were modeled using linear solid elements with reduced integration (C3D8R), and the deck slab was modeled using linear shell elements with reduced integration (S4R). The live load analysis was initially conducted on a simply supported bridge due to its simplicity for verification purposes. A three-dimensional representation of the FE model of a constant-depth spliced prestressed concrete I-girder bridge is shown in Figure 6.1. The interactions between the deck slab and the girders and between diaphragms and girders were modeled as rigid without any slip at the interfaces, simulating the full composite behavior of the girder and deck slab. The rigid connection assumption was defined using *Tie Constraints* assigned between the faces of diaphragms, girders,

and deck slab at the interface locations. The tie constraints are shown in the transverse view of the bridge model in Figure 6.2.

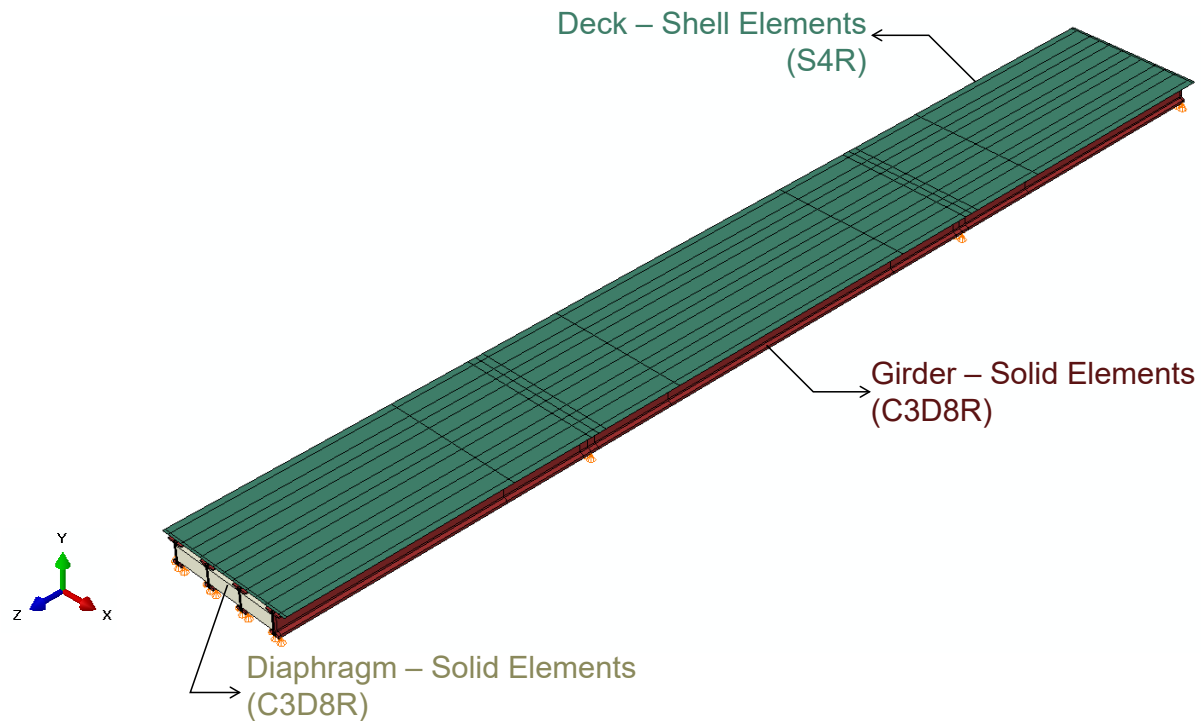


Figure 6.1. Finite element model of a constant-depth spliced prestressed concrete I-girder bridge

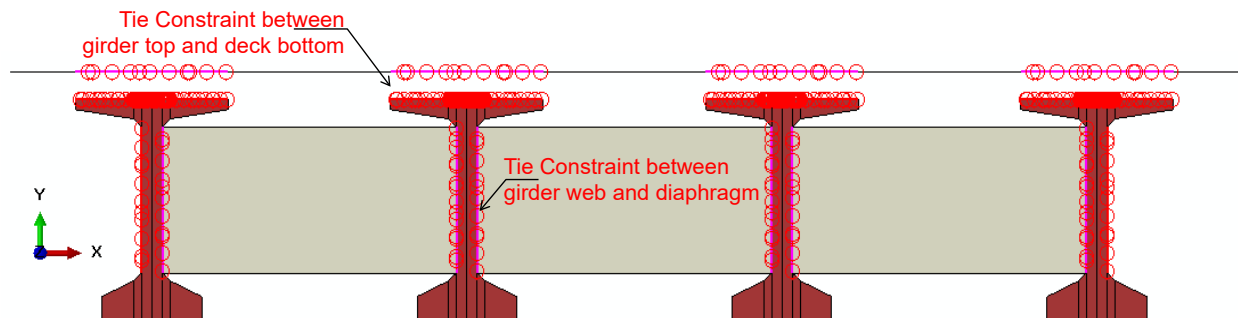


Figure 6.2. Elevation view showing tie constraints in the FE model

The concrete material property was defined as linear elastic for all the components. The stress levels during service are expected to be within the elastic limit and are consistent with the AASHTO specifications. The linear-elastic, first-order analysis was performed using the *Linear Perturbation Analysis* method in Abaqus.

6.1.1. Live Load Analysis

Zokaie et al. (1991) considered several positions of the truck loads and identified the critical positions from the preliminary analysis that were applied in the refined analysis models. However,

the procedure may be tedious in continuous bridges, especially when load distribution is dependent on the three-dimensional behavior of the bridge. This procedure involves stepping the vehicle longitudinally and transversely on the bridge deck and is more commonly referred to as “brute-force” approach (Adams et al. 2019). The computation effort of the bridge live load analysis is very high in a refined FE model when considering different vehicle configurations, varying rear axle spacing of the design truck, several combinations of vehicles and lane loads, and the number of loaded lanes. The number of load cases can be very large depending on the live load parameters and bridge geometry. Hence, it is not preferable to analyze continuous prestressed concrete bridges using the brute-force method.

6.1.1.1. Influence Surface Method

Influence lines are used in line-girder analyses to identify and apply live loads at the critical positions. The influence lines provide the critical positions in the longitudinal directions only. Since the bridge is assumed to be a single line-girder, the variation of force effect for different transverse positions is not known. When performing two-dimensional or three-dimensional analyses, influence surfaces are very helpful to visually represent the variation of a force effect along the length and width of a bridge. An influence surface is a three-dimensional plot generated by plotting the force effect at a particular point in the bridge for unit loads applied at different positions. An influence surface is similar to an influence line but applicable for two-dimensional configurations, thereby considering both longitudinal and transverse positions of wheel loads. The influence surface can be represented as a three-dimensional plot where the horizontal plane of the plot represents the bridge deck plane where the live loads are applied. The vertical ordinate at a given point on the horizontal plane represents the value of the force effect when a unit load is applied at the point on the deck surface. The vertical ordinate is also called the influence ordinate. Several commercial bridge analysis programs perform live load analysis using influence surfaces. Past research studies (Hays et al., 1989; Terzioglu et al., 2017; Okumus et al., 2018) have used the same approach for studying bridge behavior under live loads. Grubb et al. (2011) provided an example of live load analysis using influence surfaces in a curved and skewed continuous steel I-girder bridge.

The influence surface can be generated according to the Muller-Breslau principle by applying unit displacement or rotation at a particular point (point of interest) after removing the restraint for the displacement or rotation. The displaced shape is the influence surface. The ordinates of an influence surface represent the effect a specified unit load has on the force of interest for different positions. This method of deriving the influence surface requires a modification of the model (removing restraints) for each point of interest and desired response (Adams et al., 2019).

A relatively easier approach to obtain influence surfaces is by stepping a unit load in both longitudinal and transverse directions. This method is similar to the brute-force approach, but only a single unit load is required, therefore reducing the total number of load cases. In this approach, the unit load is applied at different positions along the length and width of the bridge, and the responses are recorded for each position. At the end of the analysis, the desired responses at nodes

or in elements are recorded. The measured response for a particular position of the unit load provides the influence surface ordinate at that position. The influence surface can be generated by mapping the response with the load positions on the deck surface. The computational effort is less compared to the “brute-force” approach because the analyses need to be performed only for unit load at several positions. The calculation of live load effects can be carried out mathematically by interpolation and integration of the influence surface for a design vehicle load. The influence surface approach relies on superposition and is applicable only to linear-elastic structures. It cannot be used with material nonlinearity, or when uplift occurs at supports (nonlinear boundary conditions).

6.1.1.2. Application of Influence Surface Method

A simply supported bridge with a span length of 168 ft was chosen for the development of live load analysis methodology. The bridge model consisted of four Tx84 girders spaced at 12 ft, an 8.5-in. thick deck, and an overhang width of 4.5 ft. This model is referred to as the verification model in this report. A Python script was developed to generate the FE model, apply loads, and analyze and extract results. The unit load was applied at 121 different positions on the deck surface. The loading points were equidistant along the length of the bridge between the lines of support. In the transverse direction, the load was applied along the curb edges, girder center lines, and the midpoints between them. The load positions considered in the verification model are shown in Figure 6.3.

The boundaries for the load positions were the support lines and the curb edges. The curb edges were assumed to be at 1 ft distance from the free edge, considering a concrete railing of 1-ft width at its base. The centerlines of the girders are denoted as G1, G2, G3 and G4. The left and right edges of the deck slab are denoted as L and R, respectively. The forces in the composite girders were extracted for every $1/10^{\text{th}}$ distance from the supports. The support reactions were also recorded to study the shear-reaction distribution of live loads.

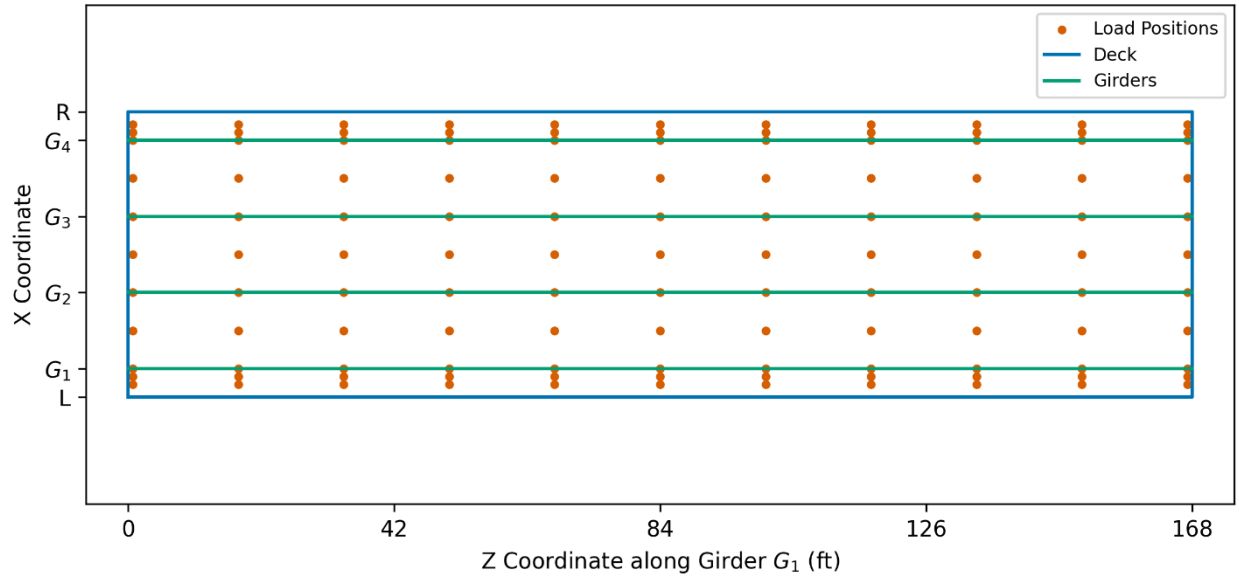


Figure 6.3. Plan view showing load positions for generating influence surfaces in the verification model

6.1.1.3. Verification of an Influence Surface

An influence surface was generated for the midspan bending moment in girder G₁ by mapping the output to the load coordinates. The influence surface is shown as a contour plot in Figure 6.4. The contour plot was developed by linearly interpolating the ordinates between the 121 load points. The values were not extrapolated beyond the boundaries of the load positions, and the ordinates were considered as zero because there are no force effects for loads acting outside the defined boundary. The influence surface shows that the refinement of load positions is sufficient for the verification model.

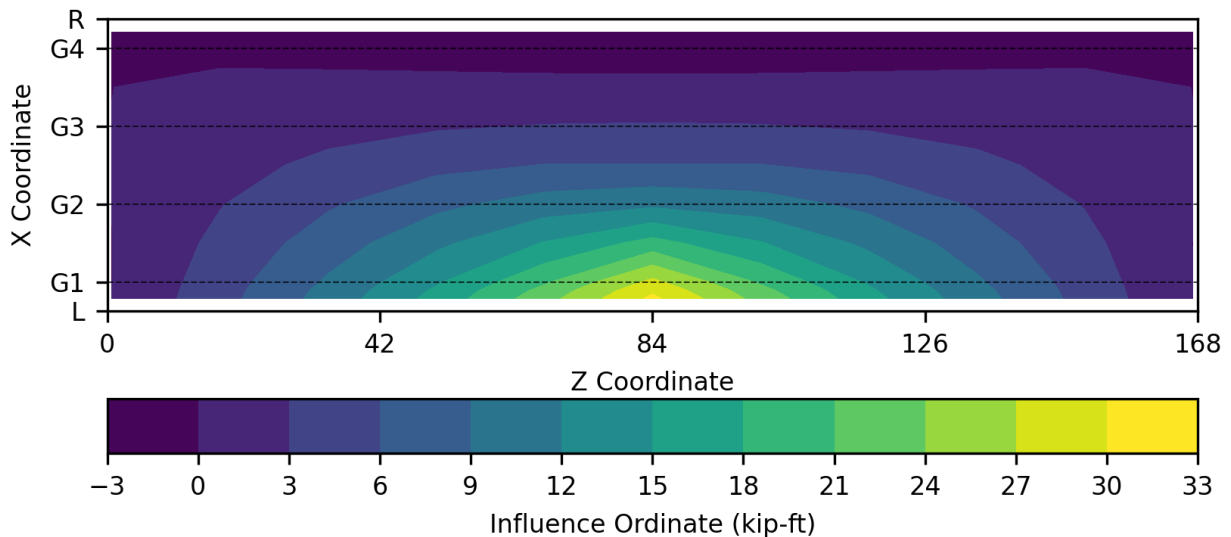


Figure 6.4. Plan view of influence surface for midspan bending moment in girder G₁ in the verification model

The influence surface was also verified by cutting sections along the longitudinal and transverse directions of the deck. The cuts in influence surface along the girder centerline approximately represent influence lines for the girders considering them as one-dimensional elements (line-girders). The influence surface cuts were made along the lines shown in Figure 6.5 and the resulting influence lines are plotted alongside the Figure 6.5.

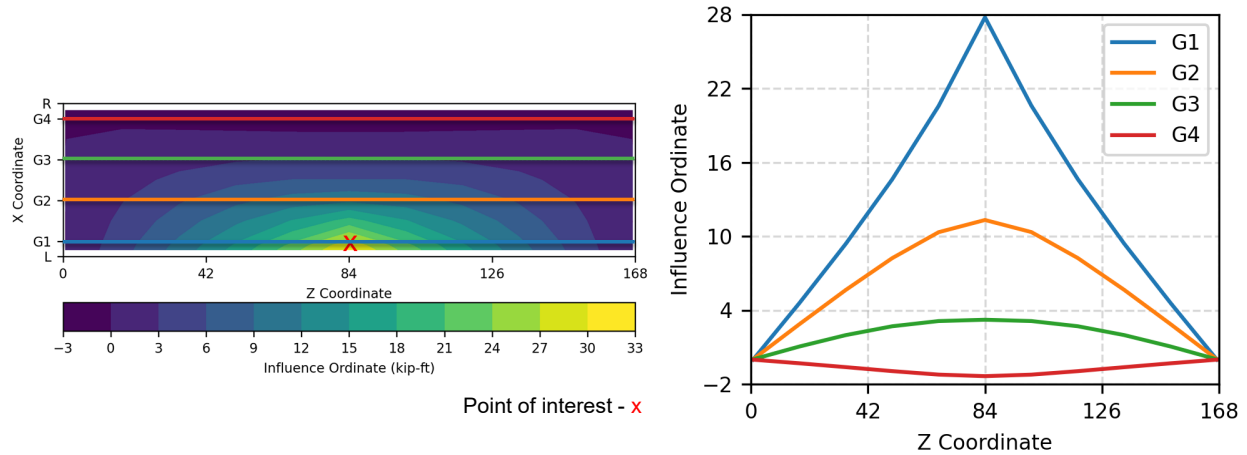


Figure 6.5. Influence lines along girder centerlines for midspan bending moment in girder G1 in the verification model

The influence surface was also cut in the transverse direction at the quarter points of the bridge length. The locations of the cut lines are shown in Figure 6.6 alongside the plot of influence ordinates at the cut lines. The influence lines along the transverse cut lines show higher influence when the load is applied in the overhang regions and lower influence as the load is applied further away from the girder G1. The reduced influence also indicates the participation of other girders for farther load positions. The influence lines along the first and third quarters of the length are identical as expected in a simply supported bridge for midspan bending moment due to symmetry.

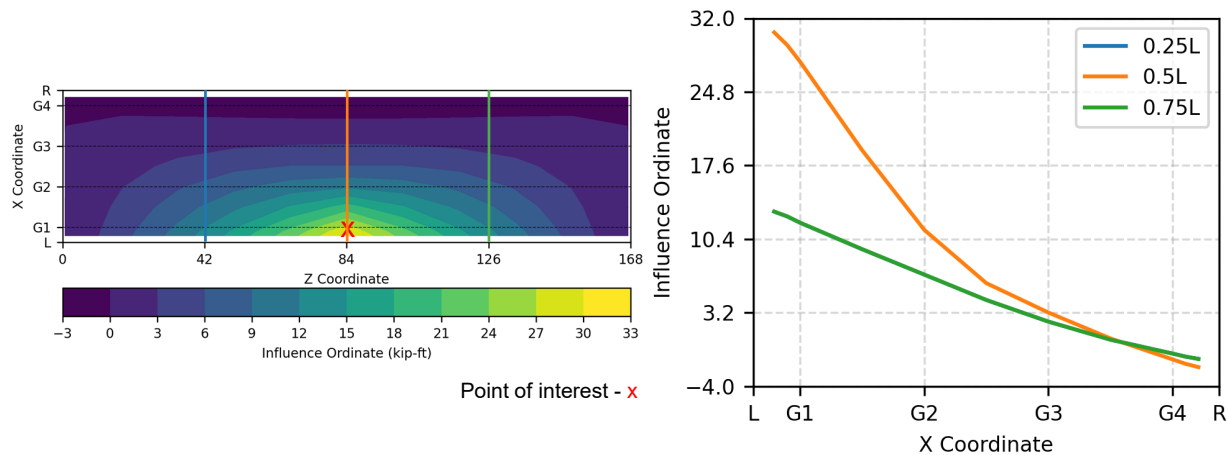


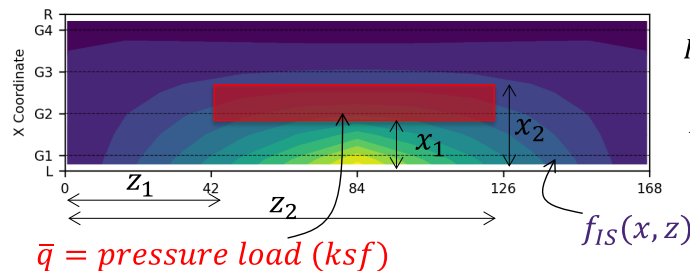
Figure 6.6. Influence lines along transverse lines at quarter points for mid-span bending moment in girder G1 in the verification model

6.1.1.4. Live Load Optimization

Live load analysis using an influence surface requires the appropriate positioning of the vehicle to result in the extreme force effects in the component being analyzed. The load positions may be known and intuitive for relatively simple bridges, but the exact location of the vehicle that maximizes or minimizes the force effects is difficult to estimate for most continuous bridges. Adeli and Ge (1989) presented a dynamic programming method for live load analysis of statically determinate and indeterminate bridges. The researchers demonstrated the procedure using influence lines and extremum functions to determine load positions for maximizing force effects. Some commercial bridge analysis programs (Autodesk 2024, LARSA 2022, LUSAS 2024, MIDAS 2024) also adopt load optimization algorithms along with the influence surfaces to find the extreme effects of live loads. The literature on load optimization is very limited. So, in place of load optimization algorithms, a “brute-force” method was used for identifying the critical live load positions.

The AASHTO (2024) design live load consists of combinations of design lane load with design truck and design tandem. The design lanes are required to be positioned across the roadway width such that force effects are maximized or minimized. Furthermore, the lane load and wheel loads are also required to be positioned within the design lanes to achieve extreme force effects. For this purpose, the design lanes were stepped by 0.5 ft transversely. The wheel loads were stepped by 0.2 ft within the lanes between the extreme edges (0.2 ft from the edge of the design lane). The responses were recorded at each step, and the lane and wheel positions that created the extreme effects were obtained from the record.

The force effects of the design truck and the design tandem were calculated by multiplying the ordinate of the influence surface at the coordinates of wheel loads for any given lane and vehicle positions. But the force effects for design lane load require a more complex approach due to its distribution and variable nature. In cases where the influence surface ordinates are both positive and negative, the lane load needs to be applied only to the regions that have positive values. The start and end points of lane load need to be optimized for extreme force effects in addition to the transverse and longitudinal positions. The force effects were calculated by using the trapezoidal rule of numerical double-integration method on both transverse and longitudinal directions. The optimization of design lane load is illustrated in Figure 6.7, where f_{IS} is the function of influence surface, $\bar{q}$ is the design lane load pressure (0.64 ksf), and the x and z axes are along the transverse and longitudinal directions of the bridge, respectively. The interval sizes are dx and dz in x and z axes, respectively.



$$F = \text{Volume of Influence Surface (IS)} \times \bar{q}$$

$$F = \bar{q} \cdot \int_{x_1}^{x_2} \int_{z_1}^{z_2} f_{IS} \cdot dx \cdot dz$$

Figure 6.7. Numerical integration for optimization of design lane load

The interval size for numerical double-integration was chosen as 0.5 ft in both directions, such that a relatively accurate solution can be obtained within reasonable computational time. The value was chosen based on a sensitivity analysis. A pressure load 0.64 ksf was applied on the deck surface in between the exterior and first interior girders as shown in Figure 6.8. The bending moment at the midspan of exterior girder (G1) was 688.8 kip-ft in the FE analysis. The live load optimization program was run on the influence surface shown in Figure 6.4 to find the start position, end position, and the bending moment at the midspan of girder G1. The interval sizes were varied from 1.0 ft to 0.05 ft, with the sizes in both directions as equal values. The calculated values are compared with the FE analysis result in Table 6.1. The errors in calculated bending moments were -1.7% for the interval sizes of 1.0 ft, 0.5 ft, and 0.2 ft. The error was reduced to -0.5% for a 0.1-ft interval and 0.1% for a 0.05-ft interval. The interval size of 0.1 ft was chosen for numerical integration since the analysis will involve multiple iterations, and the computational time was too large for smaller interval sizes. Additionally, the lane and vehicle loads prescribed in AASHTO are notional loads, and an error less than 1% is reasonable given the precision of the applied load levels.

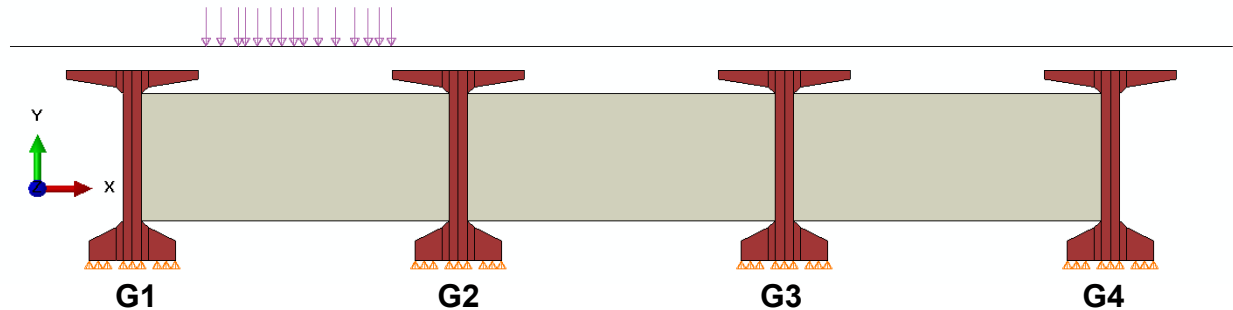


Figure 6.8. Bridge cross section showing load application for sensitivity analysis of numerical double-integration

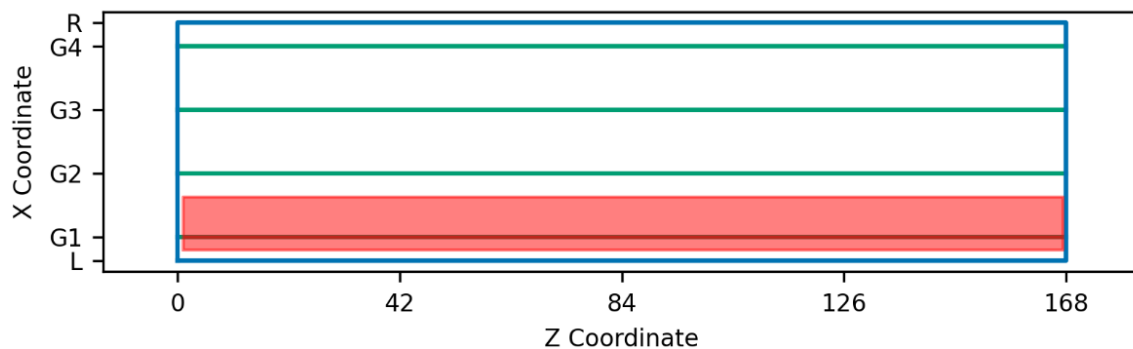
Table 6.1 Sensitivity Analysis of Numerical Integration Intervals

Interval Size (ft)	Calculation (kip-ft)	Error	FE Analysis (kip-ft)
1.0	677.4	-1.7%	688.8
0.5	677.4	-1.7%	
0.2	677.4	-1.7%	
0.1	685.3	-0.5%	
0.05	689.2	0.1%	

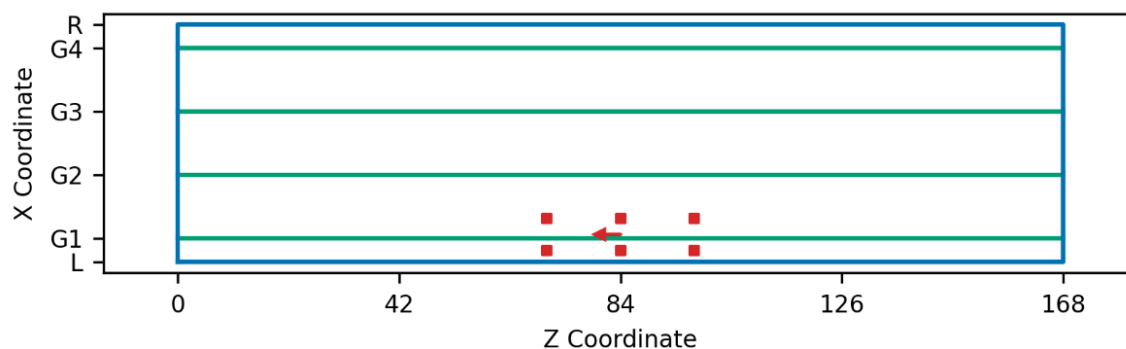
The positions of a single design lane load, design truck and design tandem for maximum bending moment in the mid-span of girder G1 are shown in Figure 6.9. The vehicular load positions obtained from the live load optimization analysis matched the expectations based on a line-girder

analysis approach. The arrows in the figure indicate the direction of movement of the truck, which help identify the heavier rear axles. The analysis was carried out by varying the rear axle spacing from 14 ft to 30 ft. However, the shortest spacing resulted in the maximum positive bending moment as expected for a simple span bridge. The direction of movement of the design tandem is not relevant because of the equal wheel loads and symmetric wheel positions about the vehicle axes.

The positions of two lanes of the design truck for maximum moment at girder G1 mid-span are shown in Figure 6.10. The center lines of the trucks were at 0.7 ft and 12.7 ft from the center line of girder G1. The position of the truck in the first lane was the same as the single-lane case. The truck in the second lane was at the closest possible distance from the girder. The observations in this preliminary live load analysis helped identify conditions that can reduce the number of iterations, thereby reducing the computational time. These observations have been incorporated in parametric analysis to increase the efficiency of live load optimization.



a) Design Lane Load



b) Design Truck

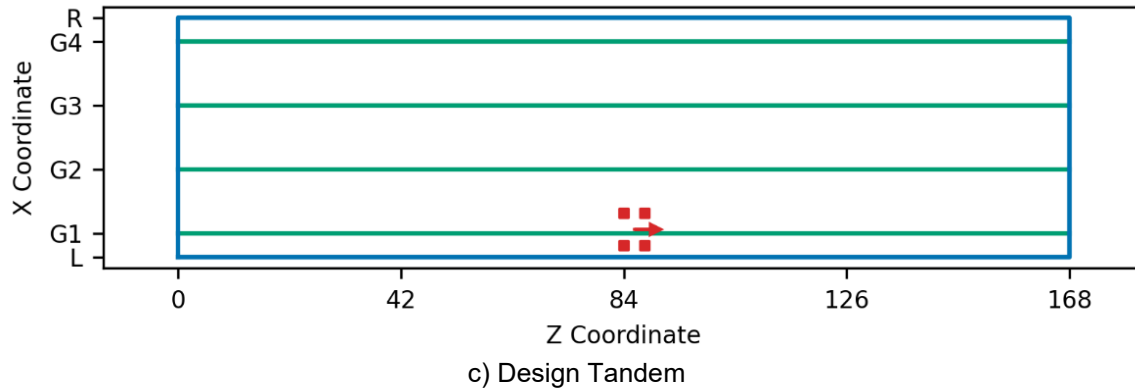


Figure 6.9. Single lane vehicle positions for maximum bending moment at mid-span of girder G1

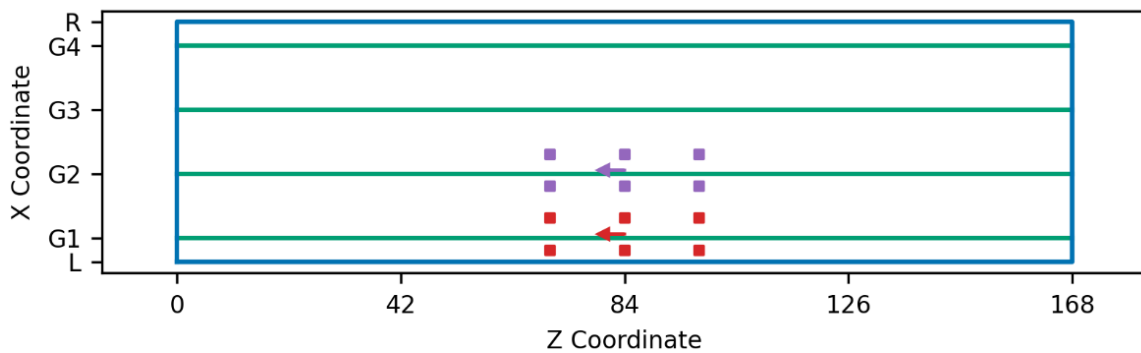


Figure 6.10. Two lane design truck positions for maximum bending moment at midspan of girder G1

6.2. Scope of Parametric Study

The bridge parameters that influence the live load distribution in girders are primarily the span length, girder spacing, girder type or stiffness, and the number of loaded lanes. The purpose of the parametric study was to develop design guidelines for the spliced prestressed concrete girder bridges that will be constructed in the future. Hence, the focus was mainly on the representative bridge systems based on those designed or constructed previously in Texas. The parametric study was divided into two categories depending on the girder type used in the bridge, namely constant-depth bridges and variable-depth bridges. In the constant-depth bridges, a single girder cross-section is used for the entire length of the bridge in all girder lines. The constant-depth bridges are used for relatively shorter span lengths with a span-to-depth ratio of 30 or less. For longer span lengths with span-to-depth ratios greater than 30, the variable-depth bridges may be preferred. In variable-depth bridges, the girder depth at the negative-moment regions near the interior supports is increased by a factor of 1.6 of the girder depth at positive-moment regions. The splice locations are assumed to be at a distance of 0.2 times the central main-span length (L) from the interior supports for both constant-depth and variable-depth bridges. The values of the parameters considered in parametric study are given in Table 6.2 for constant-depth and variable-depth bridges.

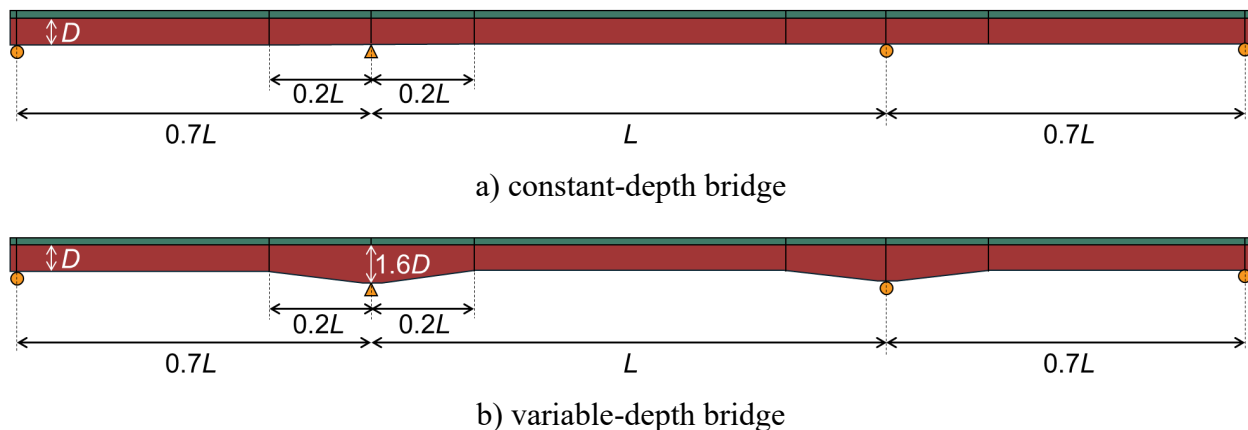


Figure 6.11 Schematic elevation views of constant-depth and variable-depth spliced I-girders with deck slab

The bridge models in the parametric study consisted of four girder lines with girders uniformly spaced. Based on the spliced girder bridges constructed in Texas, girder spacings of 8 ft, 10 ft, and 12 ft were chosen for the study. Spliced bridges with three or more continuous spans are more prevalent in Texas. Hence, three span continuous bridges with end span lengths 0.7 times the central span length (L) were analyzed. TxDOT standard deck slab thickness of 8.5 in. was assumed for all bridges. The deck overhangs had a thickness of 8.5 in and widths of 4.5 ft, measured from the centerline of the exterior girder to the deck edge. The deck slab thickness and the deck overhang width were not varied in the parametric study.

TxDOT has developed STx and HSTx I-girder cross-sections specifically for spliced girder applications (TxDOT, 2022). The HSTx girders are haunched variants of the STx girders with increased web depth. In constant-depth bridges, the continuous girders are formed by splicing the STx girders together. In variable-depth bridges, the corresponding haunched HSTx girder sections are used at the interior supports. The girder depth transitions linearly between the STx and HSTx sections. In the parametric study, STx7 and STx9 girders were chosen because they represent the range of span lengths and girder depths. In variable-depth bridges, the haunched girders had a web depth of 1.6 multiplied by that of the constant depth STx girders. The cross-sectional dimensions of the STx girders are shown in Figure 6.12. The haunched girder segment length at the interior supports was detailed as 5 ft in all variable-depth bridge models – TxDOT allows the designer to determine this dimension and potential thickening of the bottom flange in this region. An elevation view of a variable-depth STx7 bridge with a span-to-depth ratio of 35 is shown in Figure 6.13. The bridge shown in the figure had diaphragms at the supports only. The diaphragm width was detailed as 18 in. for all cases in parametric study.

was no significant reduction in the governing bending moments in the girders. Permanent interior diaphragms are not provided in simply supported concrete I-girder bridges currently in Texas. The transverse edges of the deck slab at joints are stiffened by a thickened deck slab instead of transverse diaphragms. However, diaphragms at supports are found to be necessary in spliced girder bridges due to stability issues as previously mentioned. The live load distribution factors (LLDFs) derived by Zokaie et al. (1991) and recommended by AASHTO (2024) do not distinguish between bridges with and without diaphragms or cross-frames. Hence, there is a need to study the effect of diaphragms on live load distribution and verify the AASHTO LLDFs for spliced girders with and without diaphragms. This was studied by considering four different diaphragm layouts named Layout-0, Layout-1, Layout-2 and Layout-3.

In Layout-0, there were no diaphragms considered in the model, which is the current practice for simply supported prestressed concrete I-girder bridges. Although Layout-0 is not recommended for spliced girder bridges, this layout serves as a baseline model to quantify the influence of diaphragms in live load distribution. In Layout-1, diaphragms were at the end and interior supports only. Additional diaphragms were assumed at splice locations in Layout-3 and Layout-4. In Layout-4, diaphragms were also present at the middle of the drop-in segment and between the splice locations and end supports of the end spans. The schematic representations of the diaphragm layouts are shown in three-dimensional views and plan views in Figure 6.14.

AASHTO (2024) provides reduction factors for moment and magnification factors for shear in skewed bridges. The effects of support skew in spliced girder bridges were studied by modeling the same bridge geometries with normal supports and skewed supports at 30° and 45° skew angles. In total, there were 288 models of spliced prestressed concrete I-girder bridges in the parametric study, with 96 models for each support skew angle.

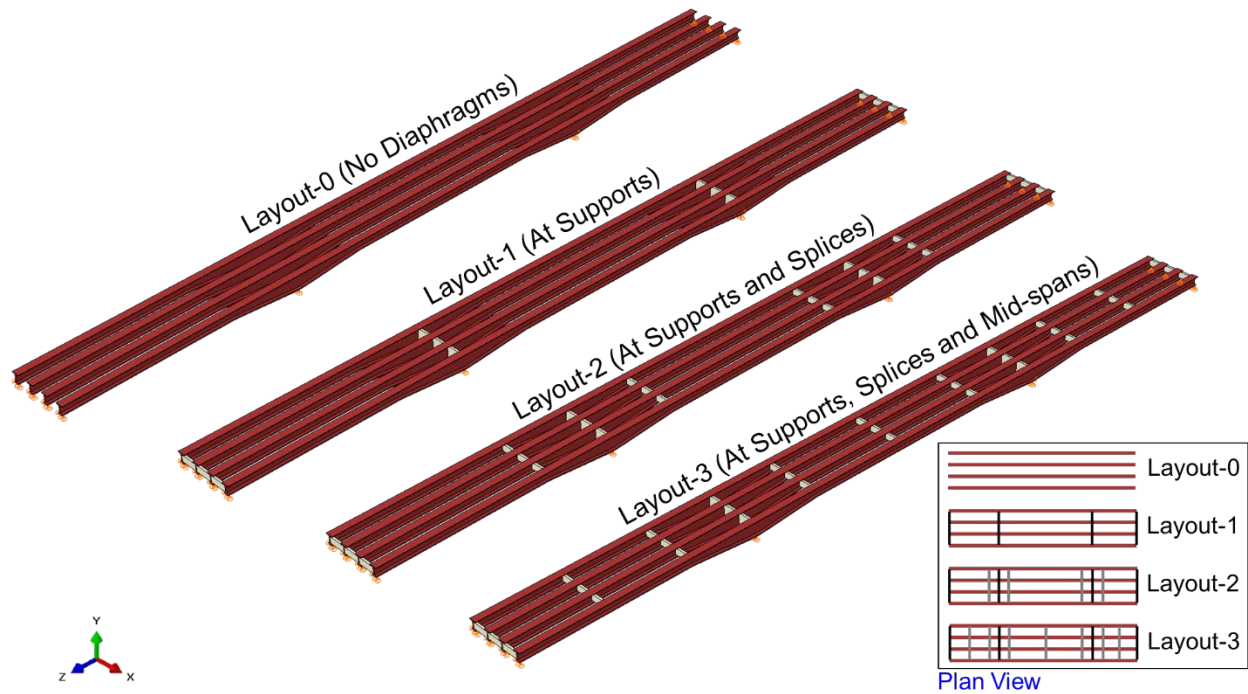


Figure 6.14. Schematic and plan views of girder-diaphragm layouts considered in the parametric study

Table 6.2 Summary of Parameters for Live Load Distribution Study

Parameter	Values
Girder types	STx7, STx9
Span-to-depth ratio (L/D)	25, 30 (constant-depth bridges) 30, 35 (variable-depth bridges)
Haunched girder depth (variable-depth bridges only)	$1.6 \times \text{Depth of prismatic girder}$
Span arrangement	$0.7L + L + 0.7L$
Number of girder lines (N_b)	4
Skew angles	0° , 30° , and 45°
Deck slab thickness (t_s)	8.5 in.
Girder spacing (S)	8 ft, 10 ft and 12 ft
Overhang width (w_{oh})	4.5 ft
Diaphragm locations	Layout 0: No diaphragms Layout 1: Support diaphragms only Layout 2: Diaphragms at supports and splice locations Layout 3: Diaphragms at supports, splice locations and middle of drop-in segments
Splice locations	$0.2L$ from interior supports

The primary objective of the study is to gauge the appropriateness of the current live load distribution factors (LLDF) for spliced prestressed concrete I-girder bridges, though they are not applicable for few bridge geometries. The bending moment and shear reactions in exterior and interior girders obtained in the analyses were compared to the values obtained from the line-girder

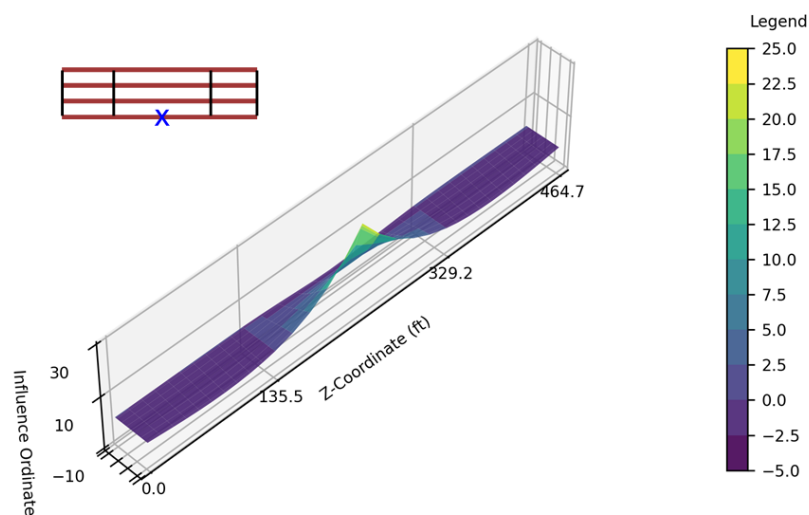
analysis according to LLDF equations in AASHTO (2024). The secondary objective of the study is to study the influence of diaphragms on load distribution.

6.3. Influence Surfaces in Spliced-Girder Bridges

The influence surfaces were generated for the spliced girder bridges following the same procedure used in the verification example explained in the previous section. Due to the symmetric span arrangement ($0.7L + L + 0.7L$), the points at the middle of the first and central spans were chosen for the positive moment, and the first interior support was chosen for the negative moment. The influence surfaces were generated for all the analyzed cases. In this section, the influence surfaces are plotted for one constant-depth girder bridge with STx7 girders spaced at 12 ft and featuring a central span length of 193 ft and diaphragms at supports only (Layout-1).

6.3.1. Bending Moments

The bending moment influence surfaces are shown as three-dimensional plots in Figure 6.15 for the exterior girders in the chosen spliced girder bridge. The influence surfaces for the positive moment at the middle of the central span and first end span are shown in Figure 6.15a and b, respectively. The influence surface for negative moment at the first interior support is shown in Figure 6.15c. The surface plots confirm there are no discontinuities in the surfaces. Furthermore, the surfaces also show the spatial variation of the influence ordinates. For example, in case of positive moment at the middle of central span, the influence surface indicates the moment will be higher when the truck is placed on the deck overhang compared to the case where the truck is placed along the centerline of the exterior girder.



a) positive moment (kip-ft) in girder G_7 at the middle of the central span

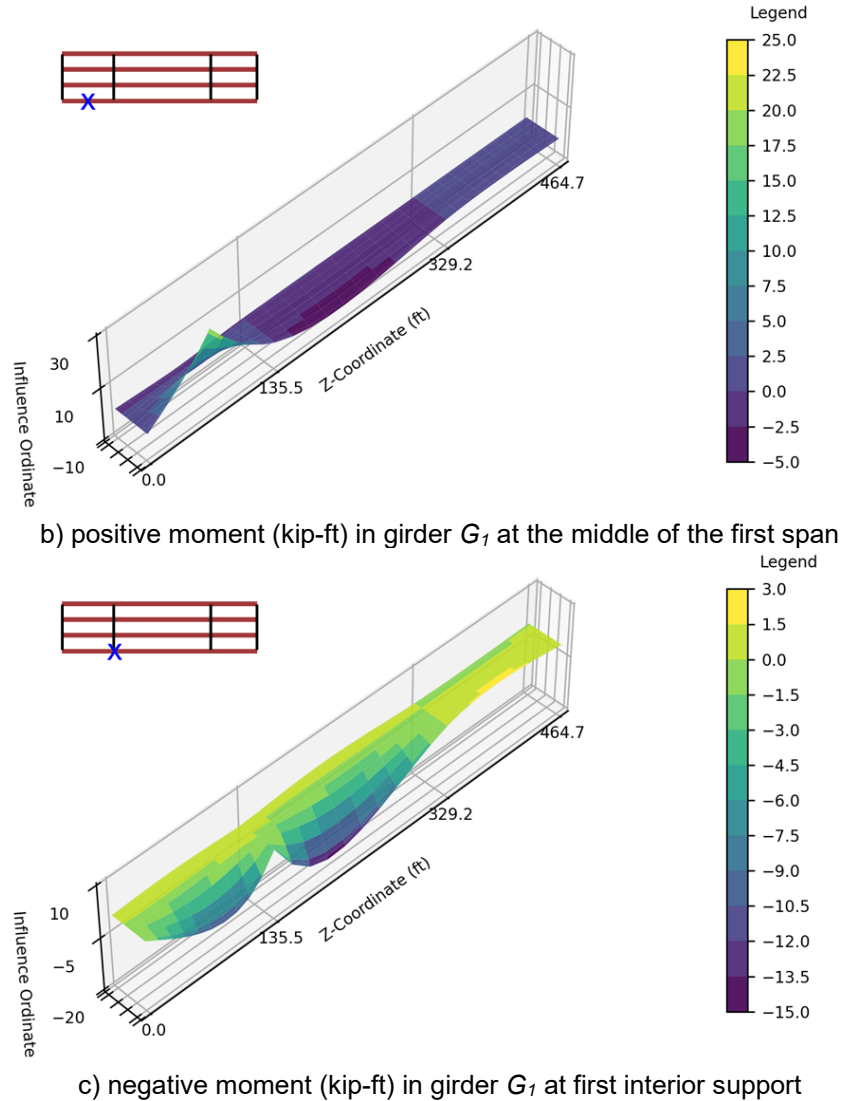


Figure 6.15. Three-dimensional plots of influence surfaces for bending moments in the exterior girder G_1

Contour plots are another form of representing the influence surfaces, where the influence ordinates are shown as contour lines. The influence surfaces shown in Figure 6.15 are shown in the form of contour plots in Figure 6.16. The legends for the contour plots are the same as given in Figure 6.15. The contour plots are easier to interpret than three-dimensional plots due to the two-dimensional format. The critical positions of the truck can be predicted using the contour plots. The plots also show the diminishing force effects for points of load away from the point under consideration. The surface cuts along the longitudinal direction on the contour plot are one-dimensional representations of the influence surface. The influence surface cuts along the centerline of the girders are plotted in Figure 6.17 for the same cases shown in Figure 6.15 and Figure 6.16.

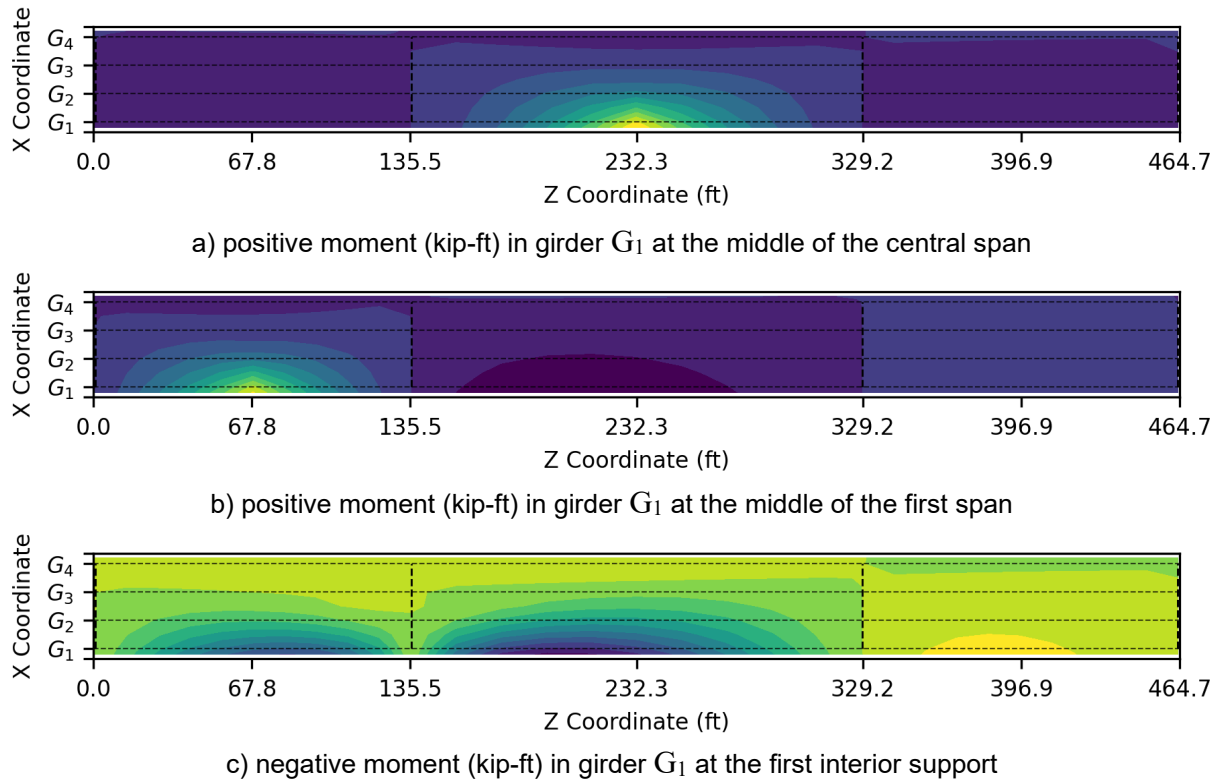


Figure 6.16. Contour plots of influence surfaces for bending moments in the exterior girder G_1

The influence surface cuts along the girder centerlines (G_1 , G_2 , G_3 , and G_4) obtained from the FE analyses are compared with the influence lines obtained from a line-girder analysis (LGA) in Figure 6.17. The influence surface cut along the centerline of exterior girder G_1 qualitatively matches the influence line diagram for the girder for all three cases of bending moment. However, the influence ordinates calculated along the cuts are significantly lower than the values obtained in line-girder analysis. The difference is due to the refined analysis approach used in FE analysis, considering the three-dimensional realistic behavior of the bridge. The deck slab and diaphragms tie the girders together and act as a system in the FE analysis. Conversely, in the line-girder analysis approach, the composite girder acts separately with only the tributary width of the deck slab. The critical positions of the vehicle loads may not be the same when using these different analysis approaches due to the difference in curvature of the influence surface and the difference in magnitude of influence ordinates. In Figure 6.17c, the peak influence ordinate on the cut along G_1 is at a point nearer to the interior support compared to the corresponding point in the line-girder analysis. Therefore, the critical positions of vehicle load are also expected to be different due to the different locations of peak response.

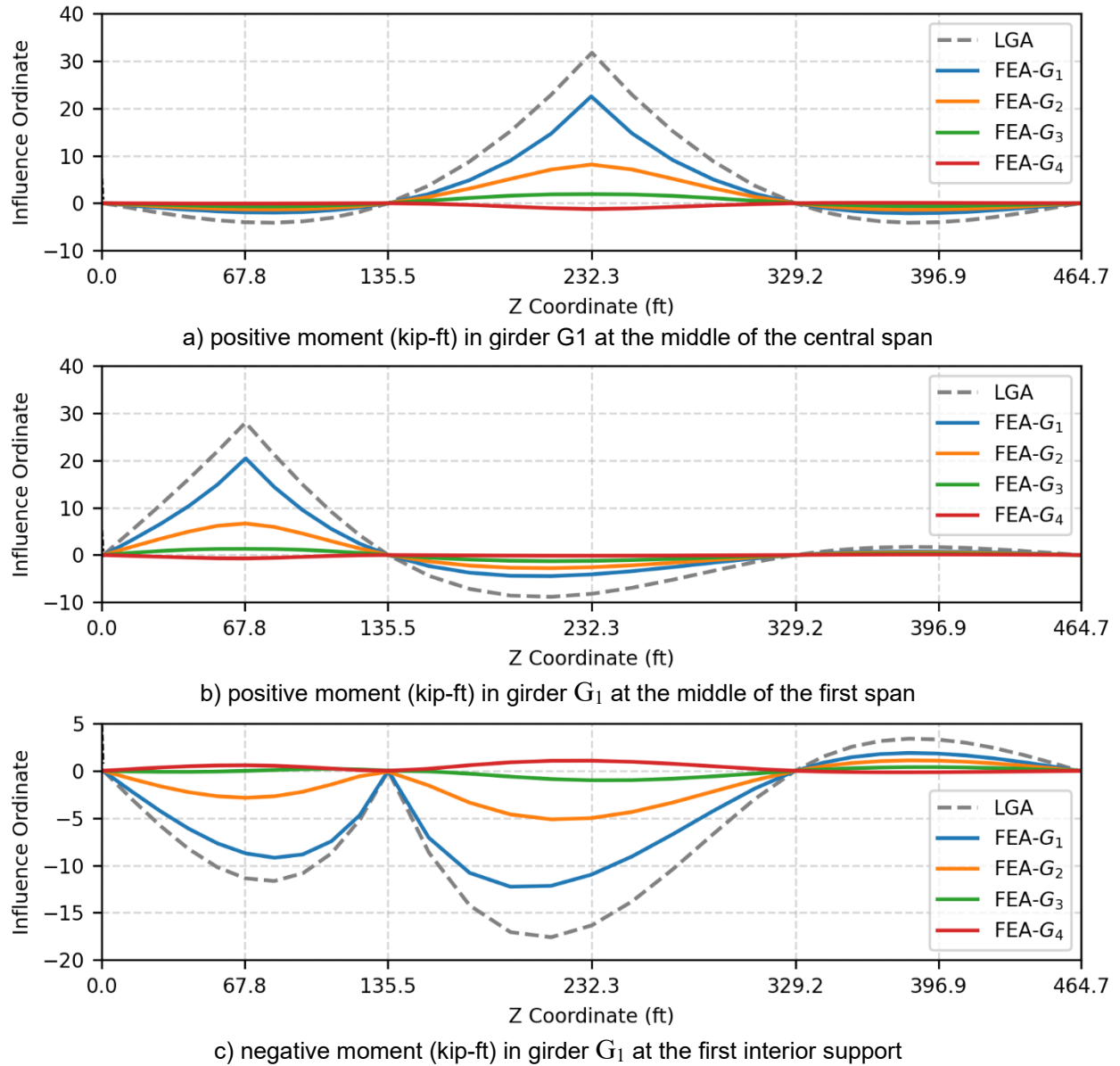


Figure 6.17. Line plots of influence surface cuts along the center line of girders for bending moments in the exterior girder G_1

6.3.2. Shear

The influence surfaces for vertical reactions at the left end and interior supports in girder G_1 are represented in the form of contour plots in Figure 6.18a and b, respectively. The contour plots show the localized nature of the influence surface. The influence ordinate values are high around the point of support, and the values are negligible if the load is located further away from the point of support.

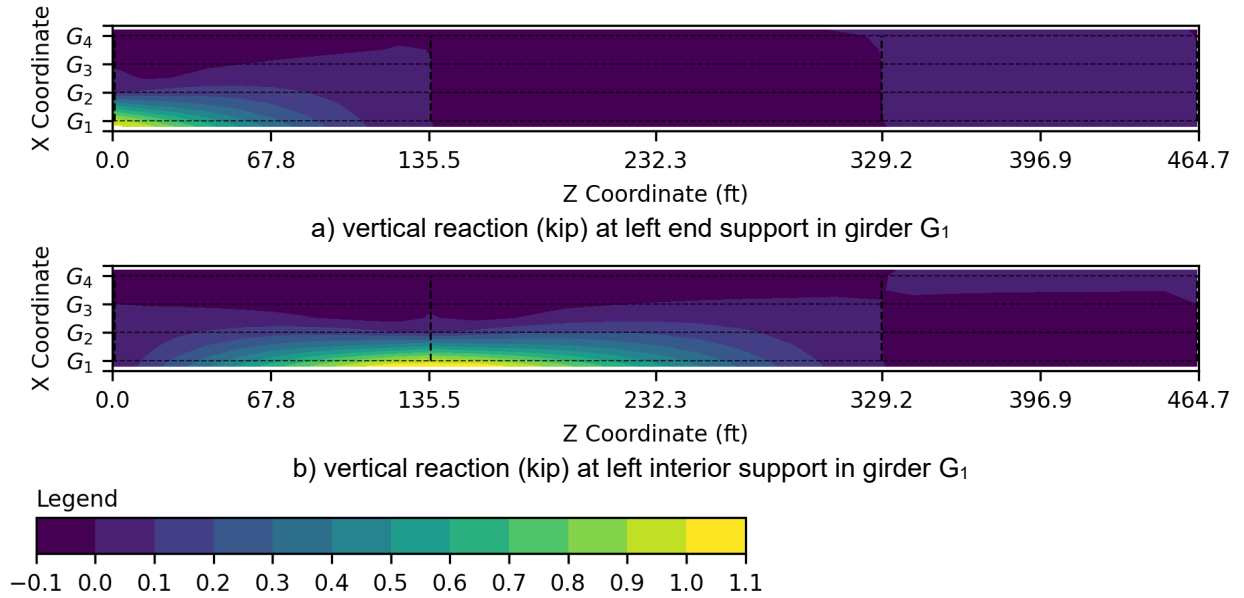


Figure 6.18. Contour plots of influence surfaces for reactions in the exterior girder G_1

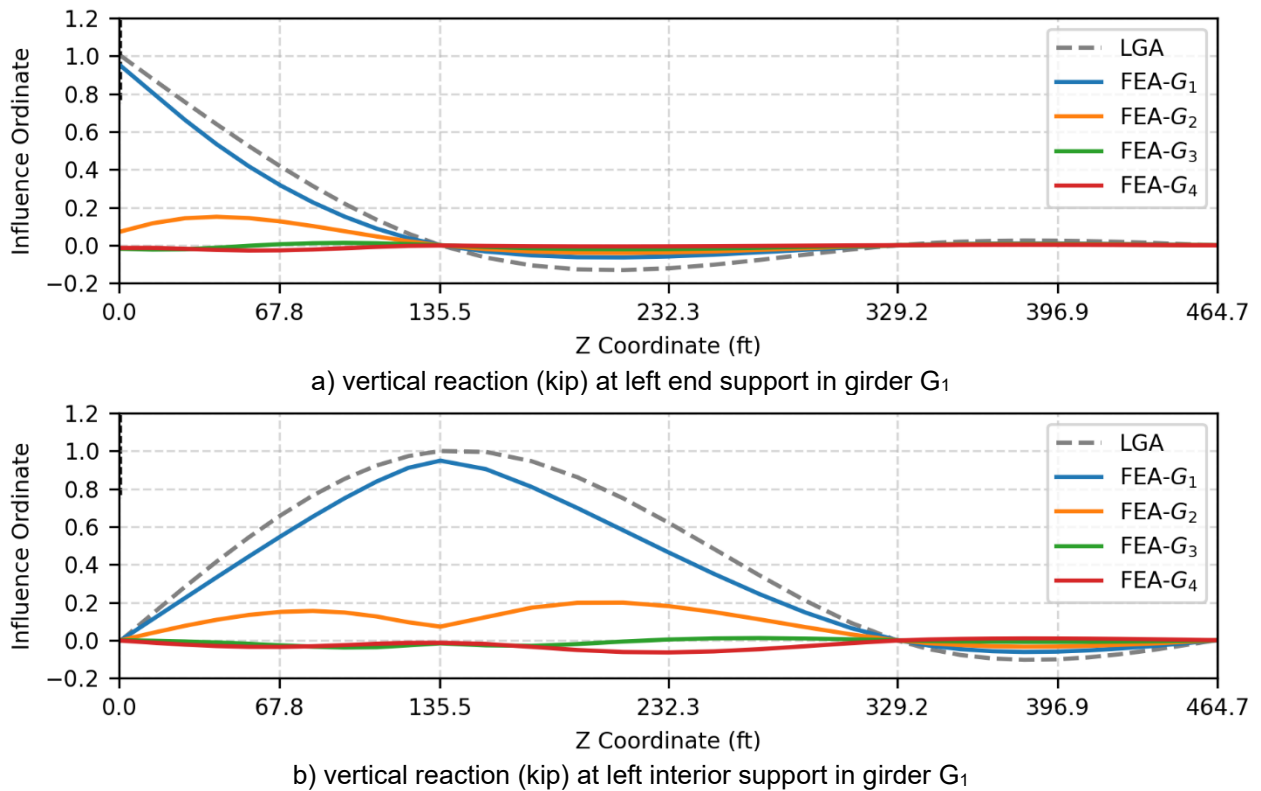


Figure 6.19. Line plots of influence surface cuts along the center line of girders for vertical reactions in the exterior girder G_1

The influence surfaces for vertical reactions were cut along the girder centerlines and plotted as line plots in Figure 6.19. The influence line diagram from a line-girder analysis is compared with the influence surface cuts in the figure. As observed in the comparison for bending moments, the

cuts along the girder G_1 approximately match the influence line diagrams for both reactions at the end and interior supports. However, the plots also show the spatial variation of the influence surface along other girders. For example, the peak of influence surface at the girder G_2 in Figure 6.19a occurs away from the support at G_2 . This indicates that the critical truck positions in lanes away from the girder G_1 may not be near the supports. Conversely, in a line-girder analysis, the critical positions of all the loaded lanes depend on the influence line diagram only. In Figure 6.19b, there are two peaks of the influence surface on the girder G_2 , both away from the interior support (P1). This also emphasizes the need to account for multiple load positions to calculate the critical load effects in the live load optimization analysis.

6.4. Live Load Distribution Factors

Line-girder analysis (LGA) using Live Load Distribution Factors (LLDF) is the most commonly used method, recommended by AASHTO, for analyzing bridge girders for live loads (Guner, 2024). The LLDFs represent the number of design lanes to be considered for the longitudinal response of a girder. In this approximate analysis approach, the longitudinal response of a single girder line due to a single lane of vehicle load is multiplied with the LLDFs to determine the total response of the bridge girder. AASHTO (2025) provides equations to calculate the LLDFs for bending moments and shear in exterior and interior girders. Additionally, correction factors are also specified for LLDFs to account for skewed supports. The LLDF equations were developed as part of the *National Cooperative Highway Research Program (NCHRP) Project 12-26* (Zokaie et al., 1991).

6.4.1. LLDFs from FE Analysis

The extreme live load effects were calculated by using the live load optimization program and influence surfaces obtained from FE analysis. According to AASHTO (2024), the live load effects calculated using any accepted method of analysis need to be multiplied by a multiple presence factor (m) to account for the probability of simultaneous lane occupation. Hence, the live load effects obtained from FE analysis are multiplied by the m values specified in Table 6.3 according to the number of loaded lanes.

Table 6.3 Multiple Presence Factors

Number of Loaded Lanes	Multiple Presence Factor, m
1	1.20
2	1.00
3	0.85
>3	0.65

The LLDFs were calculated as the ratio of forces obtained in the FE analysis for any number of design lanes (including multiple presence factors) with respect to the forces obtained in a line-girder analysis, where the vehicle loads corresponding to a single lane were applied. The equation for calculation of $LLDF_{FEA}$ is given in Equation 5-1, where m represents the multiple presence

factor, $F_{FEA, n \text{ lanes}}$ represents the force calculated from FE analysis for n number of lanes, and $F_{LGA, single \text{ lane}}$ represents the force calculated from line-girder analysis for a single lane.

$$LLDF_{FEA} = \frac{mF_{FEA, n \text{ lanes}}}{F_{LGA, single \text{ lane}}} \quad (5-1)$$

In the following sub-sections, the calculation of LLDFs is explained in detail with an example of LLDF for negative moment at the first interior support.

6.4.1.1. Line-Girder Analysis

The line-girder analysis was performed in Abaqus (Dassault Systèmes, 2024) using three-dimensional linear beam elements (B31) for a single girder line. The boundary conditions in the line-girder analysis were modeled as ideal pin and roller conditions with the first interior support pinned for translation. The section properties of the beam elements were defined according to composite section properties, assuming the tributary width of the deck slab acting compositely with the I-girders. A unit load (1 kip) was applied at every 1/10th point along the length of the line-girder in each load case. In total, there were 31 load cases for each model including the loads at the end points. The results were utilized to plot the influence line diagrams and extreme force effects were calculated by optimizing the vehicle load positions. The influence line diagrams and influence surface cuts for the spliced girder bridge chosen in Section 5.3 are shown in Figure 6.17 and Figure 6.19 for bending moments and reactions, respectively.

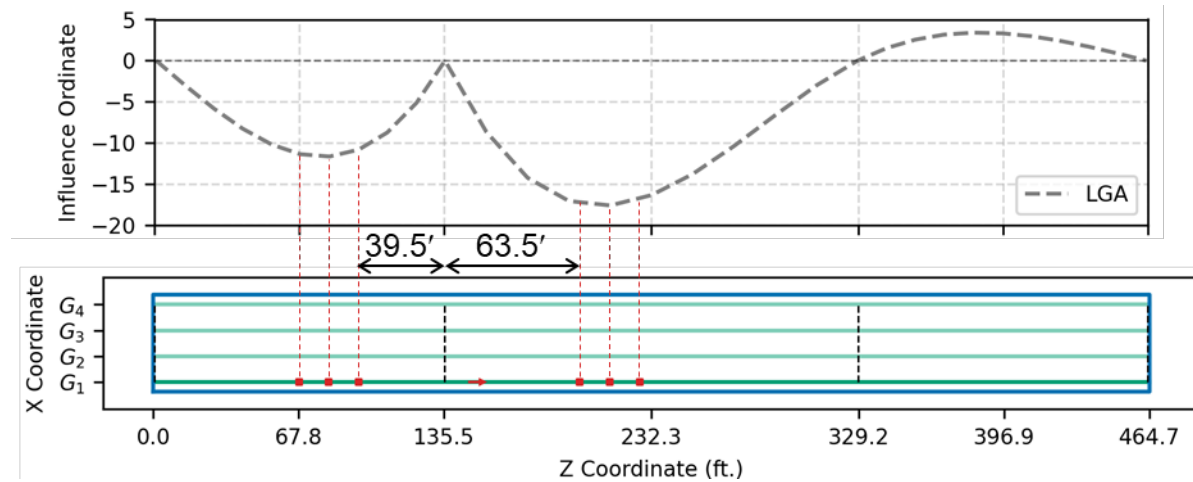


Figure 6.20. Influence line diagram and position of a single lane of design trucks for the negative bending moment at first interior support in line-girder analysis

The critical positions of the two succeeding design trucks calculated from line-girder analysis are shown in the plan view and overlaid on the line plot of the influence line diagram in Figure 6.20. The minimum negative moment at the interior support in the bridge was calculated to be -2068 kip-ft for the vehicle positions shown in Figure 6.20. In the figure, the wheel loads of each axle are lumped together at the centerline of the girder G_1 and the plot is superimposed with influence line diagram for negative moment at first interior support.

6.4.1.2. FE Analysis

The critical positions of design trucks were found using the live load optimization program for extreme force effects. The maximum number of design lanes, calculated according to AASHTO (2024), was three in bridges with 10 ft or 12 ft girder spacing but limited to two in bridges with 8 ft girder spacing. The live load optimization program, developed by the Research Team, is capable of accounting for multiple vehicle positions in determining the extreme force effects.

The positions of three lanes of succeeding design trucks for minimum negative moment at first interior support are shown in Figure 6.21. The critical positions of the design trucks in the FEA-based influence surface analysis were different from the positions obtained in influence line analysis for the first lane of design trucks. The leading truck was 5 ft closer to the first interior support in the influence surface analysis. In this case, the second and third lane design trucks were in the central span, unlike the design trucks in first lane, due to the high influence ordinates of girders G_2 and G_3 in the central span. The minimum negative moment was -1784 kip-ft for three lanes of succeeding design trucks. In this case, there are three lanes loaded, therefore the multiple presence factor (m) of 0.85 must be included in the live load moment. Hence, according to Equation 5-1, the LLDF for the three-lane case is calculated as 0.73.

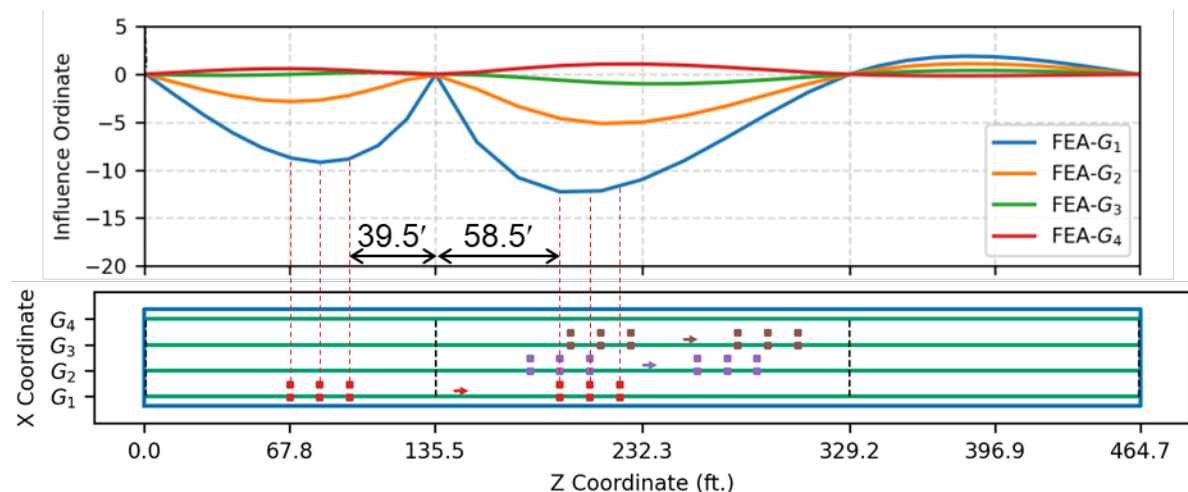


Figure 6.21. Influence surface cuts and position of three lanes of design trucks for minimum negative bending moment at the first interior support in FE analysis

6.4.2. AASHTO LLDF Equations

The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2024) provides equations to calculate the LLDFs for bending moments and shear in continuous concrete girder bridges. The multiple presence factors are included in the equations for live load distribution factors in *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2024). However, for cases where lever rule is specified by AASHTO (2024), the multiple presence factor needs to be accounted. TxDOT (2024a) recommends using m value of 1.0 for exterior girders with deck overhang width greater than half of the girder spacing under a single loaded lane. Furthermore, TxDOT (2024a) limits the LLDFs (including multiple presence factors) to a minimum value equal to the number of loaded lanes

divided by the number of girders. In this study, the main objective was to check the validity of AASHTO (2024) for calculating the LLDFs. Therefore, the LLDFs calculated from FE analyses were compared with the LLDFs calculated using AASHTO (2024) specifications only (not according to the current TxDOT practice).

6.4.3. Comparison of LLDFs from FE Analysis and AASHTO

The LLDFs calculated from the FE analyses are compared with the values calculated from AASHTO (2024) in the form of diagonal plots in this section. The constant-depth bridges with normal support lines are only shown in this section. The LLDFs were calculated for only the design truck, assuming the same distribution for design lane loads due to the high computational cost for optimizing design lane loads. Moreover, the distribution factors are the same for both design truck and design lane loads according to AASHTO (2024). The LLDFs for single lane, two lane and three lane cases were differentiated to evaluate the accuracy of the AASHTO equations. A schematic representation of a three-span continuous spliced girder bridge is shown in Figure 6.22, in which Section 1-1 and Section 2-2 represent the middle of first and second spans. The positive bending moments were evaluated for Sections 1-1 and 2-2 and negative bending moments were evaluated at first interior support (P1). The reactions at end support (A1) and first interior support (P1) were also evaluated for the comparison.

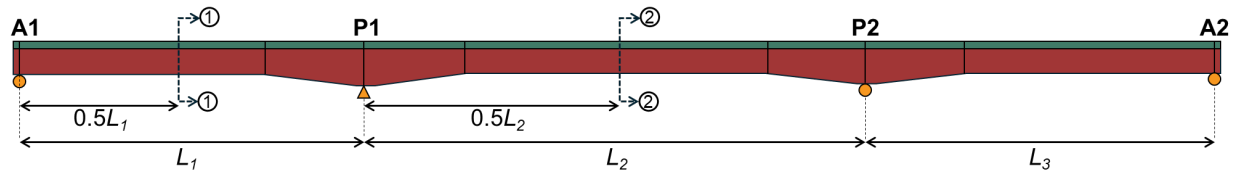


Figure 6.22. Schematic elevation of a girder in a spliced-girder bridge

6.4.3.1. Positive Moment at Section 1-1

The LLDFs obtained from FEA are compared with AASHTO equations for bending moment at section 1-1 in Figure 6.23. In case of exterior girders with a single loaded lane, the lever rule is specified for the LLDF by AASHTO (2024). The lever rule consistently overestimated the distribution factor compared to the FEA results. The highest LLDF value is generally used for the design of girders. The AASHTO LLDFs in the figure indicates the governing LLDFs were mostly from the single lane case using lever rule. But in FEA LLDFs the two-lane cases had the highest value among all three cases. The AASHTO equations for multiple lanes were less conservative than the single-lane case for exterior girders.

The LLDF equations specified for interior girders closely match the LLDFs from FEA. The differences between AASHTO and FEA LLDFs were smaller in interior girders than the differences in exterior girders. The multiple-lane cases had higher LLDFs than single-lane cases, both in AASHTO predictions and FEA results. The FEA results for two-lane and three-lane design trucks were approximately equal and larger than the single-lane truck loads. The FEA LLDFs in the exterior girders were higher than the values of the interior girders, possibly due to the wider

overhangs. The differences in LLDFs for the same bridge geometry with different diaphragm layouts were negligible for the moment at section 1-1. Nonetheless, the LLDFs in bridges with diaphragms were lower than the bridges without diaphragms.

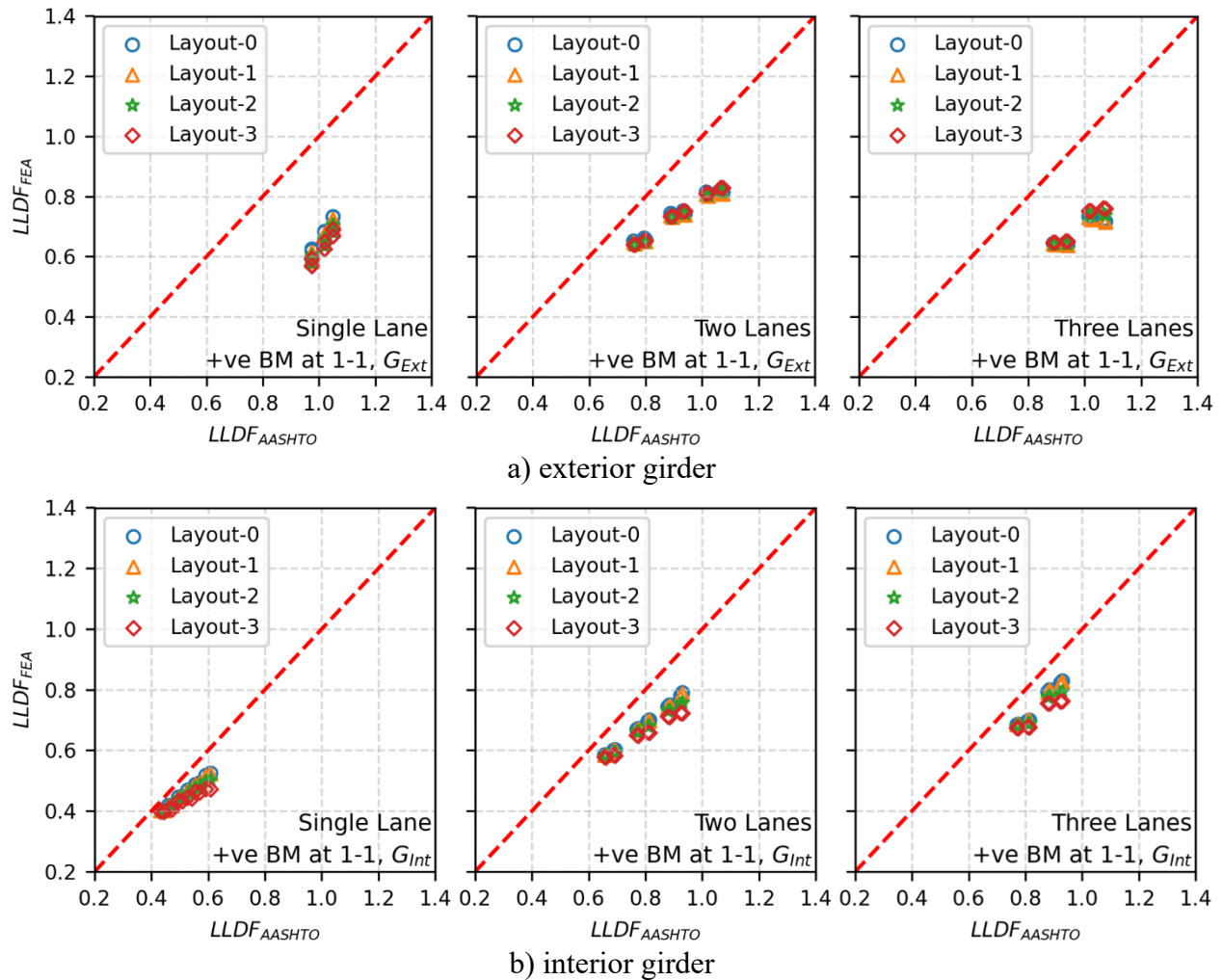


Figure 6.23. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 1-1

6.4.3.2. Positive Moment at Section 2-2

The LLDFs from FEA are compared with the AASHTO values in Figure 6.24 for positive bending moment at the middle of the central span (section 2-2). In general, there was little difference between the LLDFs for the positive moments at section 1-1 and section 2-2. The relative difference between FEA and AASHTO LLDFs were also same for both moments. The lever rule for the exterior girders consistently overestimated the distribution factor for single-lane cases. However, the two-lane cases had the highest LLDFs in exterior girders according to AASHTO and FEA. The interior girder LLDFs from FEA were approximately equal to the AASHTO values. However, the exterior girders govern the design for live loads due to higher LLDFs.

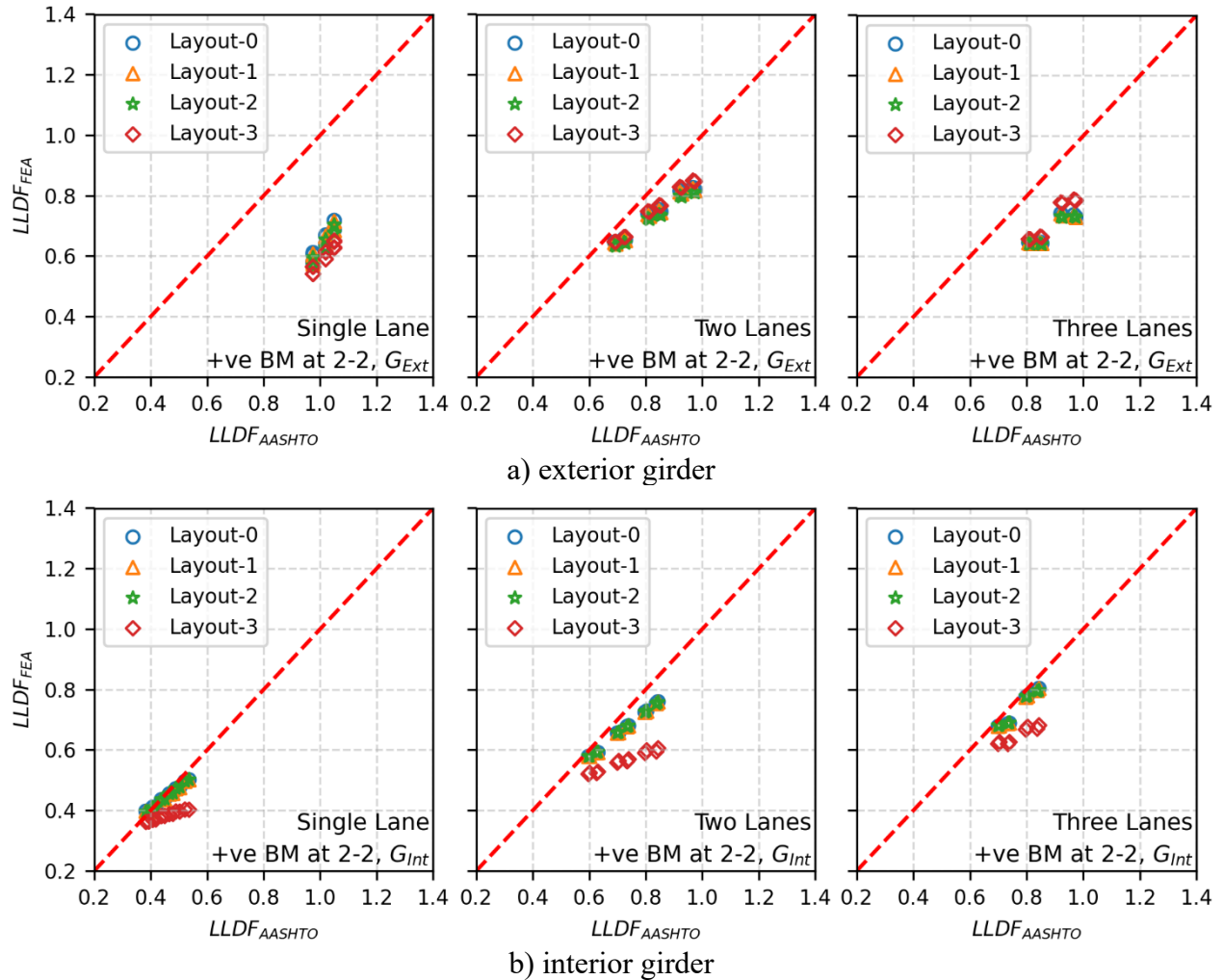


Figure 6.24. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 2-2

The key difference between moments at section 1-1 and 2-2 was observed in bridges with Layout-3. At section 1-1, there were no diaphragms exactly at that location. In bridges with Layout-3, the diaphragms were exactly located at the mid-span of the central span. Therefore, the LLDFs for positive moments at section 2-2 were different from the values in bridges with other diaphragm layouts. The exterior girder LLDFs were higher in Layout-4 for multiple loaded lanes. The LLDFs in Layout-3 were lower than Layout-1 by 8 to 20% for interior girders and by 6 to 9% for exterior girders in constant-depth bridges with normal supports. The difference in LLDFs was higher in bridges with wider girder spacings. This indicates the effects of diaphragms are proportional to the girder spacing. The effect of diaphragms is explained in more detail in Section 5.5.

6.4.3.3. Negative Moment at Support P1

The negative bending moment between the points of contraflexure in a continuous bridge is generally governed by the case where two succeeding design trucks are in each lane on adjacent spans. According to AASHTO (2024), the minimum negative moment was calculated by taking 90% of the effect of two design trucks in both FEA and line-girder analysis. The LLDFs obtained

from FEA are compared with the AASHTO LLDFs in Figure 6.25 for exterior and interior girders in bridges under two succeeding design truck loadings. The lever rule was very conservative for single-lane cases in exterior girders, as observed for positive moments. For other cases, the LLDFs from AASHTO equations were closer to the FEA LLDFs, especially for interior girders.

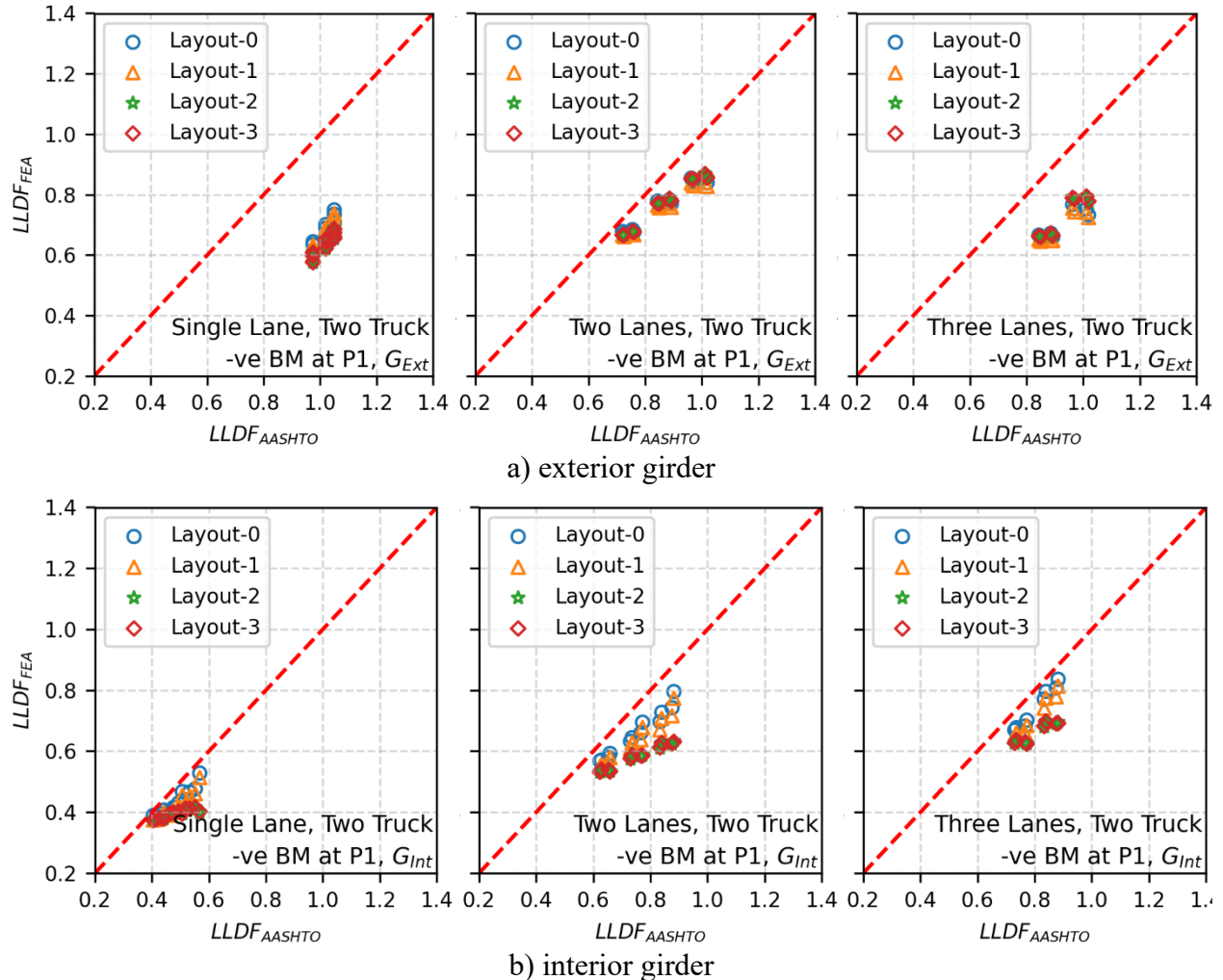


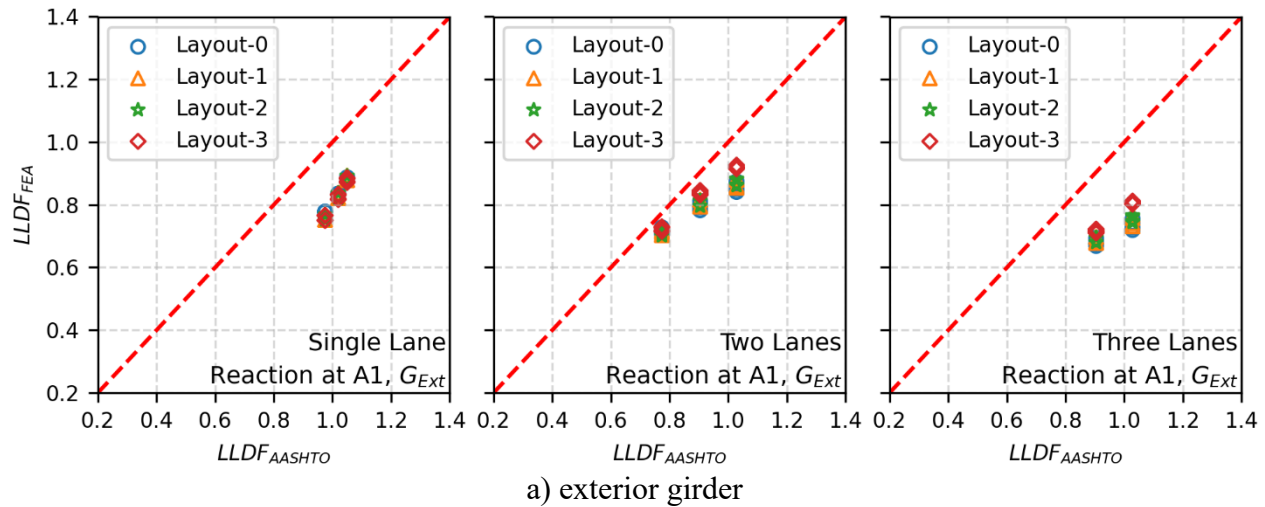
Figure 6.25. Comparison of LLDFs from FEA and AASHTO for negative bending moment at support P1

The exterior girder LLDFs were higher than the values in interior girders, and the cases with two lanes of two design trucks governed. On average, the negative bending moments and the corresponding LLDFs in interior girders were 20% lower than the moments in exterior girders. The LLDFs for interior girders in bridges with Layout-2 and Layout-3 were significantly lower (1 to 22%) than the values in bridges with only support diaphragms. The change in load distribution for negative moments at interior supports was due to the presence of diaphragms at splice locations, which were very near to interior supports. The support diaphragms in Layout-1 resulted in 2% to 4% lesser moments compared to Layout-0.

6.4.3.4. Shear Reaction at A1

The support reactions were assumed to represent the shear distribution factors in this study. The LLDFs calculated from the reactions at end support A1 in FEA are compared with the AASHTO values in Figure 6.26. The lever rule was better in predicting the shear reactions in the exterior girder than the moments for a single lane of a design truck. AASHTO equations for interior girder shear LLDFs are dependent only on the girder spacing and not on the longitudinal stiffness or the girder length. The LLDFs from FEA were also primarily dependent on the girder spacing. The end support reaction in exterior girders was mostly governed by the single-lane case when the multiple presence factor of 1.2 is considered. The interior girder reactions were governed by two lanes of a design truck. Overall, the AASHTO LLDFs were conservative for most cases except for bridges with Layout-0.

The diaphragms play an important role in load distribution for shear reactions. The reactions were 16 to 32% lower in bridges with Layout-1 when compared to bridges with Layout-0. The difference was mainly due to the diaphragms at the support locations. The interior diaphragms had a negligible effect on the reaction at the end supports. The FEA LLDFs exceeded the AASHTO values for interior girders by 4 to 12% for cases without support diaphragms. AASHTO (2024) recommends providing deep and rigid end diaphragms at the supports in concrete girder bridges for proper load distribution. Hence, the AASHTO equations may not be applicable for calculating shear in interior girders when there are no support diaphragms. Furthermore, the girders and bearings may be designed more economically if support diaphragms are provided.



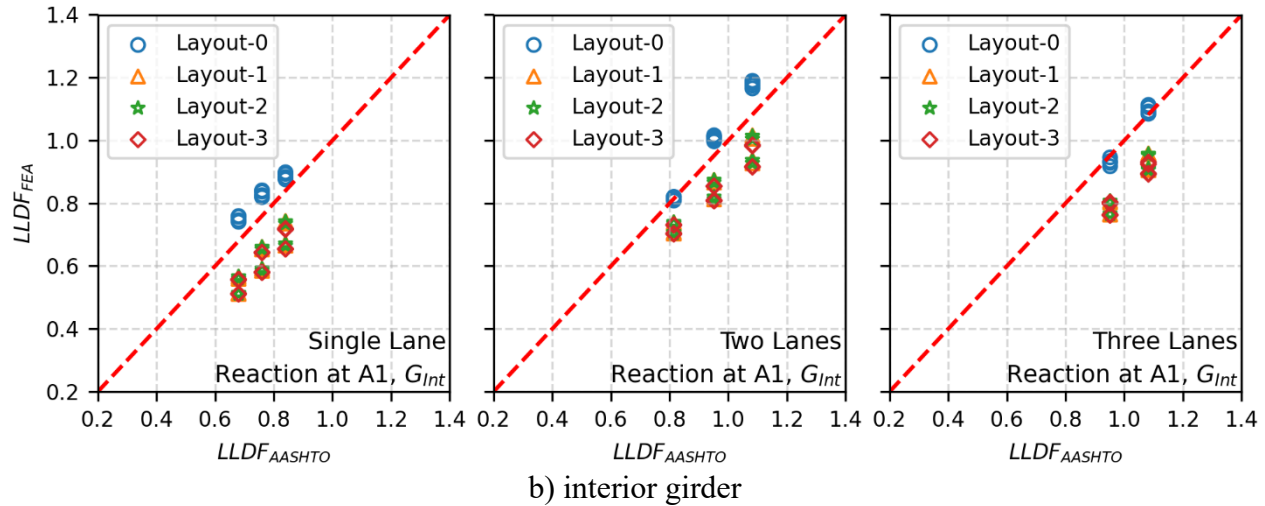


Figure 6.26. Comparison of LLDFs from FEA and AASHTO for vertical reactions at end support A1

6.4.3.5. Shear Reaction at P1

The shear reaction at the interior support is governed by the two design trucks located in adjacent spans. The LLDFs obtained from FEA for shear reaction at the first interior support P1 are compared against the AASHTO LLDFs in Figure 6.27. The support reaction at P1 in the exterior girder was higher than in the interior girders. The exterior girders were governed by the two loaded lanes and the interior girders by two and three loaded lanes. The LLDFs from FEA were approximately equal to or less than the AASHTO LLDFs. The difference between the LLDFs in exterior girder for bridges with different diaphragm layouts was negligible. But in general, the bridges with diaphragms at the splices (Layout-2 and Layout-3) had higher LLDFs in exterior girders and lower LLDFs in interior girders.

In bridges with support diaphragms, the reactions were 7 to 15% lower in the interior girder and up to 4% lower in the exterior girder when compared to bridges without diaphragms. Furthermore, in bridges with additional interior diaphragms at the splices, the reactions were reduced by 17 to 28% in interior girders and by 5 to 7% in exterior girders. The reactions in bridges with Layout-3 were approximately equal to the values in bridges with Layout-2. The diaphragms at the middle of the drop-in segment had less influence on the load distribution for reaction at interior supports. The effects of different diaphragm layouts are discussed in the next section.

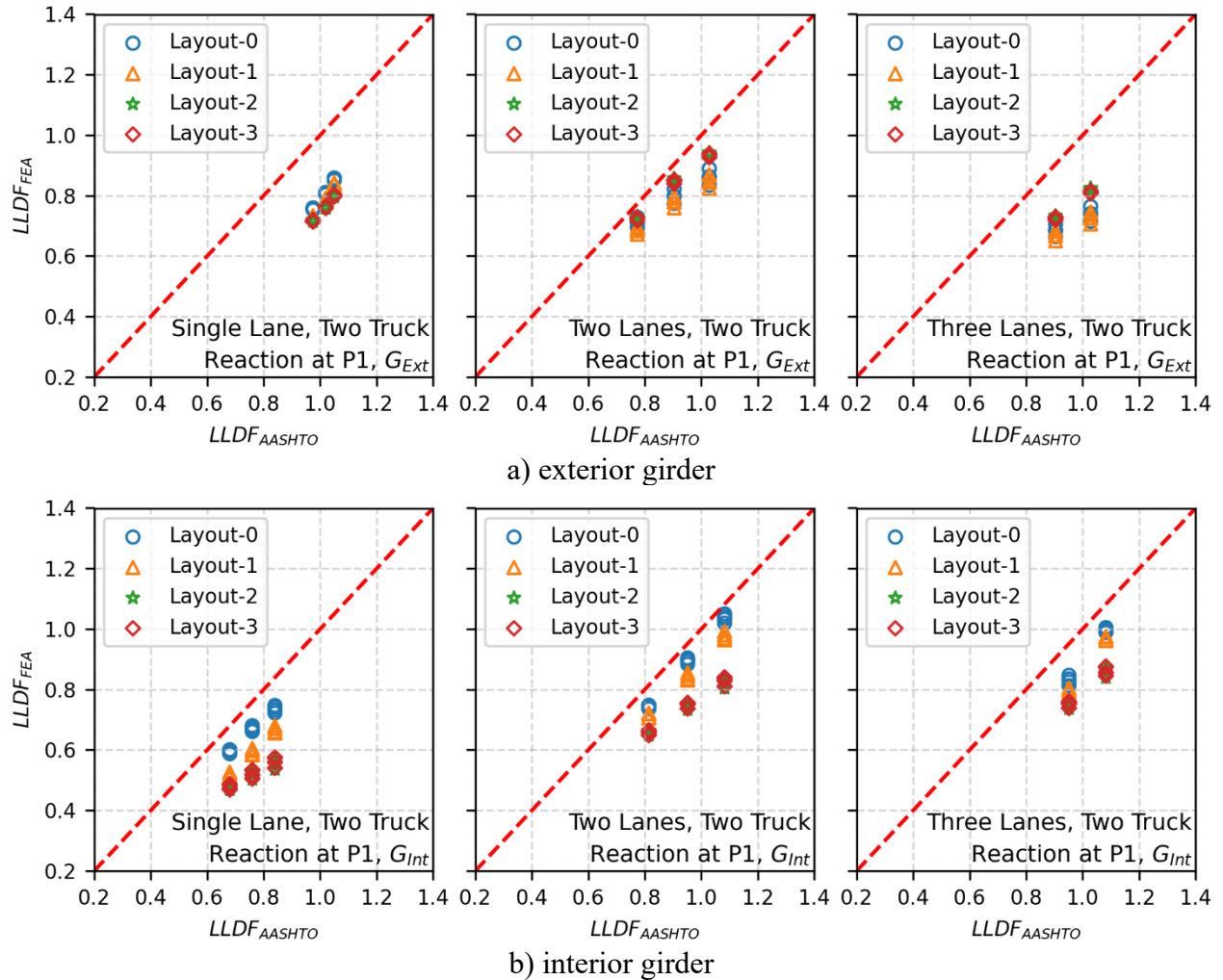


Figure 6.27. Comparison of LLDFs from FEA and AASHTO for vertical reactions at interior support P1

6.5. Effect of Girder Depth Variation

The increased girder depth near the interior supports in variable-depth bridges may be economical for long-span spliced girder bridges. In the parametric study for bridges with span-to-depth ratio of 30, both constant-depth and variable-depth girders were modeled. The live load effects are compared in both bridge types for the same bridge geometry in this section. AASHTO (2024) recommends considering average longitudinal stiffness when there is variation of stiffness along the bridge length. Furthermore, the longitudinal stiffness parameter (K_g) for some variable-depth bridges exceeds the maximum limits specified for using the AASHTO LLDF equations. The comparison of LLDFs obtained from FEA and AASHTO are also discussed in this section.

6.5.1. Comparison of Constant-Depth and Variable-Depth Bridges

The LLDFs were calculated following the procedure explained in the previous section. However, in the line-girder analysis for variable-depth bridges, the variation in girder section properties were considered for the tapered regions. The pier segment was divided into ten segments and the girder

section at the middle of the segment was considered for each segment. A schematic representation showing the variation of girder section properties in the line-girder model of a variable-depth bridge is shown in Figure 6.28. The LLDFs from FEA for variable-depth and constant-depth bridges of the same geometry were compared by calculating the ratio of the corresponding values. The minimum and maximum values of the LLDF ratios for the live load effects in exterior and interior girders are summarized in Table 6.4.



Figure 6.28. Schematic representation of a line-girder model for a variable-depth bridge

Table 6.4 Minimum and Maximum Ratios of $LLDF_{variable-depth}$ to $LLDF_{constant-depth}$

Live Load Effect			Layout-0	Layout-1	Layout-2	Layout-3
Positive Moment at 1-1	Exterior Girder	min	0.97	0.97	0.97	0.97
		max	1.01	1.01	1.01	1.01
	Interior Girder	min	1.01	1.01	1.01	1.01
		max	1.02	1.02	1.02	1.02
Positive Moment at 2-2	Exterior Girder	min	1.00	1.00	1.00	1.01
		max	1.03	1.03	1.03	1.03
	Interior Girder	min	1.03	1.03	1.03	1.01
		max	1.04	1.04	1.04	1.02
Negative Moment at P1	Exterior Girder	min	0.99	0.98	0.98	0.99
		max	1.06	1.04	1.04	1.04
	Interior Girder	min	1.01	0.99	0.96	0.96
		max	1.08	1.06	1.02	1.02
Reaction at A1	Exterior Girder	min	0.99	0.98	0.98	0.98
		max	1.00	1.00	1.00	1.00
	Interior Girder	min	1.00	1.00	1.00	1.00
		max	1.01	1.01	1.01	1.01
Reaction at P1	Exterior Girder	min	0.99	0.99	1.00	1.00
		max	1.02	1.04	1.04	1.05
	Interior Girder	min	1.02	0.91	0.92	0.91
		max	1.06	0.97	0.97	0.97

6.5.1.1. Bending Moments

The ratios of variable-depth bridge LLDFs with respect to constant-depth bridge LLDFs are plotted in Figure 6.29 and Figure 6.30, for positive bending moments at Section 1-1 and 2-2, respectively. The LLDFs for positive moments were approximately equal in variable-depth and constant-depth bridges. The difference between the LLDFs in variable-depth and constant-depth bridges was not more than 4% in all cases for positive bending moments. The LLDFs for positive moments in the middle of the central span were marginally higher than the LLDFs for positive moments in the end

span. The ratio of LLDFs was less than 1.0 for the positive moment at section 1-1 in exterior girders only. The ratios were approximately equal in bridges with different diaphragm layouts because the diaphragm layouts had the same effects irrespective of the variation in girder stiffness.

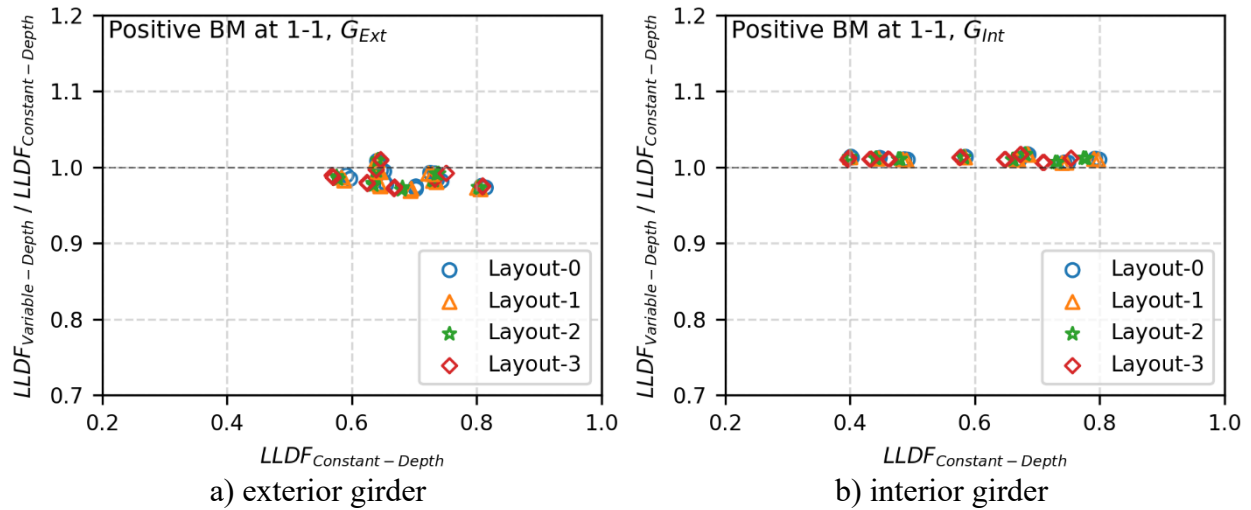


Figure 6.29. Effect of girder depth variation on positive moment at section 1-1

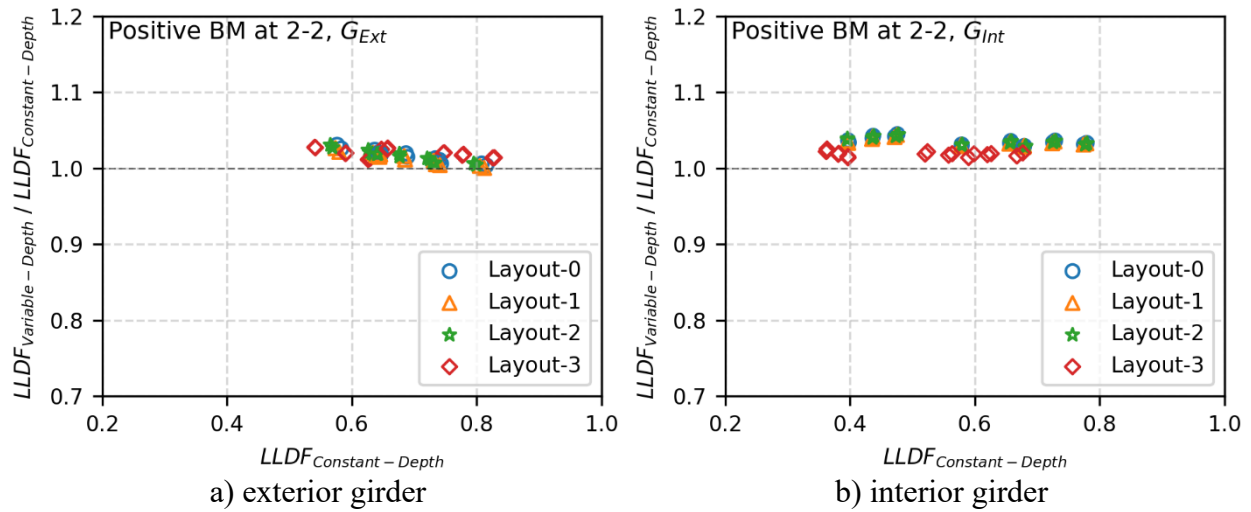


Figure 6.30. Effect of girder depth variation on positive moment at section 2-2

The increased girder depth at the interior supports influenced the LLDFs for negative moments at that location. The ratio of LLDFs for variable-depth and constant-depth bridges are plotted in Figure 6.31 for the negative bending moments. The LLDFs are plotted only for the two succeeding design trucks cases. The largest difference between the LLDFs (maximum 11%) was seen in bridges without any diaphragms (Layout-0). The ratios for Layout-2 and Layout-3 were approximately equal. The LLDFs for the support moments in variable-depth bridges were higher than constant-depth bridges by not more than 8% for bridges with support diaphragms.

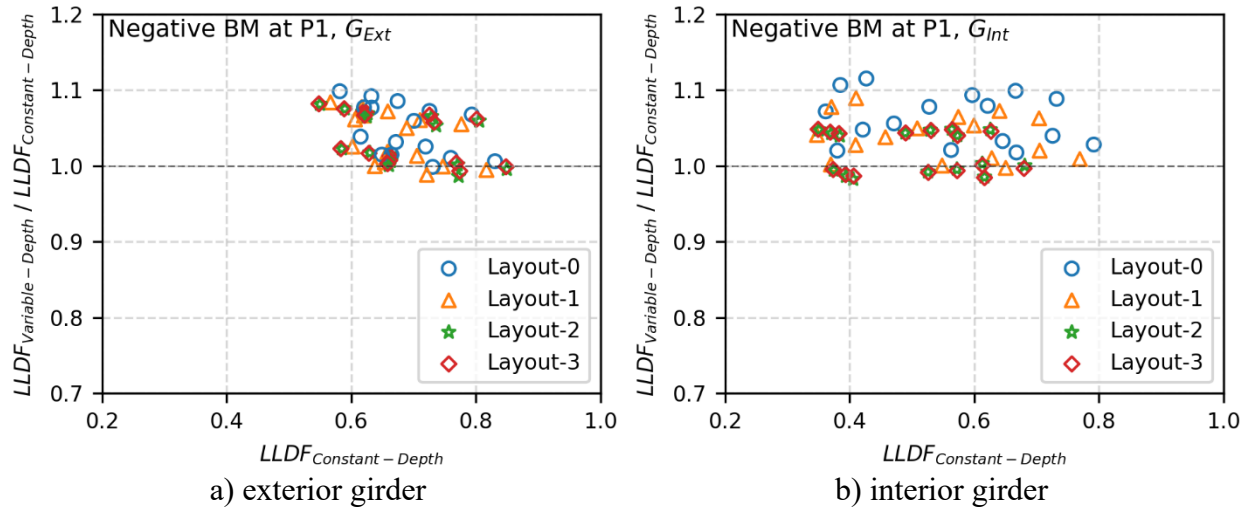


Figure 6.31. Effect of girder depth variation on negative moment at interior support P1

6.5.1.2. Shear Reactions

The ratio of shear reaction LLDFs in variable-depth bridges with respect to the values in constant-depth bridges of the same geometry are plotted in Figure 6.32 and Figure 6.33, for end supports and interior supports, respectively. The girder depth variation had a negligible difference in LLDFs for reactions at end supports as observed in the Figure 6.32 for both exterior and interior girders. However, the increased girder stiffness near the interior supports influenced the LLDFs for vertical reactions at the interior supports, especially in the interior girders. The corresponding LLDFs were lower than the values in constant-depth bridges for all the bridge models with support diaphragms in this parametric study. The largest difference in LLDFs was observed in the bridge model with widely spaced ($S = 12$ ft), in which the LLDFs for interior support reaction reduced by 9% when variable-depth girder segments were used in place of constant-depth girders. Furthermore, the LLDF ratios were approximately equal for bridges with Layout-2 and Layout-3, both of which had diaphragms at the splices.

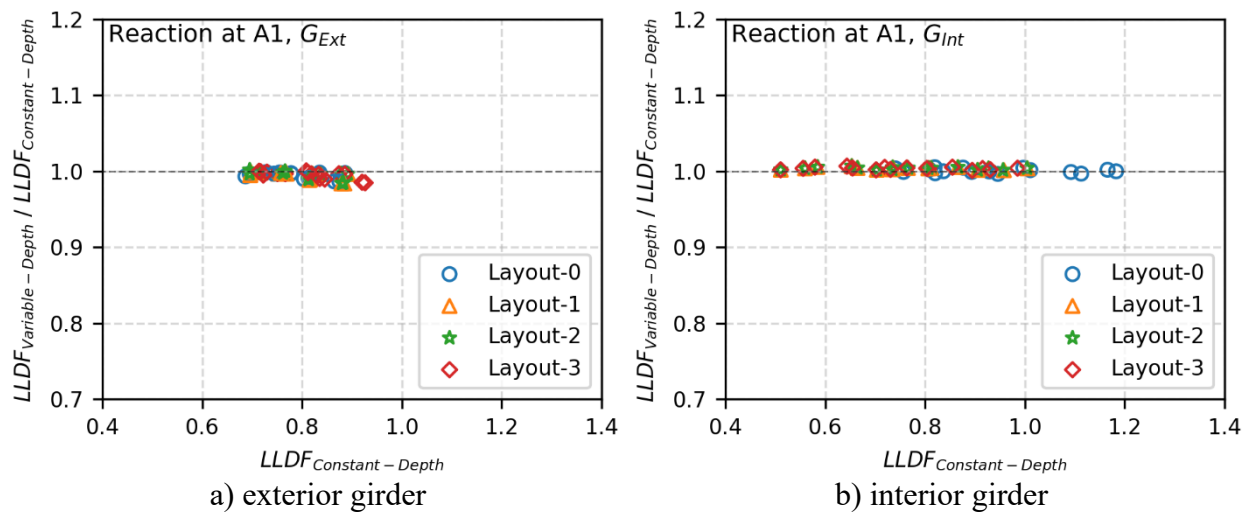


Figure 6.32. Effect of girder depth variation on vertical support reaction at end support A1

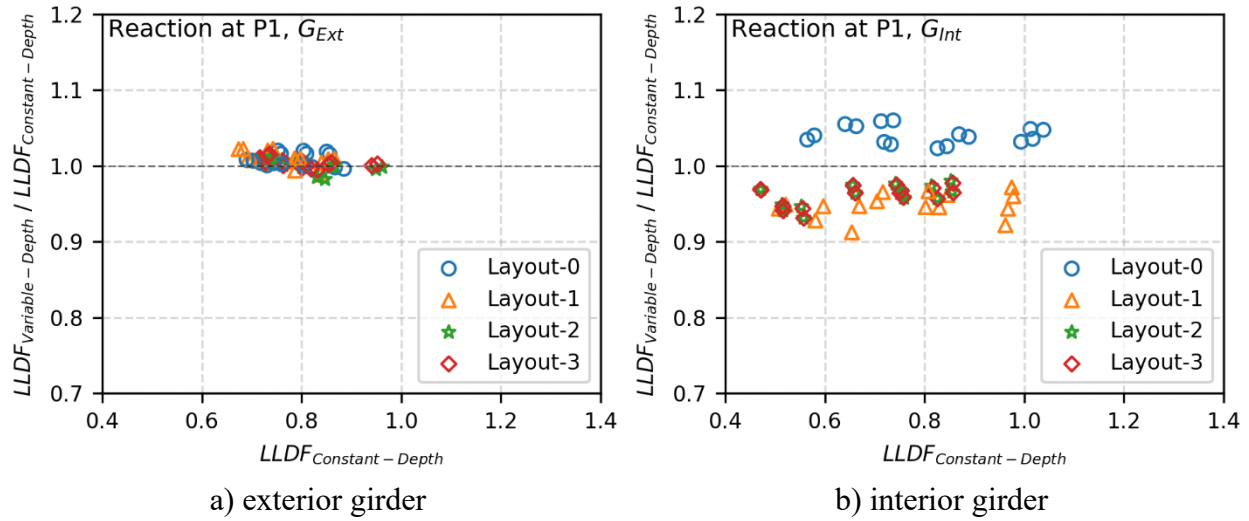


Figure 6.33. Effect of girder depth variation on vertical support reaction at interior support P1

6.5.2. Comparison of LLDFs from FE Analysis and AASHTO

The LLDF equations specified by AASHTO (2024) do not differentiate between constant-depth and variable-depth girders. However, for girders with variation in moment-of-inertia such as in variable-depth girder bridges, the commentary *C4.6.2.2.1* allows the use of the distribution factors with average longitudinal stiffness parameter (K_g). The average K_g value for variable-depth girders was calculated as the weighted average of the K_g values for each segment of the girders. The average K_g values for bridges with STx7 and STx9 girders were 8,977,920 and 16,713,893 in.⁴, both exceeding the maximum limit (7,000,000 in.⁴) specified by AASHTO for using LLDF equations. The AASHTO LLDFs were calculated for the variable-depth bridges even though the equations are not applicable. The LLDFs calculated from FEA are compared with the AASHTO values in this section by calculating the ratio of the corresponding values. In this section, only the maximum values obtained in single-lane and multi-lane cases are considered for brevity. Nonetheless, only the maximum values are of concern for design purposes.

6.5.2.1. Bending Moments

The ratios of FEA LLDFs to AASHTO values are plotted against the AASHTO LLDFs for positive bending moment at section 1-1 and 2-2 in variable-depth bridges with normal supports in Figure 6.34 and Figure 6.35, respectively. The plots indicate the AASHTO LLDFs for exterior girders are too conservative compared to the FEA results for the bridges considered in this study. The high level of conservatism may be due to the wide deck overhang which was not varied in the parametric study. The equations specified for moment in interior girders were conservative by 11 to 21% for the positive bending moment at section 1-1. For positive moment at section 2-2, the AASHTO LLDFs were approximately equal to the values calculated from FEA for variable-depth bridges with span-to-depth ratio of 35 and diaphragm layouts without mid-span diaphragms (Layout-0, Layout-1 and Layout-2). Although the span lengths for the bridges in the study were longer than the limits specified by AASHTO, the LLDF equations may be used for calculating live load

moments. The interior girder mid-span moments in bridges with Layout-3 were lower than the bridges with other diaphragm layouts. The LLDF equations specified in *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2024) were derived assuming the diaphragms do not participate in live load distribution (Zokaie et al., 1991). Therefore, the AASHTO LLDFs for positive moment at section 2-2 were 9 to 21% higher than the values obtained from FEA in bridges with Layout-3.

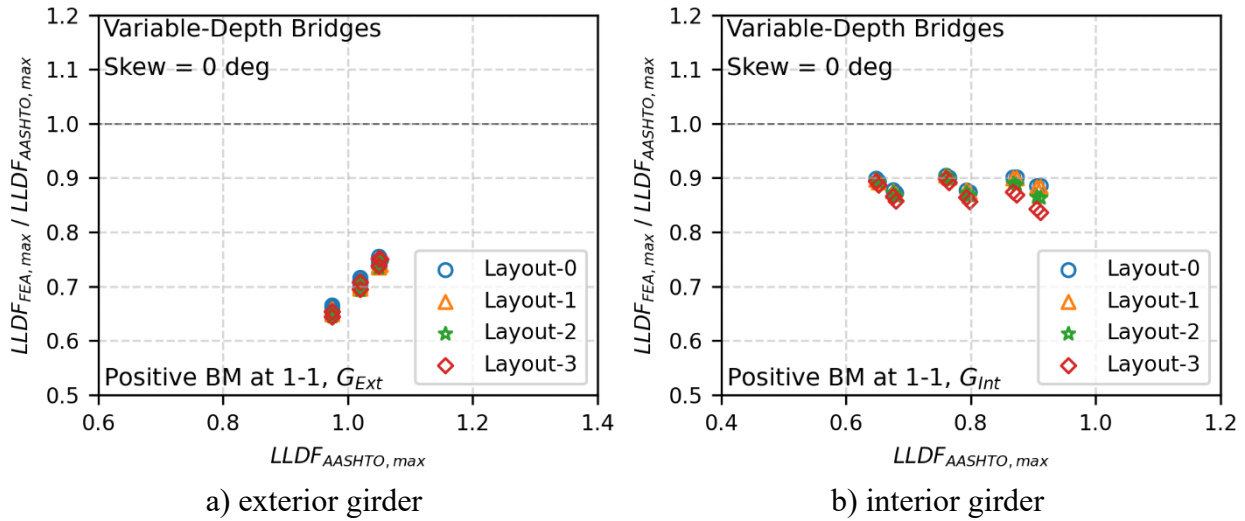


Figure 6.34. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 1-1

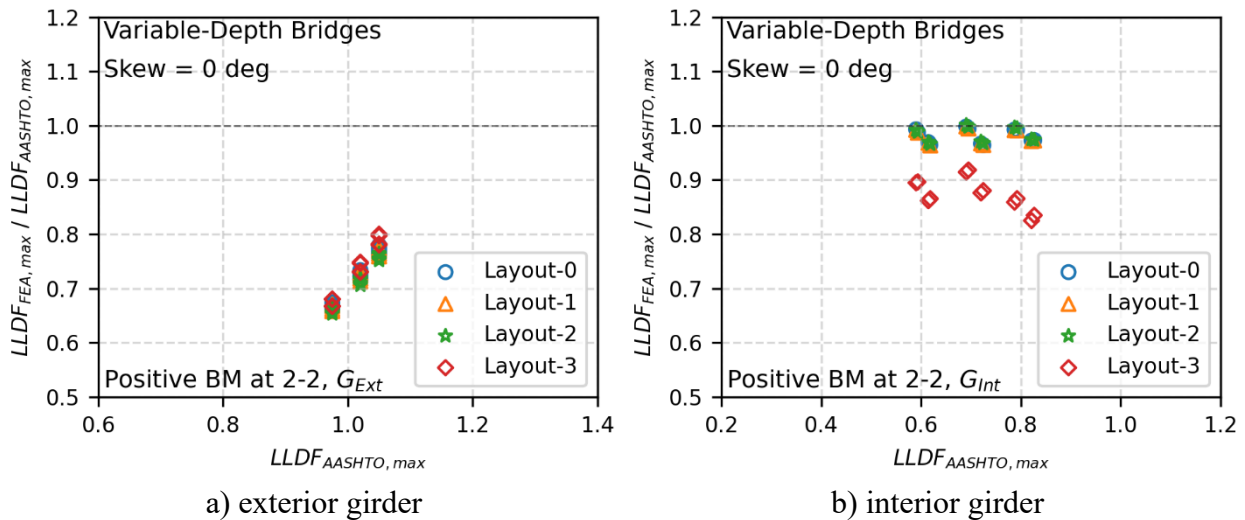


Figure 6.35. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 2-2

The ratio of $LLDF_{FEA}$ to $LLDF_{AASHTO}$ for negative bending moment at the interior support P1 are plotted in Figure 6.36. AASHTO LLDFs were 16 to 47% higher than the values obtained in FEA for exterior girders, similar to the positive moment LLDFs. In case of interior girders, the AASHTO LLDFs were approximately equal to the FEA values for few bridges with Layout-0. The support diaphragms reduce the moment at the interior supports resulting in conservative estimates by AASHTO equations. The LLDFs for support moments were not influenced by the interior

diaphragms at mid-span. In bridges with diaphragms at the supports and splices, the AASHTO equations result in 11 to 28% higher LLDFs than FEA.

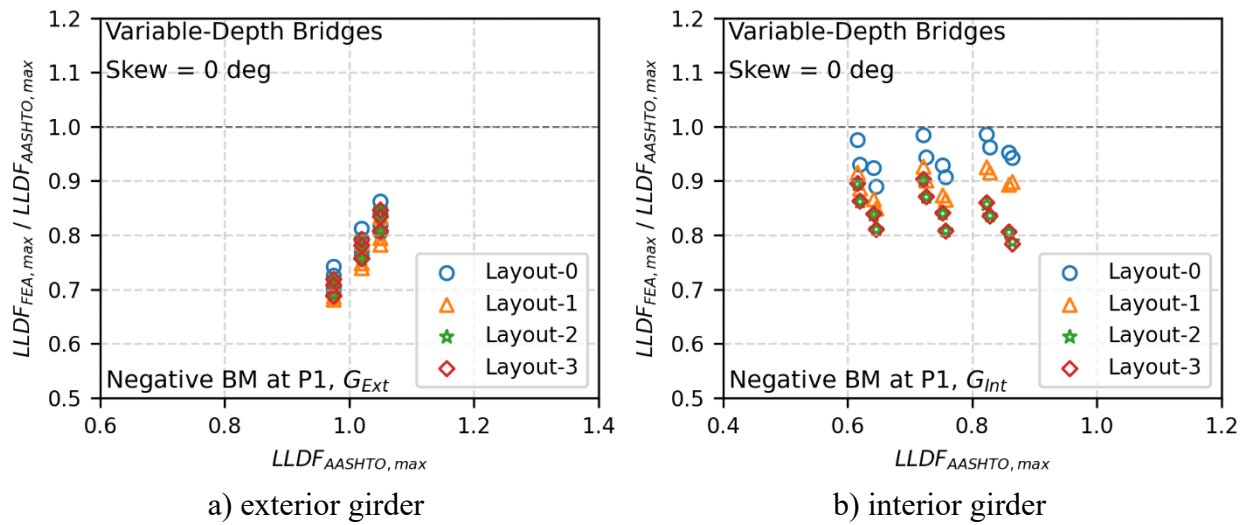


Figure 6.36. Comparison of LLDFs from FEA and AASHTO for negative bending moment at support P1

6.5.2.2. Shear Reactions

The LLDFs for shear reaction at end support obtained from FEA were lower than the AASHTO values for variable-depth bridges similar to the constant-depth bridges. The ratio of LLDFs from FEA with respect to AASHTO values are plotted in Figure 6.37 and Figure 6.38 for end support and interior support reactions in variable-depth bridges. The ratios range between 0.75 and 0.95 for reactions in exterior girder, therefore the AASHTO LLDFs are conservative by 5 to 31% compared to FEA results. The ratio of LLDFs for end support reactions were lower than the values for interior support reactions.

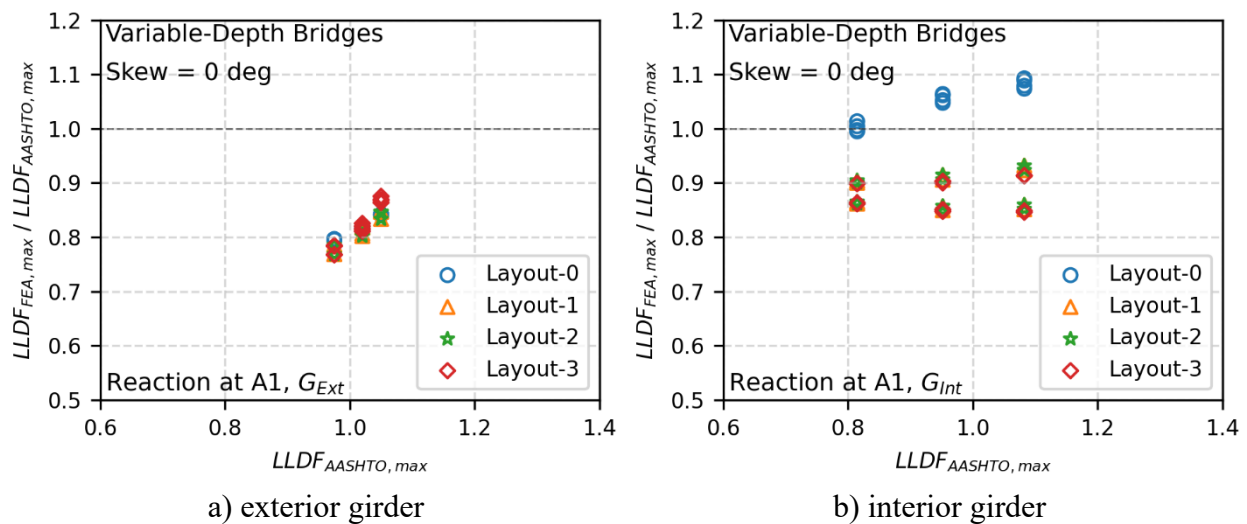


Figure 6.37. Comparison of LLDFs from FEA and AASHTO for vertical reactions at end support A1

The AASHTO LLDFs were lesser than the FEA values for the interior girders in bridges without support diaphragms. As explained in the previous section for constant-depth bridges, the AASHTO equations are not applicable for bridges without support diaphragms. In all other cases, the AASHTO LLDFs for interior girder were conservative by 11 to 24% for interior support reactions and by 14 to 30% for end support reactions when compared to FEA values. The ratios were approximately the same for bridges with other diaphragm layouts.

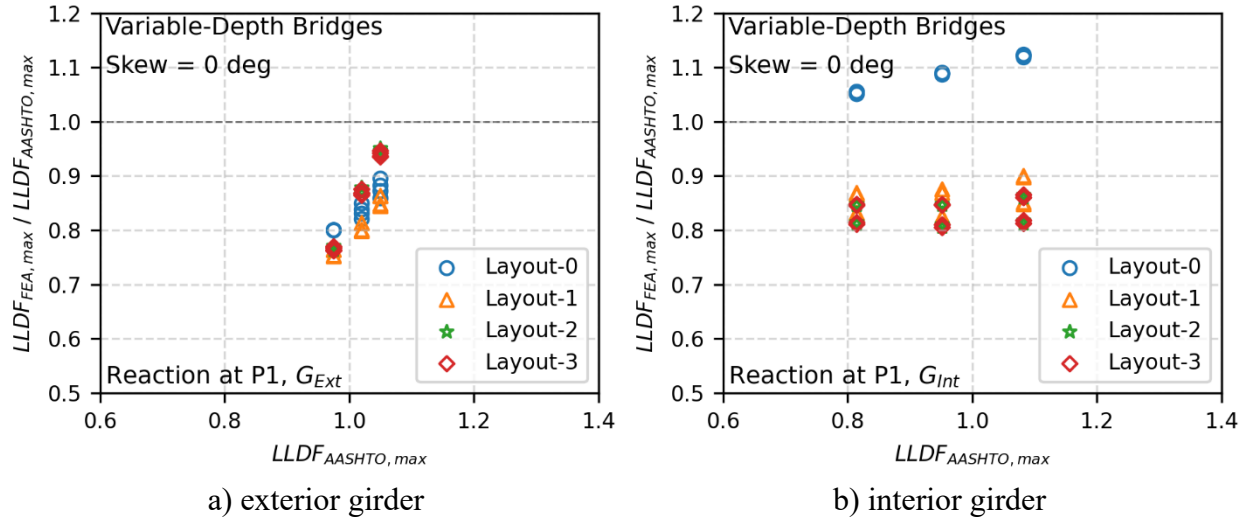


Figure 6.38. Comparison of LLDFs from FEA and AASHTO for vertical reactions at interior support P1

6.6. Effect of Diaphragms

Diaphragms in concrete girder bridges provide alternative load paths in addition to the deck slab. Four different diaphragm layouts investigated in parametric study. AASHTO (2024) recommends providing diaphragms at all supports, which corresponds to Layout-1 in this study, to ensure proper load distribution. The bridges with Layout-0 had no diaphragms. Simply supported concrete I-girder bridges in Texas are built with no permanent diaphragms. However, long-span spliced girder bridges require diaphragms at least at the supports to ensure stability during construction. Hence, the effects of different diaphragm layouts were studied with respect to the Layout-1 configuration, which is a minimum requirement for spliced girder bridges, both according to AASHTO and TxDOT practices. The LLDFs for bridges with different layouts were normalized with respect to the corresponding bridges with Layout-1. Therefore, the normalized LLDFs will be always equal to 1.0 for bridges with Layout-1. The normalized LLDFs of constant-depth spliced bridges with normal supports are compared in the following sections to understand the influence of diaphragms. There were 12 bridge geometries with four different diaphragm layouts studied.

6.6.1. Positive Moment at Section 1-1

The variation of LLDFs for positive moment at section 1-1 in exterior and interior girders is shown in Figure 6.39. Each bridge geometry with four configurations of diaphragm layouts is plotted as a line in this figure. The moments at midspan are not influenced by the presence of support

diaphragms, because the exterior and interior girder moments were approximately equal in bridges with Layout-0 and Layout-1. The diaphragms at splices reduced the moments in the exterior girder for all single-lane cases and for a few two-lane cases. But for most cases where multiple lanes were loaded, the exterior girder moments were reduced due to the addition of diaphragms at splices and at the middle of spans. In the exterior girder, the maximum normalized LLDF for Layout-3 was 1.06 for three-lane cases and the minimum value was 0.95 for single-lane cases. The minimum normalized LLDF for interior girders was 0.90 and 0.92 for single-lane and multiple-lane cases, respectively. The interior girder bending moment LLDF reduced for all cases in bridges with Layout-2 and Layout-3, when compared to bridges with Layout-0 or Layout-1. The diaphragm at the middle of the first span in Layout-3 was closer but not exactly at the location of section 1-1. Therefore, the influence of mid-span diaphragm was not as significant as seen in section 2-2.

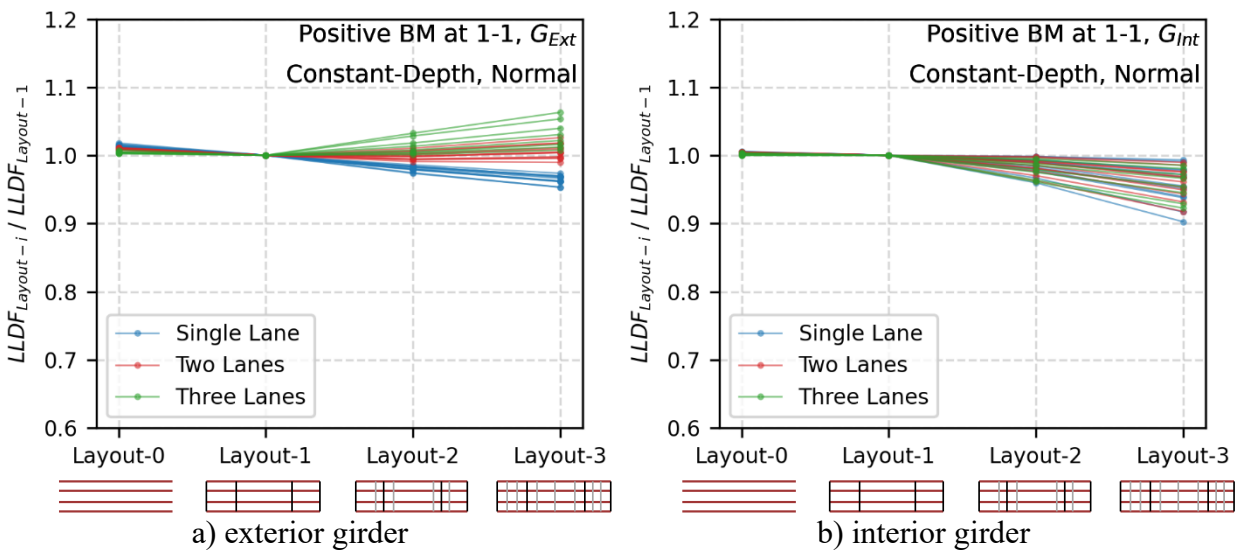


Figure 6.39. Effect of diaphragm layout on positive moment at section 1-1

6.6.2. Positive Moment at Section 2-2

The variation of normalized LLDFs between bridges with different layouts is shown in Figure 6.40 for positive bending moment at section 2-2 in exterior and interior girders. The LLDFs were approximately equal for bridges with Layout-0, Layout-1 and Layout-2. The largest difference was seen in bridges with Layout-3 due to the diaphragms at the middle of the drop-in segments. The diaphragm was located exactly at section 2-2; therefore, the live load distribution was significantly influenced by the diaphragm in Layout-3. The differences were qualitatively similar to the differences in moment at section 1-1. In the exterior girder, the single-lane cases had lesser normalized LLDFs, and the multiple-lane cases had higher values. The minimum value was 0.91 and the maximum value was 1.08. The interior girder had reduced LLDFs in Layout-3. The lowest normalized values in the interior girder were 0.80, 0.79, and 0.84 for single-lane, two-lane and three-lane cases, respectively. The normalized values in Layout-2 were not less than 0.99, indicating the difference was mainly due to the diaphragm at the midspan.

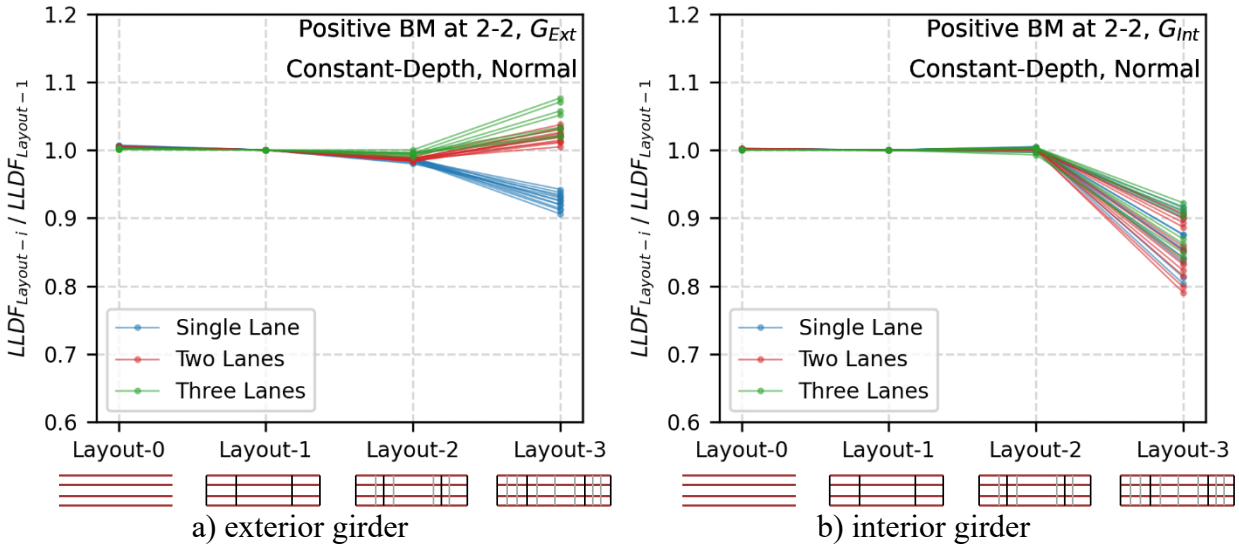
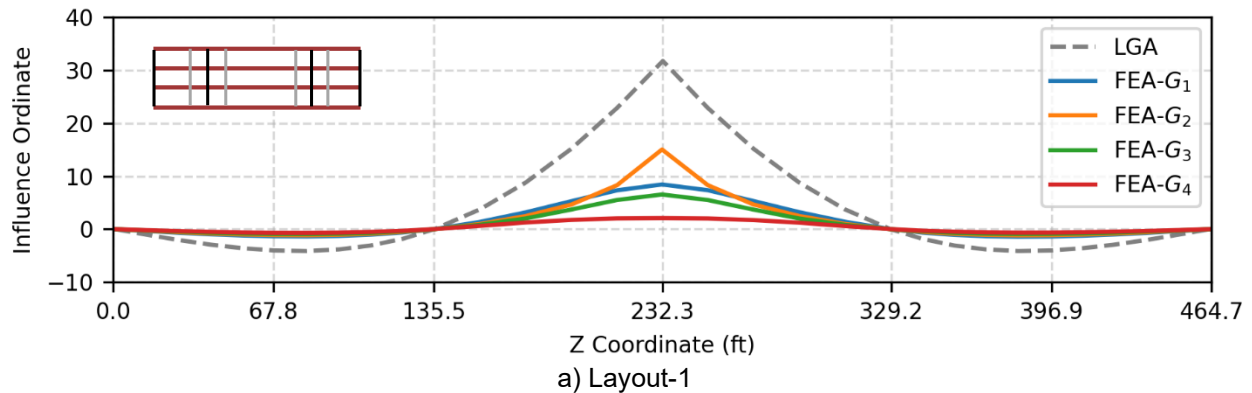


Figure 6.40. Effect of diaphragm layout on positive moment at section 2-2

The influence of the mid-span diaphragm on the moment at section 2-2 was investigated using the influence surfaces. The line plots of the longitudinal cuts on the influence surface for positive moment in the interior girder G2 at section 2-2 are shown in Figure 6.44 for a bridge with Layout-2 and Layout-3. The only difference between the layouts was the additional interior diaphragms at the middle of drop-in segments. The line plots for girder G2 indicate lesser moment in the bridge with Layout-3 when the unit load is exactly at mid-span. Additionally, the influence ordinate of girder G1 is higher than the bridge without mid-span diaphragms. This comparison of influence surface indicates the change in the load path due to the diaphragm. Moreover, the critical vehicle position for positive moments at section 2-2 is at the middle of the span length based on the influence surface. Therefore, the moments were significantly reduced in bridges with Layout-3.



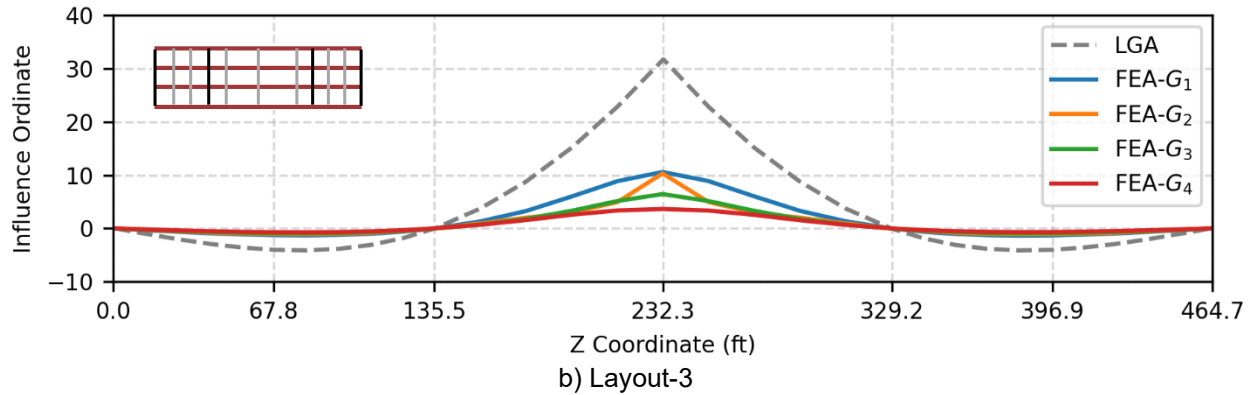


Figure 6.41. Longitudinal cuts on the influence surface for vertical reaction (kip) at left end support A1 in exterior girder G1

6.6.3. Negative Moment at Support P1

The normalized LLDFs for negative moments at first interior support P1 are plotted in Figure 6.42. The negative moment LLDFs in bridges with Layout-0 had higher values than the same bridge geometries with Layout-1, however, the increase was not significant. The LLDFs decreased or increased significantly in cases where diaphragms were considered at the splice locations in addition to support diaphragms. For exterior girders, the LLDFs are reduced in bridges with Layout-2 or Layout-3 for single-lane cases because of the less influence of diaphragms when the design truck is on the overhang region. The lowest normalized LLDF in exterior girder was 0.93 for single-lane cases, and highest value was 1.07 for three-lane cases. The largest reduction in interior girder LLDF was found in the single-lane case with a normalized LLDF of 0.78. The values in bridges with Layout-2 and Layout-3 were approximately equal for most cases. The diaphragms at the splices are closer to the interior supports where the negative moments were calculated. Therefore, Layout-2 and Layout-3 had significant influence in the distribution of load for negative moments. The additional interior diaphragms at the mid-span does not affect the load distribution in the girders.

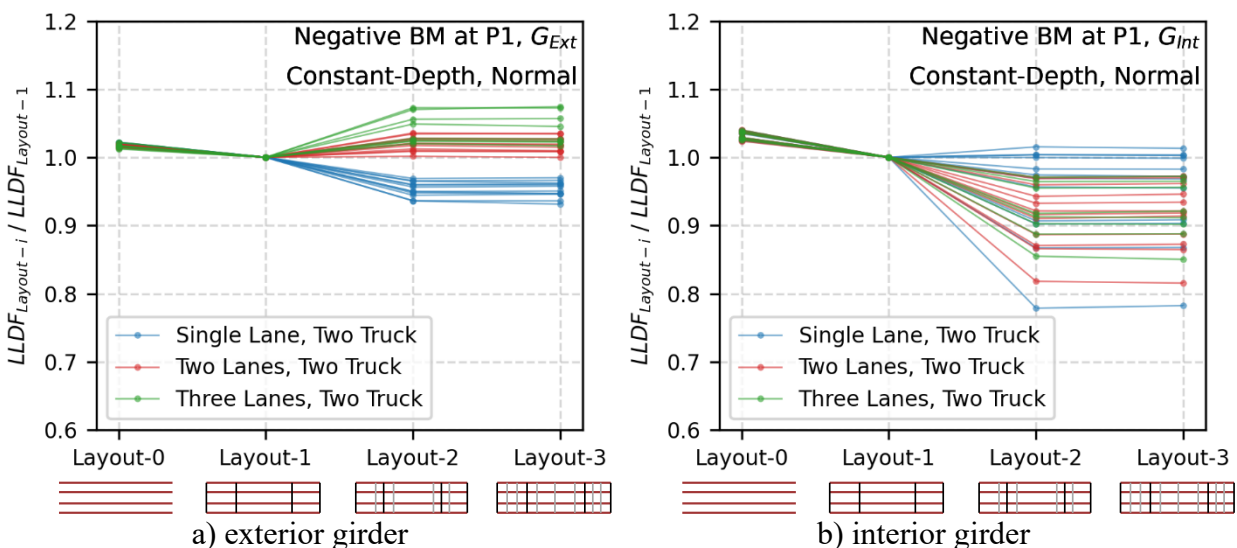


Figure 6.42 Effect of diaphragm layout on negative moment at interior support P1

6.6.4. Reaction at Support A1

The normalized LLDFs of reactions at the end support A1 are plotted in Figure 6.43. The exterior girder reactions were approximately equal for the layouts without mid-segment diaphragms. In bridges with Layout-3, the LLDFs were higher than in other cases, especially for multiple loaded lanes. It was due to the mid-segment diaphragm distributing the truck loads from the interior of the bridge to the exterior girder.

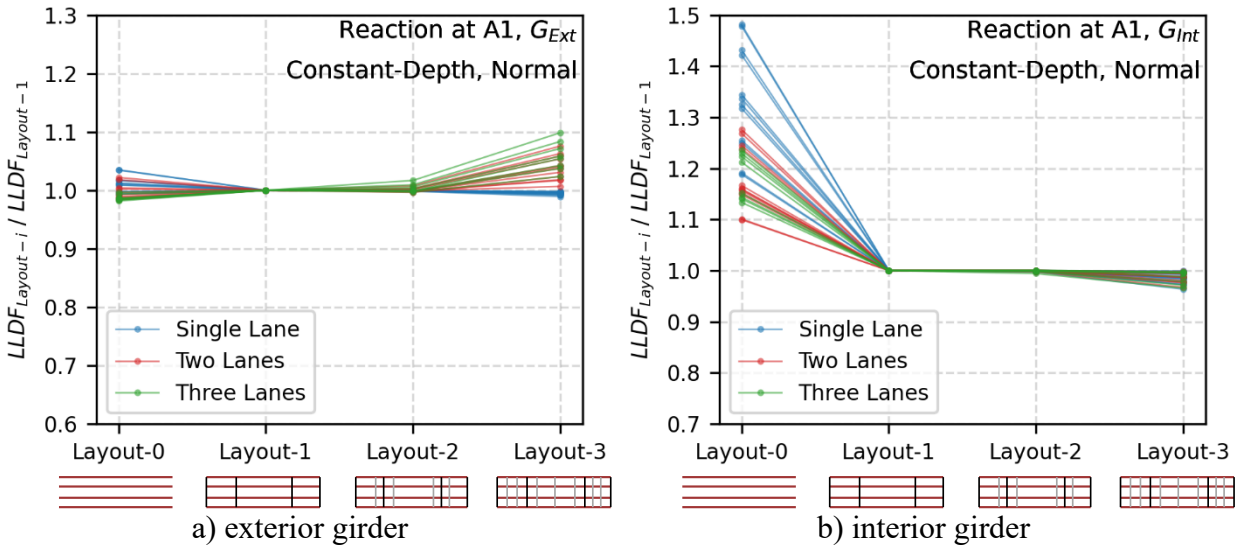


Figure 6.43. Effect of diaphragm layout on vertical reaction at end support A1

The plot for interior girder (Figure 6.43b) emphasizes the importance of support diaphragms in distributing the loads between the girders. The reactions in bridges with Layout-0 were 19 to 48% higher than the values in bridges with support diaphragms only (Layout-1) under a single loaded lane. In cases of multiple lanes, the reactions were higher by 10 to 28%. The interior diaphragms at supports had a negligible effect on interior girder reactions. However, the mid-segment diaphragms reduced the reaction due to a better load distribution. The lowest normalized LLDF was 0.96 for bridges with Layout-3. The influence surfaces provide a better understanding of the change in load distribution due to the support diaphragms. The longitudinal cuts along the centerline of the girders on the influence surface for vertical reaction at the support A1 in the interior girder G2 is shown in Figure 6.44. The influence line diagram does not change for the same bridge with different diaphragm layouts. The cut along the girder G2 shows the higher reactions in Layout-0 compared to Layout-1, when the loads are near the girder G2. The cut along the girder G1 reveals the load distribution by the support diaphragms. The influence ordinate is equal to zero at the end support of Girder G1 in Layout-0. But in Layout-1, the influence ordinates are higher due to the support diaphragms.

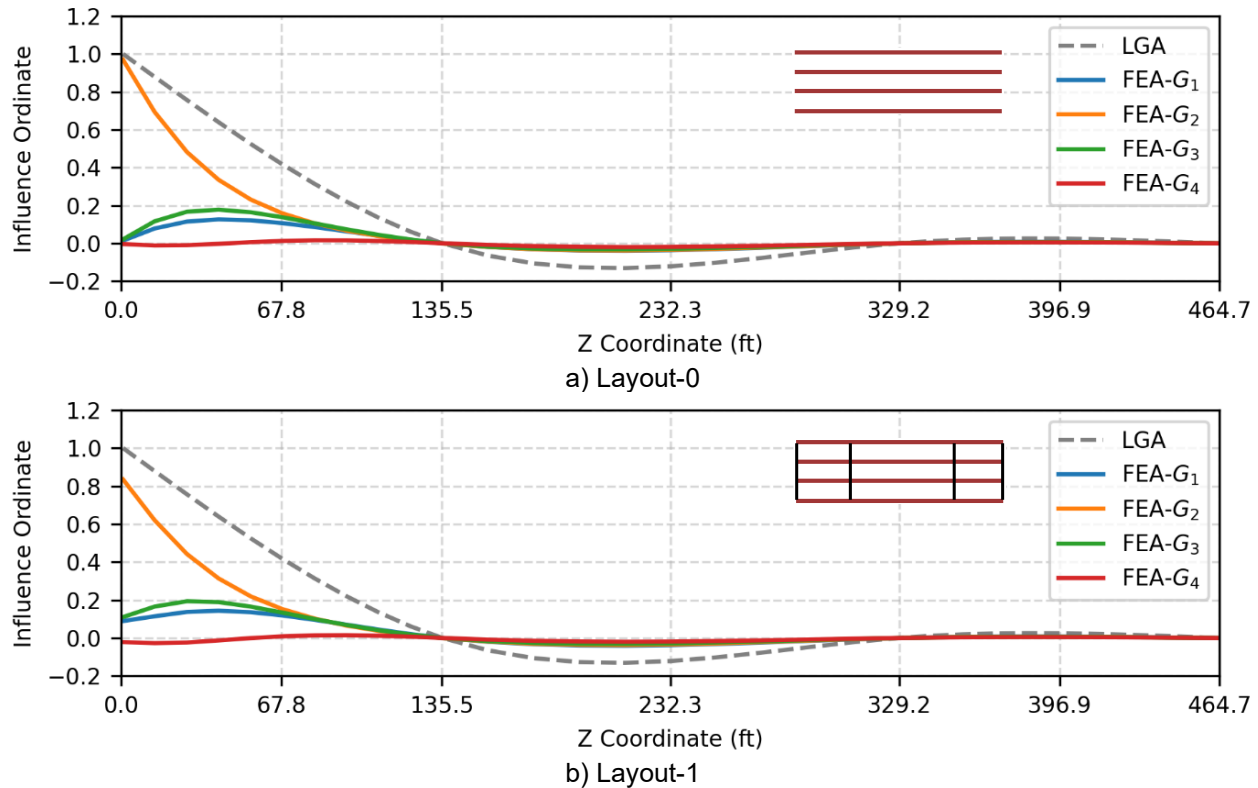


Figure 6.44. Longitudinal cuts on the influence surface for vertical reaction (kip) at left end support A1 in exterior girder G1

6.6.5. Reaction at Support P1

The variation of normalized LLDFs for vertical reaction at the interior support P1 is shown in Figure 6.45. The support diaphragms are necessary for the load distribution at the interior supports, especially for interior girders. But the normalized LLDFs were not as high as the values seen in reaction at the end supports. The highest value in bridges with Layout-0 was 1.17 for single-lane cases and 1.08 for multi-lane truck loads. The exterior girder reactions in single-lane cases were lower (0.95 to 0.98) in bridges with interior diaphragms, however, for multiple loaded lanes, the reactions were higher (1.03 to 1.15). The normalized LLDFs of interior girders were higher in Layout-0 and lower in Layout-2 and Layout-3 than the values in Layout-1. The maximum normalized LLDF for interior girder was 1.17 in bridges with Layout-0. In bridges with interior diaphragms at splices, the normalized LLDFs were between 0.81 and 0.95.

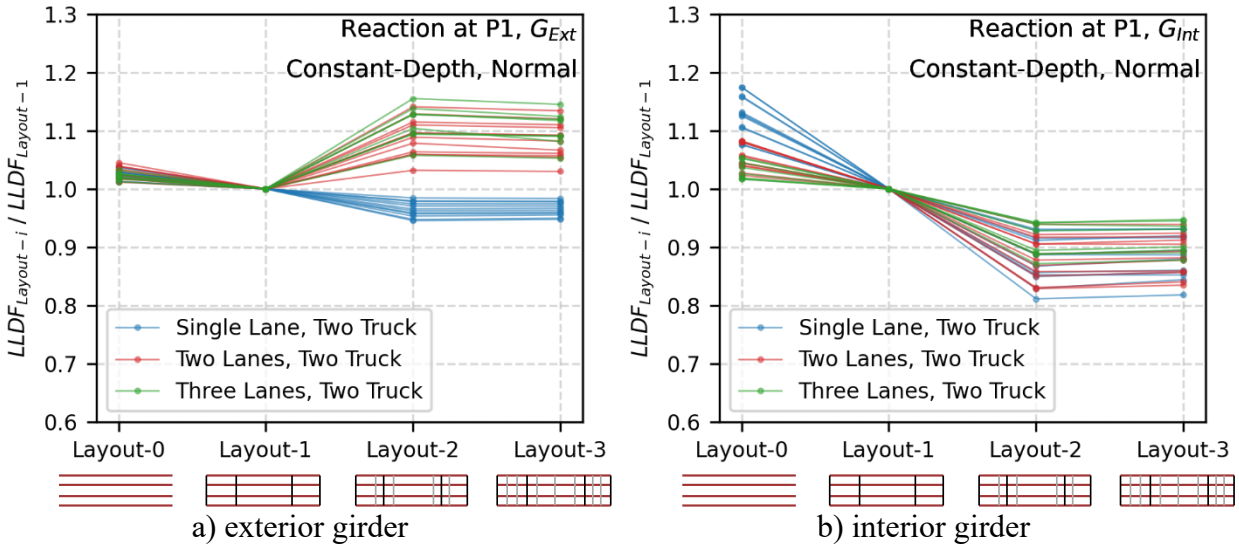


Figure 6.45. Effect of diaphragm layout on vertical reaction at interior support P1

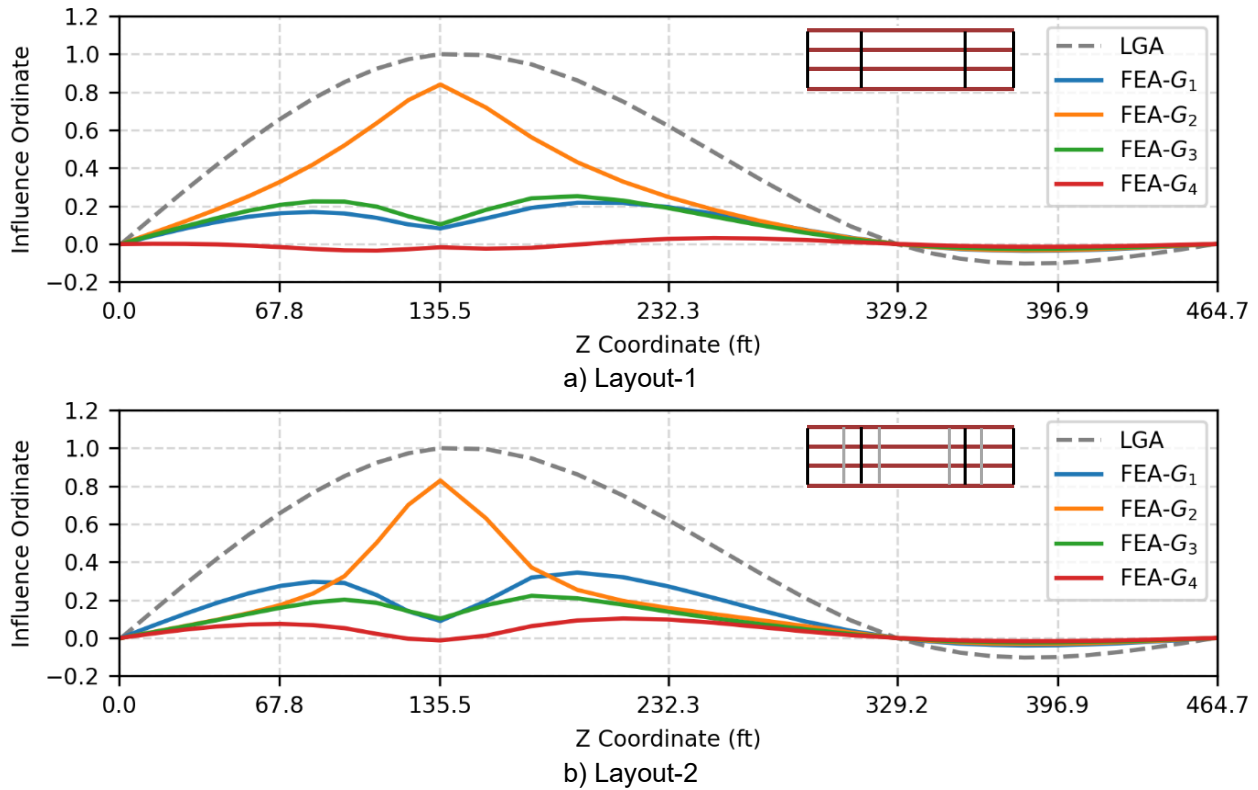


Figure 6.46. Longitudinal cuts on the influence surface for vertical reaction (kip) at interior support P1 in exterior girder G1

The role of diaphragms at splices can be understood from the influence surface cuts shown in Figure 6.46 for a bridge with Layout-1 and Layout-2. Both bridges had diaphragms at the supports, but the bridge with Layout-2 had additional interior diaphragms at the splices. The ordinates at the support P1 in the interior girder G2 were approximately equal in both cases. However, the values reduced significantly for the Z-coordinates further away from the interior support in Figure 6.46b.

The maximum reaction at support P1 occurs when there are two succeeding trucks in adjacent spans. Therefore, when the trucks are at adjacent spans, the reactions due to their loads are lower in the case of Layout-2 than in Layout-1. The load distribution from interior to exterior girders can also be seen in Figure 6.46 from the higher ordinates for girder G1 in Layout-2 compared with Layout -1, especially near the splice locations.

6.7. Skewed Bridges

The live load bending moments are reduced in bridges with skewed support lines when designed using the approximate analysis method of AASHTO (2024). However, the spliced girder bridges in the TxDOT 0-7115 project exceed the allowable limits for using the approximate analysis methods. Furthermore, the diaphragm layouts may influence the load distribution in skewed bridges. Therefore, the effect of support skew was investigated for spliced girder bridges with 30- and 45-degree angles in the parametric study. The bridges were modeled with parallel skew supports, and the diaphragms were also oriented parallel to the support lines. The FE models were developed using the same methodology adopted for normal-support bridges. The positions of unit loads applied for influence surface analysis were defined as shown in Figure 6.47 for a spliced girder bridge with 30-degree skew and Layout-3 diaphragm arrangement. The ratio of LLDFs from FE analysis of skewed bridges with respect to the LLDFs in normal-support bridges were calculated to study the effects of support skew.

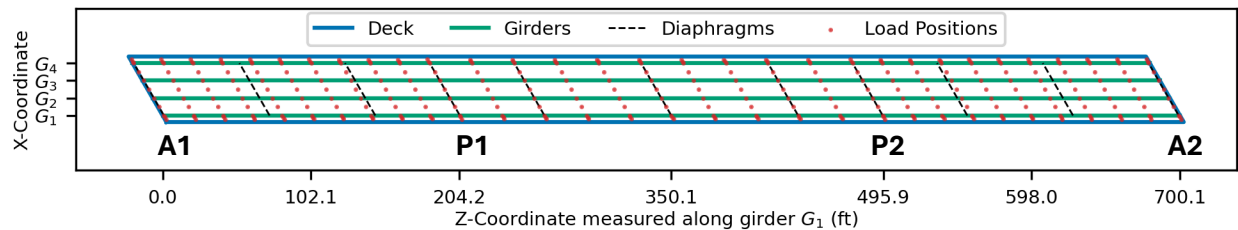


Figure 6.47. Load positions in a skewed spliced girder bridge

6.7.1. Effects of Support Skew on Moments

6.7.1.1. Positive Moments at Section 1-1 and 2-2

The ratio of LLDFs in skewed bridges with 30 skew angle with respect to the LLDFs in normal bridges are plotted in Figure 6.48, Figure 6.49, and Figure 6.50, for positive moment at section 1-1, positive moment at section 2-2 and negative moment at interior support P1, respectively. The ratios larger than 1.0 indicate an increase in LLDFs due to skewed supports, whereas ratios smaller than 1.0 indicate a reduction in LLDFs. The ratios are plotted for bridges with all four diaphragm layouts. In general, there was a decrease in bending moments in most cases. However, the diaphragms influenced the change in bending moments at section 2-2 and at support P1. In the previous sections, the different behavior of bridges with Layout-3 was observed for positive moment at section 2-2. Similarly, for negative moments at the interior support, the bridges with Layout-2 and Layout-3 had LLDFs significantly different from the bridges with Layout-0 and

Layout-1. In case of positive moment at section 1-1, the diaphragm layouts caused a small difference. Therefore, the moment reduction in skewed bridges was also consistent among bridges with different diaphragm layouts as observed in Figure 6.48a, and Figure 6.48b. The positive moment LLDFs were reduced by not more than 5% for 30-degree skew angle. Due to the interior diaphragms closer to the positive moment sections, the bridges with Layout-3 had the lowest ratios.

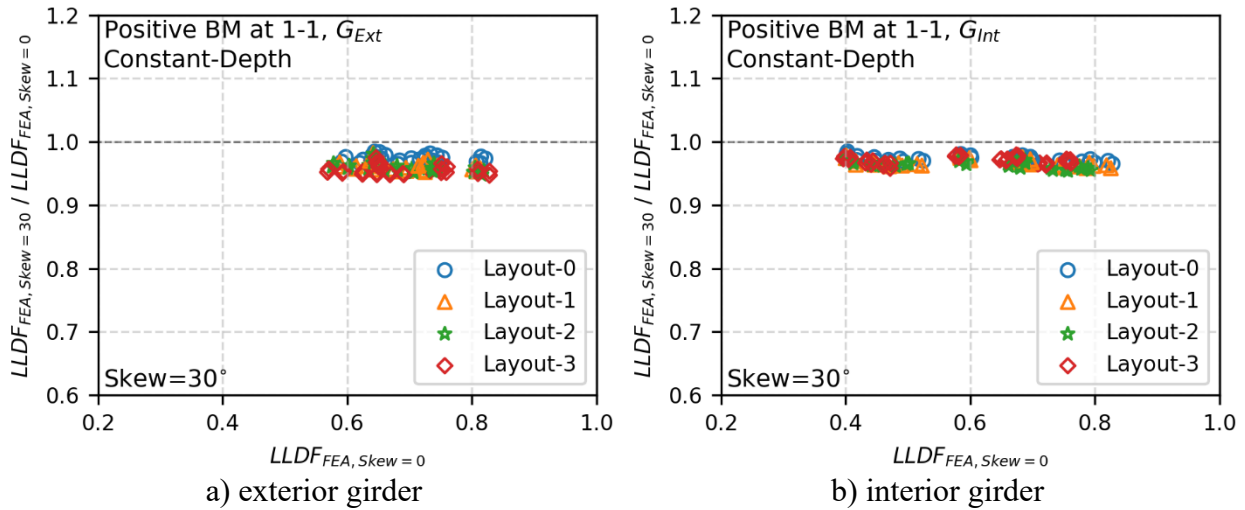


Figure 6.48 Normalized LLDFs for positive moment at section 1-1 in bridges with 30-degree skew

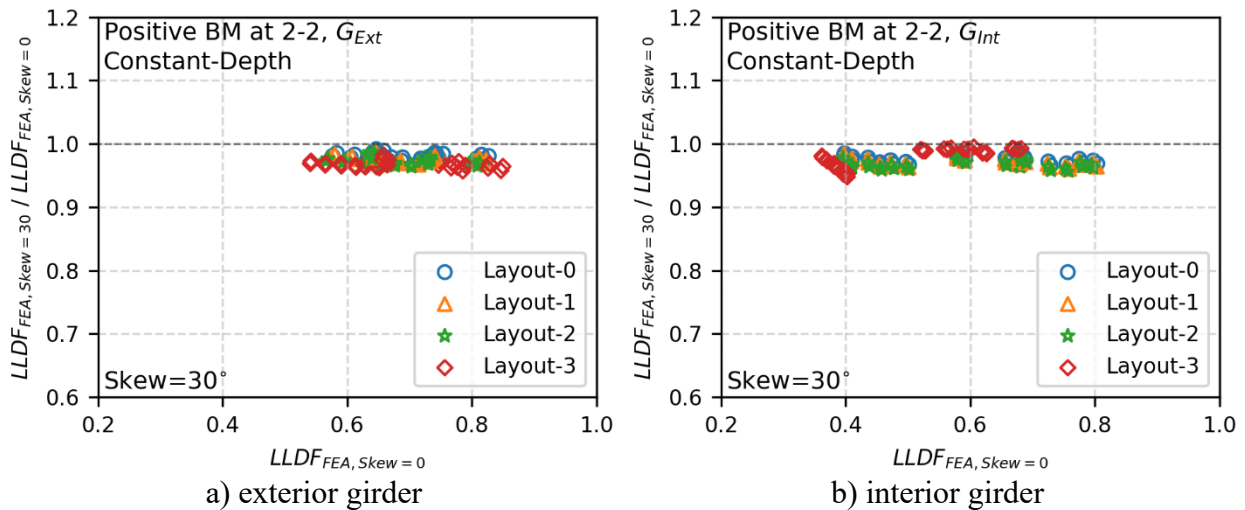


Figure 6.49. Normalized LLDFs for positive moment at section 2-2 in bridges with 30-degree skew

6.7.1.2. Negative Moment at Interior Support P1

The interior support negative moments in exterior girders of skewed bridges were higher than the values in normal bridges. The live load effects due to two succeeding trucks in each lane were only considered in the plots shown in Figure 6.50a, and Figure 6.50b. The ratio of LLDFs was between 0.97 and 1.15 for the exterior girders and between 0.95 and 1.07 for interior girders. The higher values in exterior girders were observed in bridges without interior diaphragms (Layout-0 and Layout-1). If interior diaphragms are provided, the loads are distributed to other girders thereby

reducing the moment in exterior girder. Due to the load distribution, the LLDF ratios for the interior girder were lesser in bridges with Layout-0 and Layout-1 and approximately unchanged from normal-support bridges for Layout-2 and Layout-3.

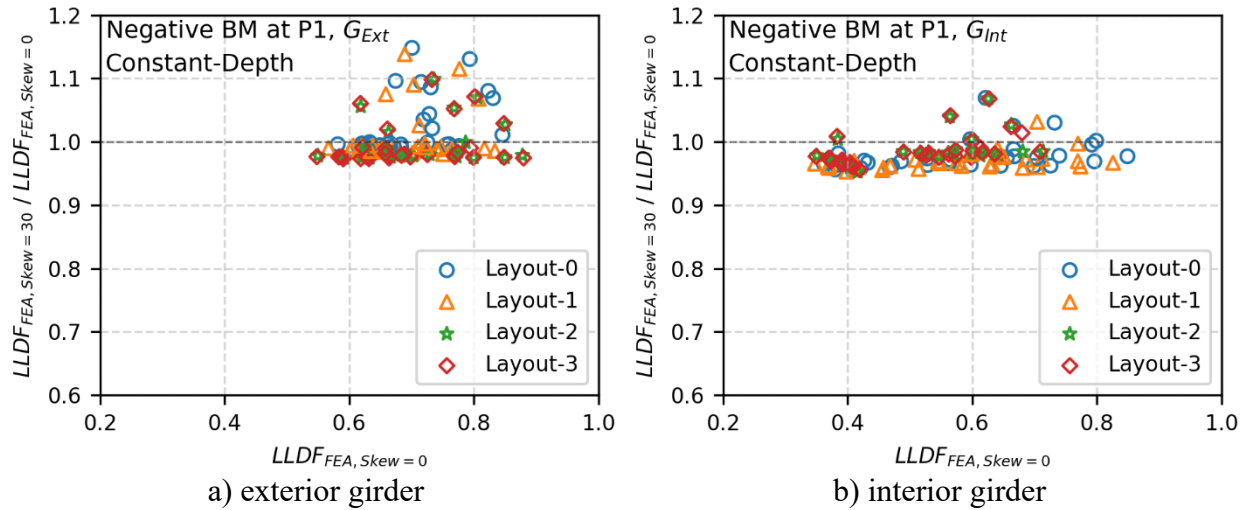


Figure 6.50. Normalized LLDFs for negative moment at support P1 in bridges with 30-degree skew

6.7.2. Effects of Support Skew on Shear Reactions

The support shear at obtuse corner of a skewed bridge is generally higher than the shear in a normal bridge. The LLDFs for reactions in spliced girder bridges were also higher when the support lines were skewed. The LLDF ratios for 30-degree skew angles are plotted in Figure 6.51 and Figure 6.53 for vertical reactions at the end support and interior support, respectively.

6.7.2.1. End Support Reactions

The support diaphragms were found important for load distribution in the previous sections. The LLDFs normalized for support skew effects were higher in bridges with support diaphragms (Layout-1, Layout-2, and Layout-3) than the bridges without any diaphragm (Layout-0). The LLDF ratios for exterior girder reactions in skewed bridges with Layout-0 were not more than 1.06 for 30-degree skew, whereas bridges with other layouts were up to 1.14. The support skew had similar effects on the interior girder reactions at the end supports. The largest LLDF ratio was 1.17 for the interior girder reactions at support A1.

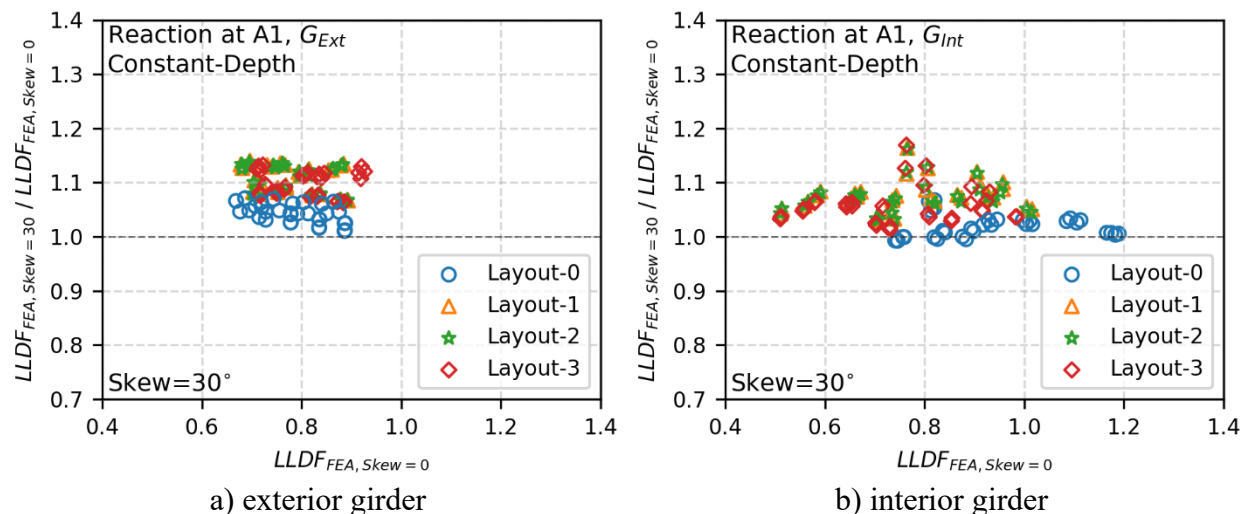


Figure 6.51. Normalized LLDFs for reactions at the obtuse end support A1 in bridges with 30-degree skew

The location, layout and orientation of diaphragms can significantly increase the shear LLDFs. For example, in the interior girder reactions, the three-lane cases higher LLDF ratios (maximum 1.17) than single-lane or two-lane cases (maximum 1.08). The design truck positions for a three-lane case that resulted in the highest LLDF ratio for interior girder G2 are shown in Figure 6.52. The design truck in the third lane was near the mid-segment diaphragm because of the load path provided by the diaphragm to the interior girder G2. Therefore, it is important to consider the diaphragm effects more accurately using refined analysis methods.

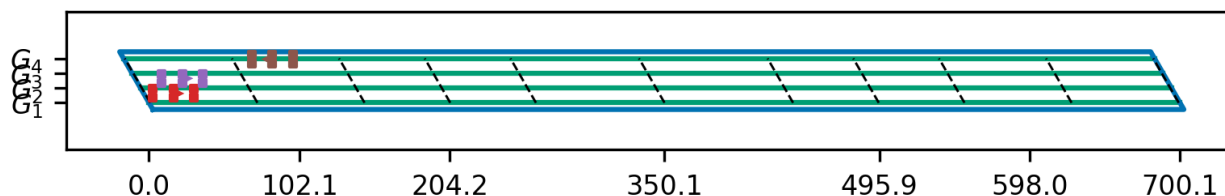


Figure 6.52. Three-lane design truck positions for vertical reaction at support A1 in the interior girder of a skewed (30-degree) bridge

6.7.2.2. Interior Support Reactions

The LLDFs for interior support reactions in skewed bridges were significantly higher than normal bridges. The LLDFs normalized for support skew effects are plotted in Figure 6.53. The LLDF ratios were between 1.03 and 1.15 for exterior girders and between 0.95 and 1.03 for interior girders in 30-degree-skew bridges. The highest amplification of shear reactions was seen in bridges with only support diaphragms (Layout-1). There are two trucks in adjacent spans for maximum reaction at the interior support. The critical positions of the trucks are near the splices. Therefore, the diaphragms at splices provide additional load paths for the design trucks to the supports. Similar to the end support reactions, the highest amplification of interior girder reaction at interior supports was in three-lane cases. The critical positions of the design trucks for the highest amplification of interior girder reaction are shown in Figure 6.54.

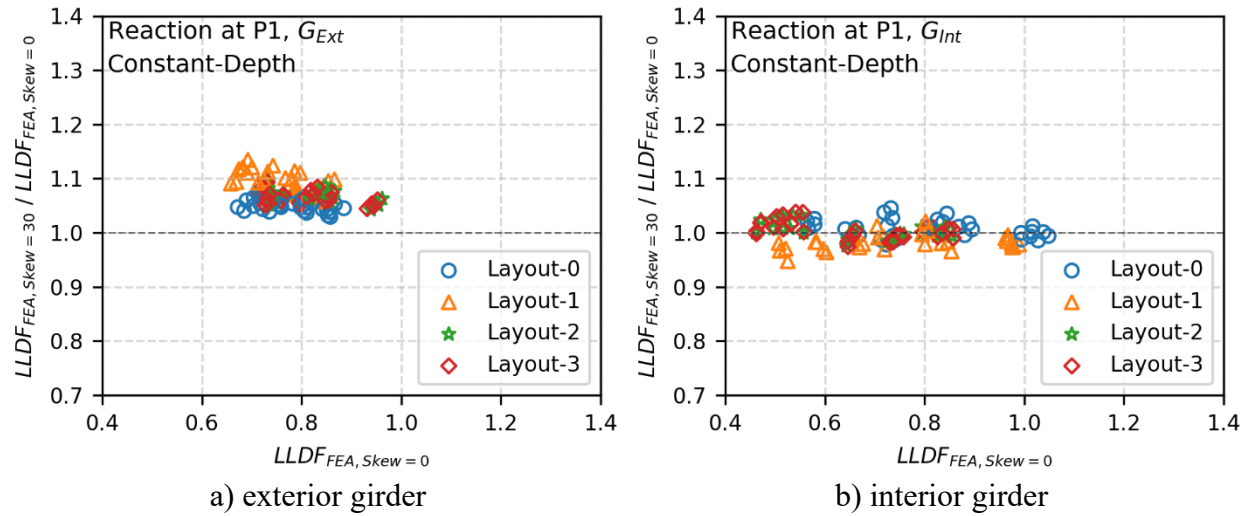


Figure 6.53. Normalized LLDFs for reactions at the obtuse interior support P1 in bridges with 30-degree skew

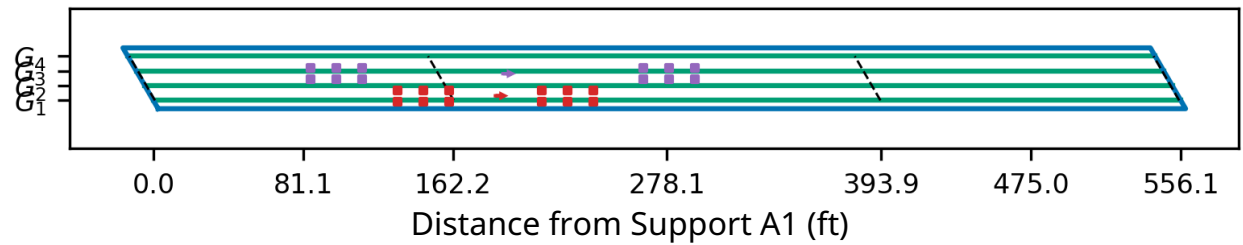


Figure 6.54. Three-lane design truck positions for vertical reaction at support P1 in the interior girder of a skewed (30-degree) bridge

6.7.3. Comparison of AASHTO and FEA LLDFs in Skewed Bridges

In a line-girder analysis, the effect of skewed supports is not modeled explicitly. AASHTO (2024) considers the skew effects with correction factors on LLDF equations. The bending moments are reduced in accordance with the reduction factors specified in *Table 4.6.2.2.2e-1* and shear or reactions at the obtuse corners are increased in accordance with correction factors specified in *Table 4.6.2.2.3c-1*. The LLDFs obtained from FEA are compared with the AASHTO values (including correction for skew effects) for skewed spliced girder bridges in this section. The maximum LLDF from FEA and AASHTO were only considered for the comparison. The results of variable-depth bridges with 30-degree skew are discussed in this section.

6.7.3.1. Bending Moments

The ratio of maximum LLDF from FEA to the AASHTO value is plotted in Figure 6.55 and Figure 6.56, for positive bending moments at section 1-1 and 2-2, respectively. The LLDFs for exterior girder positive moments from AASHTO equations were greater than the values obtained from FEA. The AASHTO values were 26 to 54% greater than FEA values for positive moments. A multiple presence factor of 1.20 was considered in AASHTO LLDFs for single-lane cases.

However, according to current TxDOT practice (TxDOT, 2024a), the multiple presence factor for single-lane cases is equal to 1.0, when the deck overhang width exceeds half of the girder spacing. Bridge models with 8-ft girder spacing and 4.5-ft deck overhang width satisfy the criteria for reduced multiple presence factors. The conservative estimates of LLDFs for the exterior girders indicate the reduction of multiple presence factors for single-lane cases may be acceptable for the spliced girder bridges considered in parametric study.

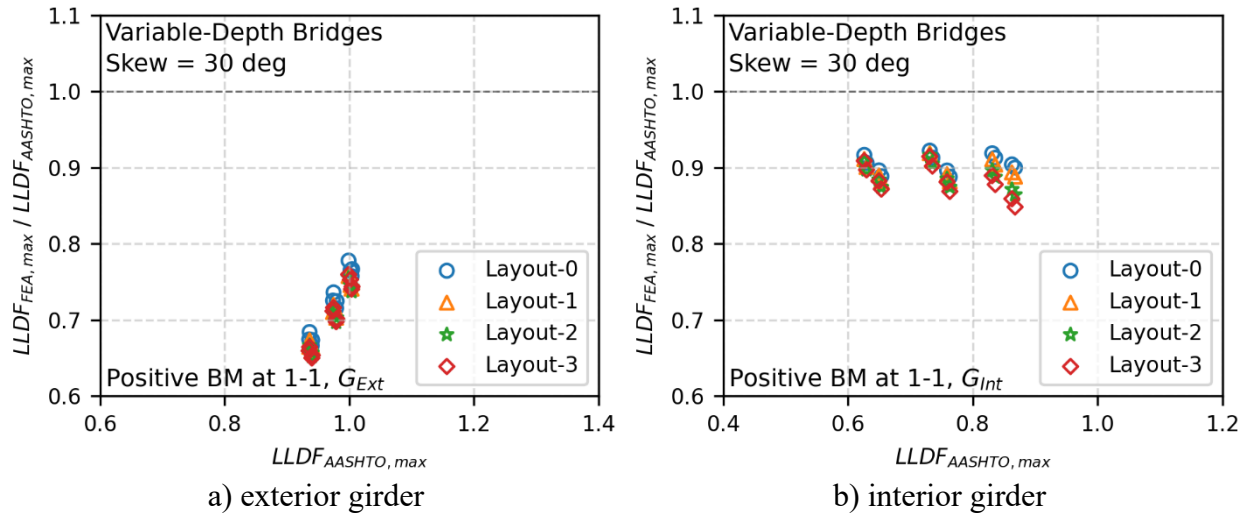


Figure 6.55. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 1-1 in 30-degree skew variable-depth bridges

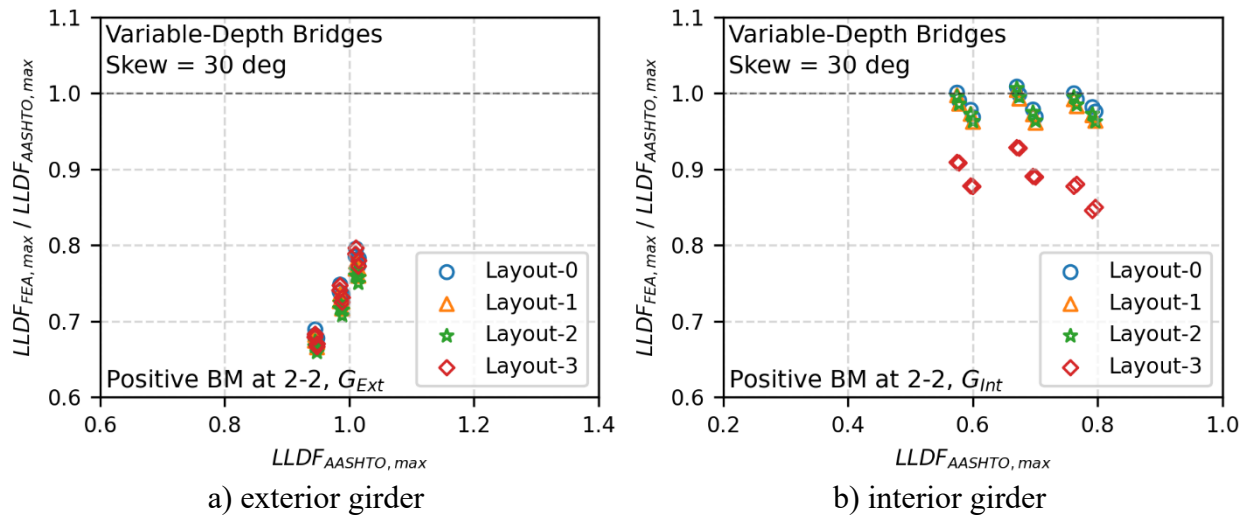


Figure 6.56. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 2-2 in 30-degree skew variable-depth bridges

The AASHTO LLDFs for positive moments in the interior girder were greater or equal to the FEA LLDFs. For positive moment at section 1-1, AASHTO values were 8 to 18% greater than FEA results. In bridges with longer central span lengths ($L/D = 35$) and without mid-span diaphragms, the FEA LLDFs were approximately equal to the AASHTO LLDFs. The LLDF equations provide

conservative estimates for the mid-span positive moments when diaphragms are present at the location.

The ratio of FEA LLDFs to AASHTO LLDFs are plotted in Figure 6.57 for negative moments at interior support. The negative moment LLDFs calculated using AASHTO equations were greater than FEA values for both exterior (16 to 46%) and interior girders (4 to 27%). However, the values for exterior girders were more conservative than the interior girders. The cases with diaphragms at the splices (Layout-2 and Layout-3) resulted in lower LLDFs compared to bridges with only support diaphragms (Layout-1) or no diaphragms (Layout-0).

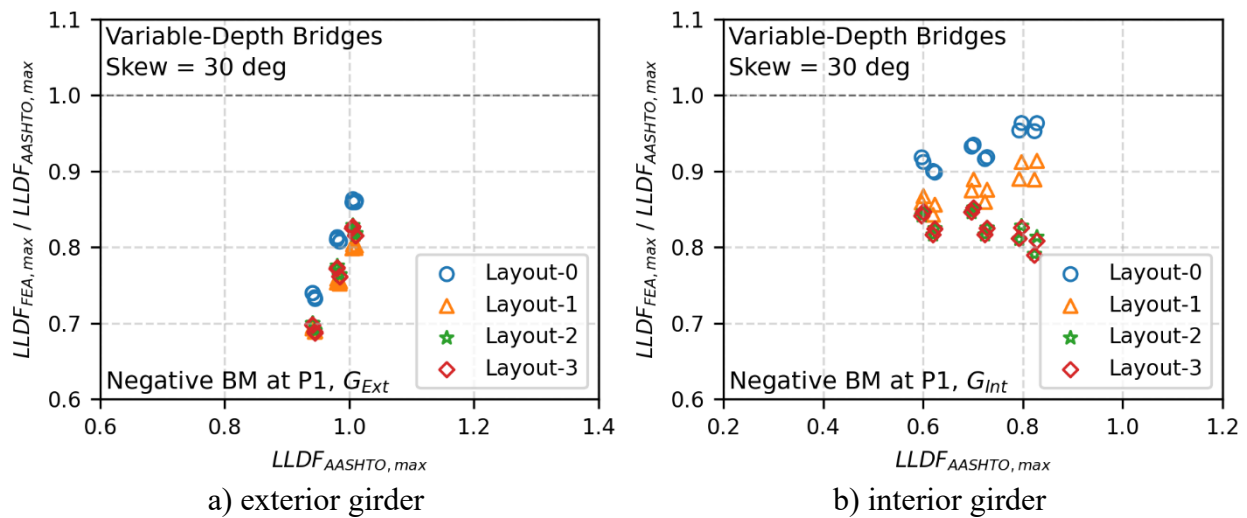


Figure 6.57. Comparison of LLDFs from FEA and AASHTO for negative bending moment at support P1 in 30-degree skew variable-depth bridges

6.7.3.2. Shear Reactions

The shear reactions obtained from FEA are compared with AASHTO values in Figure 6.58 and Figure 6.59, for end support and interior support vertical reactions, respectively. The AASHTO LLDFs for end support reactions were 7 to 29% greater than FEA results in exterior girder. In case of interior girders, the AASHTO values were greater than FEA results for the bridges with support diaphragms. However, for the bridges without any diaphragms, the AASHTO values were lower than the FEA values up to 6%. As explained earlier for normal bridges, the AASHTO equations are not applicable for bridges without support diaphragms.

The LLDFs derived from FEA for interior support reactions were lower than the AASHTO values for two succeeding design truck cases. The interior support reactions are generally larger when two trucks are considered in adjacent span when compared to single or multiple lanes of single truck. The comparison of LLDFs showed AASHTO was conservative by 6 to 29% for the exterior girder and by 20 to 39% for the interior girder.

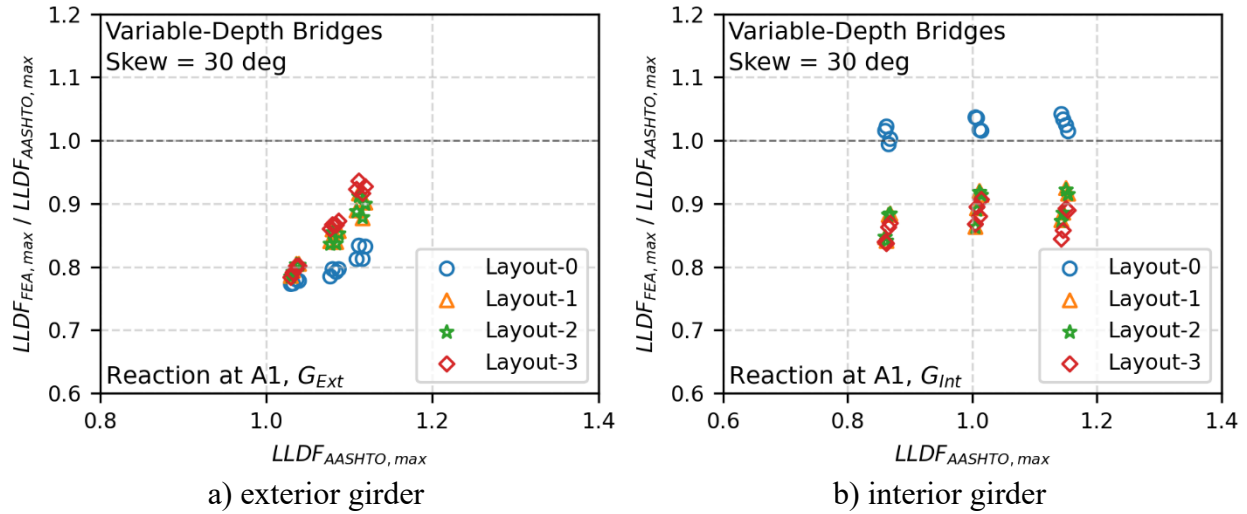


Figure 6.58. Comparison of LLDFs from FEA and AASHTO for vertical reaction at end support A1 in 30-degree skew variable-depth bridges

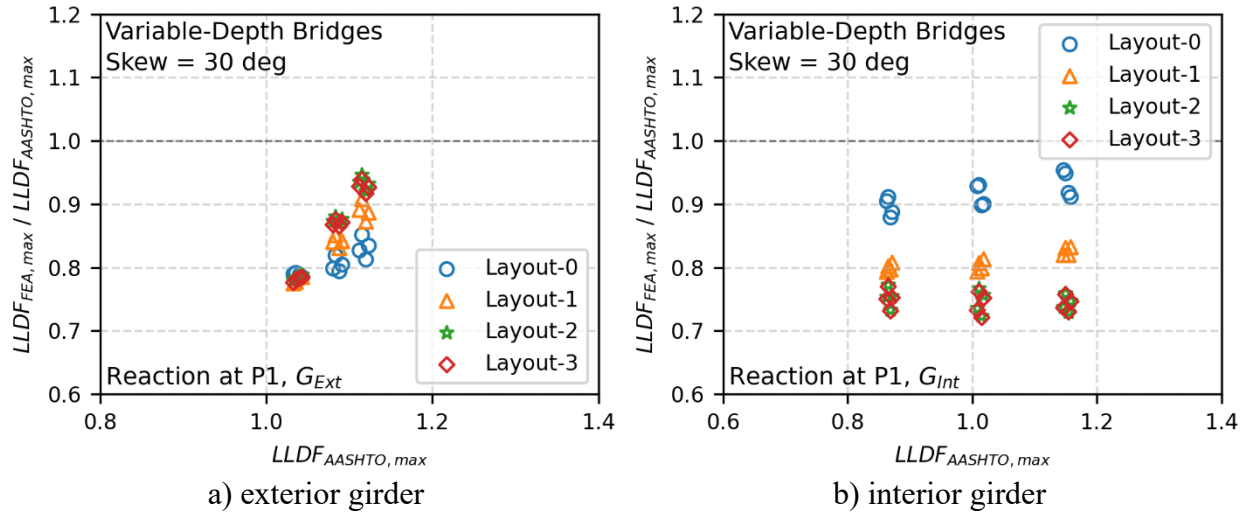


Figure 6.59. Comparison of LLDFs from FEA and AASHTO for vertical reaction at interior support P1 in 30-degree skew variable-depth bridges

6.8. Chapter Summary

This chapter summarizes the numerical study on live load distribution for spliced concrete girder bridges. The study was necessary due to the limitations of the current AASHTO (2024) specifications for application in spliced girder bridges. An extensive parametric study was carried out by considering a wide range of parameters. The parameters and their range were chosen to represent the spliced girder bridges that have been constructed in Texas in the past. Additionally, the influence of diaphragm layouts on the load distribution was studied by considering four different layouts.

The live load analyses were performed for 288 bridge models using an approach based on influence surfaces. The influence surfaces were generated by performing static analysis for unit loads at

several positions on the bridge. The extreme force effects due to design truck load were calculated by optimizing the position of the trucks. The LLDFs were calculated for moments and shear reactions at critical positions from FEA and line-girder analyses. The LLDFs calculated for representative bridges were compared with the values predicted by AASHTO (2024) equations. The AASHTO equations were conservative for most cases. However, AASHTO equations were unconservative for the bridges with no support diaphragms. The FE analysis showed support diaphragms are necessary for better load distribution at the supports. The current specifications for LLDF are appropriate for spliced girder bridges with normal support lines. The variation in girder stiffness in variable-depth bridges was studied by comparing with constant-depth bridges of the same geometry. There were negligible differences in the LLDFs for moments and end support reactions due to the change in girder stiffness. The interior support reactions were significantly reduced in variable-depth bridges when compared to constant-depth bridges.

The diaphragm layout played an important role in load distribution, especially when the diaphragms are near the points of interest. The diaphragms helped distribute the load to the interior girders. The variation of moments and shear reactions for different diaphragm layouts were discussed. The interior diaphragms may be chosen based on the requirements for stability during construction, but support diaphragms are recommended for better load distribution between the girders.

The skewed support lines generally resulted in lower moments and higher shear reactions than the corresponding values in normal bridges. However, for negative moments at interior support the diaphragms at splices increased the moments for many cases. The shear reactions were larger in skewed bridges and their magnitudes were also influenced by the diaphragms. The effects of skewed supports were investigated for 30-degree and 45-degree skew angles. The AASHTO equations with correction factors for moments and shear reactions resulted in conservative estimates for the distribution factors.

Based on the observations in the parametric study, the RT recommends using approximate analysis method specified by AASHTO (2024) for spliced girder bridges that are within the geometric limits studied in the TxDOT 0-7115 project. Support diaphragms are required to distribute the loads to the girders more efficiently. AASHTO equations can be used for shear and reactions only when support diaphragms are provided. The designer may choose to provide diaphragms at splices to reduce the negative moments and shear at the interior supports, even though they may not be necessary for stability during construction. The mid-span diaphragms may be chosen for long-span spliced girder bridges to reduce positive moments. However, a refined analysis approach is necessary to account for the beneficial effects of diaphragms in live load distribution.

Chapter 7. Recommended Design Details and Methodology

Based upon the results from the parametric FEA studies, recommended details and design recommendations were developed. The design recommendations are divided into two areas – a simplified procedure and a more rigorous approach that makes use of three-dimensional FEA models. While most designers will opt for the simplified approach, rigorous modelling is recommended for some cases. The simplified stability checks are given in reference to the design ranges analyzed as part of this research project. Designers can use this simplified approach for STx I-girder bridges to determine critical stability cases and capacity given the required restrictions. Due to unique buckling behavior of U-girders, refined analysis may often be prudent. The design recommendations are provided in respect to diaphragm layout, stability during erection and deck pour, live-load distribution, and refined analysis.

7.1. Recommended Diaphragms Layouts

Based on the results from the stability and live load distribution analyses, at minimum, diaphragms at the supports and splice points are recommended. While precast diaphragms or even steel diaphragms might be used, the results of the parametric studies would be consistent with cast-in-place diaphragms. The diaphragms should be at least the full depth of the girder webs; however, deeper diaphragms that include part of the flanges of the girders are beneficial. Full-depth diaphragms maximize the torsional brace stiffness and allow the reinforcing steel in the diaphragm to be developed outside of the girder splice zone and reduce congestion of reinforcement. Figure 7.1 shows the various layouts that were considered in the parametric studies on stability and live load distribution. The recommended configuration with diaphragms at the supports and splice points are shown in Layout-2.

The parametric results demonstrated that Layout-2 significantly improved the stability of the system relative to Layout-0 and Layout-1 by increasing the lateral torsional buckling capacity. This increase is caused primarily by reducing the unbraced length and secondarily introducing marginal warping restraint at the supports (when diaphragm thickness was at least 18 in.). These diaphragms are also necessary for distributing the live load. The current configurations generally utilized by TxDOT consist of Layout 3, which provides additional stability and good live load distribution performance. The researchers do not advise the use of Layout-0 (no diaphragms) or Layout-1 (diaphragms only at supports). Although the eigenvalue solution and nonlinear geometrical results showed reasonable behavior with Layout-1 (only at the supports), having at least one intermediate diaphragm assists in controlling cross-sectional load position effects, since a significant amount of the load comes from the cast-in-place concrete deck that is applied at the top of the girder.

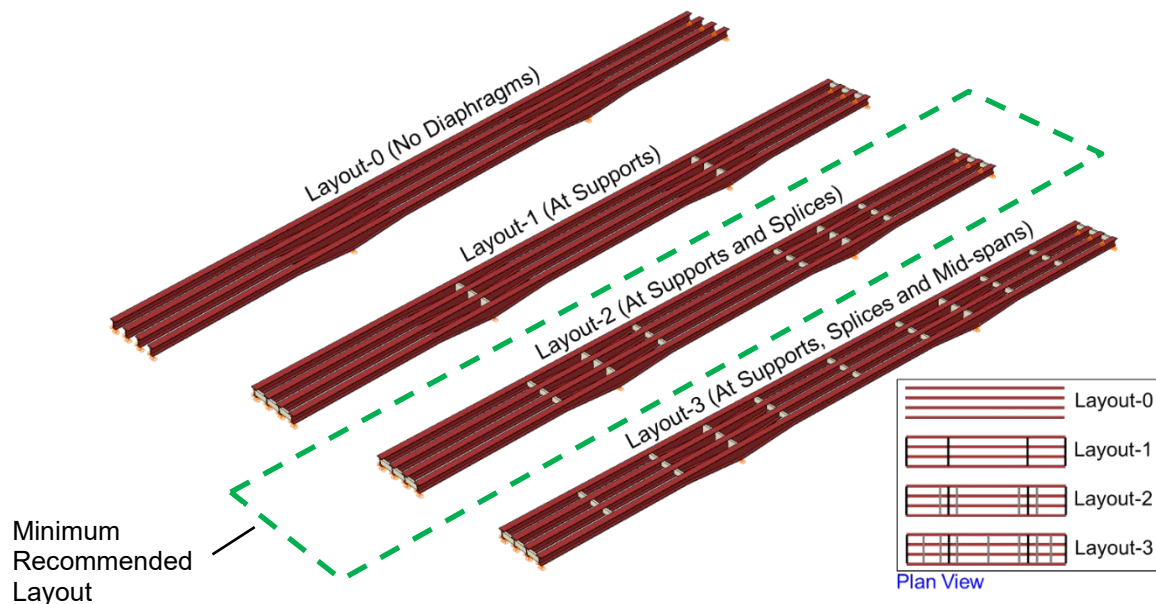


Figure 7.1. Isometric view of diaphragm layouts and the preferred option

7.2. Check Girder Stability during Erection and Deck Pour

The stability appraisal of a bridge system during erection in this section relies on the commonly employed construction practices, as shown in Figure 7.2. The span length ratios contribute to the critical load case that governs the stability of the system. In particular, when the ratio of the exterior (end) span to the interior (middle) span is approximately 70% or higher, the load cases that typically govern the maximum induced forces are Case 1 and Case 2. On the other hand, when the end span ratio is less than 70% of the middle span, Case 4 may control. For girder systems with typical span length configurations of $0.7L$ - $1.0L$ - $0.7L$, where L denotes the length of the middle span, Case 1 will govern.

In the evaluation of stability limit states during girder erection and deck casting, a dead load factor of 1.4 that is consistent with AASHTO BDS (2024) Section 3.4.2.1 should be utilized on the appropriate construction loads. Although the construction load factor in AASHTO is specified for steel superstructure units, the intent of the load factor is for cases in which stability may control the design. The load factor captures uncertainties in the placement of loads that might adversely impact the stability of the system. Because the limit states being evaluated for the prestressed concrete girder segments involve stability, the use of the 1.4 factor is therefore consistent.

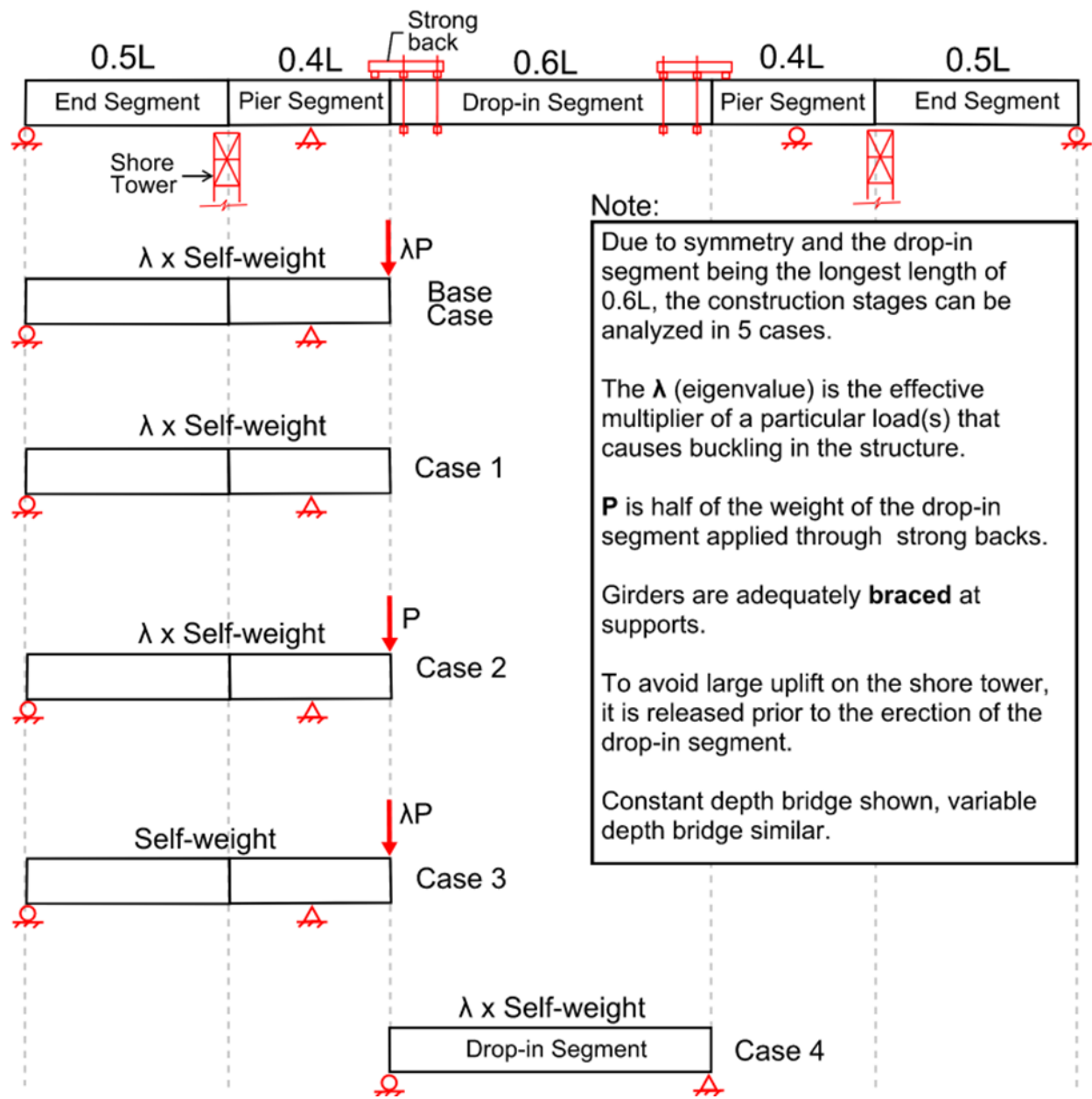


Figure 7.2. Typical girder stability load cases during erection

The stability assessment of the system during deck pour accounts for the critical continuous deck-pour construction sequence and relies on the typical load cases shown in Figure 7.3. The configurations of the specific load cases were found to generally induce the maximum possible moment in the systems. Additionally, these analyses assumed that the concrete was plastic with no stiffness, thus offering no top flange lateral restraint. In general, when using staged concrete pours, concrete placed more than 24 hours before another deck placement can be considered to provide continuous lateral restraint to those regions of the structure due to the stiffening of the concrete, thereby improving system stability. Although 24 hours may seem like a short time relative to conventional 28-day concrete strengths, concrete stiffens relatively quickly and will provide

substantial restraint to the girders. In addition to directly restraining the regions of the span with previously cured concrete, adjacent sections of the girders will possess increased torsional stiffness due to warping restraint provided by the now composite regions of the bridge. For three-span girder systems with spans close to the typical span configurations of $0.7L$ - $1.0L$ - $0.7L$, Case 2 will control. For span configurations where the end span is greater than 70% of the middle span, Cases 1 may control. For cases where the end spans are less than 70% of the middle span, Case 2 will control.

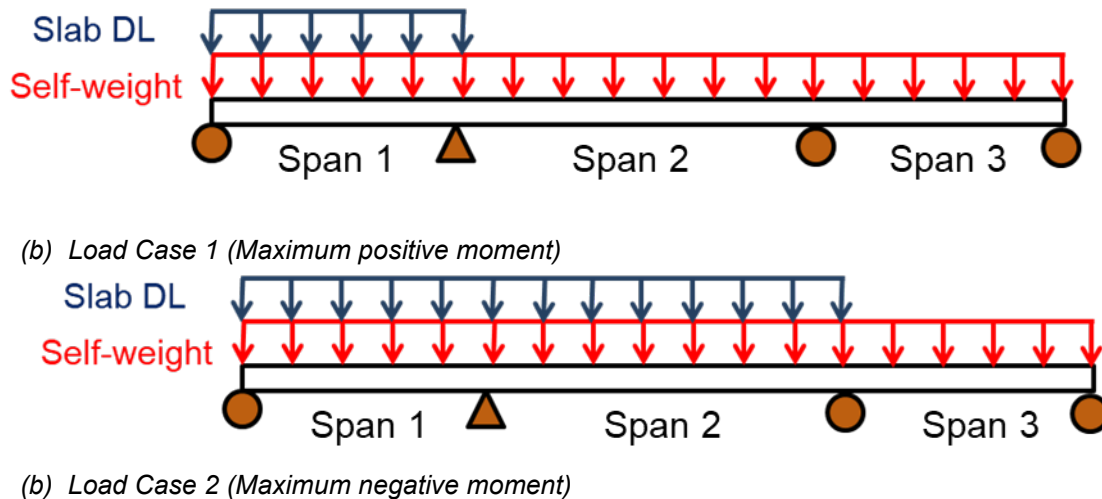


Figure 7.3. Load case under construction load during deck pour

A similar approach can be taken for two span cases, where the longer of the two spans will be the most critical for stability due to typical moment distribution and unbraced length. Applying the slab dead load solely over the longer span, or either of the spans in the case of balanced span lengths, will produce the largest positive bending moment. Conversely, applying the slab over both spans will generate the greatest negative bending moment. Both should be checked for design.

Once the critical cases for erection and deck pour are determined, the lateral torsional buckling capacity of the girder system must be calculated and checked against the factored construction moment. The procedure to be used generally depends on the girder types and the girder geometries.

7.2.1. Check I-girder Buckling Capacity

For STx7 and STx9 I-girders consisting of main span lengths between 193 ft and 340 ft, approximately $0.7L$ - $1.0L$ - 0.7 span configurations, girder spacing within the range of 8 to 12 ft and an 8.5 in. deck, a conservative simplified approach can be utilized to evaluate stability. For I-girders that significantly extend outside this design range, a refined analysis per Section 7.4 is recommended, but the guidance herein should still be used as a point of reference for nonconforming systems.

To calculate the buckling capacity of a STx I-girder, the full expression by Galambos (1968) can be used. This equation includes the warping and Saint-Venant torsion terms modified by a monosymmetry parameter. However, due to the monosymmetry parameter introduced by

Galambos (1968) and the warping term having a negligible effect on the buckling capacity results, this parameter can be ignored for the recommended design range when the values from Table 7-1 are used.

As was demonstrated in the parametric studies, the stability of the system with cracking must be assessed in a way to account for possible localized negative and positive bending region cracks via a reduction factor as cracking in these regions can impact the buckling resistance of the section. If cracks are observed from transportation or early stages of erection or construction that extend through the flanges and into the web, the girder is recommended to be repaired – if possible – or replaced due to the potential of significant reductions in the buckling capacity. Considering the uncracked buckling capacity of the system as M_o , the capacity of the system is reduced to $0.6M_o$ (a 40% reduction) when cracks extend to the center of gravity. If the crack depth equals the top flange thickness, the corresponding system capacity is reduced to $0.8M_o$ (a 20% reduction). Although many spliced concrete girders utilize haunched sections around intermediate supports, if the haunched section is part of the critical unbraced length for LTB, the section can be idealized with a prismatic section using the depth that dominates the majority of the unbraced length.

Although steel girder stability provisions often apply C_b factor greater than 1.0, a conservative value of unity is recommended for spliced concrete girders, given the reduced number of diaphragms in these systems relative to the steel girders. In addition, because a significant proportion of the construction moment results from deck concrete loads acting at the top of the section, adopting a C_b provides a conservative means to account for the influence of the load position effects.

Note that the buckling capacity of a given segment is a function of the corresponding unbraced length. The LTB resistance should be compared with the design maximum construction moment along the segment that is being evaluated. As stated above, the design construction moments should be based upon a load factor of 1.4 applied to the girder self-weight and all construction loads. The researchers have determined, for the design range tested, that the global system buckling mode, which is a limit state in AASHTO BDS (2024) Section 6.10.3.4.2, does not control. As this stability limit generally applies to narrow I-girder units with 2 or 3 girders, an addendum will be included to R1B in the future covering the stability of these narrow girder systems.

$$M_o \approx C_b \frac{\pi}{L_b} \sqrt{EI_y GJ} \rightarrow 0.8M_o \geq M_u \quad (7.1)$$

Where:

- o M_o = The lateral torsional buckling capacity between brace points
- o C_b = The moment gradient factor (recommended to be taken as 1)
- o L_b = Length between braces (flexure)
- o E = Young's Modulus
- o G = Shear Modulus
- o J = Saint-Venant torsional constant
- o I_y = Moment of inertia about the weak axis.

The approximate equations of the Saint-Venant torsional constant, J , given in AASHTO BDS (2024) equations C4.6.2.2.1-1 and C4.6.2.2.1-2 result in errors around 25% when applied to STx girders. Thus, the torsional properties for the STx girders were determined and the recommended properties for use in design are presented in Table 7-1.

Table 7-1 STx girder torsional constant

Girder	J ($\times 10^3 \text{ in}^4$)
Tx28	17.4
Tx34	18.1
Tx40	18.8
Tx46	26.2
Tx54	27.1
Tx63	28.7
Tx70	29.7
STx7	52.7
STx8	59.7
STx9	62.7

7.2.2. Evaluation of Stability of Spliced U-girders

The conventional LTB solutions that are utilized for LTB of I-shaped girders do not generally provide good solutions for U-girders, since the out of plane direction of many of these girders is actually the major axis. Similar findings are reported in Yura and Widiyanto (2005). A refined analysis is generally recommended to evaluate the stability of spliced U-girders under the full construction load. The stability of U-shaped girders is enhanced significantly if a topping slab is first casted on the U-girder before installing the entire deck load.

For a refined analysis, considering the uncracked buckling capacity of the system as M_o , the capacity of the system is reduced to $0.50M_o$ (a 50% reduction) when cracks extend to the center of gravity. If the crack depth is limited to solely the top flange thickness, the capacity is reduced to $0.8M_o$ (a 20% reduction).

7.2.3. The Impact of Girder Deformations on Constructability

Results from the parametric finite element studies included initial imperfection levels of $L/1000$ and $L/500$. $L/1000$ imperfection is consistent with the PCI imperfection of $1/8$ " per 10 ft of length that produces an imperfection of $L/960$. The larger imperfection of $L/500$ was utilized simply to understand the behavior. Based upon the FEA solutions, the recommended limit on the initial lateral imperfection of the I and U-girders should not exceed $L/1000$, which again is consistent with current fabrication tolerances. If this limit is met, for the typical design range with cracking limited to the flanges, additional twist and sway of the I-girders is negligible at the predicted construction loading. This is generally the case for U-girders during erection. However, for the same restrictions, U-girders during deck pour under asymmetric loading can typically experience twists and lateral displacements of up to two times the initial imperfection. As such, designers should check that lateral displacement and corresponding twist of $L/500$ do not cause constructability issues for U-girders. This increased susceptibility to the initial imperfection in U-girders is primarily caused by localized cross-sectional distortion due to cracking of the bottom flange.

The 1.4 load factor discussed earlier in the chapter is only for the strength limit states and should not be used to evaluate deformation limits. The load levels that should be used to evaluate girder deformations are the actual expected construction loads.

To counteract constructability issues, a staged deck pour that allows concrete to harden over the supports or tied-in precast /cast-in-place partial lid slabs are recommended. These methods provide end warping restraint to the system and significantly increase the torsional stiffness of the system by pseudo-closing the section. This increased stiffness thus reduces lateral displacements and twist of the girder.

7.3. Determine Live Load Distribution Factors

The parametric study on live load distribution spliced concrete I-girder bridges covered a wide range of bridge geometries representing the current inventory in Texas. The FE analysis method using influence surfaces and live load optimization analysis is an acceptable refined analysis approach. The approximate analysis method specified by AASHTO BDS (2024) is based upon line-girder analysis and LLDFs to approximately calculate the design forces due to live loads. This simple analysis approach may be preferable in design practice. The comparison of LLDFs obtained from FEA and AASHTO for the representative bridges showed AASHTO LLDFs are conservative or approximately equal to FEA LLDFs for most cases. However, there are some limitations for the conclusion and certain conditions need to be met to follow AASHTO's approximate analysis

method. The conditions to be satisfied for using the approximate analysis method according to *A4.6.2.2 of AASHTO LRFD Bridge Design Specifications* (AASHTO, 2024) are given below:

1. Diaphragms should be provided at the supports to ensure proper load distribution.
2. The number of girders (N_b) shall be at least equal to four and the spacing shall not exceed 12 feet.
3. The span-to-depth ratio shall not exceed 30 in case of constant-depth bridges and 35 in case of variable-depth bridges, where the depth is calculated including the depth of the girder and deck slab and span length is measured between the support centerlines.
4. The deck overhang width should not exceed 4.5 feet, where the width is measured from the centerline of exterior fascia girder to the free edge of the deck slab.
5. In case of variable-depth bridges, the average longitudinal stiffness parameter shall be calculated based on the weighted average of the values for the entire girder length. The variation in girder stiffness has to be considered in the line-girder analysis by meshing the haunched segment into multiple smaller segments and defining average section properties for each segment.
6. In skewed bridges, the skew angle shall be limited to 45 degrees.
7. For exterior girders, the current practice of using multiple presence factor of 1.0 for single lane loading may be satisfactory.
8. The design the interior girders may be treated separately from exterior girders due to very conservative LLDFs for exterior girders compared to interior girders in long-span spliced girder bridges.
9. The favorable effects of intermediate diaphragms on bending moments, shear and reactions shall not be accounted when using approximate analysis method.

7.4. FEA Guidance and Benchmarking

The stability assessment of systems that fall outside the scope of this study must be determined using a refined finite element analysis (FEA) approach. Commercial bridge design software packages are available that perform buckling analyses using a variety of element configurations, including beam, shell, and solid elements. The analyses methodologies required for the appraisal of the stability response entail an eigenvalue buckling analysis and a large deformation, incremental analysis. The eigenvalue buckling analysis can be used to capture the elastic buckling response of the system, while the incremental analysis can be customized to capture the system response by accounting for geometric and material nonlinearities – i.e., all sources of nonlinearity entailed in the problem setting. The following steps are recommended when evaluating the stability of spliced prestressed concrete girder systems:

1. A good starting point for a stability analysis is to use Equation 7.1, with accurate torsional properties for I-shaped girders, to calculate the theoretical buckling capacity for a simply supported girder under uniform moment. This provides early insight into the viability of the structure type and LTB resistance relative to the expected construction loads.
2. Select a benchmark case from Appendix D and analyze all erection and deck placement load cases by linear-elastic eigenvalue buckling analysis via the software of choice.
3. Compare the eigenvalue buckling analyses for all cases and identify differences. The benchmark cases should be met before conducting a refined analysis with a new model. Potential sources of error include:
 - Need to discretize the system into smaller finite element mesh sizes
 - Load applications at centroids of elements instead of actual location on the cross-section
 - Boundary conditions contributing unintended and unrealistic restraint
 - The eigenvalue scaling all loads on the structure rather than the load of interest
4. Perform eigenvalue buckling of all construction conditions and deck placement load cases.
 - The eigenvalue buckling capacity must exceed the construction load level. If the system is subject to cracking, the location and depth of the cracks should be explicitly accounted for. On the other hand, a simplified approach to account for the cracking effects is to reduce the eigenvalues obtained from the intact system. In this regard, the reduction factors for the I-shaped girder systems shall be 60% (40% reduction) and 80% (20% reduction) when cracks extend to the center of gravity or top flanges, respectively. The corresponding reduction factors for the U-shaped girder systems shall be 50% (50% reduction) and 80% (20% reduction).
5. Perform a large displacement analysis of the critical construction case with an introduced imperfection. Imperfections can be included in the model geometry by directly manipulating the mesh node locations or by altering the initial state. The magnitude of the imperfection should be of $L/1000$ and the construction load should be applied. The large displacement analysis must consider geometric stiffness terms and update the global stiffness matrix at every increment. Loading increments in the large displacement analysis should at maximum be 10% of the construction load to prevent numerical errors. Large displacement analyses should make use of factored loads and a load factor of 1.4 is recommended, which is consistent with AASHTO LRFD BDS (2024) Section 3.4.2.1.
 - Based upon the results of the large displacement analysis, a comparison with design construction forces during construction can be made. Additionally, an evaluation of the lateral displacements and twist of each girder should be made to evaluate constructability issues. The deformation comparison should be made at expected

construction load levels (load factor of 1.0) If stability or constructability is a concern, additional stiffness must be added by way of more diaphragms along the span and/or top flange lateral restraint.

Chapter 8. Summary and Conclusions

8.1. Summary

This study focused on the stability and live load distribution in spliced-concrete girder bridges. Spliced-concrete girder systems extend the possible span lengths of these bridges compared to the geometries that have been used for decades. Stability of prestressed concrete girders has historically been related to roll stability and conditions where biaxial bending during shipping, lifting or setting of girders has exceeded material stress limits. The span lengths of precast prestress girder span lengths have historically been limited to approximately 150 ft., primarily due to transportation limitations. However, with the recent advances in the industry and in Texas design and construction practices using spliced concrete girders, bridges with significant increases in span lengths are now being constructed. These trends towards longer bridge spans lead to significant increases in the girder slenderness, thereby introducing concerns about the stability of these spliced concrete bridge systems. The requirements for bracing in these girders to ensure stability and predictable response during construction have not been established. Additionally, there is no explicit modern code guidance for lateral-torsional buckling of concrete girders. With regards to design force distributions, the applicability of AASHTO live load distribution factors on these girders is also unknown because many of the spliced-girder bridges do not satisfy the limits outlined in the AASHTO BDS (2024).

The research presented in this report was part of a research project (TxDOT 0-7115) sponsored by the Texas Department of Transportation (TxDOT). The primary objectives of the project were to understand the stability of long-span prestressed concrete I- and U-girders during erection and construction, and the distribution of live load in the completed bridge. Furthermore, the role of diaphragms in stability and live-load distribution was also studied. A full-scale experimental testing program was necessary to evaluate the stability of the typical girders utilized in Texas. The full-scale tests were conducted at the Ferguson Structural Engineering Laboratory to understand the flexural and torsional behavior of I- and U-girders under a variety of load conditions including both vertical and lateral loads. The girders were tested in both the uncracked and cracked conditions under various combinations of flexural and torsional loading.

The flexural behavior was studied by applying the vertical loads concentric to the girder center line and combined flexural-torsional behavior was studied by applying eccentric-vertical loads. The vertical bending tests were conducted for loads beyond the cracking limits to study the differences between uncracked and cracked state. The girder specimens showed significant reduction in stiffness after the first crack occurred as observed from the moment-curvature plots. However, the severe stiffness reduction was localized at the crack locations. The load-displacement plots considering the entire span of the girder indicated small difference in stiffness after the onset of cracking. The changes in stiffness were captured by considering the local curvatures at locations with embedded vibrating wire gauges. Each of the tests were generally repeated three times to ensure consistency in the data. During tests in which significant cracking occurred, subsequent load-deformation curves generally began with the identical stiffness as the

first cycle; however, the non-linear behavior occurred at loads lower than the cracking load. The torsion-twist plots from the combined bending-torsion tests showed significant reduction in torsional stiffness at the crack locations. One of the key observations from the tests was the visibility of cracks. After the load was removed, the cracks closed due to the prestressing effects. The cracks were not visible with naked eye; however, the subsequent loading cycle had reduced stiffness due to the cracks. The importance of this with regard to stability is the indication that the cracked nature of the girders is not generally visible to the naked eye. As a result, although inspectors and construction personnel may believe that the girders are in the uncracked state – in reality, they may be cracked. Therefore, the parametric studies that were used to understand the stability behavior of the girders were based upon models with i) uncracked section properties, and ii) cracked section properties.

The experimental results were used for the development and validation of FE models. A continuum-based three-dimensional FE modeling approach was developed using non-linear material properties for concrete, reinforcing steel and prestressing strands. An implicit-static analysis approach was adopted considering the non-linear geometric effects. The numerical analyses of the experimental specimens were found capable of capturing the response of the systems subjected to flexure and combined flexure-torsion. The adoption of a continuum-based, three-dimensional model was indispensable for evaluating the torsional response of the girders under the combined action of flexural and torsional stresses. The computational framework successfully captured the local (moment-curvature and torque-twist) and global (load-deflection) responses of the systems, providing a numerical means for assessing the lateral-torsional stability of the girders.

The FE modeling technique was adopted to conduct two comprehensive parametric studies for girder stability. The modelling studies reflected 3 different stages for modelling the behavior 1) stability behavior during erection, 2) stability behavior during construction, and 3) live load distribution under in-service conditions. The first parametric study evaluated the stability response of multi-span spliced girders during erection and the second study was on the single-girder and multi-girder bridge systems during deck placement. The geometric parameters and their range were chosen based on the past spliced-girder bridges constructed in Texas and typical practices of TxDOT.

The stability of the girder systems were evaluated first by considering critical buckling moments for the girders followed by large displacement analyses on imperfection girder systems. The Eigenvalue buckling analyses were conducted to calculate the critical buckling moment capacities for lateral-torsional buckling during different stages of erection and deck construction. The stability was further investigated by conducting second-order geometric-nonlinear analysis considering geometric imperfections in the girder. As noted above, the lateral-torsional response was evaluated for uncracked and cracked conditions for the girders. The uncracked girders were stabilized with lateral restraints at the support locations only. The buckling moment capacities were lower in cracked girders due to the reduced stiffness after cracking. The buckling capacities were evaluated for single and multi-girder systems to compare the effects of diaphragms and check

for global buckling mechanisms. The influence of diaphragms at supports, splices and mid-spans was also evaluated.

The live load distribution in spliced-girder bridges was studied using an influence-surface based approach. A series of linear static analysis was performed for unit loads at different positions in the bridge. The influence surfaces were developed based on the response under unit loads. A live load optimization program was developed to identify critical design truck positions for extreme force effects. The force effects were normalized with respect to line-girder analysis to calculate the live load distribution factors (LLDF). The calculated LLDFs for bending moments and shear reactions were compared with the values obtained from AASHTO (2024) equations. The effects on LLDFs due to variation in girder depth, diaphragm layouts and skewed supports were studied. The LLDFs obtained from FE analyses were lower or approximately equal to the AASHTO values for most cases. The key conclusions from the project are discussed in the next section.

8.2. Conclusions

The research project resulted in a significant improvement in the understanding of the stability behavior of spliced-concrete girder bridges. The existing guidelines for live load analysis were compared with a refined analysis method. Important conclusions derived from the experimental and numerical studies are presented in this section and the design recommendations are summarized.

8.2.1. Full-Scale Bending Tests on I- and U-Girders

The full-scale experimental study on the flexural and combined flexure-torsion behavior of I- and U-girders provided good insight in to the likely behavior of the girders during erection and deck placement. The tests played a vital role in determining the stiffness of the girders in both the uncracked and cracked condition so that accurate models could be developed. Once validated, the models allowed detailed parametric studies to be conducted. In addition to providing data for validation of FE models, the following conclusions were derived.

- Vibrating wire gauges provided accurate measurement of strains throughout the girder cross-section. The instrumentation plans provided means for the appraisal of the system response from local (vibrating wire gauge measurements) and global perspective (linear potentiometer measurements). Instrumentation at the support was found crucial to isolate the rigid body motion of the girder during the test.
- The crack trajectory can serve as a quantitative indicator for assessing the load level carried by the specimen. At lower load levels, flexural cracks with a straight orientation form, followed by shear-flexural cracks with a straight-inclined orientation at higher load levels as the cracks extend into regions with higher shear stresses.
- The localized flexural and torsional stiffness of the cross-section at the crack locations were reduced significantly after cracking. However, the stiffness of the girder in the

global response was reduced by a smaller margin. Although the localized cracking did not significantly impact the global stiffness, that damage can impact the stability behavior and should be considered.

- As applied loads were reduced on the specimens, the cracks closed and essentially were invisible to the naked eye since they were compressed by the prestressing forces. However, upon reloading of the girder, the effects of reduced stiffness were present at load levels below the cracking moment. Therefore, the absence of cracks during visual inspection may not be sufficient to assume uncracked section properties when considering girder stability.
- The validated FEA models were found capable of capturing the response of the systems subjected to flexure and combined flexure-torsion. The adoption of a continuum-based, three-dimensional model was indispensable for evaluating the torsional response of bridge systems under the combined action of flexural and torsional stresses. The computational framework successfully captured the local (strain gauge) and global (load–deflection) responses of the systems, providing a numerical means for assessing the girder stability in the parametric studies.

8.2.2. Stability During Girder Erection

The construction process for spliced-girder bridges have many stages that need to be evaluated for stability under temporary conditions as compared to conventional simply-supported girder bridges. The following key findings and recommendations were identified from the parametric analyses.

- A load factor of 1.4 is suggested on all self-weight loads during the erection process since the limit state under consideration involves girder stability. The factor of 1.4 is consistent with AASHTO BDS (2024) Section 3.4.2.1 for steel superstructures. Although the systems in this study are prestressed concrete elements, the limit state is related to stability. The intent of the 1.4 load factor in AASHTO is related to uncertainty in the application of the loads and the susceptibility of the systems to local and global instabilities. Therefore, using this load factor is consistent with the limit state being evaluated.
- The spliced I- and U-girder bridges considered in this study exhibit stable behavior during the many erection stages evaluated.
- The closed-form solutions for lateral torsional buckling of singly symmetric steel I-girders are applicable for concrete I-girders, if the correct torsional properties are calculated. The simplified methods specified by AASHTO (2024) for the calculation of torsional properties had some measurable differences between values determined from a finite element model. The calculated values for common I-girders utilized in Texas are provided in Chapter 7.

- Girders must be sufficiently braced laterally when they are supported on temporary or permanent supports.

8.2.3. Stability During Deck Slab Placement

The concrete deck placement stage is often the most critical stage for stability due to the relatively long unbraced lengths and large construction moments is generally during concrete deck placement for the continuous-spliced girders. The following conclusions were derived based on the parametric study on stability during deck placement.

- A load factor of 1.4 is suggested on all self-weight loads during the deck construction process since the limit state under consideration involves girder stability. The factor of 1.4 is consistent with AASHTO BDS (2024) Section 3.4.2.1 for steel superstructures. Although the systems in this study are prestressed concrete elements, the limit state is related to stability. The intent of the 1.4 load factor in AASHTO is related to uncertainty in the application of the loads and the susceptibility of the systems to local and global instabilities. Therefore, using this load factor is consistent with the limit state being evaluated.
- The study of I-girders during deck placement revealed that eigenvalue buckling analyses of a single girder line (no system action) is generally sufficient, as the results closely match those of a multi-girder system under the critical load case. The critical load case was when the deck slab concrete is placed in the first two spans of the three-span continuous bridge (Load Case 2). The single girder behavior is actually conservative in a three-dimensional analysis since the solid diaphragms tend to provide warping restraint that improves the buckling response. Therefore, conducting an analysis on a single girder is conservative for the limit state of conventional LTB.
- The FE models neglecting diaphragms were not found overly conservative when determining the critical buckling load. Therefore, a critical load approach such as by eigenvalue buckling analysis or closed-form solution of a single girder is considered an acceptable modeling approach.
- The self-weight of the girder acts on the centroid of the girder but the wet-concrete load during deck placement is applied on the girder top flange. The top-flange loading creates a destabilizing effect on the girder compared to centroidal load as observed in past studies. Therefore, if refined buckling analyses results are not available, the RT recommends using theoretical solutions for lateral-torsional buckling with an effective moment-gradient factor (C_b) of 1.0, which will reflect the destabilizing load position and moment gradient effects. Although steel girder systems often make use of C_b factors greater than 1, those systems have many more intermediate braces between supports and load position has less of an impact.

- The critical buckling moment significantly exceeded the expected applied moment in all cases of uncracked girders. Therefore, the stability of STx girders is not a major concern for bridges with span-to-depth lesser than 35.0 for variable-depth and 30.0 for constant-depth spliced-concrete bridges.
- Although the analyses showed reasonable strengths with diaphragms only at the support conditions, the researchers recommend requiring a diaphragm at either midspan or the location of the cast-in-place splice. Providing a diaphragm at the splice is likely more conducive for most applications since reinforcing steel can be tied into the girder and the contractor is casting concrete at that location already. Stability performance improves significantly when intermediate diaphragms are included between the supports. The current TxDOT practice of including diaphragms at splices and midspan is a good practice that promotes a stable system. TxDOT can likely consider removing the requirements of diaphragms at midspan; however, the researchers recommend requiring the diaphragm at the splice locations within each span.
- When including diaphragms up to the splice locations, for the design range tested, the global system buckling mode, which is a limit state in AASHTO BDS (2024) Section 6.10.3.4.2, was found to not control. This allows for the use of a simplified buckling expression as conventional lateral torsional buckling controlled.
- The impact of initial imperfections on lateral displacement was assessed. Values of $L/1000$ and $L/500$ initial imperfections were considered in the analysis. The $L/1000$ limit is consistent with the PCI tolerance of $1/8''$ per 10 ft of girder length ($L/960$). The researchers recommend keeping the limit of $L/1000$ for the imperfection in practice. Larger imperfections lead to larger deformations during the deck cast, which can impair the constructability of the girder systems. If analyses are conducted to evaluate the girder deformations during deck casts, the deformations should be evaluated at the expected maximum loads (do not use the load factor of 1.4 that is intended for a strength limit state).

8.2.4. Effect of Cracking in Girder Stability

There is the likelihood of crack initiation in girders during lifting, transport, and erection, even if the cracks are not visible to the naked eye. These cracks may be harder to detect due to the prestressing effect found to cause crack closure upon unloading, as observed in both the test and FE analysis. The occurrence of cracking can significantly impact the load-carrying capacity of the system due to the substantial reduction in stiffness. The spliced girders were analyzed for stability considering the reduced stiffness. The following conclusions are made based upon the results of the parametric study:

- In a bridge system, cracking leads to redistribution of moments and affects the stability of the structure, depending on the depth of cracks. The capacity and deformability of the

bridge system are significantly reduced as the crack depth in both the positive and negative moment regions approaches the center of gravity. However, the stability of the system is not significantly affected if the crack depth remains limited to the flange regions.

- The stability of the I-section bridges was affected to some extent by the girder cross-section. As the depth of the I-girder increases, the magnitude of capacity reduction increases correspondingly.
- The capacity of the U-girder bridges is more susceptible to cracking due to the open configuration of the section. After crack initiation, significant cross-sectional distortion may occur.

8.2.5. Live Load Distribution

An extensive parametric study was carried out to evaluate the approximate analysis method recommended by AASHTO (2024) for spliced girder bridges in Texas. The following conclusions were derived from the parametric FEA studies:

- Live load analysis using influence surfaces was efficient in determining the critical force effects in the girders for a very large number of detailed FE models.
- Live Load Distribution Factors (LLDFs) calculated from the AASHTO equations for bending moments and shear reactions were mostly conservative or approximately equal to the values calculated from FE analyses.
- LLDF equations using average stiffness properties and span lengths exceeding the limits specified by AASHTO may be used for spliced-girder bridges for design purposes.
- The variation in girder stiffness near the interior supports had negligible difference in the LLDFs, if the variation is also modeled in the line-girder analysis.
- The diaphragm layouts play a significant role in load distribution. Diaphragms at the supports are required and that requirement should not be removed in the future. The researchers recommend at least one intermediate diaphragm be required between each support (either at the splice or at midspan) to provide The intermediate diaphragms at splices or midspan help reduce the negative moments at the support for the interior girders. The diaphragm at mid-span help distribute the moments to the girders more efficiently; however, providing a diaphragm at either midspan or the splices are satisfactory. AASHTO equations do not differentiate the bridges with diaphragms, however, the LLDFs may be more conservative for many cases.
- Skewed supports reduce moments and increase shear reactions in the girders. The diaphragms add more complexity in the load distribution due to the different load paths offered by the diaphragm in addition to the skewed supports and deck slab. The existing

correction factors to account the effects of support skew are applicable for the spliced girder bridges.

8.3. Scope for Future Research

The research work resulted in a significant improvement in the current knowledge on the stability and live-load behavior of spliced concrete girder bridges. However, there are several limitations to the study due to the assumptions in the analysis and geometric limits considered in the parametric studies. The limitations open up areas for future research on the spliced girder bridge systems. Some of the potential areas for future study are discussed below:

- The second-order analyses showed large lateral displacements during deck placement can occur for longer spans. The analyses did not consider the influence of intermediate diaphragms and connection details of precast deck panels on the girder, which may significantly reduce the lateral deformations. In TxDOT Project 0-5706, Yang et al. (2010) showed the distribution and stiffness of top bracing bars influence the girder behavior during deck overhang construction. A detailed numerical study on spliced girder bridges including the precast deck panels and the connections may result in more accurate estimates for lateral deformations.
- There are guides for evaluating stability of prestressed girders during lifting. The current guidance is based on allowable material stresses and does address the roll stability of the girders during lifting. The lifting case is unique because there is no restraint on lateral movement and the unbraced length is equal to the entire girder length that is being lifted. Additionally, the number and placement of the lifting points affect the stability of the girder. The stability of steel girders during erection was studied in TxDOT Project 0-5574 (Stith et al., 2010). Similarly, benefits can be achieved by studying the behavior of the concrete girder segments during lifting; particularly if the segment lengths continue to get longer.
- A simplified approach was adopted in this study to account for the effects of cracking on girder stability. However, the effects of cracking are more complex and evolving during the load application. A detailed numerical study including the evolution of cracking and the prestressing effects may provide a better understanding of the girder stability.
- Although the researchers are attempting to consider the susceptibility of the prestressed girder units to global buckling modes (similar to the steel provisions in AASHTO BDS (2024) Section 10.3.4.2), more detailed investigation may be warranted. Recent work on steel girder systems (Fish et al. (2025) has significantly improved the prediction abilities of the system mode limit state. The span lengths of the spliced-concrete girders are increasing and these girders systems may exhibit a similar limit state, particularly for narrow girder units consisting of 4 girders or less.

- The development of Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC) has enabled the design of slender concrete girders for spans longer than conventional prestressed concrete girders. The long-span spliced girders considered in this study were satisfactory for stability during construction. Slender UHPFRC girders may need to be evaluated for stability due to the difference in stiffness properties compared to the conventional prestressed concrete girders.
- Current AASHTO equations for LLDFs are not applicable for the range of span lengths and stiffness of spliced-concrete girder bridges. Although the equations are satisfactory for design purposes, there is a scope for design optimization if the equations are modified for long-span bridges. A more rigorous study on live load distribution of long-span concrete and steel girder bridges is necessary to extend the LLDF equations beyond the current limits for their application.

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Appendix A: Experimental Testing Results and Validation of Strain Gauge Data

This appendix presents the experimental testing results of the I-girder specimens and summarizes the validation results for the strain values obtained from the vibrating wire gauges installed on the Tx-U40, Tx28, and Tx34 specimens.

A.1 Experimental Testing Results for I-Girders

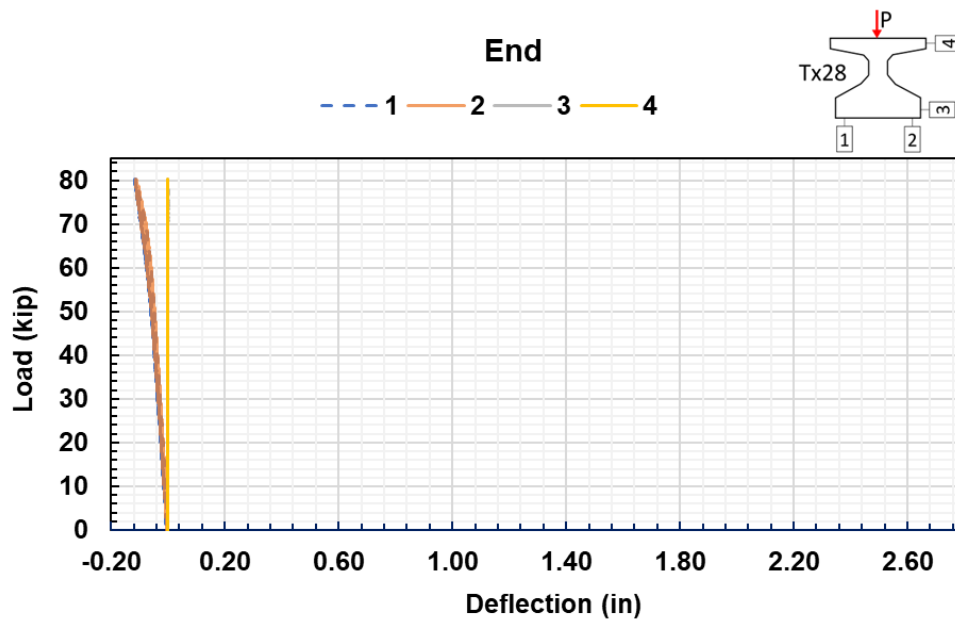


Figure A.1. Tx28 bending to cracking test graph of load-deflection at girder end

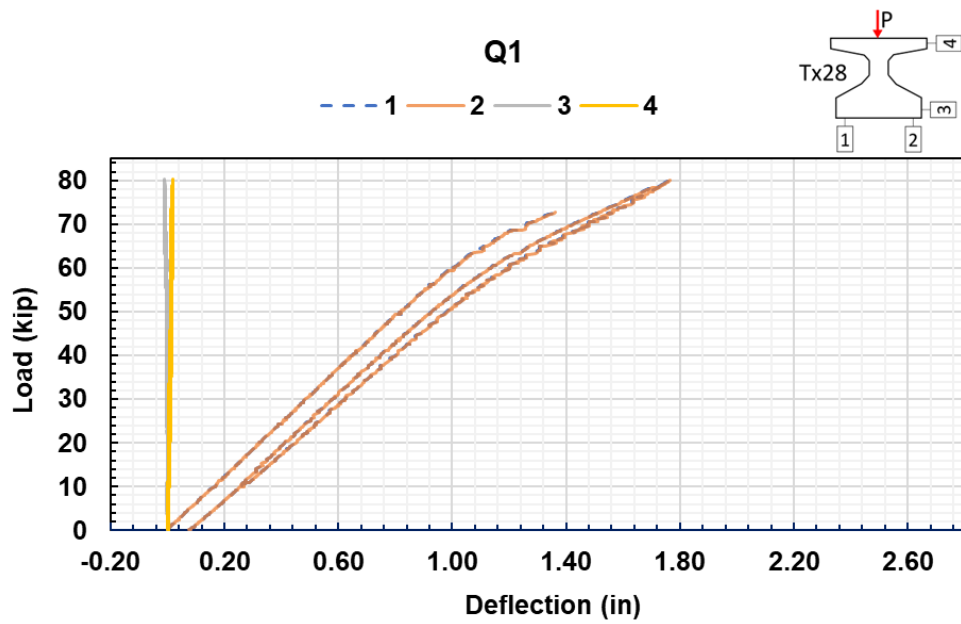


Figure A.2. Tx28 bending to cracking test graph of load-deflection at first quarter point (Q1)

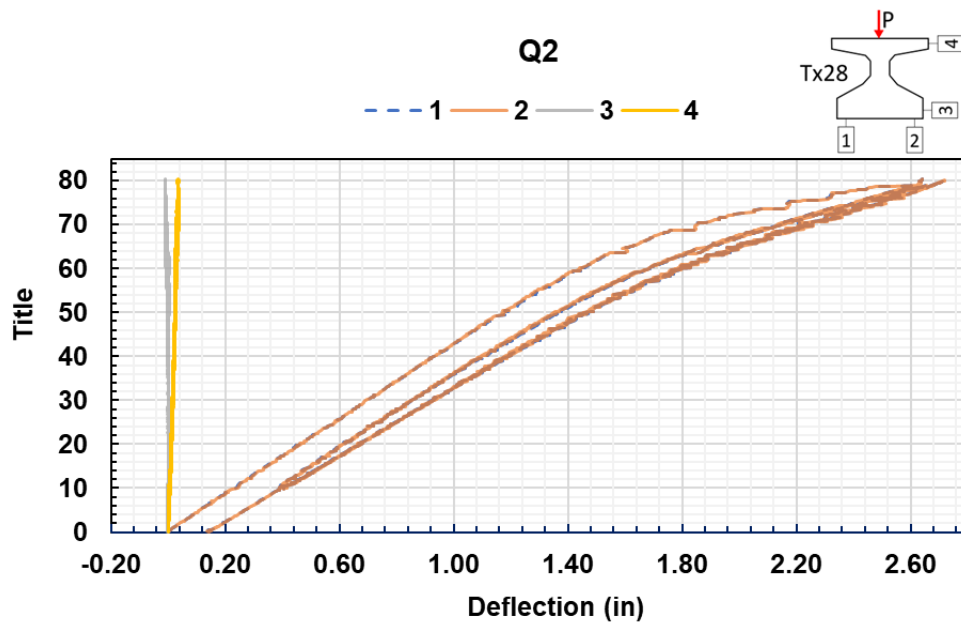


Figure A.3. Tx28 bending to cracking test graph of load-deflection at midspan (Q2)

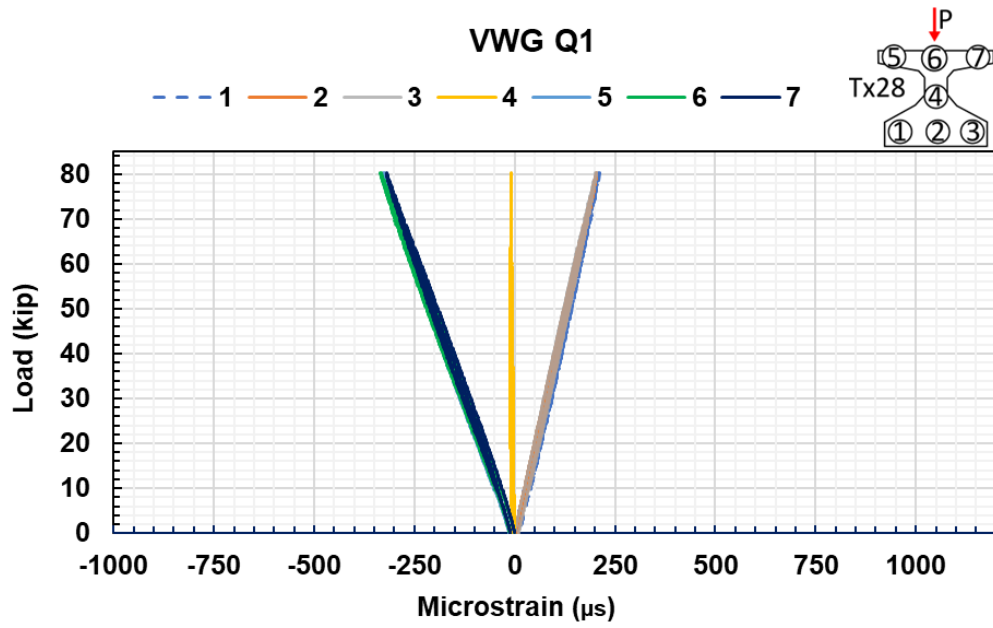


Figure A.4. Tx28 bending to cracking test graph of load-strain at first quarter point (Q1)

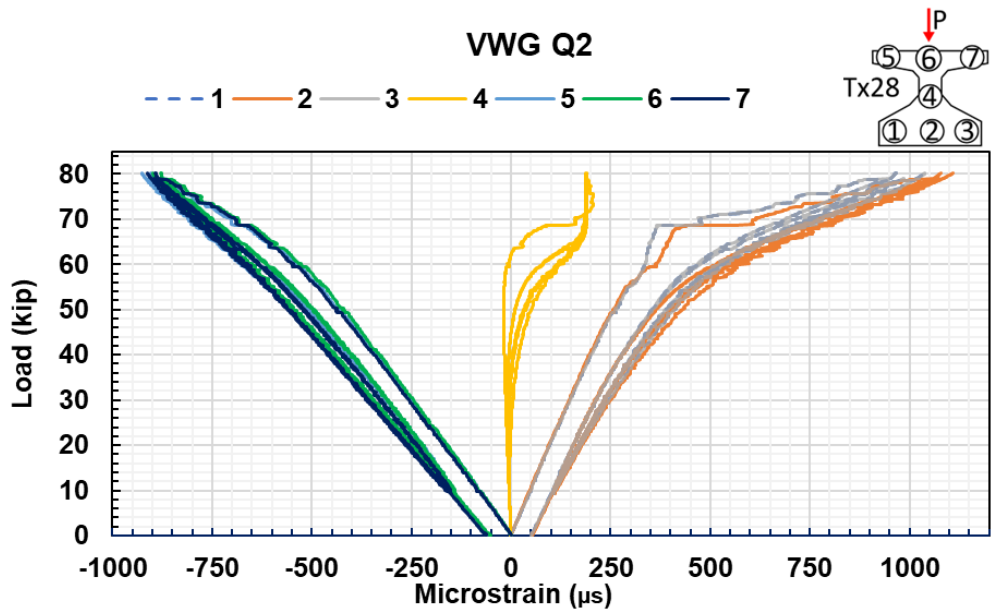


Figure A.5. Tx28 bending to cracking test graph of load-strain at midspan (Q2)

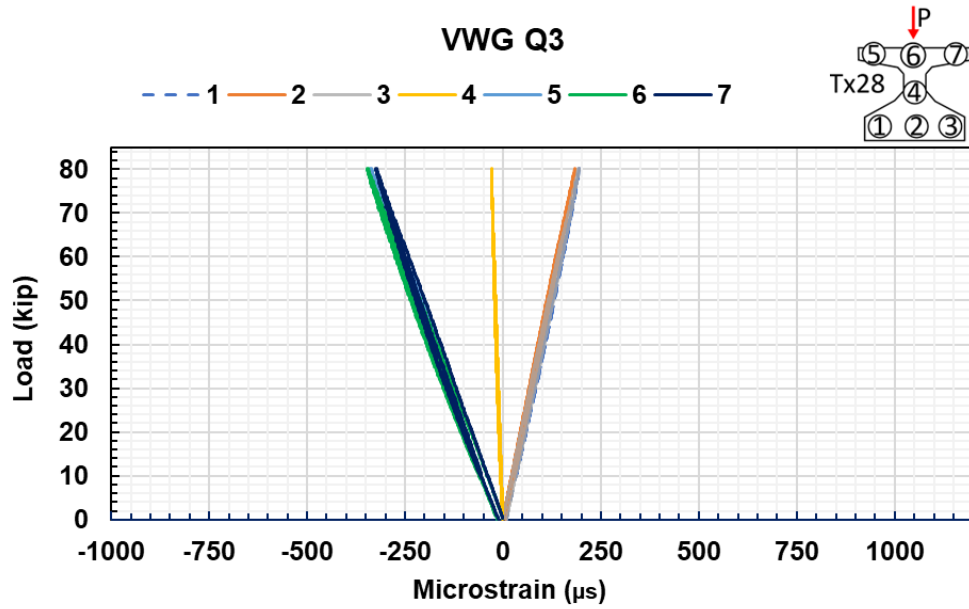


Figure A.6. Tx28 bending to cracking test graph of load-strain at second quarter point (Q3)

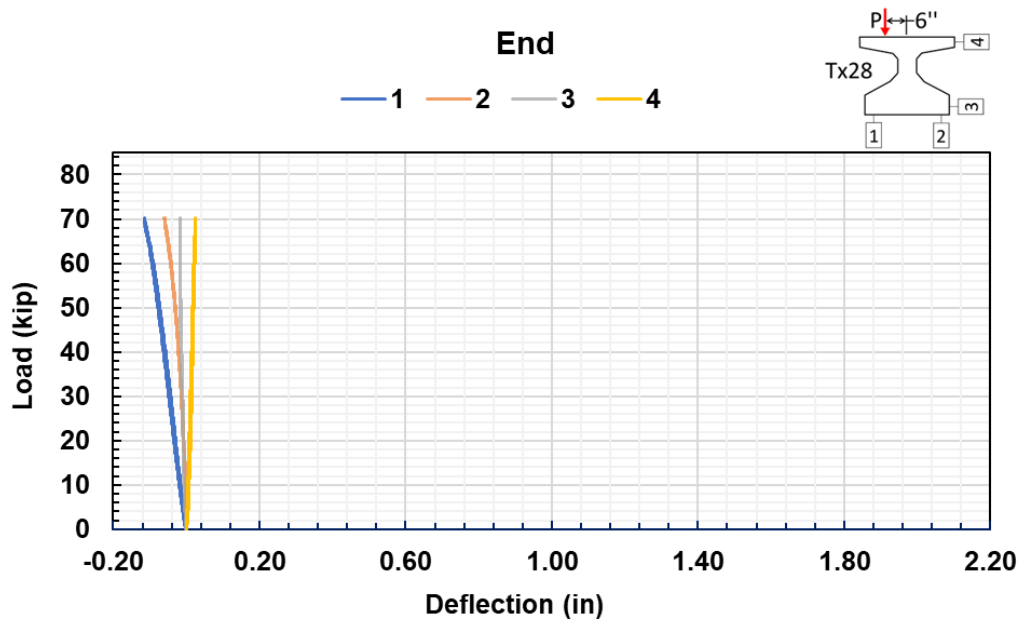


Figure A.7. Tx28 combined bending and torsion test after cracking graph of load-deflection at girder end

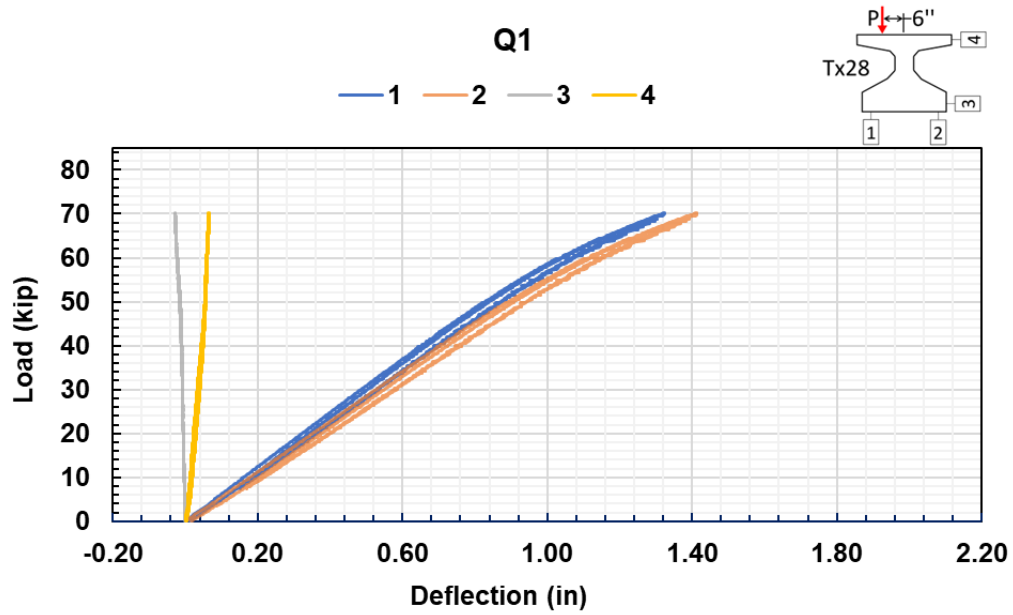


Figure A.8. Tx28 combined bending and torsion test after cracking graph of load-deflection at first quarter point (Q1)

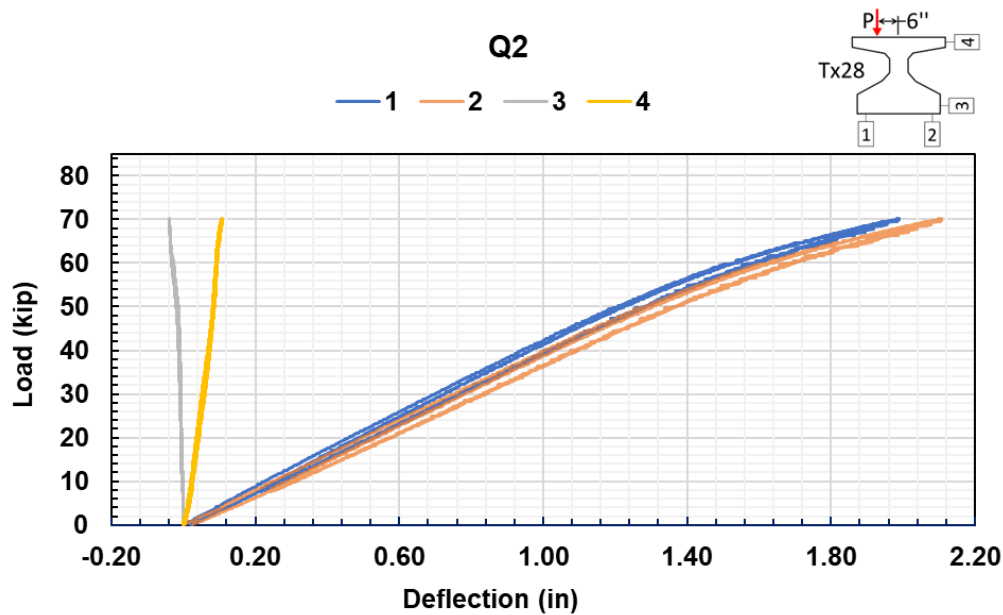


Figure A.9. Tx28 combined bending and torsion test after cracking graph of load-deflection at midspan (Q2)

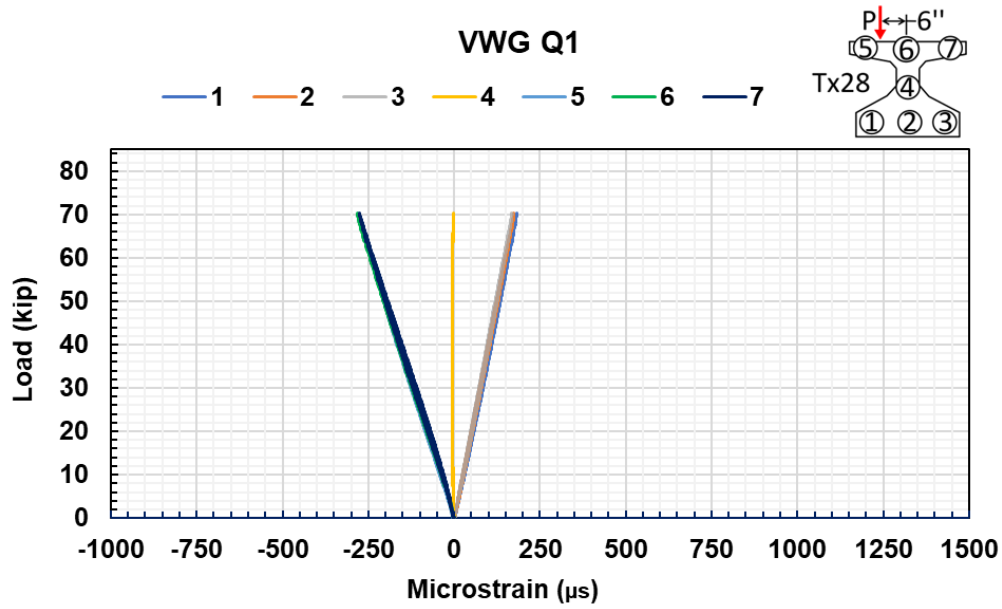


Figure A.10. Tx28 combined bending and torsion test after cracking graph of load-strain at first quarter point (Q1)

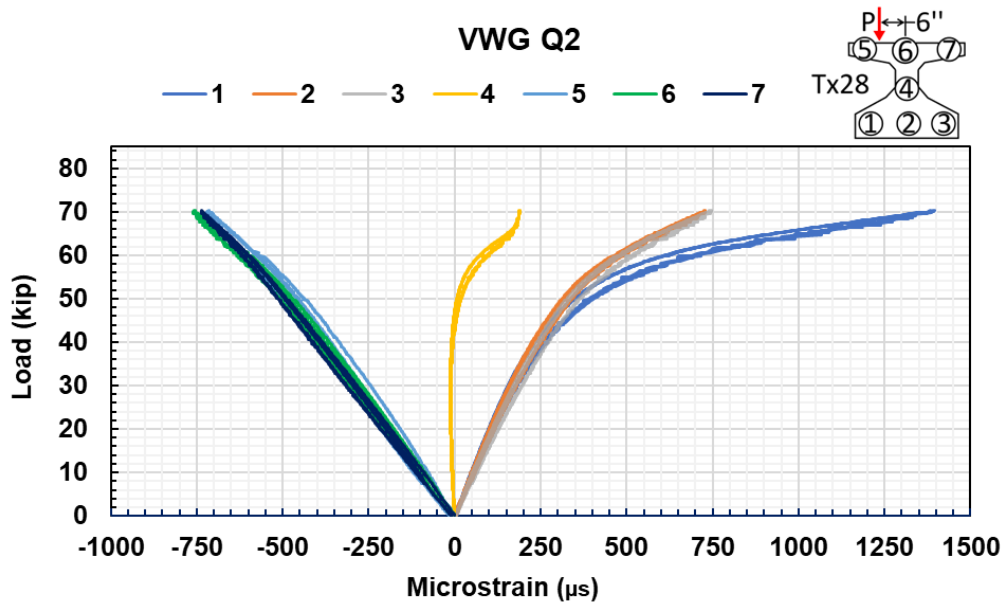


Figure A.11. Tx28 combined bending and torsion test after cracking graph of load-strain at midspan (Q2)

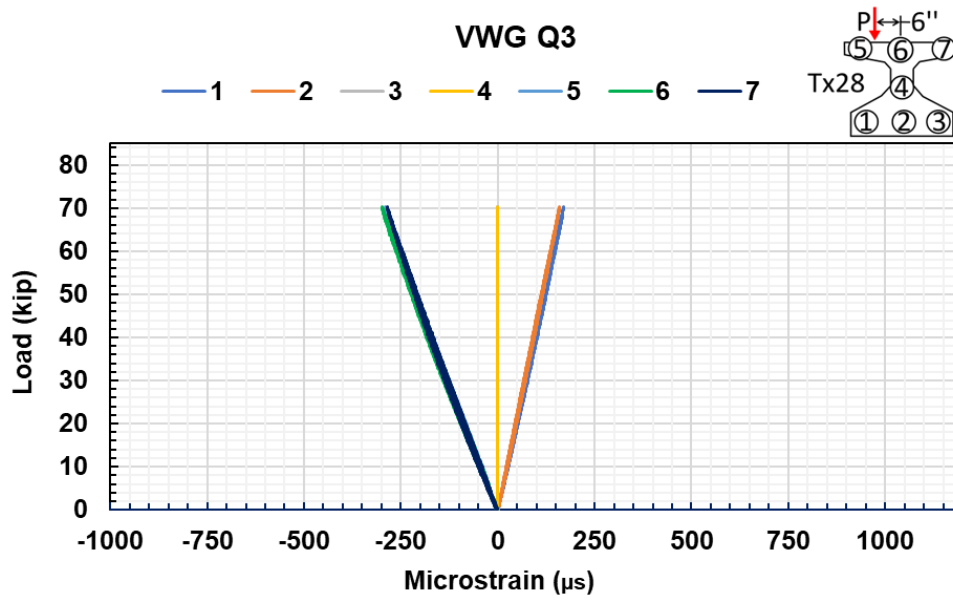


Figure A.12. Tx28 combined bending and torsion test after cracking graph of load-strain at second quarter point (Q3)

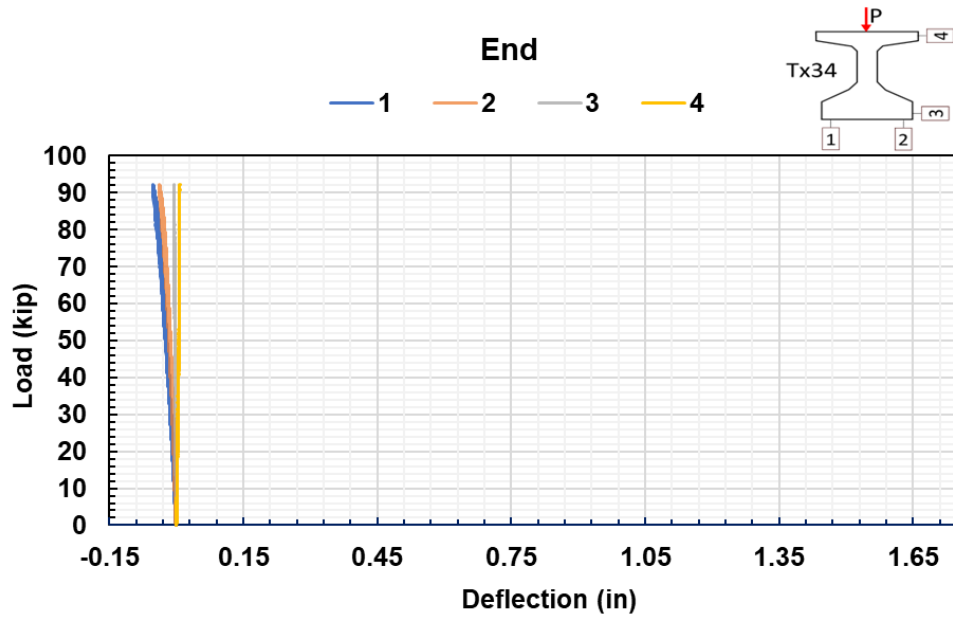


Figure A.13. Tx34 bending to cracking test graph of load-deflection at girder end

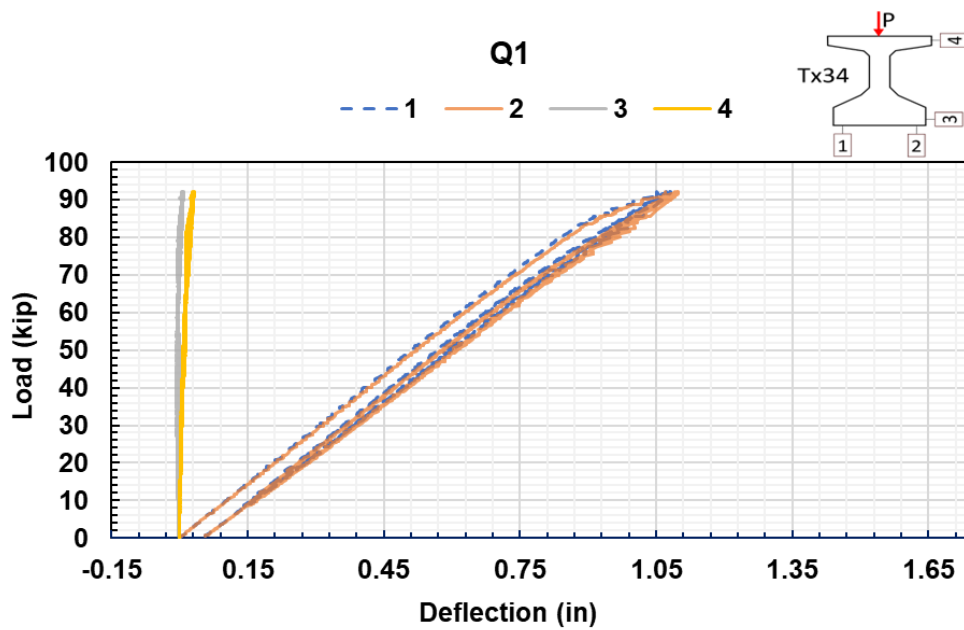


Figure A.14. Tx34 bending to cracking test graph of load-deflection at first quarter point (Q1)

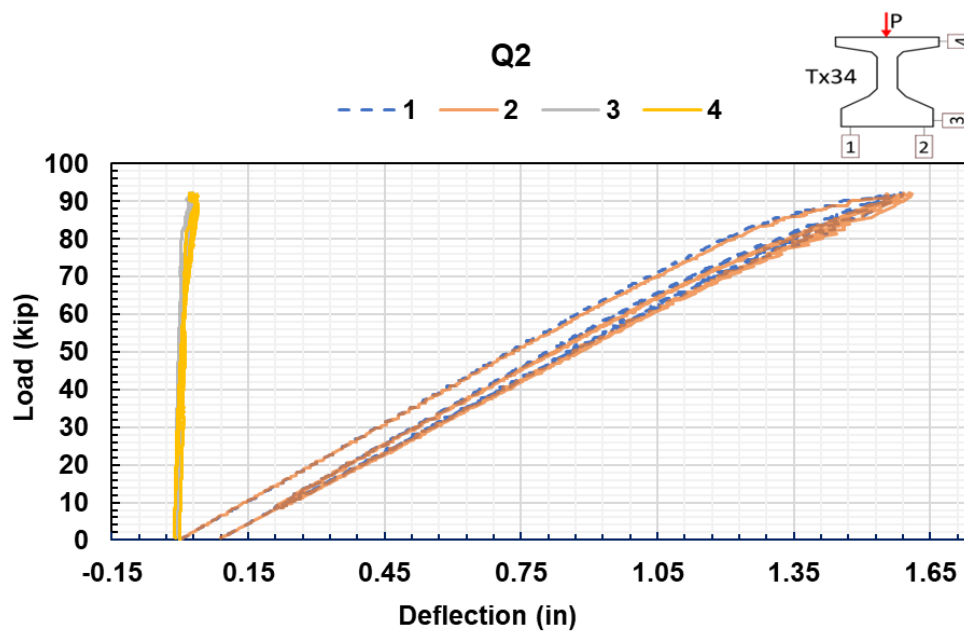


Figure A.15. Tx28 bending to cracking test graph of load-deflection at midspan (Q2)

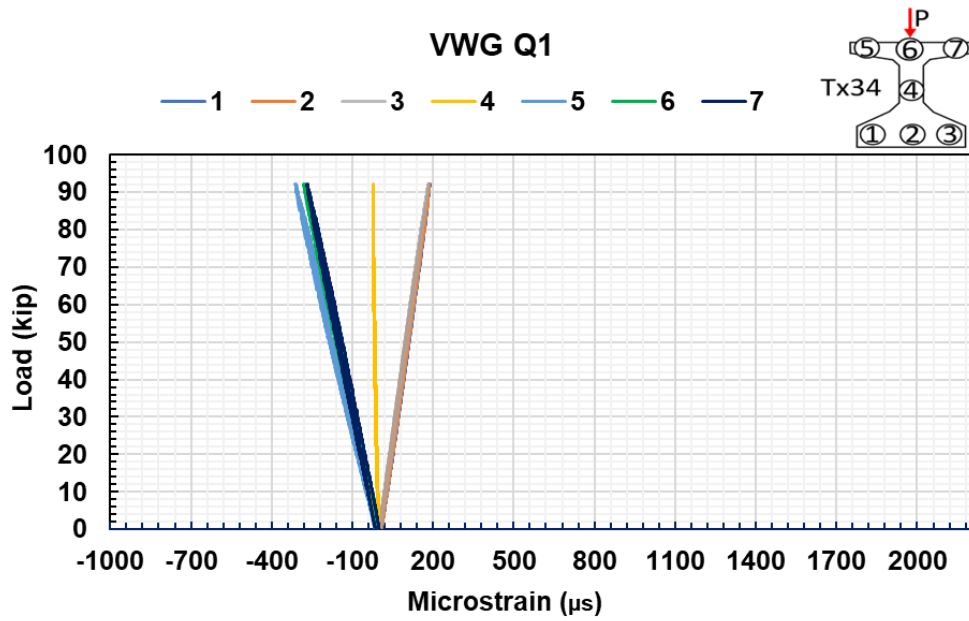


Figure A.16. Tx34 bending to cracking test graph of load-strain at first quarter point (Q1)

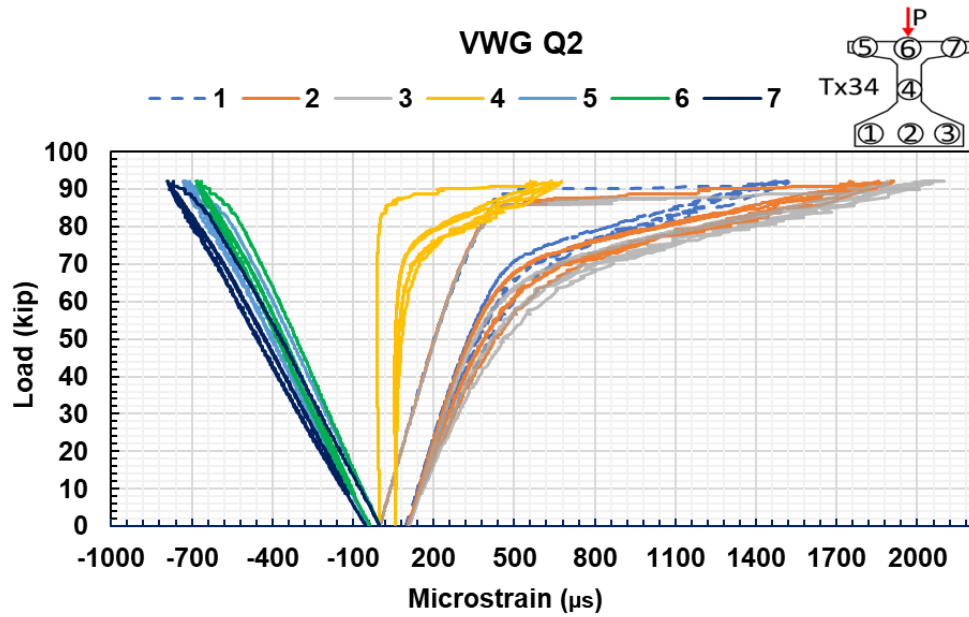


Figure A.17. Tx34 bending to cracking test graph of load-strain at midspan (Q2)

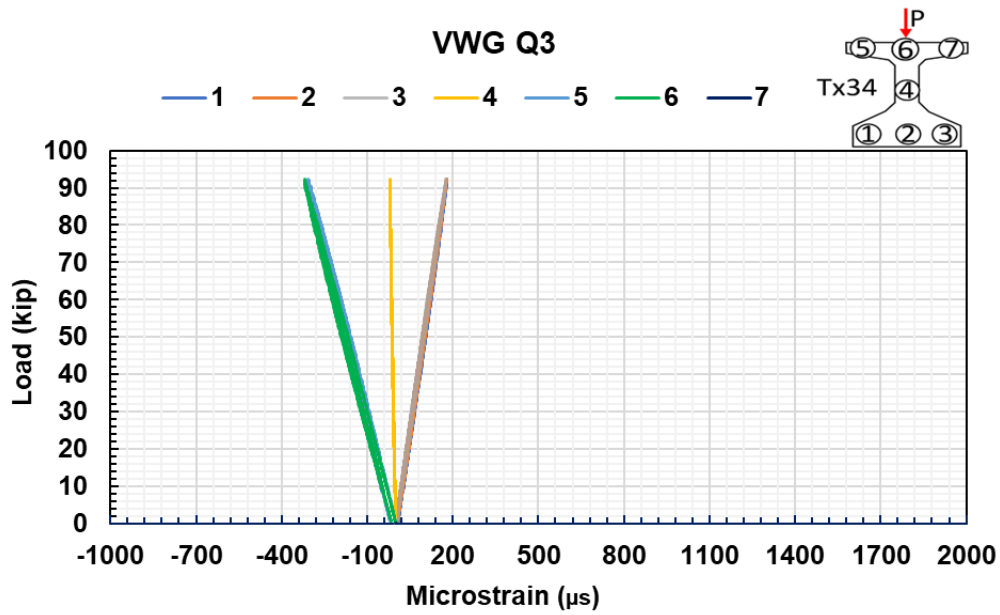


Figure A.18. Tx34 bending to cracking test graph of load-strain at second quarter point (Q3)

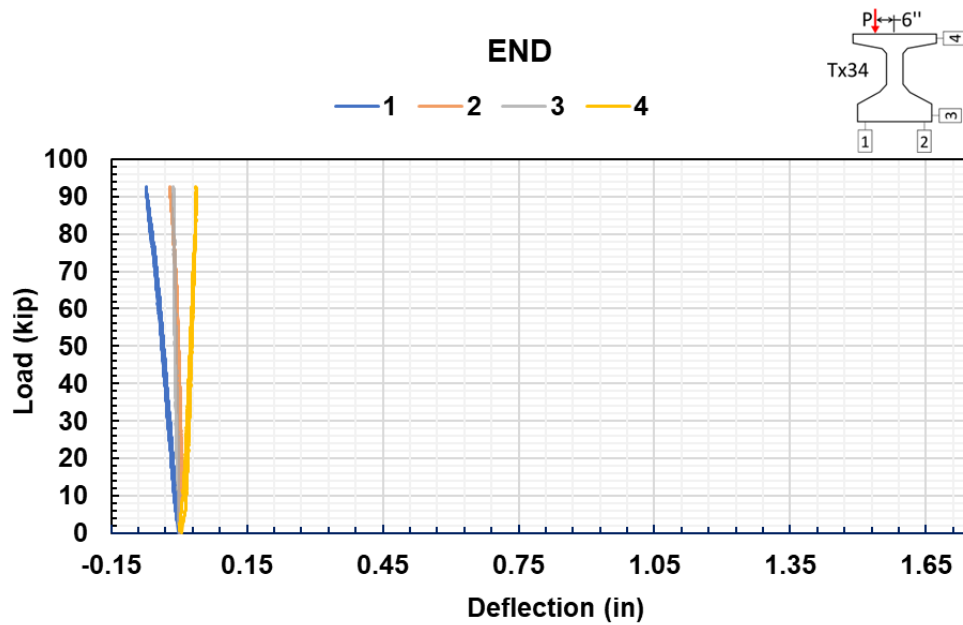


Figure A.19. Tx34 combined bending and torsion test after cracking graph of load-deflection at girder end

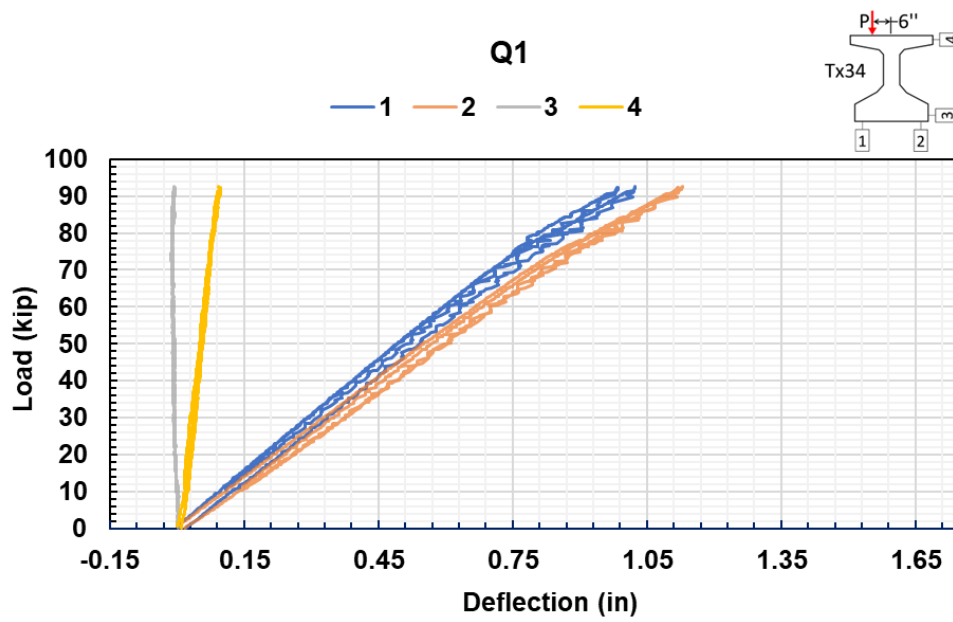


Figure A.20. Tx34 combined bending and torsion test after cracking graph of load-deflection at first quarter point (Q1)

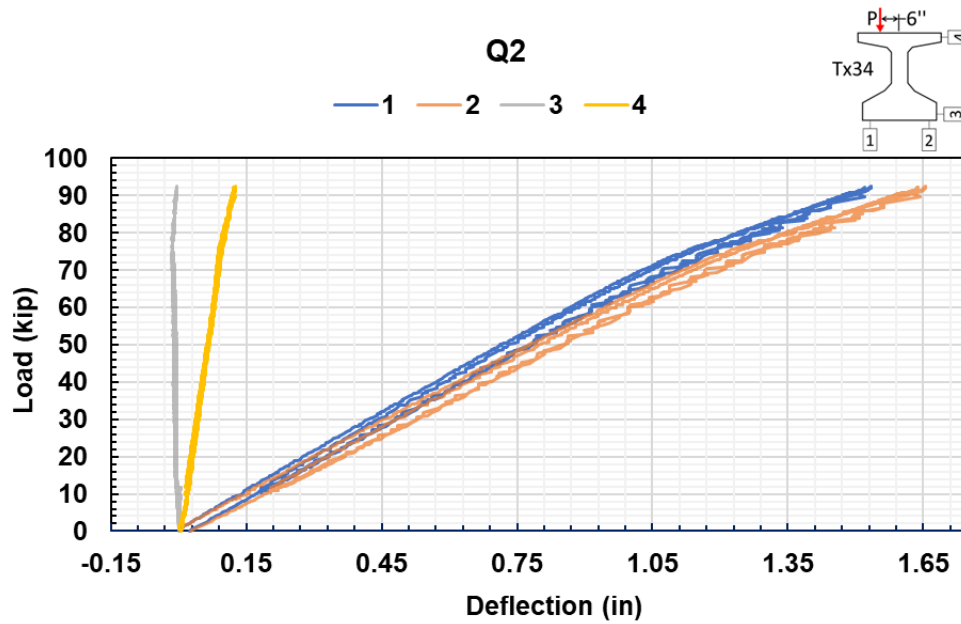


Figure A.21. Tx34 combined bending and torsion test after cracking graph of load-deflection at midspan (Q2)

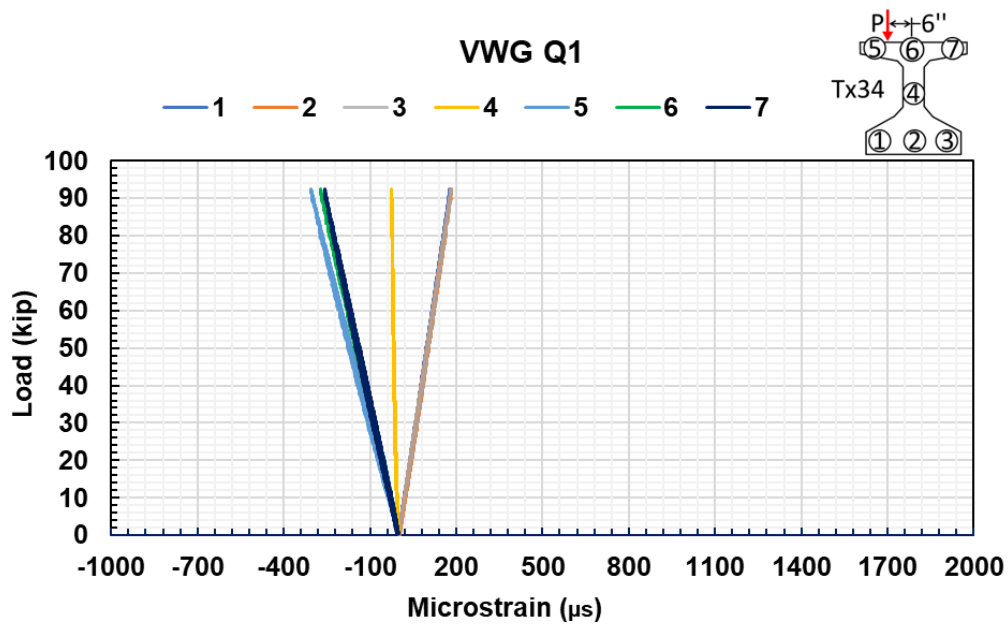


Figure A.22. Tx34 combined bending and torsion test after cracking graph of load-strain at first quarter point (Q1)

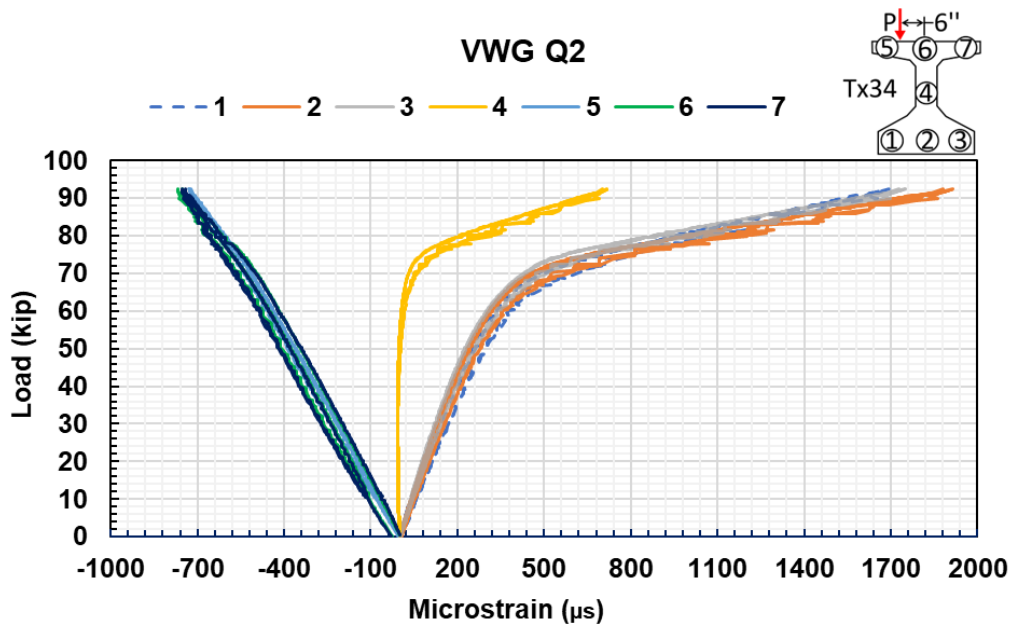


Figure A.23. Tx34 combined bending and torsion test after cracking graph of load-strain at midspan (Q2)

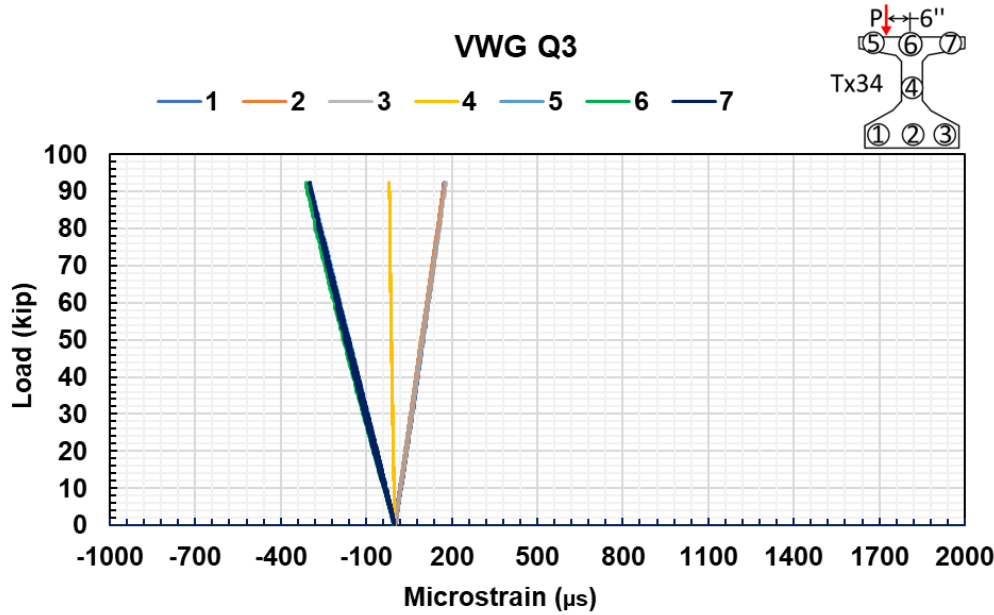


Figure A.24. Tx34 combined bending and torsion test after cracking graph of load-strain at second quarter point (Q3)

A.2 Validation of Strain Gauge Data

The vibrating wire strain gauges were positioned across the section height at the second quarter point (Q3) along the girder length – symmetrically aligned with those at the first quarter point. Similar to the validation scheme adopted for gauges at the first and second quarter points, the ϵ_{xx} component was investigated, per Figure 4.5 and Figure 4.8. The strain gauge values obtained from vibrating wire gauges for fibers across the section height are compared with their numerical counterparts for Tx-U40 girder in Figures A.25. The experimental measurements were linear as cracking had not yet initiated at this location. The analysis effectively captured the strain values observed in the experiment for all fibers on the top and bottom flange portions, thereby validating the employed numerical approach. The gauge readings are positive on the bottom flange, in contrast to those on the top flange, due to the state of stress developed in these regions, with tension in the former and compression in the latter.

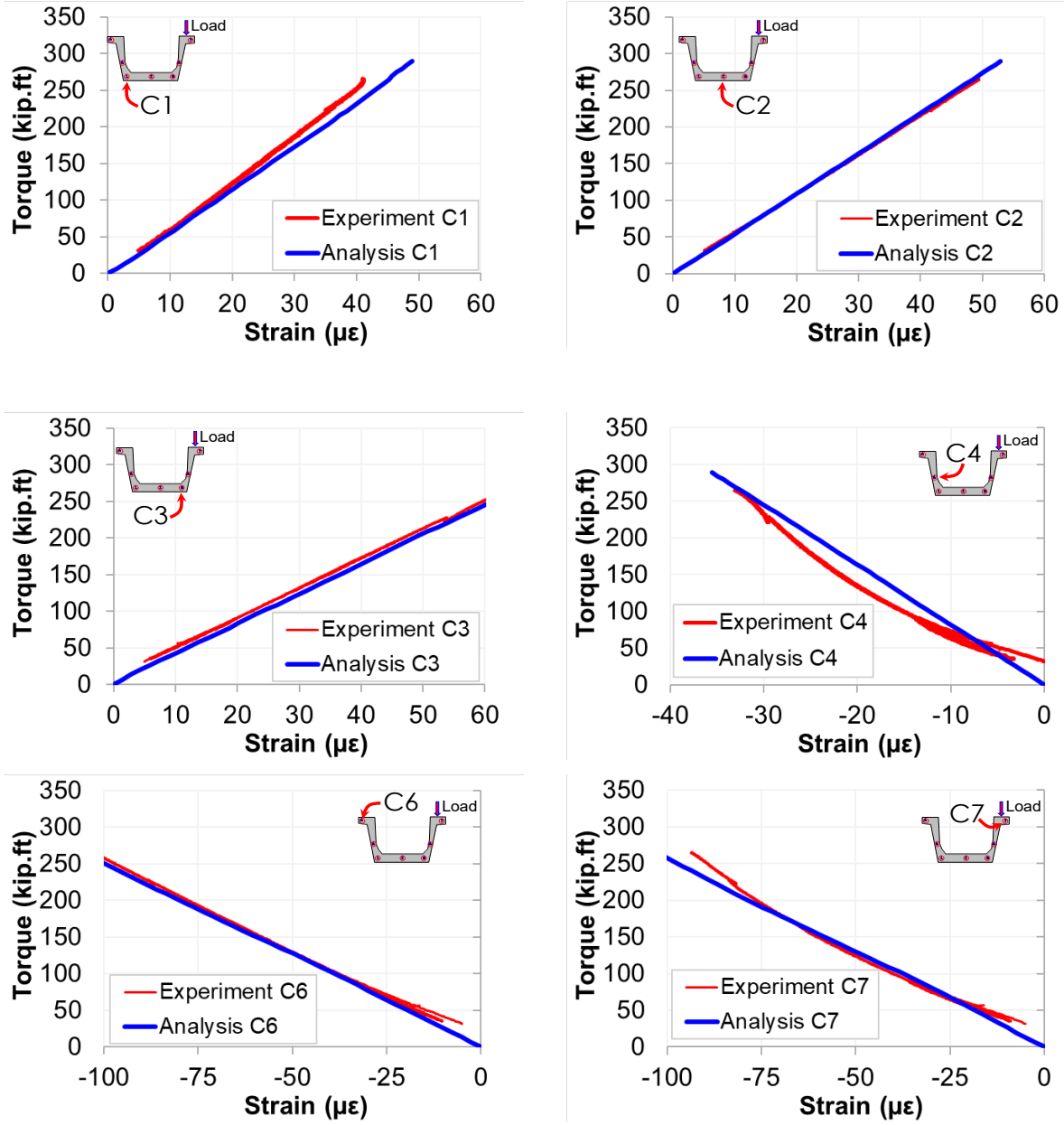


Figure A.25. Numerical and experimental strain values at different locations in the cross-section at the second quarter point (Q3) of the Tx-U40 specimen

Figure A.26 compares the experimental data obtained from vibrating wire gauges installed at the second quarter point (Q3) of the Tx34 girder with their numerical counterparts. The analysis gave a near-perfect match of the experimental response for all fibers on the top and bottom flanges, thereby validating the adopted numerical approach. The gauge readings are positive on the bottom flange, in contrast to those on the top flange, given the state of stress developed in these regions, with tension in the former and compression in the latter.

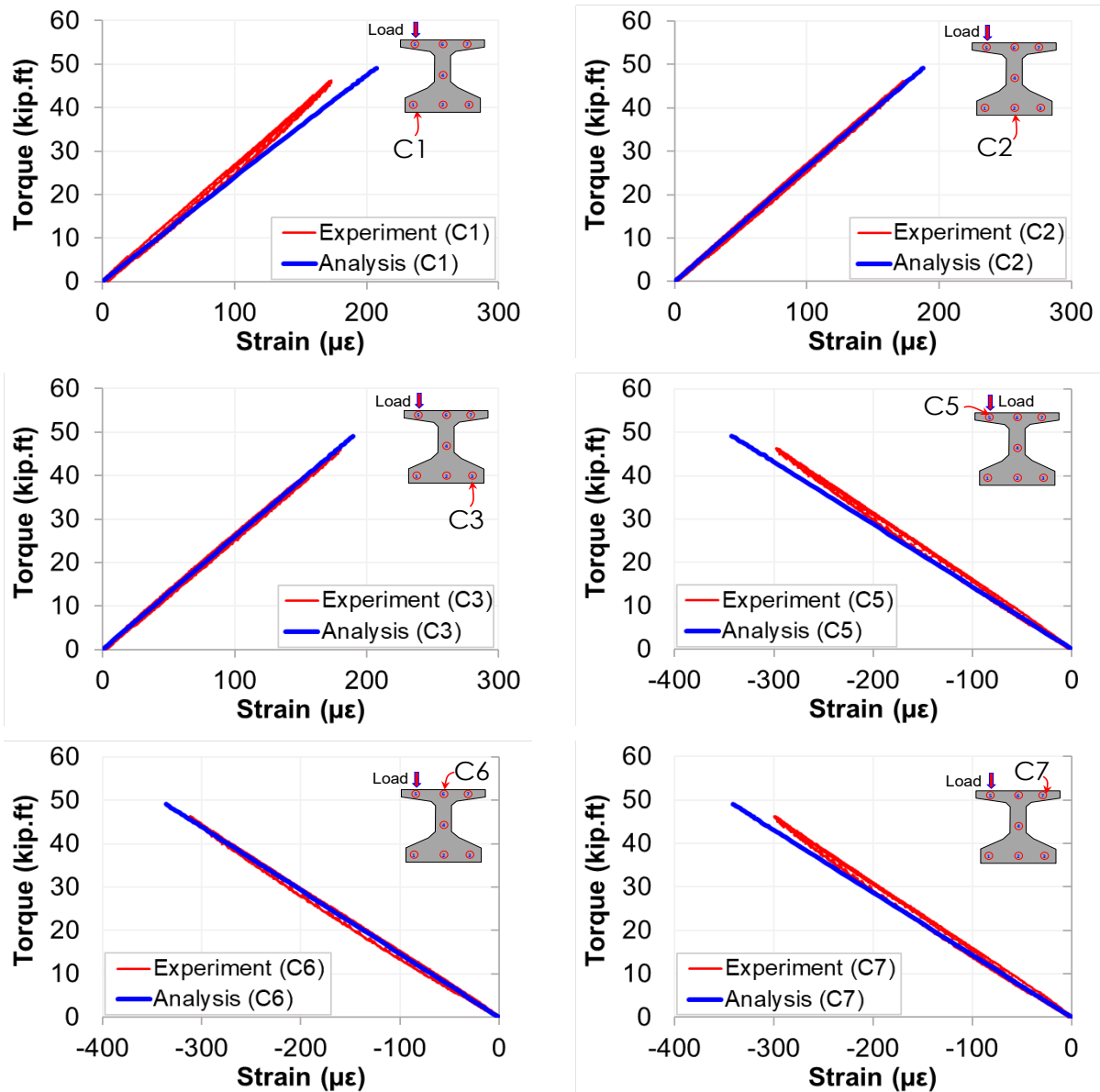


Figure A.26. Numerical and experimental strain values at different locations in the cross-section at the second quarter point (Q3) of the Tx34 specimen

Subsequently, the strain gauge values obtained from vibrating wire gauges for fibers across the section height are compared with their numerical counterparts for the Tx28 girder in Figure A.27. Similar to the conclusions reached for other specimens, the analysis effectively captured the strain

values observed in the experiment for all fibers in the top and bottom flanges. The developed states of stress were tension in the bottom flange and compression in the top flange, resulting in positive and negative strain readings, respectively – as shown in the figure.

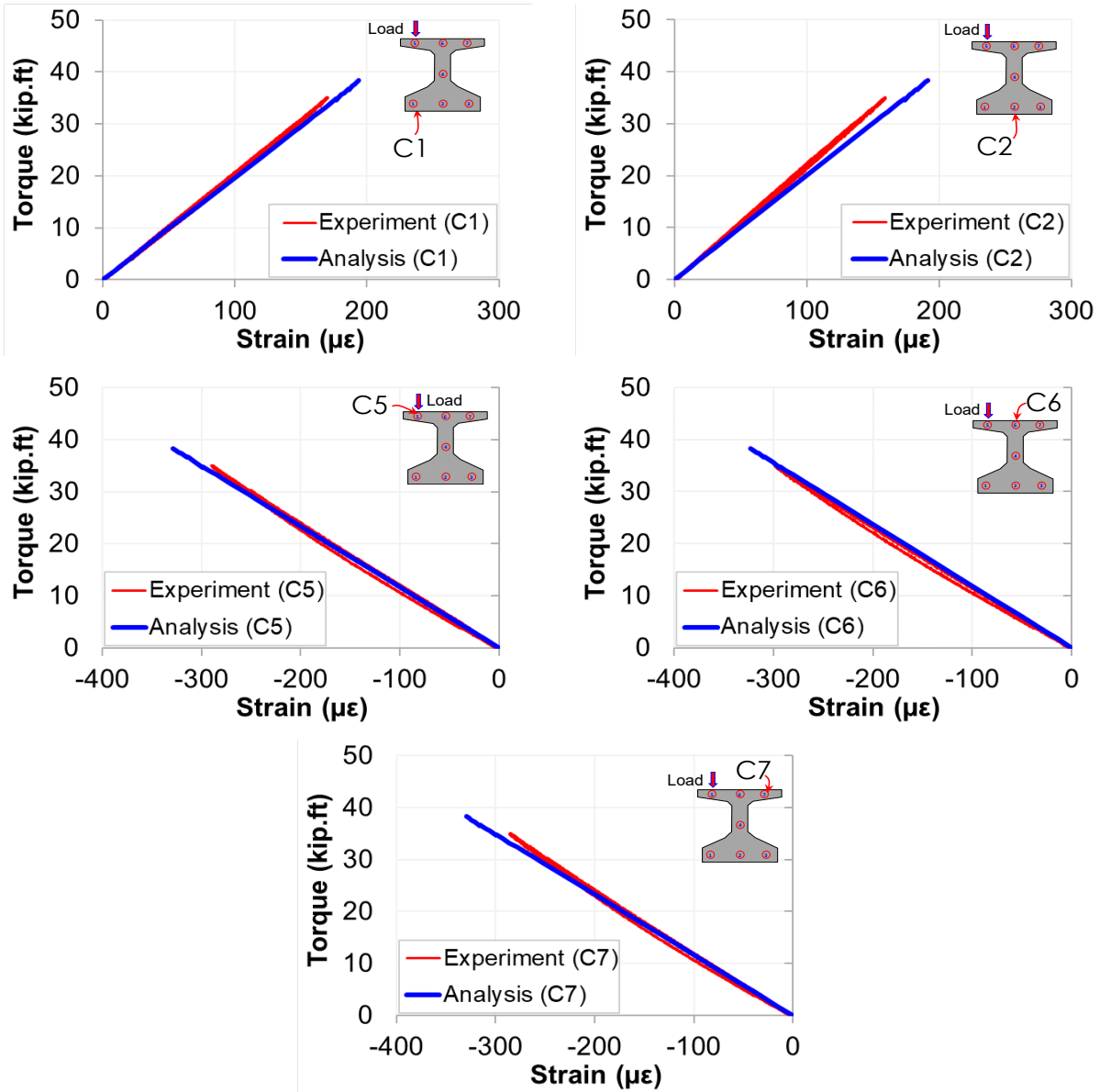
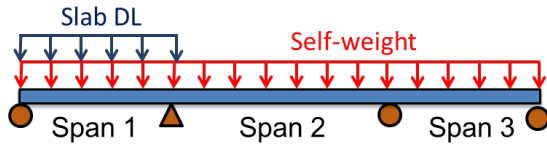


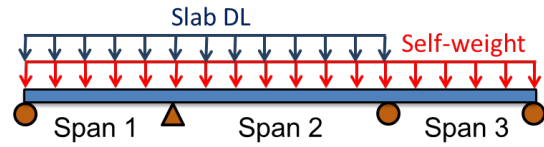
Figure A.27. Numerical and experimental strain values at different locations in the cross-section at the second quarter point (Q3) of the Tx28 specimen

Appendix B: Eigenvalue buckling analysis of multi-girder systems

- Constant-depth girder



Load case 1

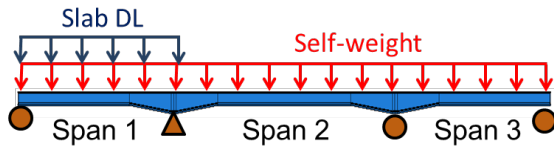


Load case 2

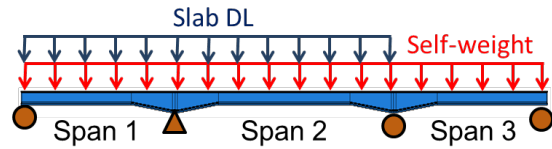
Load case	Section	L/D	Girder spacing (ft)	Eigenvalue	Buckling span	Buckling girder	C_b	$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$	$\frac{M_{cr, FEA}}{C_b M_{cr, UM}}$
1	STx7	25	8	21.00	2	Exterior	2.40	51,054	1.47	0.61
			10	20.19	1	Exterior	1.84	68,494	1.37	0.74
			12	18.51	1	Interior	1.71	71,498	1.42	0.83
		30	8	12.26	2	Exterior	2.42	42,538	1.48	0.61
			10	11.93	1	Exterior	1.85	57,879	1.40	0.75
			12	10.97	1	Interior	1.72	60,581	1.46	0.85
	STx9	25	8	9.79	2	Exterior	2.34	44,075	1.46	0.62
			10	9.78	2	Exterior	2.39	43,506	1.44	0.60
			12	9.51	1	Interior	1.79	61,936	1.42	0.79
		30	8	5.65	2	Exterior	2.36	36,371	1.45	0.62
			10	5.67	2	Exterior	2.41	36,005	1.44	0.60
			12	5.59	2	Exterior	2.46	35,070	1.40	0.57
2	STx7	25	8	10.70	2	Exterior	1.82	50,592	1.46	0.80
			10	10.03	2	Interior	1.81	50,963	1.47	0.81
			12	9.10	2	Interior	1.80	50,515	1.46	0.81
		30	8	6.27	2	Exterior	1.83	42,355	1.47	0.80
			10	5.88	2	Interior	1.82	42,647	1.48	0.81
			12	5.34	2	Interior	1.81	42,333	1.47	0.81
	STx9	25	8	5.30	2	Exterior	1.83	43,172	1.43	0.78
			10	4.99	2	Interior	1.82	43,428	1.44	0.79
			12	4.56	2	Interior	1.81	43,071	1.43	0.79
		30	8	3.09	2	Exterior	1.84	35,992	1.44	0.78
			10	2.91	2	Interior	1.83	36,205	1.45	0.79
			12	2.66	2	Interior	1.82	35,963	1.44	0.79

•

- Variable-depth girder



Load case 1

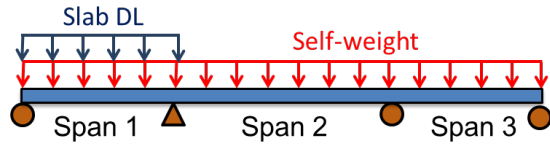


Load case 2

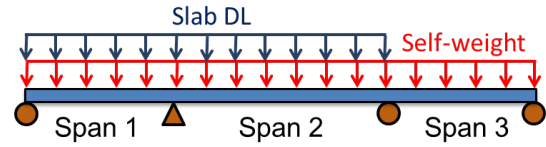
Load case	Section	L/D	Girder spacing (ft)	Eigenvalue	Buckling span	Buckling girder	C_b	$M_{cr, FEA}$ (k-ft)	$\frac{M_{cr, FEA}}{M_{cr, UM}}$	$\frac{M_{cr, FEA}}{C_b M_{cr, UM}}$
1	STx7	30	8	14.86	1	Exterior	2.26	60,367	1.45	0.64
			10	13.93	1	Exterior	2.23	60,045	1.44	0.65
			12	12.75	1	Interior	2.16	62,919	1.51	0.70
		35	8	9.38	1	Exterior	2.26	52,043	1.47	0.65
			10	8.79	1	Exterior	2.23	51,780	1.46	0.66
			12	8.06	1	Interior	2.16	54,343	1.54	0.71
	STx9	30	8	7.56	2	Exterior	3.17	33,575	1.34	0.42
			10	7.07	1	Exterior	2.27	51,715	1.43	0.63
			12	6.66	1	Exterior	2.24	51,369	1.42	0.63
		35	8	4.63	2	Exterior	3.17	28,031	1.31	0.41
			10	4.46	1	Exterior	2.27	44,462	1.44	0.64
			12	4.14	1	Interior	2.20	46,949	1.53	0.69
2	STx7	30	8	8.27	2	Exterior	2.64	42,365	1.47	0.56
			10	7.72	2	Interior	2.62	42,642	1.48	0.57
			12	6.99	2	Interior	2.60	42,390	1.47	0.57
		35	8	5.16	2	Exterior	2.65	36,094	1.47	0.55
			10	4.82	2	Interior	2.63	36,333	1.48	0.56
			12	4.37	2	Interior	2.60	36,161	1.47	0.57
	STx9	30	8	4.14	2	Exterior	2.66	36,560	1.46	0.55
			10	3.81	2	Interior	2.64	36,076	1.44	0.55
			12	3.55	2	Interior	2.62	36,638	1.46	0.56
		35	8	2.53	2	Exterior	2.66	30,481	1.43	0.54
			10	2.38	2	Interior	2.64	30,666	1.43	0.54
			12	2.17	2	Interior	2.62	30,527	1.43	0.55

Appendix C: Riks analysis of single-girder systems

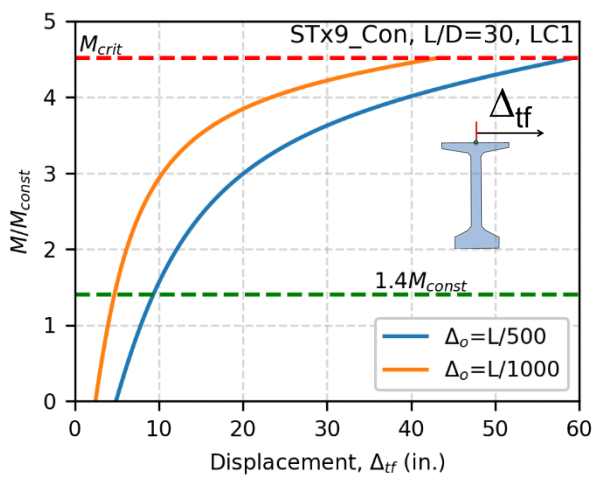
- Constant-depth girder with no crack



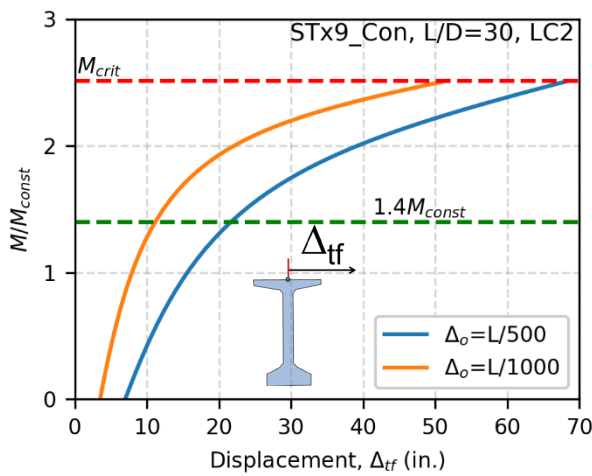
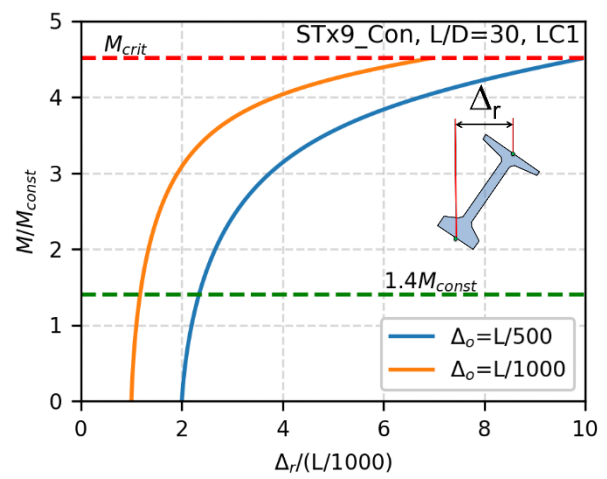
Load case 1



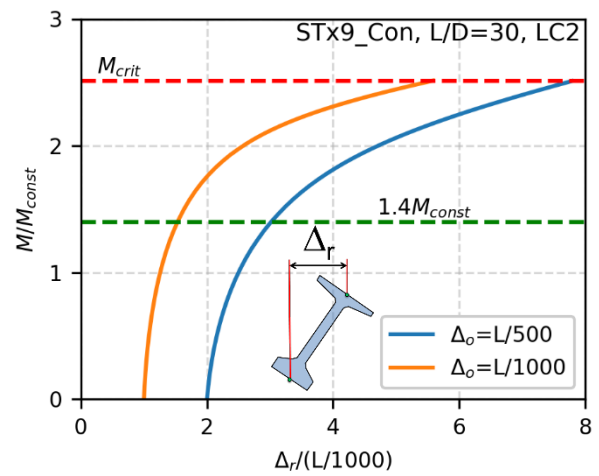
Load case 2



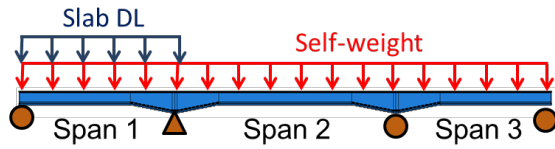
(a) STx9, L/D = 30, S = 12ft, Load case 1



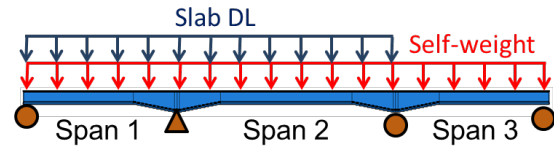
(b) STx9, L/D = 30, S = 12ft, Load case 2



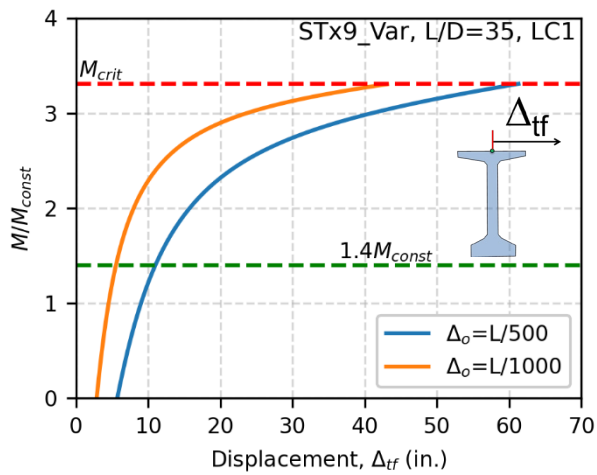
- Variable-depth girder with no crack



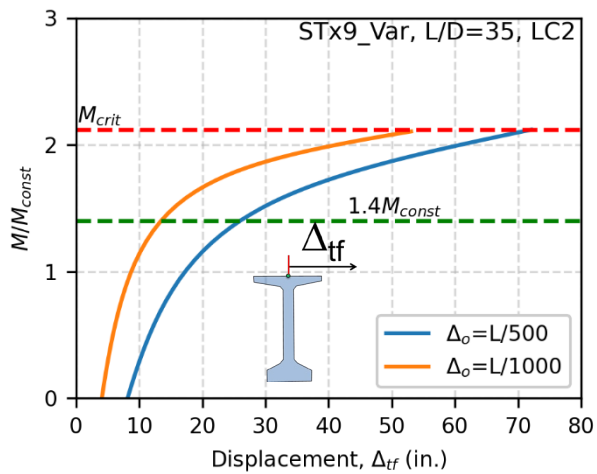
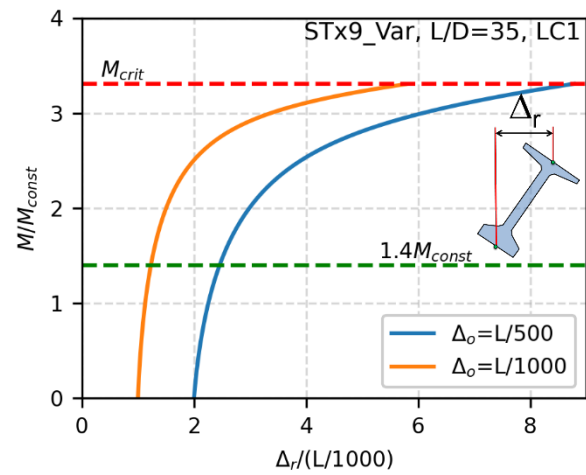
Load case 1



Load case 2



(a) STx9, L/D = 35, S = 12ft, Load case 1



(b) STx9, L/D = 35, S = 12ft, Load case 2

Appendix D: Benchmark FEA Examples

This appendix presents two benchmark cases for design and construction engineers to use when evaluating the stability of spliced girder systems with different software. Abaqus (2023) was used throughout this report for parametric studies. The benchmarks allow for the comparison of different commercial software and modeling schemes. These modeling schemes for I and U-girders may use different finite element analysis (FEA) element types and element orders. This report used solid, “brick”, hexahedral elements with high mesh densities to accurately capture the stability behavior of the girders. Other schemes may use different solid elements, beam elements, shell/plate elements, and/or a hybrid use of multiple element types.

This appendix used two analysis methods in evaluating the stability of the benchmarks:

- i. Linear-elastic eigenvalue buckling analysis by the subspace iteration method. Where the eigenvalue is a multiple of the load(s) applied to the structure that causes the structure to buckle.
- ii. Geometric nonlinear analysis by Riks’ Arc-length method. An imperfection was applied to the system, the load was iteratively increased, and equilibrium on the deformed shape was solved in each step. The large displacement response of the structure at key locations can then be examined for stability behavior.

The Abaqus-specific parameters in Table were used in the benchmark models of the I and U-girder bridges.

Table D-1 Parameters used in Abaqus Modeling

Parameter	Description
Gravity	386 in/s ²
Concrete Mass Density	$2.25 \times 10^{-4} \frac{s^2 lbf}{in in^3}$
Modulus of Elasticity	5,260,000 psi
Poisson's Ratio	0.2
Abaqus Solid Element (C3D8R)	An 8-node linear brick, reduced integration, with hourglass control element

D.1. Benchmark 1: STx9 Three Span I-Girder Bridge

A three-span, constant-depth I-girder bridge using the STx9 girder section and other properties is presented in Figure D.1. Figure D.1(a) displays the layout of the bridge with the span configuration of 204 ft – 291 ft – 204 ft. The segments of the bridge that are spliced to achieve the overall lengths are also denoted. The STx9 girder dimensions are listed in Figure D.1(b) (after TxDOT standard

drawings). The hexahedral FEA meshing scheme and idealized boundary conditions are presented in Figure D.1(c).

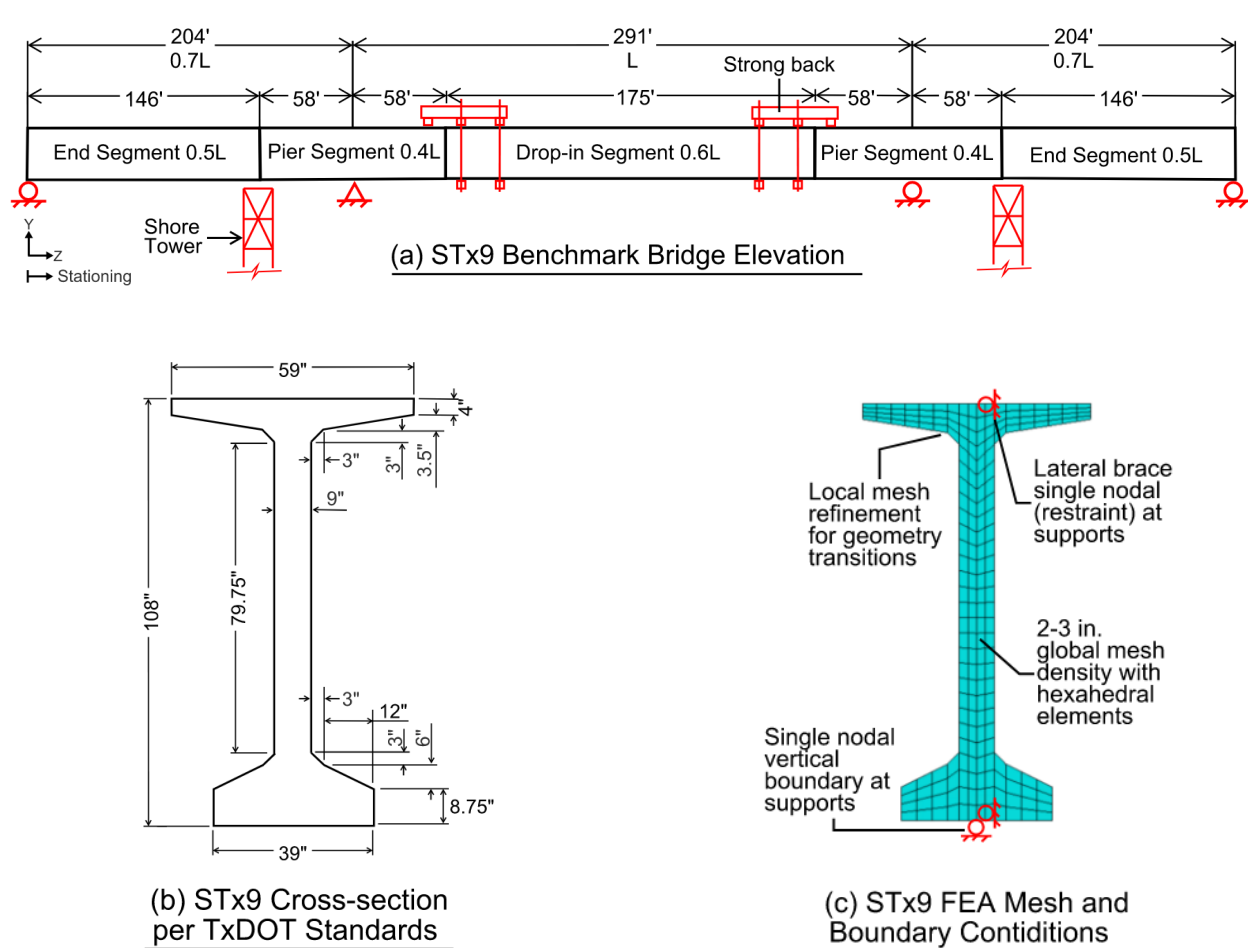


Figure D.1. STx9 benchmark bridge with span arrangement of 204'-291'-204'

D.1.1. Benchmark 1: STx9 Eigenvalue Buckling Analysis During Erection

During the erection and splicing of individual segments, the controlling stability condition is generally not readily apparent. Therefore, five different load cases were investigated to assess the stability of the components as the structure is being built. Many of these cases also include the load from the drop-in portion of the main span as it is briefly supported by temporary works (strong backs). The strong backs are supported by the previously erected end and pier segments. The results of these analyses are shown in Figure D.2., and the controlling load case was Load Case 1 – self-weight of the end segment and pier segment with the shoring tower removed. All load cases analyzed had eigenvalues much greater than 1.0, indicating stable behavior when initial imperfections are not considered.

STx9: $L/D = 30$, $L = 291$ ft, D includes an 8.5 in. slab

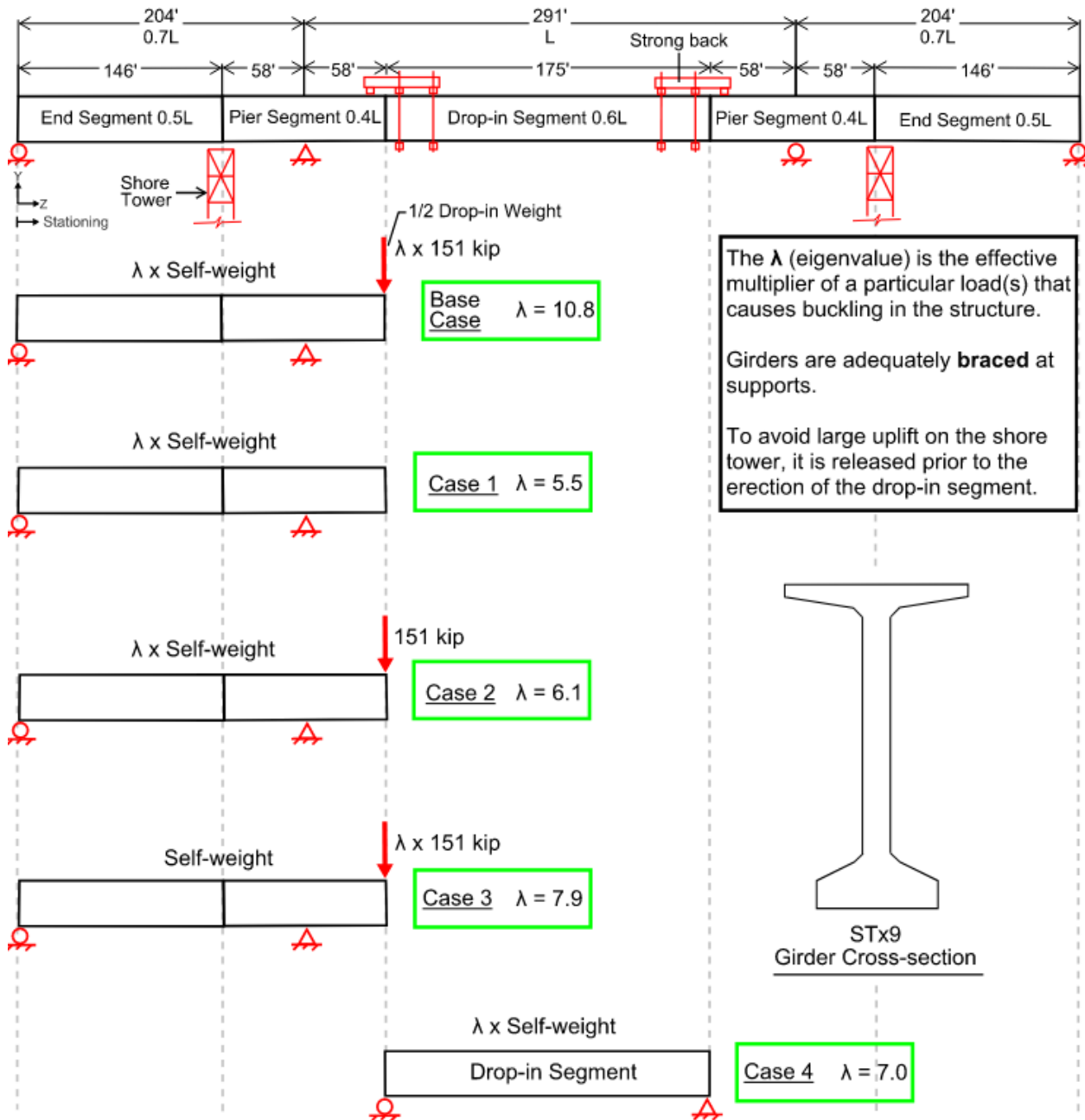
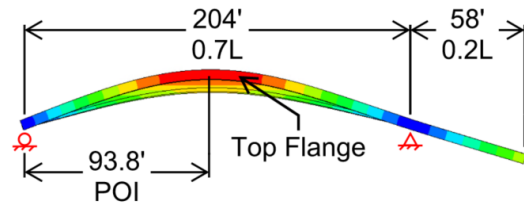


Figure D.2. STx9 eigenvalue buckling analysis during erection

D.1.2. Benchmark 1: STx9 Stability Considering Initial Imperfections During Erection

An eigenvalue buckling analysis should always be performed prior to a geometric nonlinear analysis, which considers imperfections for stability analysis. This is recommended for two reasons: (1) The eigenvalue buckling analysis provides a load magnitude where the system buckles. (2) The eigenvalue analysis produces the buckling mode shape (eigenvector). The eigenvector mode shape produces relative magnitudes of the structural displacements when buckling occurs. These relative displacements represent the movement of the structure as it buckles. The eigenvector also aids in the understanding of where initial imperfections in the system will have a destabilizing effect.

The critical load case for Benchmark 1 was Load Case 1. The controlling eigenvalue buckling analysis is the load case used for geometric nonlinear analysis. For Load Case 1, it is not easily recognized where the distribution of imperfections would cause the most destabilizing effects because of the simply supported girder with cantilever end layout. Therefore, the eigenvector mode shape in Figure D.3 was used. This mode shape was scaled by the imperfections of $L/500$ and $L/1000$ for analysis.



(a) STx9 Plan view of buckling mode

$$L_{\text{end_span}}/500 = 204'/500 = 0.41' = 4.9"$$

$$L_{\text{end_span}}/1000 = 204'/1000 = 0.20' = 2.4"$$

(b) Initial imperfection values of the end span

Figure D.3. STx9 end span mode shape and initial imperfection magnitudes

The load-displacement curves at the point of interest (POI) are displayed in Figure D.4. The load-displacement figures normalize the bending moment in the vertical axis with the expected construction load (service load), M_{const} . For Load Case 1, this includes only the girder self-weight. The horizontal axis plots the absolute lateral displacement (total sway) of the top flange during incremental loading at the point of interest – listed as Δ_{tf} . Two additional reference lines are included in the plots. The first is M_{crit} , which is the critical buckling moment from eigenvalue buckling analysis. The second is $1.4M_{\text{const}}$, which includes the dead load component (DC) load factor recommended in this report.

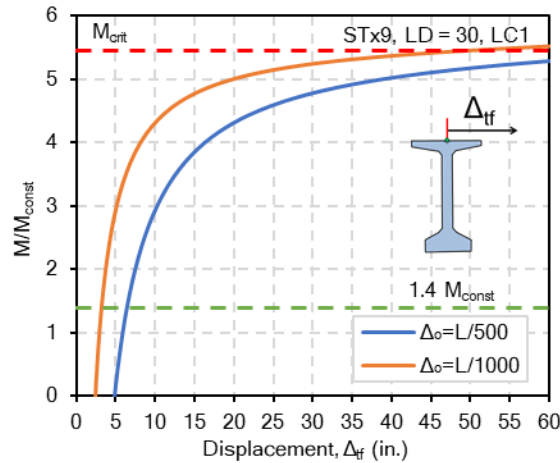
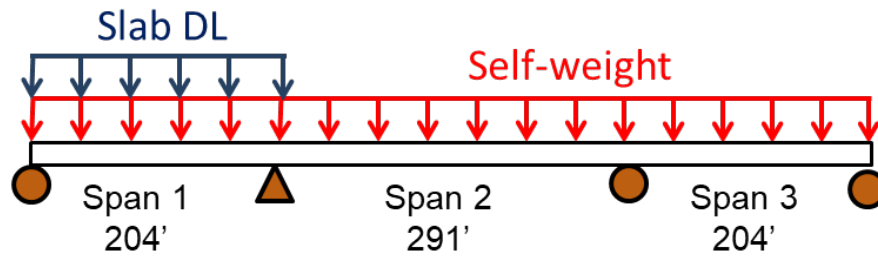


Figure D.4. Load-displacement curves for STx9, LD = 30 during erection for Load Case 1

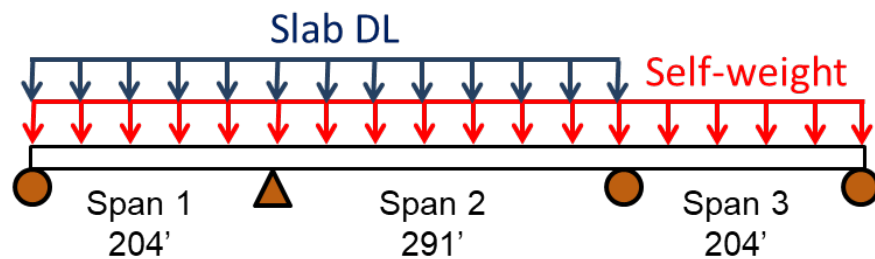
It is important to note that both load-displacement curves in Figure D.4, including initial imperfections, reach the eigenvalue theoretical buckling load of 5.5 multiplied by the self-weight of the girder for Load Case 1. However, both cases of imperfections would undergo lateral displacements greater than 40 in. to reach that capacity, presenting constructability issues. Compared to the expected service load of $M/M_{const} = 1.0$, the L/1000 and L/500 cases increase in lateral deflections by 0.55 in. and 1.0 in., respectively. Benchmark 1 only has idealized bracing at supports (lateral only and no warping). It is important for design engineers, construction engineers, and contractors to evaluate both the buckling strength of these systems and the constructability/serviceability of the structures for temporary and final conditions.

D.1.3. Benchmark 1: STx9 Eigenvalue Buckling Analysis During Deck Placement

Two load cases are considered during the placement of a 12 ft tributary width deck for Benchmark 1. The girders are braced laterally at support points and do not consider any internal diaphragms within the spans. The load cases considered represent the critical conditions for the maximum possible positive and negative moments. Load Case 1 occurs under the self-weight of the fully erected girder line, with Span 1 supporting the wet concrete deck placement load. Load Case 2 occurs under the self-weight of the fully erected girder line, and wet concrete load is applied to Spans 1 and 2. The implementation of loading has been treated differently for each component. The girder self-weight is applied as a gravity load, while the slab dead load is applied as a uniform pressure over the top flange of the girder. The wet concrete load assumes an 8.5 in. slab with a concrete unit weight of 150 lb/ft³ over the tributary girder spacing of 12 ft. The load cases are shown in Figure D.5 and the application of loads in Figure D.6.



(a) Deck placement Load Case 1



(b) Deck placement Load Case 2

Figure D.5. Load cases under construction during deck placement

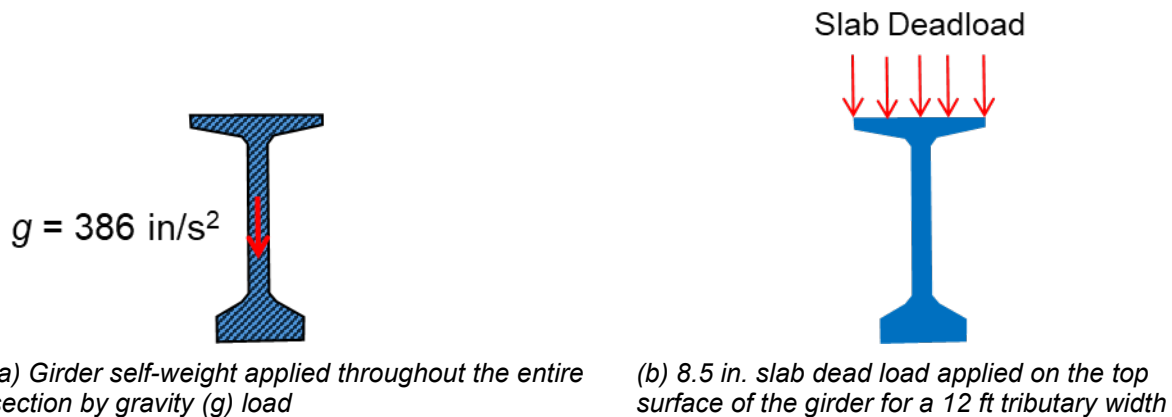


Figure D.6. Application of loads during deck placement

The eigenvalue buckling analysis for the two load cases is shown in Figure D.7. The eigenvalue (λ) is the multiple of both girder self-weight and wet concrete load that causes the system to buckle. Load Case 2 had the lowest eigenvalue of 2.5 and, therefore, is the critical load case.

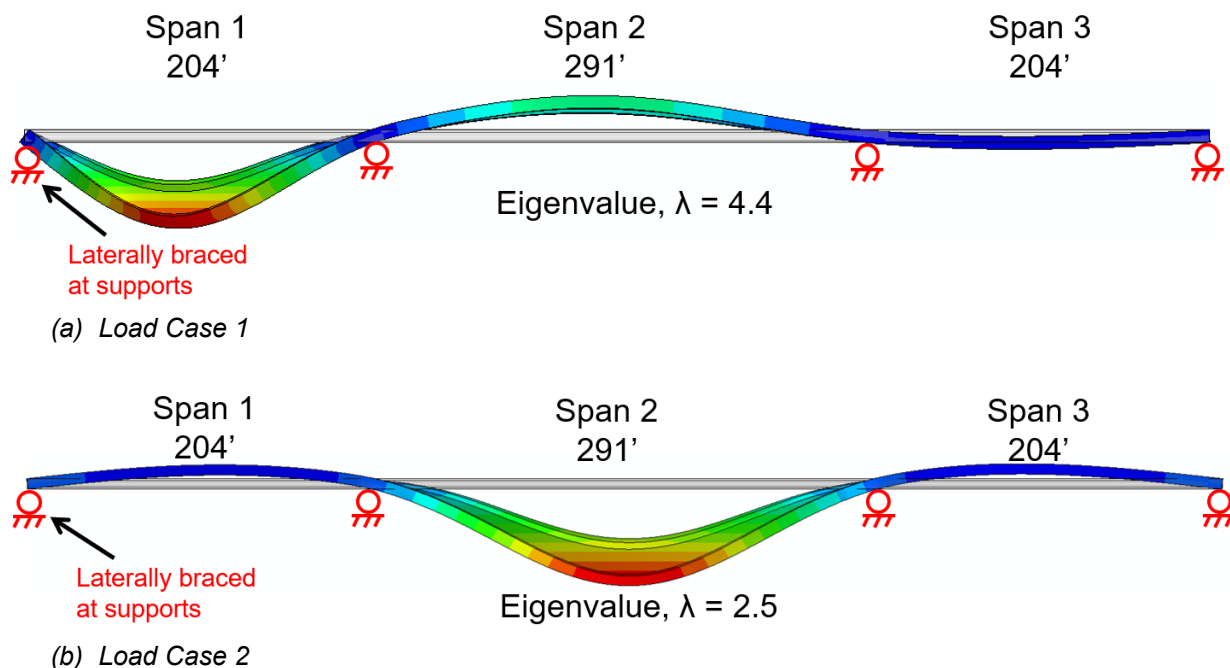


Figure D.7. Plan view of buckling modes and eigenvalue analysis

D.1.4. Benchmark 1, STx9 Stability Considering Initial Imperfections During Deck Placement

The load cases during deck placement for the eigenvalue buckling analysis are used in the geometric nonlinear analysis, considering initial imperfections. The imperfections along the girder line are applied as shown in Figure D.8. The imperfection twists about the middle of the bottom flange and is fitted across the girder line using a sine function.

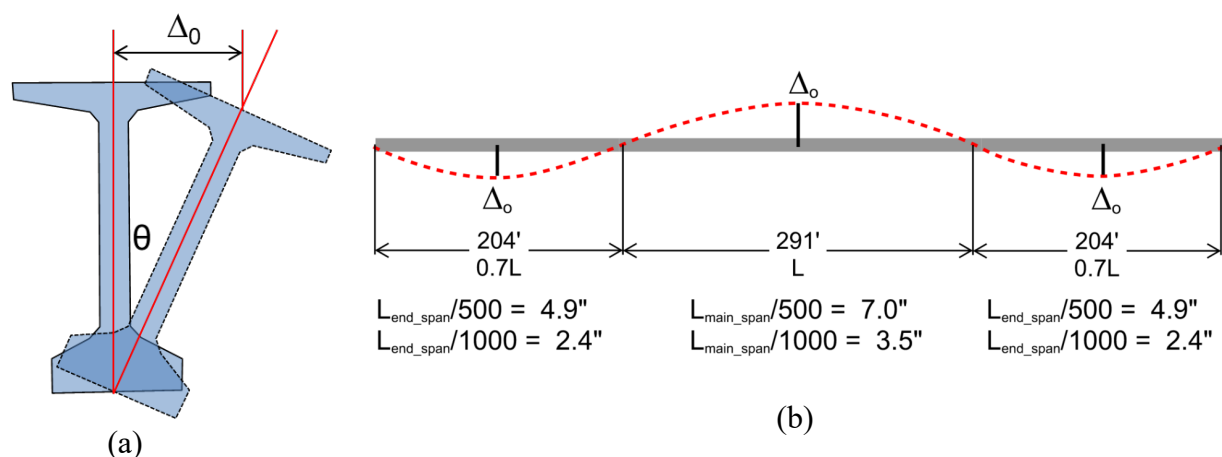


Figure D.8. (a) Cross-sectional imperfection shape and (b) plan view of imperfection distribution along the length of the girder

The results of the geometric nonlinear analysis for both load cases during deck placement are shown in Figure D.9. The point of interest for Load Case 1 is in Span 1, and the point of interest for Load Case 2 occurs in Span 2.

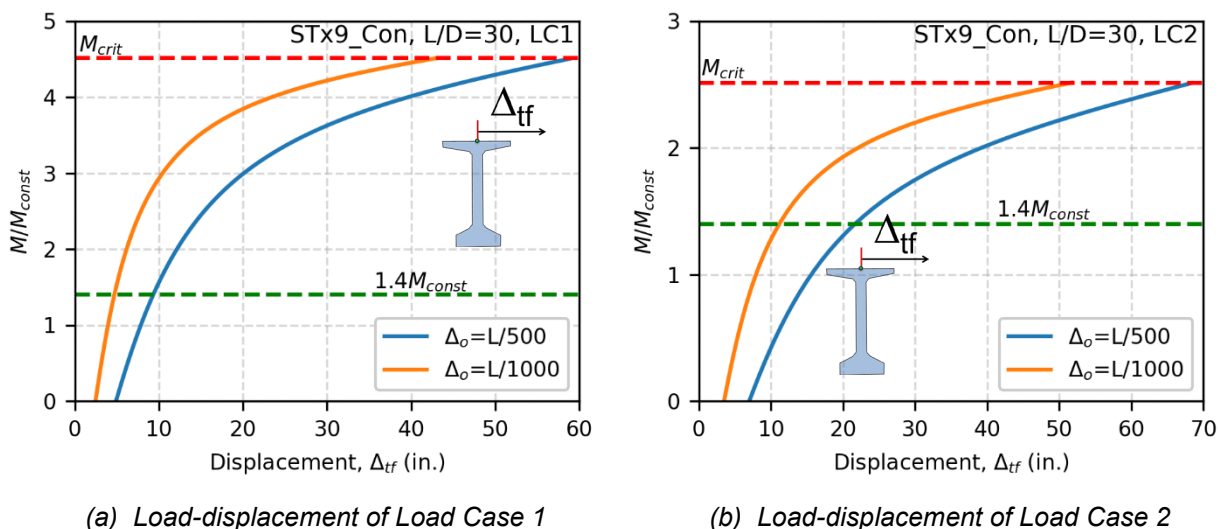


Figure D.9. Load-displacement curves for STx9, LD = 30 during deck placement for Load Cases 1 and 2.

Similar to the erection load case, both load cases in the load-displacement curves in Figure D.9, including initial imperfections, reach the eigenvalue theoretical buckling load. Where Load Case 1 had a theoretical eigenvalue of 4.4 multiplied by the girder and wet concrete load, and Load Case 2 had a theoretical eigenvalue of 2.5 multiplied by the girder and wet concrete load. However, both cases of imperfections would undergo lateral displacements greater than 40 in. to reach those capacities, presenting constructability issues. For the controlling load case, Load Case 2, the expected lateral displacement for the loading of $M/M_{const} = 1.0$, the L/1000 and L/500 cases increase in lateral deflections by 4.3 in. and 8.9 in., respectively. Benchmark 1 only has idealized bracing at supports (lateral only and no warping). It is important for design engineers, construction engineers, and contractors to evaluate both the buckling strength of these systems and the constructability/serviceability of the structures for temporary and final conditions.

D.1.5. Benchmark 1, STx9 Stability During Construction Summary

A summary of Benchmark 1 analyses is presented in Table D-2. The eigenvalue (λ) shown in the table is the multiplier applied to the load(s) that cause the system to buckle. Table D-2 also includes general properties for design. The recommended dead load component load factor (γ_{DC}) is 1.4. Additionally, a reduction factor (R) related to cracked sections and the impact on the system's stability capacity is recommended: Where the cracking is limited to the thickness of the flanges in positive and negative moment regions, use a capacity reduction factor of $R_{cr,flanges} = 0.80$. Ideally, there should not be any cracking in the prestress segments during construction; however, because

of the decrease in system stiffness and buckling capacity that occurs when cracking is present, using the reduction factor is prudent. An example calculation for design purposes, Example Calculation D.1, is provided for Load Case 2 during deck placement.

Example Calculation D.1: Benchmark 1 Load Case 2 during deck placement

Demand: $M_u = \gamma_{DC} M_{const} = 1.4 M_{const}$

Capacity: $\phi M_n = \phi R_{cr,flanges} \lambda M_{const} = 1.0 \cdot 0.80 \cdot 2.5 \cdot M_{const} = 2.0 M_{const}$

Demand to Capacity Ratio: $\frac{M_u}{\phi M_n} = 0.7 < 1, \therefore \text{Adequate Capacity}$

Table D-2 Benchmark 1 Summary Table

	Benchmark 1 Item	Result	Description
Stability During Erection	Base Load Case	$\lambda = 10.8$	
	Load Case 1	$\lambda = 5.5$	
	Load Case 2	$\lambda = 6.1$	
	Load Case 3	$\lambda = 7.9$	
	Load Case 4	$\lambda = 7.0$	
	Load Case 1 Additional Lateral Deflection	L/1000: 0.55 in. L/500: 1.0 in.	Expected additional lateral deflection during erection.
Stability During Deck Placement	Load Case 1	$\lambda = 4.4$	
	Load Case 2	$\lambda = 2.5$	
	Load Case 2 Additional Lateral Deflection	L/1000: 4.3 in. L/500: 8.9 in.	Expected additional lateral deflection during deck placement.
General Properties for Design	γ_{DC}	1.4	Recommended dead load component load factor during construction
	$R_{cr,flanges}$	0.8	Stability capacity reduction factor for cases where cracking is expected in positive and negative moment areas through the depth of the flanges.

D.2. Benchmark 2, Tx-U96-9 Three Span U-Girder Bridge

A three-span, constant-depth U-girder bridge using the Tx-U96-9 girder section and other properties is presented in Figure D.10. Figure D.10(a) displays the layout of the bridge with the span configuration of 183 ft – 261 ft – 183 ft. The segments of the bridge that are spliced to achieve the overall lengths are also denoted. The Tx-U96-9 girder dimensions are listed in Figure D.10(b) (after TxDOT standard drawings). The hexahedral FEA meshing scheme and idealized boundary conditions are presented in Figure D.10(c).

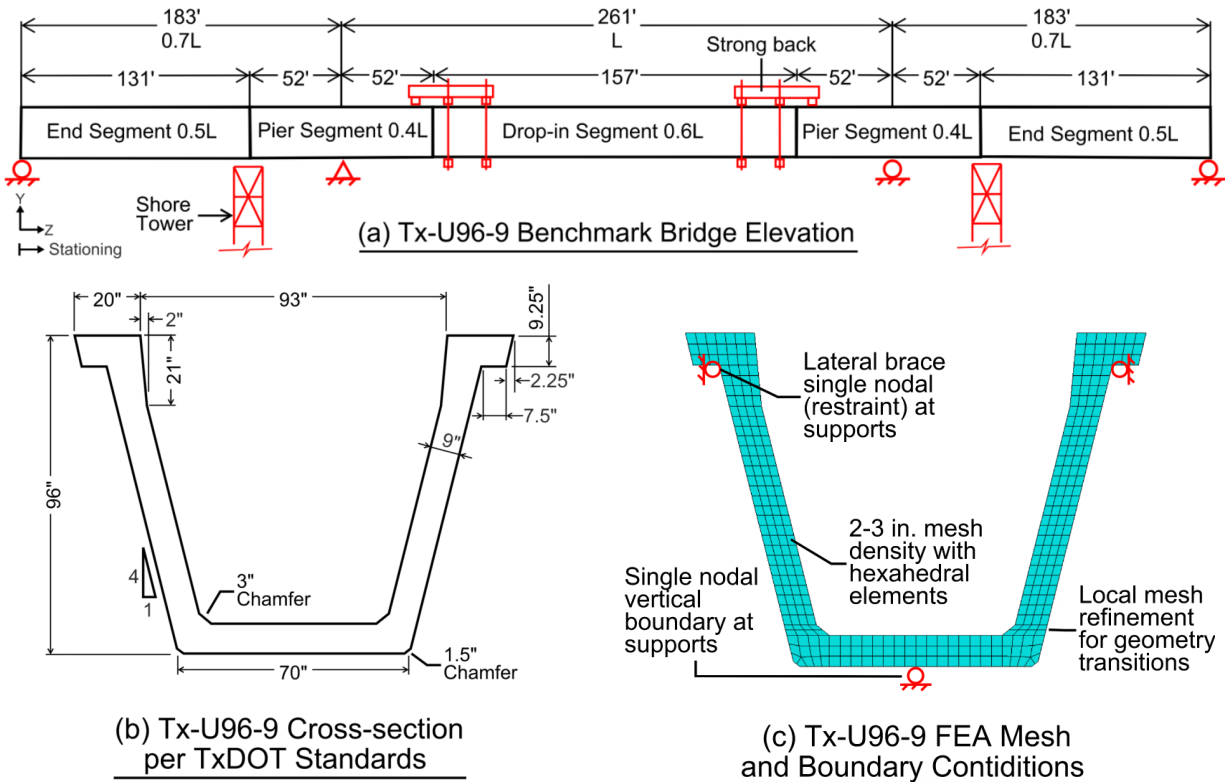


Figure D.10. Tx-U96-9 benchmark bridge with span arrangement of 183'-261'-183"

D.2.1. Benchmark 2, Tx-U96-9 Eigenvalue Buckling Analysis During Erection

Similar to Benchmark 1, during the erection and splicing of individual segments, the controlling stability condition is generally not readily apparent. Therefore, five different load cases were investigated to assess the stability of the components as the structure is being built. Many of these cases also include the load from the drop-in portion of the main span as it is temporarily supported by temporary works (strong backs). The strong backs are supported by the previously erected end and pier segments. The results of these analyses are shown in Figure D.11., and the controlling load case was Load Case 1 – self-weight of the end segment and pier segment with the shoring

tower removed. All load cases analyzed had eigenvalues much greater than 1.0, indicating stable behavior when initial imperfections are not considered.

Tx-U96-9: $L/D = 30$, $L = 261$ ft, D includes a 8.5 in. slab

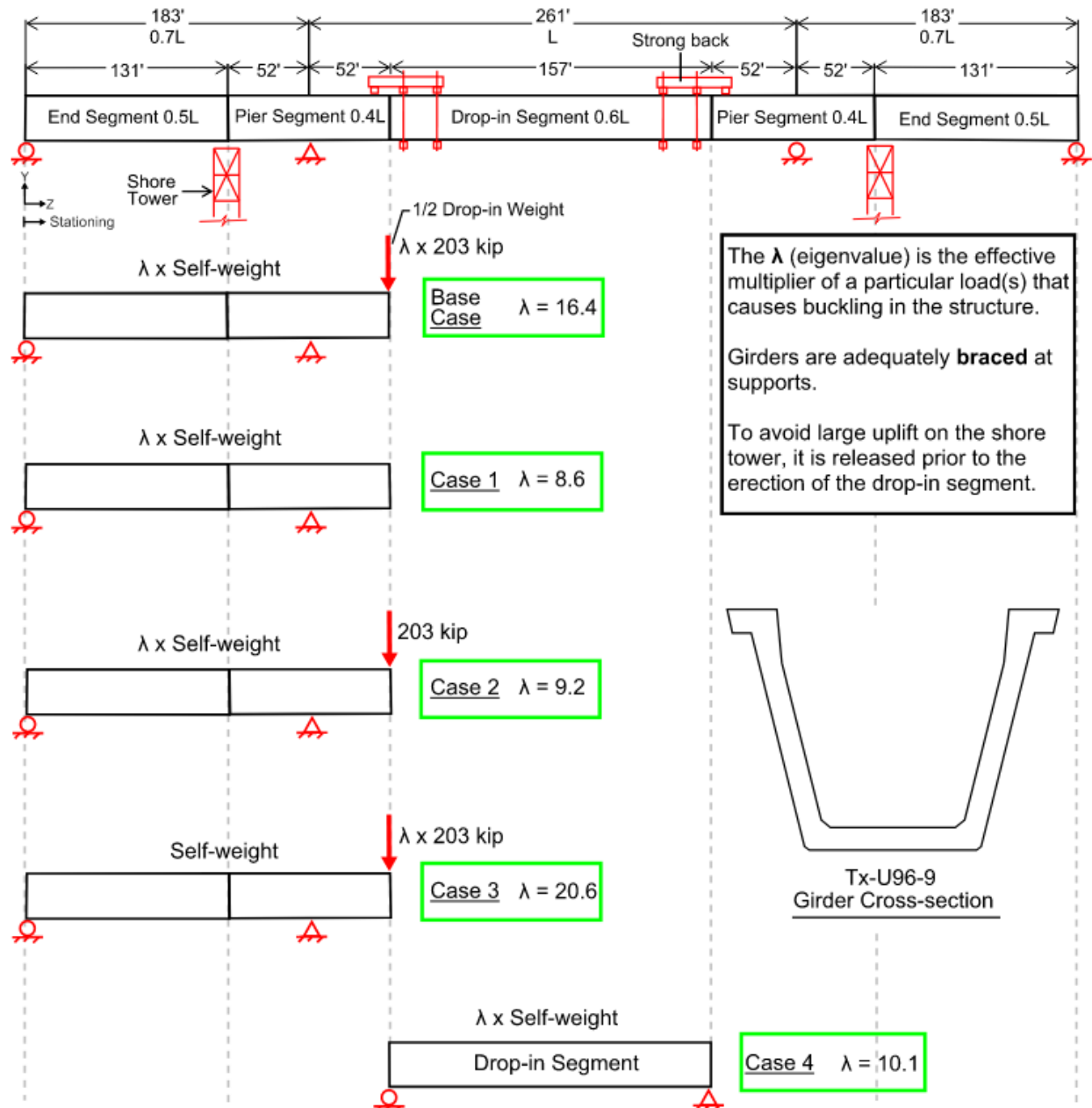


Figure D.11. Tx-U96-9 eigenvalue buckling analysis during erection

D.2.2. Benchmark 2, Tx-U96-9 Stability Considering Initial Imperfections During Erection

The critical load case for Benchmark 2 was Load Case 1. The controlling eigenvalue buckling analysis is the load case used for geometric nonlinear analysis. For Load Case 1, it is not easily recognized where the distribution of imperfections would cause the most destabilizing effects because of the simply supported girder with cantilever end layout. Therefore, the eigenvector mode shape in Figure D.12 was used. This mode shape was scaled by the imperfections of $L/500$ and $L/1000$ for analysis.

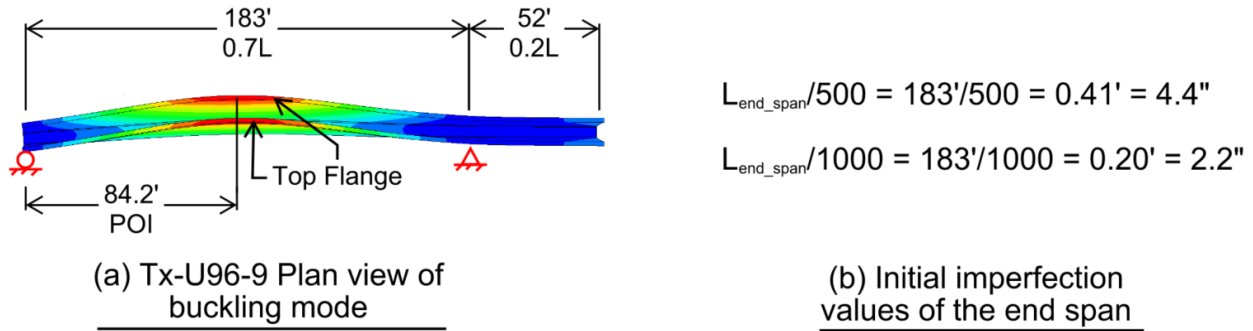
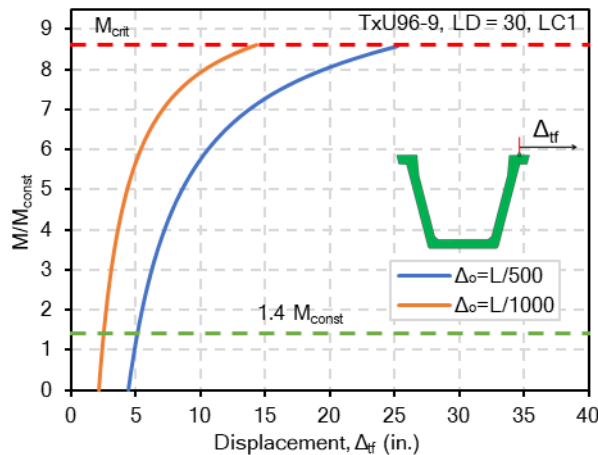


Figure D.12. Tx-U96-9 end span mode shape and initial imperfection magnitudes

The load-displacement curves at the point of interest (POI) are displayed in Figure D.13. The load-displacement figures normalize the bending moment in the vertical axis with the expected construction load (service load), M_{const} . For Load Case 1, only girder self-weight was applied. The horizontal axis plots the absolute lateral displacement (total sway) of the top flange during incremental loading at the point of interest – listed as Δ_{tf} . Two additional reference lines are included in the plots. The first is M_{crit} , which is the critical buckling moment from eigenvalue buckling analysis. The second is $1.4 M_{\text{const}}$, which includes the dead load component (DC) load factor recommended in this report.



Both load-displacement curves in Figure D.13, including initial imperfections, reach the eigenvalue theoretical buckling load of 8.6 multiplied by the self-weight of the girder for Load Case 1. However, both cases of imperfections would undergo lateral displacements greater than 15 in. to reach that capacity. Compared to the expected service load of $M/M_{const} = 1.0$, the L/1000 and L/500 cases increase in lateral deflections by 0.25 in. and 0.48 in., respectively. Benchmark 2 only has idealized bracing at supports (lateral only and no warping). Similar to Benchmark 1, design engineers, construction engineers, and contractors need to evaluate both the buckling strength of these systems and the constructability/serviceability of the structures for temporary and final conditions.

D.2.3. Benchmark 2, Tx-U96-9 Eigenvalue Buckling Analysis During Deck Placement

Two load cases are considered when placing a 22 ft 6 in. tributary width deck for Benchmark 2. The girders are braced laterally at support points and do not consider any internal diaphragms within the spans. The load cases considered represent the critical conditions for the maximum possible positive and negative moments. Load Case 1 occurs under the self-weight of the fully erected girder line, with Span 1 supporting the wet concrete deck placement load. Load Case 2 occurs under the self-weight of the fully erected girder line, and wet concrete load is applied to Spans 1 and 2. The implementation of loading has been treated differently for each component. The girder self-weight is applied as a gravity load, while the slab dead load is applied as a uniform pressure over the top flange of the girder. The wet concrete load consists of an 8.5 in. slab with a concrete unit weight of 150 lb/ft³ over the tributary girder spacing of 22 ft 6 in. The load cases are shown in Figure D.14 and the application of loads in Figure D.15.

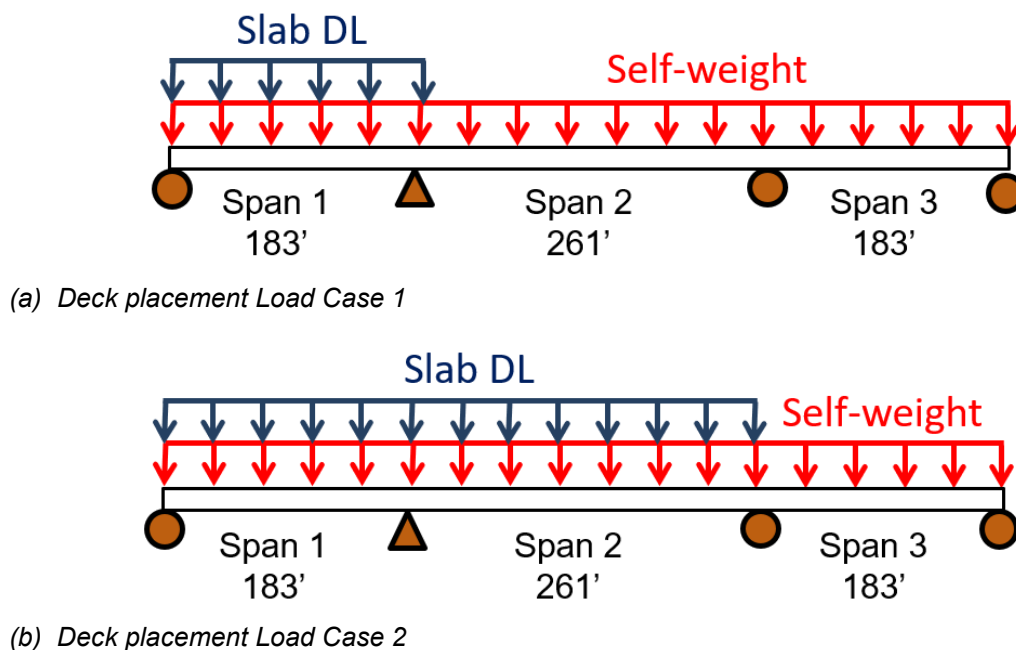


Figure D.14. Load cases under construction during deck placement

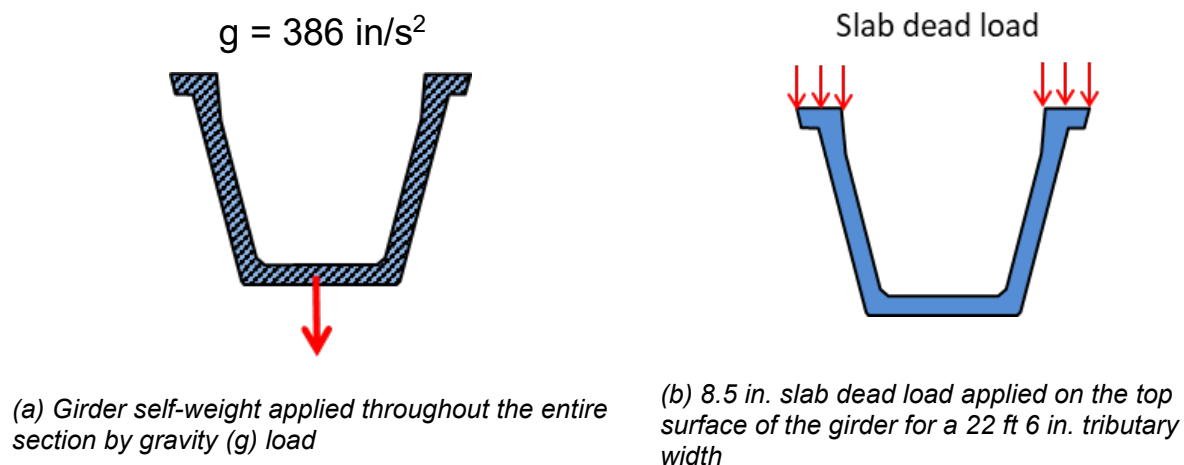


Figure D.15. Application of loads during deck placement

The eigenvalue buckling analysis for the two load cases is shown in D.16. The eigenvalue (λ) is the multiple of both girder self-weight and wet concrete load that causes the system to buckle. Load Case 2 had the lowest eigenvalue of 3.2 and, therefore, is the critical load case.

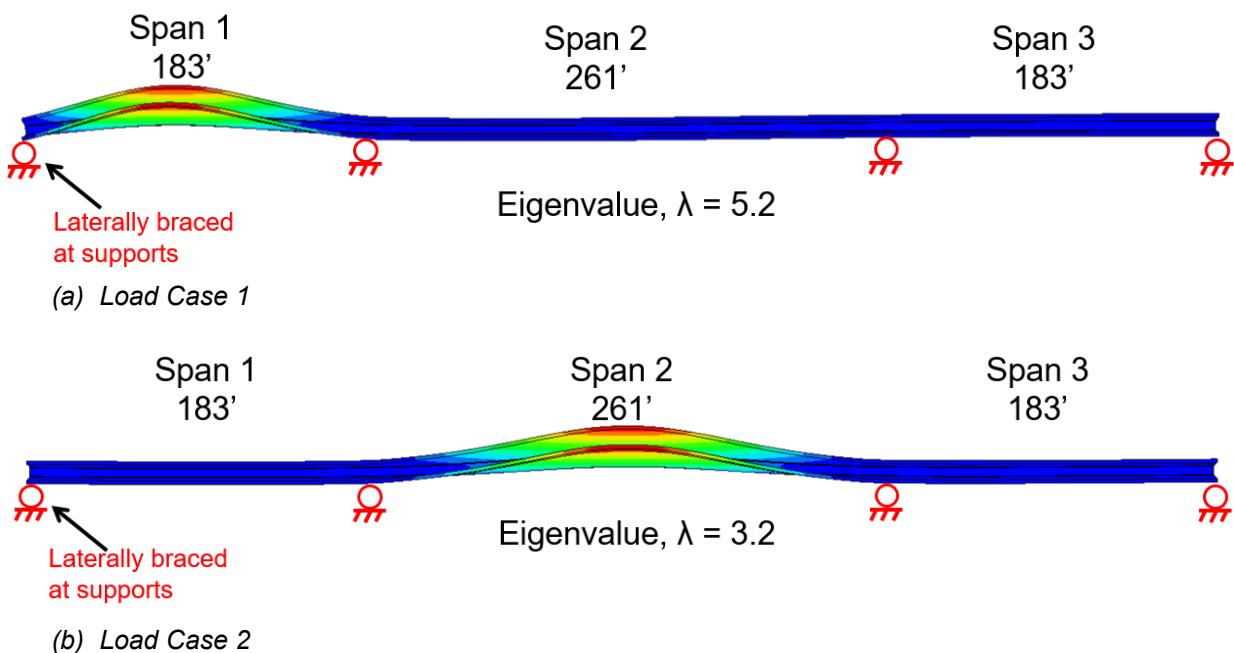


Figure D.16. Plan view of buckling modes and eigenvalue analysis

D.2.4. Benchmark 2, Tx-U96-9 Stability Considering Initial Imperfections During Deck Placement

The load cases during deck placement for the eigenvalue buckling analysis are used in the geometric nonlinear analysis, considering initial imperfections. The imperfections along the girder line are applied as shown in Figure D.17. The imperfection twists about the middle of the bottom flange and is fitted across the girder line using a sine function.

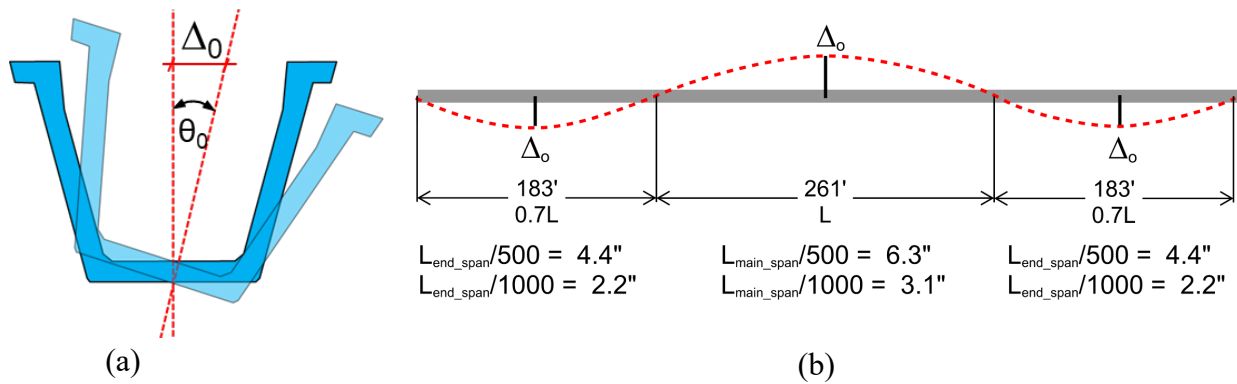


Figure D.17. (a) Cross-sectional imperfection shape and (b) plan view of imperfection distribution along the length of the girder

The results of the geometric nonlinear analysis for both load cases during deck placement are shown in Figure D.18. The point of interest for Load Case 1 is in Span 1, and the point of interest for Load Case 2 occurs in Span 2.

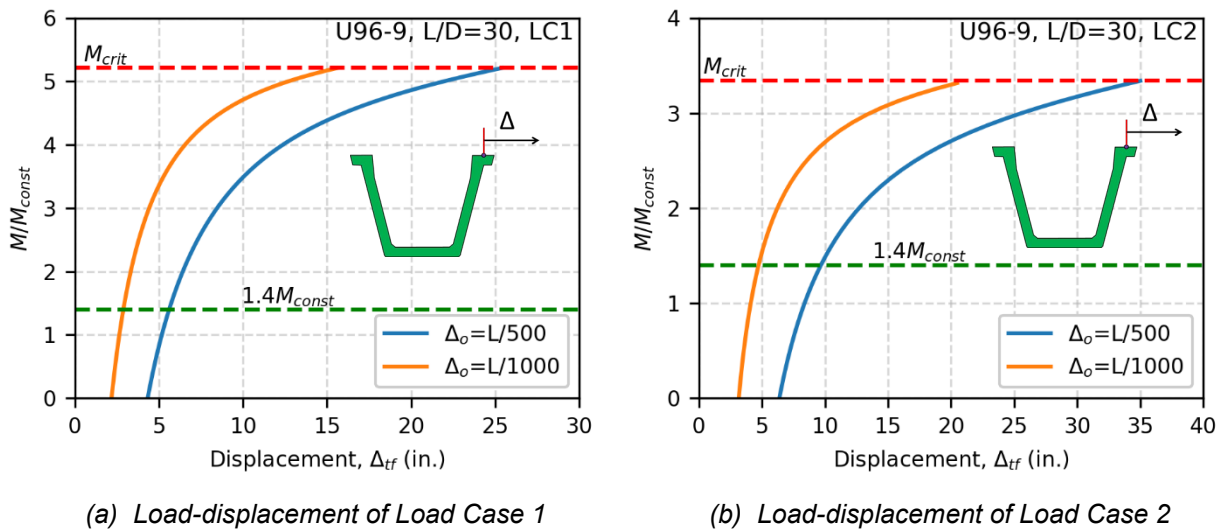


Figure D.18. Load-displacement curves for Tx-U96-9, L/D = 30 during deck placement for Load Cases 1 and 2.

Both load cases in the load-displacement curves in Figure D.18, including initial imperfections, reach the eigenvalue theoretical buckling load. Where Load Case 1 had a theoretical eigenvalue of 5.2 multiplied by the girder and wet concrete load, and Load Case 2 had a theoretical eigenvalue of 3.2 multiplied by the girder and wet concrete load. However, both cases of imperfections would undergo lateral displacements greater than 15 in. to reach those capacities, presenting constructability issues. For the controlling load case, Load Case 2, the expected lateral displacement for the loading of $M/M_{const} = 1.0$, the L/1000 and L/500 cases increase in lateral deflections by 1.1 in. and 1.9 in., respectively. Benchmark 2 only has idealized bracing at supports (lateral only and no warping). These magnitudes are relatively small, but both the buckling strength of these systems and the constructability/serviceability of the structures for temporary and final conditions need to be considered.

D.2.5. Benchmark 2, Tx-U96-9 Stability During Construction Summary

A summary of Benchmark 2 analyses is presented in Table D-3. The eigenvalue (λ) shown in the table is the multiplier applied to the load(s) that causes the system to buckle. Table D-3 also includes general properties for design. The recommended dead load component load factor (γ_{DC}) is 1.4. Additionally, a capacity reduction factor (R) related to cracked sections and the impact on the system's stability capacity is recommended: Where the cracking is limited to the thickness of the flanges in positive and negative moment regions, use a capacity reduction factor of $R_{cr,flanges} = 0.80$. Ideally, there should not be any cracking in the prestress segments during construction; however, because of the decrease in system stiffness and buckling capacity that occurs when cracking is present, using the reduction factors is prudent. An example calculation for design purposes, Example Calculation D.2, is provided for Load Case 2 during deck placement.

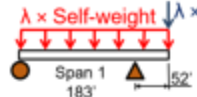
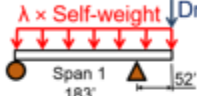
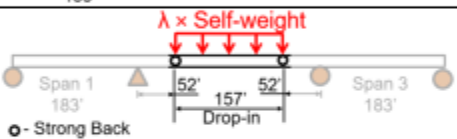
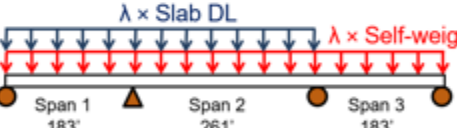
Example Calculation D.2: Benchmark 2 Load Case 2 during deck placement

Demand: $M_u = \gamma_{DC} M_{const} = 1.4 M_{const}$

Capacity: $\phi M_n = \phi R_{cr,flanges} \lambda M_{const} = 1.0 \cdot 0.80 \cdot 3.2 \cdot M_{const} = 2.6 M_{const}$

Demand to Capacity Ratio: $\frac{M_u}{\phi M_n} = 0.5 < 1, \therefore \text{Adequate Capacity}$

Table D-3 Benchmark 2 Summary Table

	Benchmark 2 Item	Result	Description
Stability During Erection	Base Load Case	$\lambda = 16.4$	
	Load Case 1	$\lambda = 8.6$	
	Load Case 2	$\lambda = 9.2$	
	Load Case 3	$\lambda = 20.6$	
	Load Case 4	$\lambda = 10.1$	
	Load Case 1 Additional Lateral Deflection	L/1000: 0.3 in. L/500: 0.5 in.	Expected additional lateral deflection during erection - service loads.
Stability During Deck Placement	Load Case 1	$\lambda = 5.2$	
	Load Case 2	$\lambda = 3.2$	
	Load Case 2 Additional Lateral Deflection	L/1000: 1.1 in. L/500: 1.9 in.	Expected additional lateral deflection during deck placement - service loads.
General Properties for Design	γ_{DC}	1.4	Recommended dead load component load factor during construction.
	$R_{cr,flanges}$	0.8	Stability capacity reduction factor for cases where cracking is expected in positive and negative moment areas through the depth of the flanges.

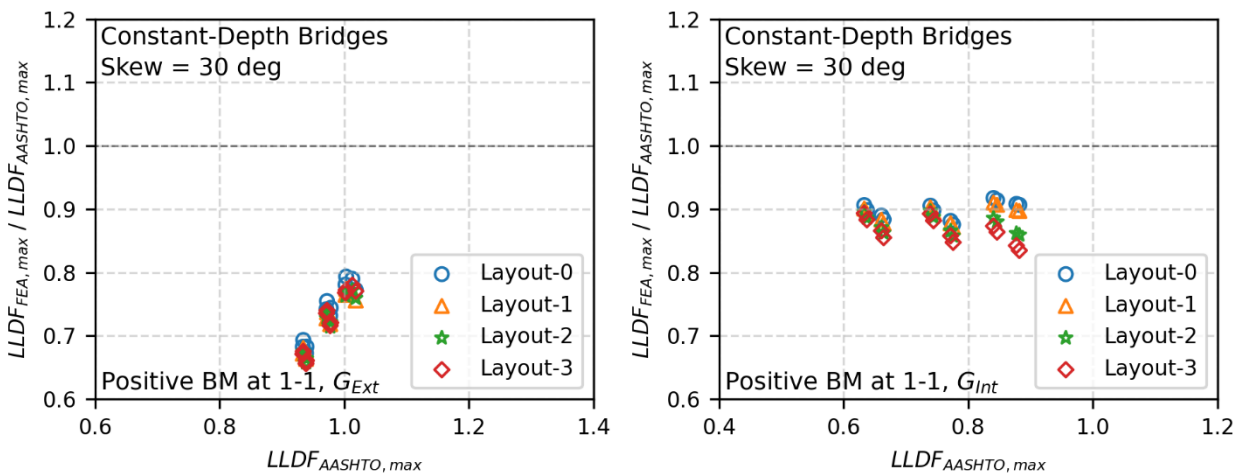
Appendix E: Comparison of AASHTO and FEA LLDFs

The discussion for live load distribution in Chapter 6 was limited to a few cases for brevity. The results of other cases are presented in this appendix.

E.1. Comparison of AASHTO and FEA LLDFs

In Chapter 6, the comparison of LLDFs from AASHTO and FEA was provided only for constant-depth bridges with normal supports and for variable-depth bridges with 30-degree skew supports. This appendix section presents the comparison for other cases.

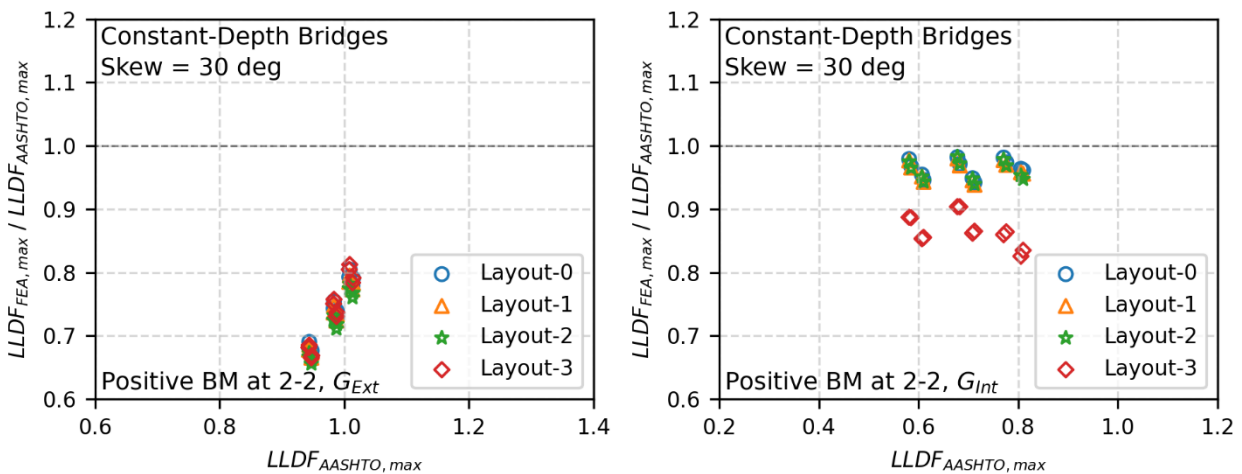
E.1.1 Constant-Depth Bridges with 30-degree Skew



a) exterior girder

b) interior girder

Figure E.1. Comparison of maximum LLDFs from FEA and AASHTO for positive bending moment at section 1-1 in 30-degree skew constant-depth bridges



a) exterior girder

b) interior girder

Figure E.2. Comparison of LLDFs from FEA and AASHTO for positive bending moment at section 2-2 in 30-degree skew constant-depth bridges

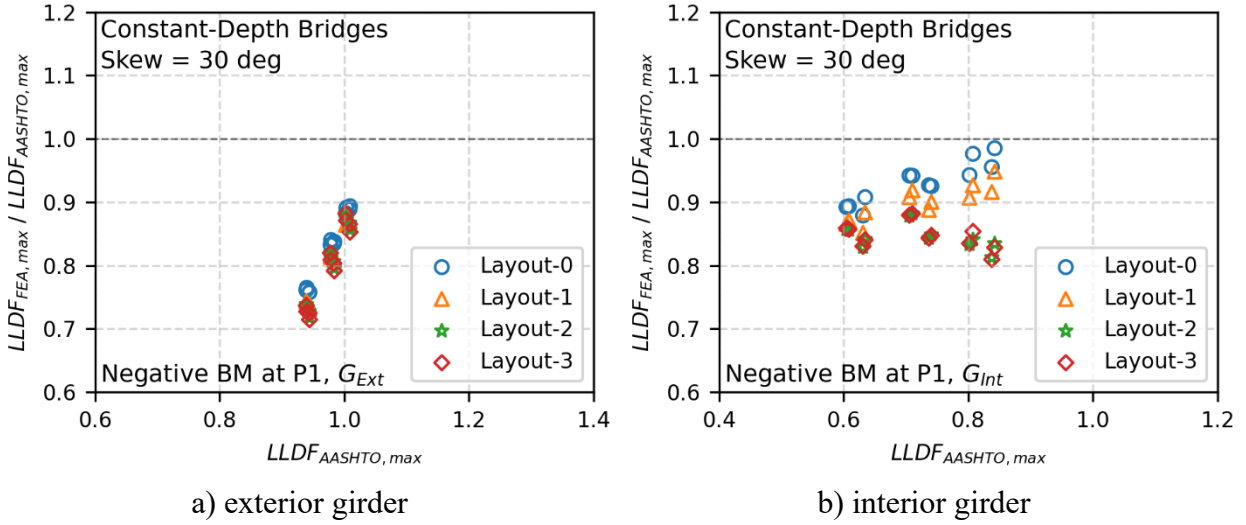


Figure E.3. Comparison of maximum LLDFs from FEA and AASHTO for negative bending moment at support P1 in 30-degree skew constant-depth bridges

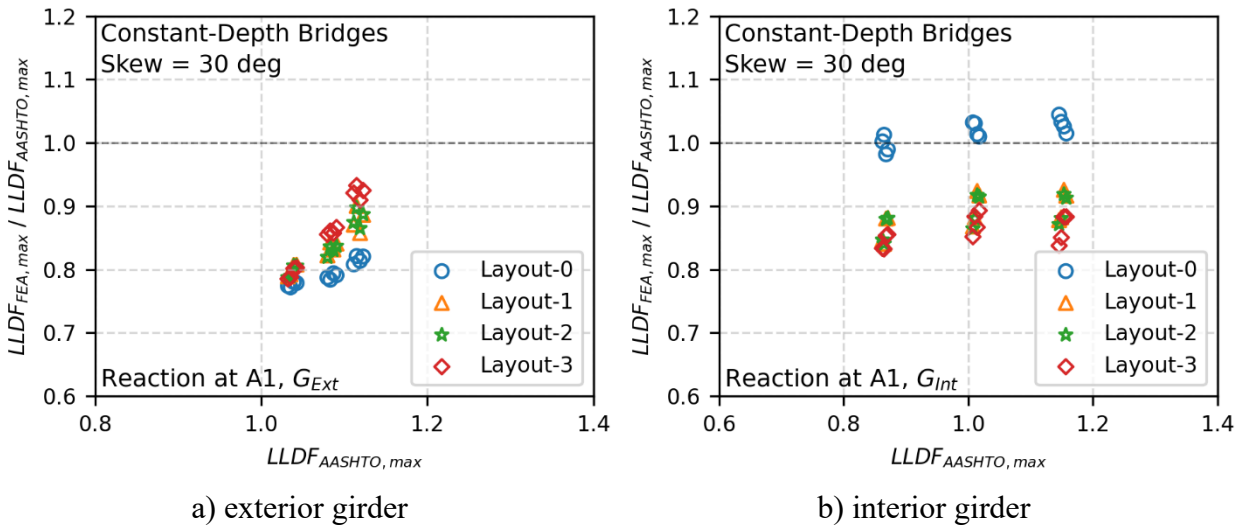
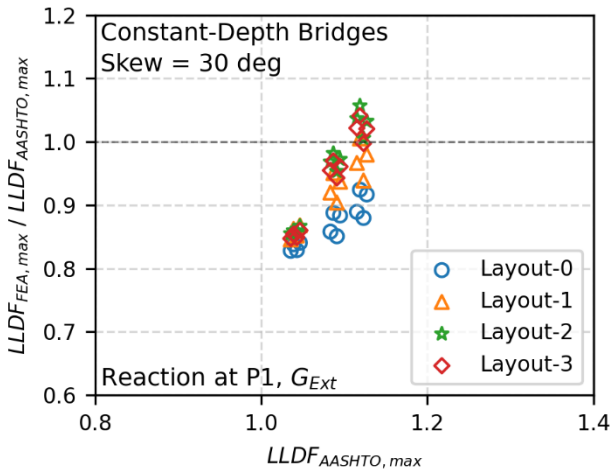
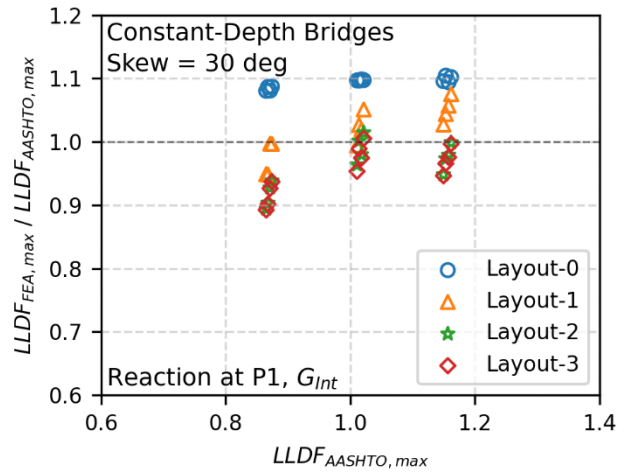


Figure E.4. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at end support A1 in 30-degree skew variable-depth bridges



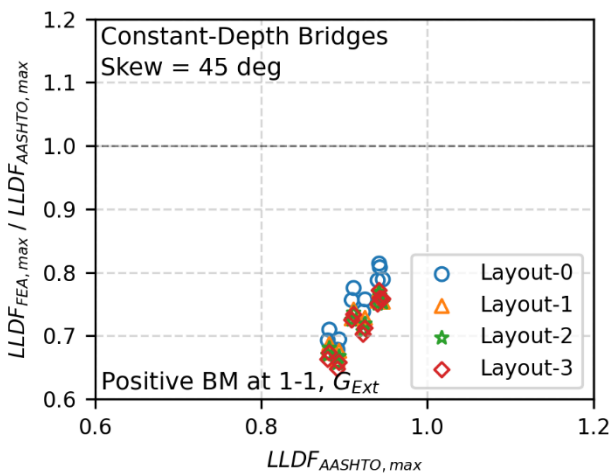
a) exterior girder



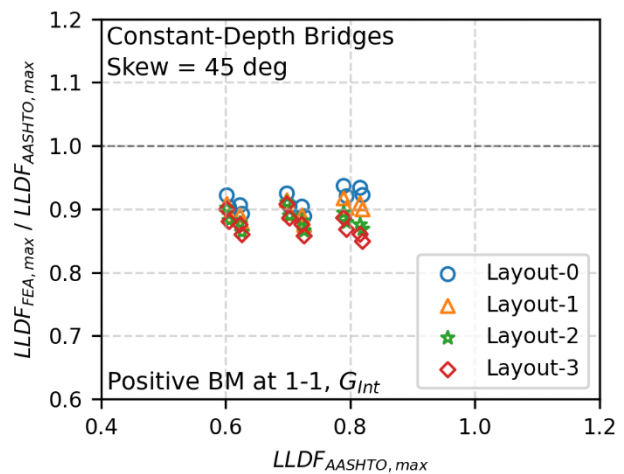
b) interior girder

Figure E.5. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at interior support P1 in 30-degree skew constant-depth bridges

E.1.2. Constant-Depth Bridges with 45-degree Skew

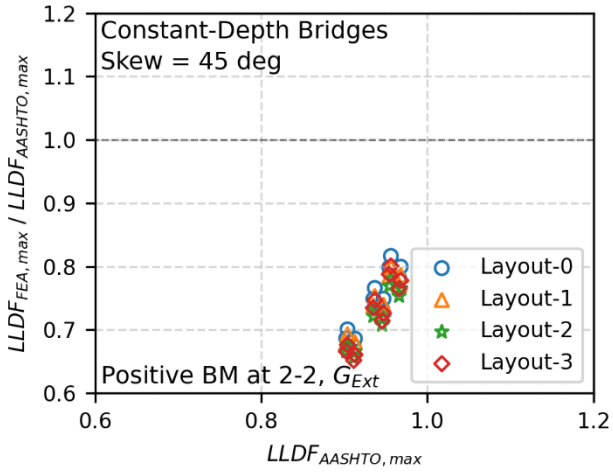


a) exterior girder

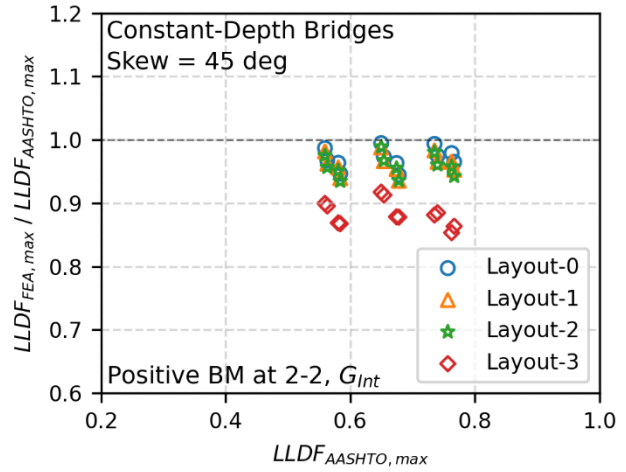


b) interior girder

Figure E.6. Comparison of maximum LLDFs from FEA and AASHTO for positive bending moment at section 1-1 in 45-degree skew constant-depth bridges

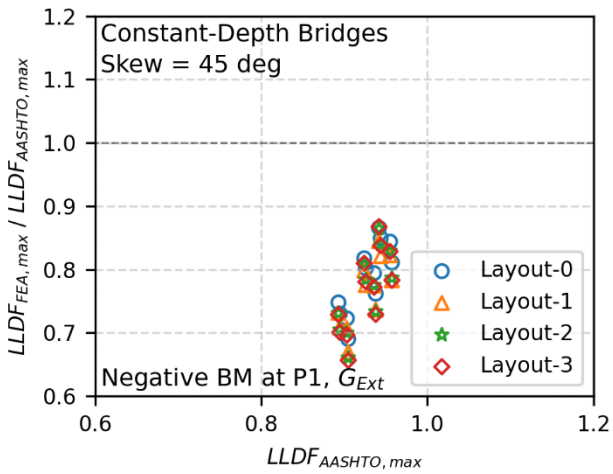


a) exterior girder

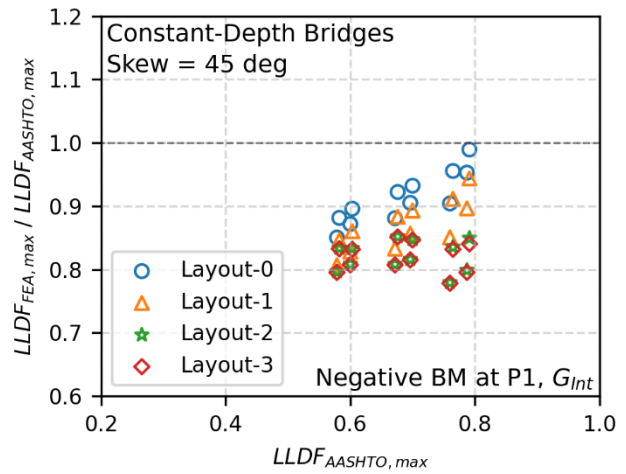


b) interior girder

Figure E.7. Comparison of maximum LLDFs from FEA and AASHTO for positive bending moment at section 2-2 in 45-degree skew constant-depth bridges

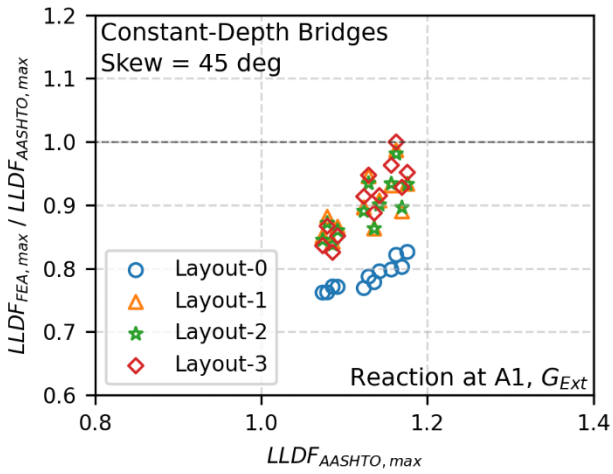


a) exterior girder

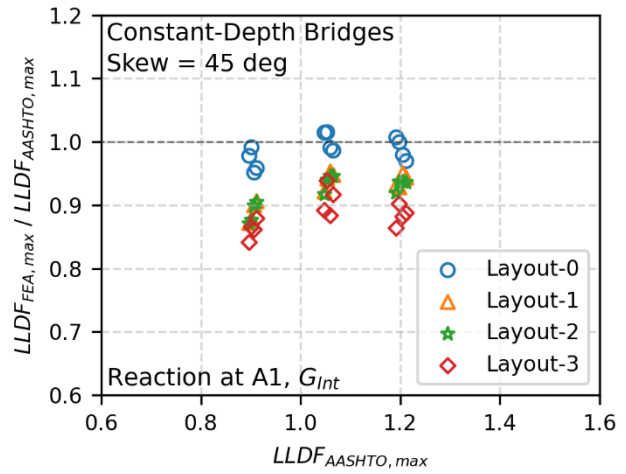


b) interior girder

Figure E.8. Comparison of maximum LLDFs from FEA and AASHTO for negative bending moment at support P1 in 45-degree skew constant-depth bridges

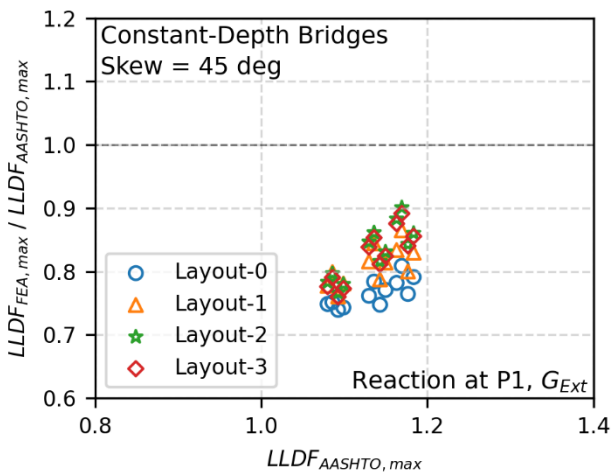


a) exterior girder

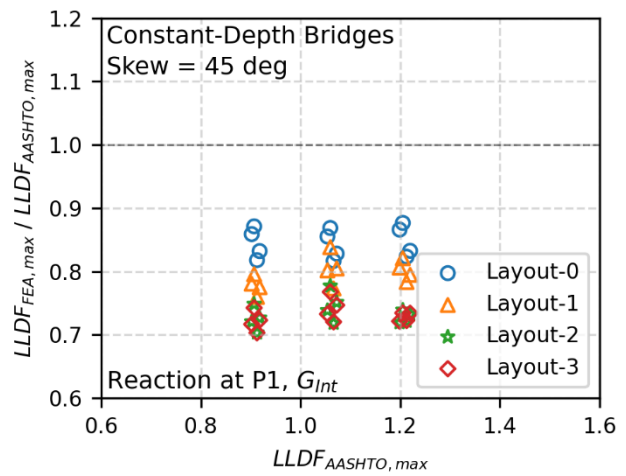


b) interior girder

Figure E.9. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at end support A1 in 45-degree skew variable-depth bridges



a) exterior girder



b) interior girder

Figure E.10. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at interior support P1 in 45-degree skew constant-depth bridges

E.1.3. Variable-Depth Bridges with 45-degree Skew

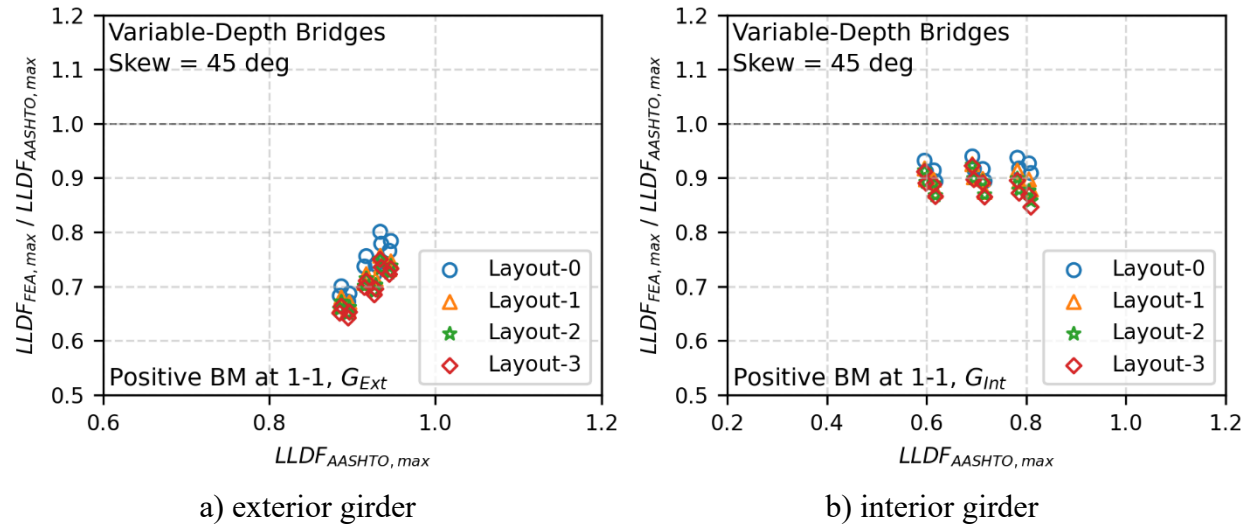


Figure E.11. Comparison of maximum LLDFs from FEA and AASHTO for positive bending moment at section 1-1 in 45-degree skew variable-depth bridges

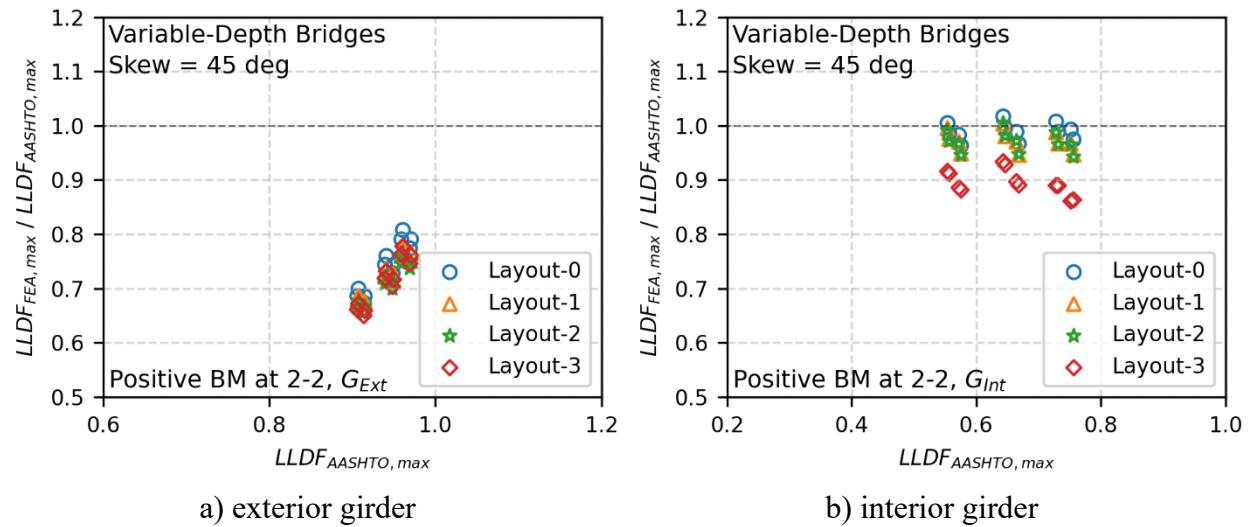
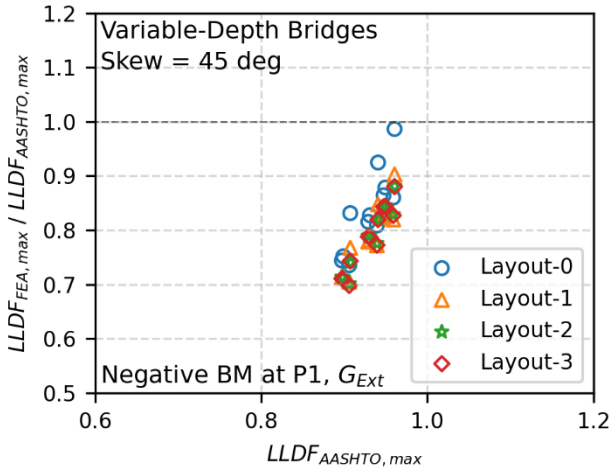
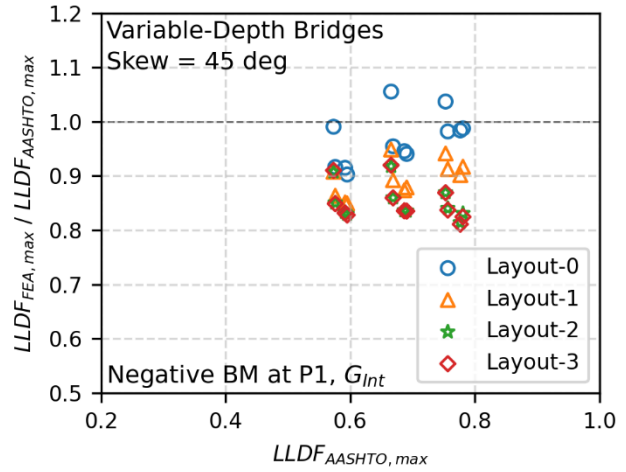


Figure E.12. Comparison of maximum LLDFs from FEA and AASHTO for positive bending moment at section 2-2 in 45-degree skew variable-depth bridges

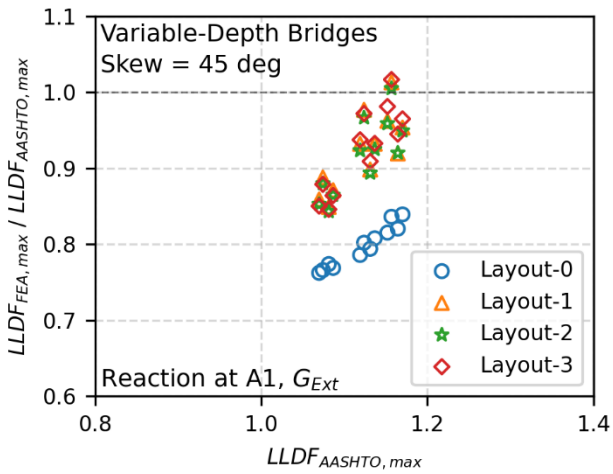


a) exterior girder

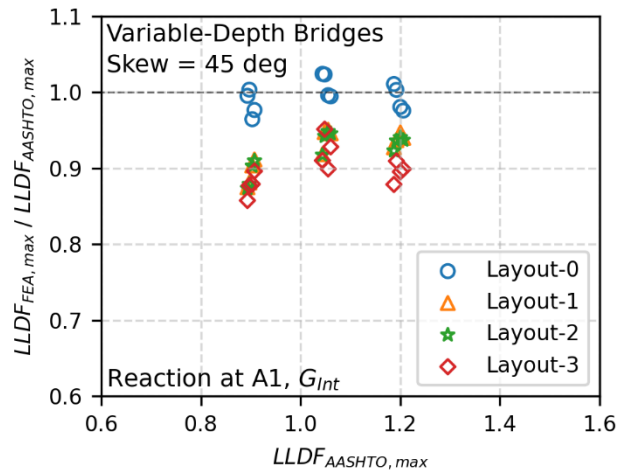


b) interior girder

Figure E.13. Comparison of maximum LLDFs from FEA and AASHTO for negative bending moment at support P1 in 45-degree skew variable-depth bridges



a) exterior girder



b) interior girder

Figure E.14. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at end support A1 in 45-degree skew variable-depth bridges

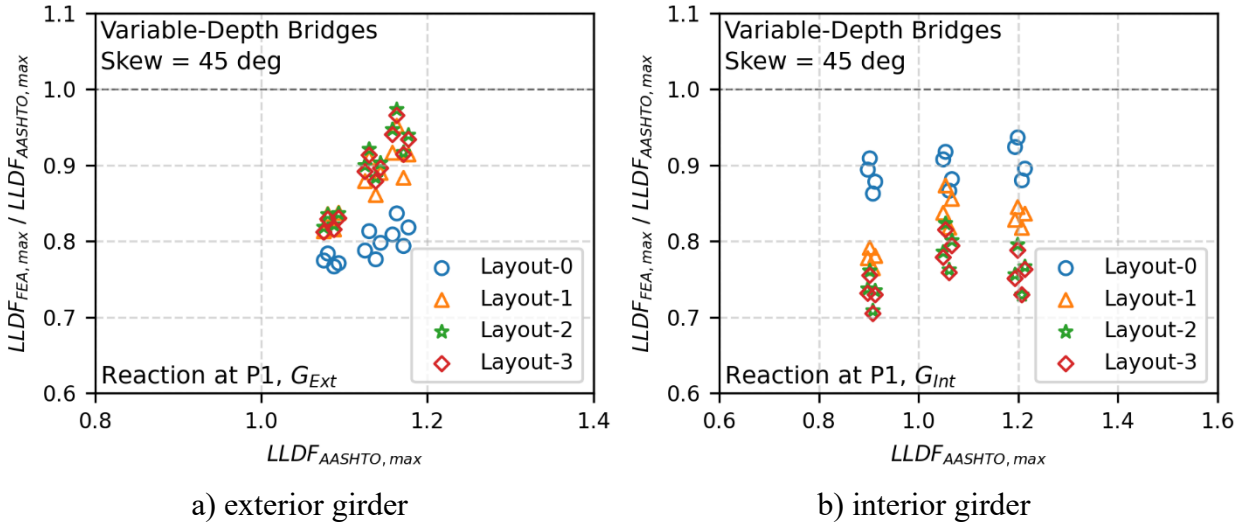


Figure E.15. Comparison of maximum LLDFs from FEA and AASHTO for vertical reaction at interior support P1 in 45-degree skew variable-depth bridges

E.2. Effect of Skewed Supports

The LLDFs for skewed bridges normalized with respect to normal support bridges are presented below for additional cases.

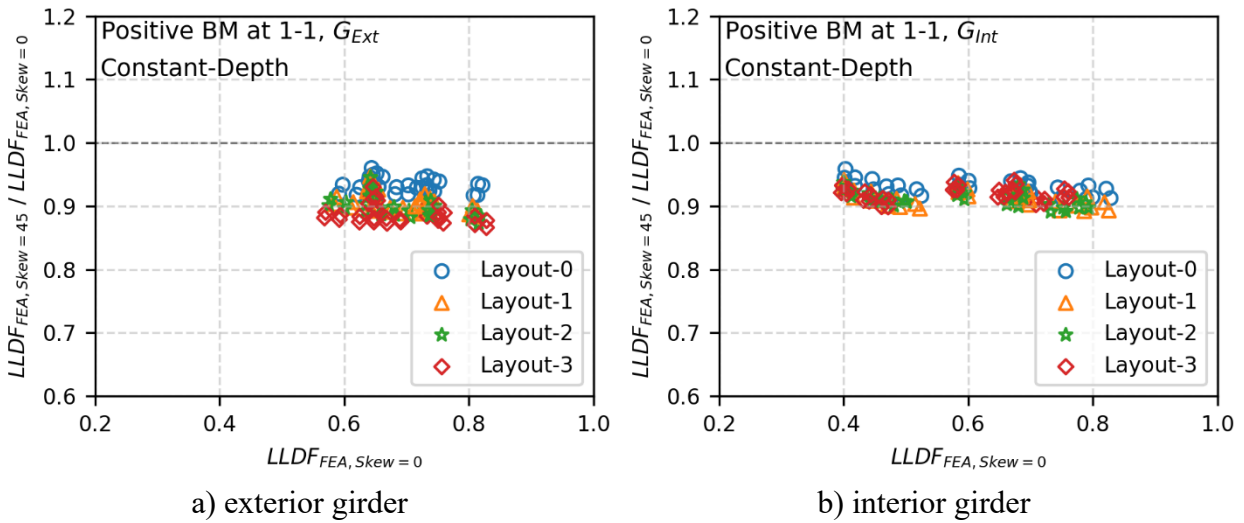
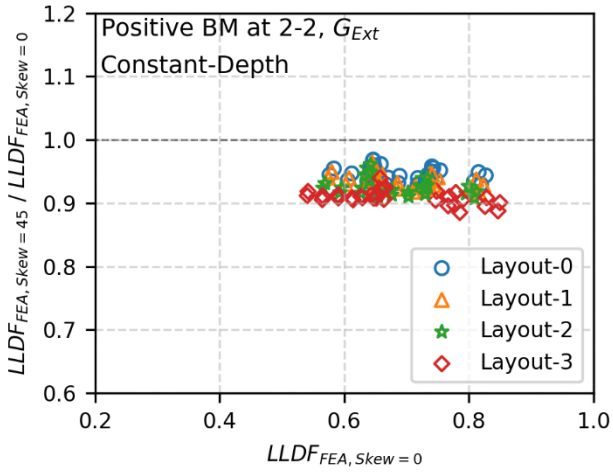
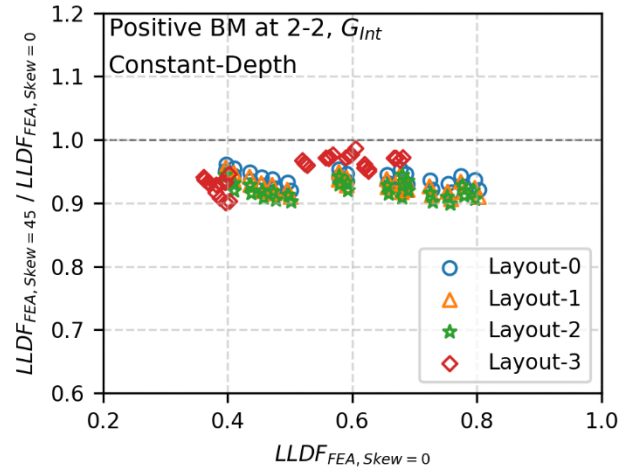


Figure E.16. Normalized LLDFs for positive moment at section 1-1 in constant-depth bridges with 45-degree skew

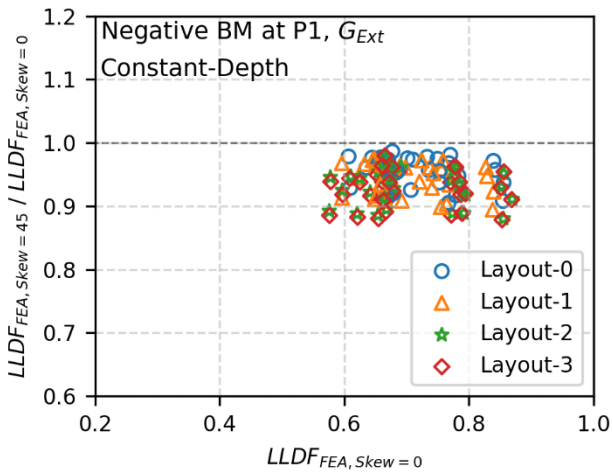


a) exterior girder

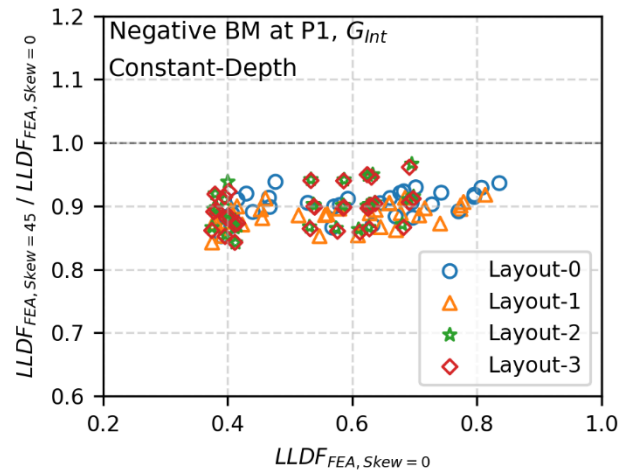


b) interior girder

Figure E.17. Normalized LLDFs for positive moment at section 2-2 in constant-depth bridges with 45-degree skew

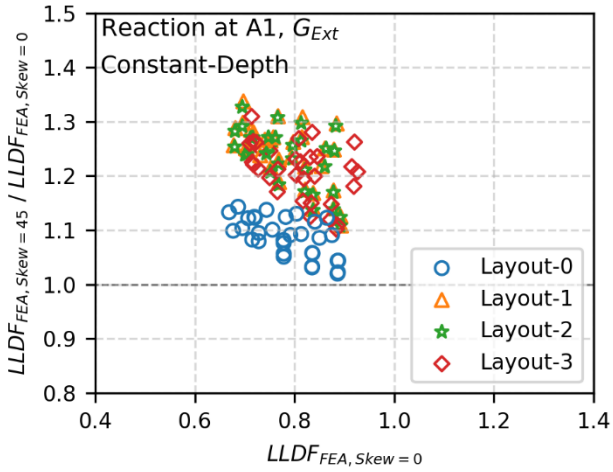


a) exterior girder

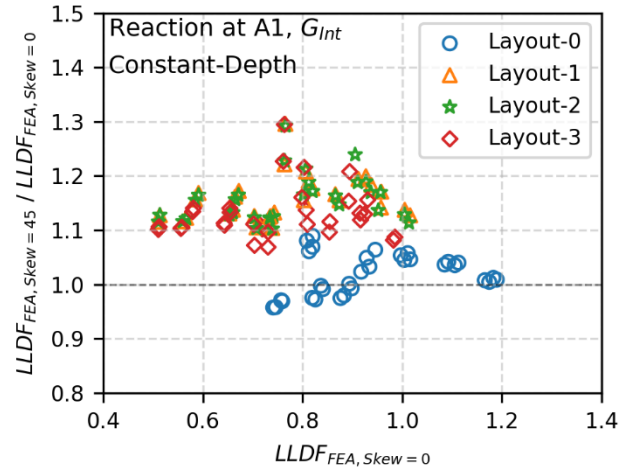


b) interior girder

Figure E.18. Normalized LLDFs for negative moment at support P1 in constant-depth bridges with 45-degree skew

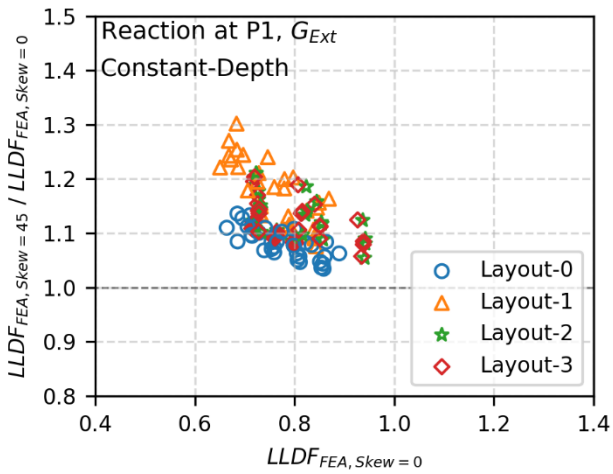


a) exterior girder

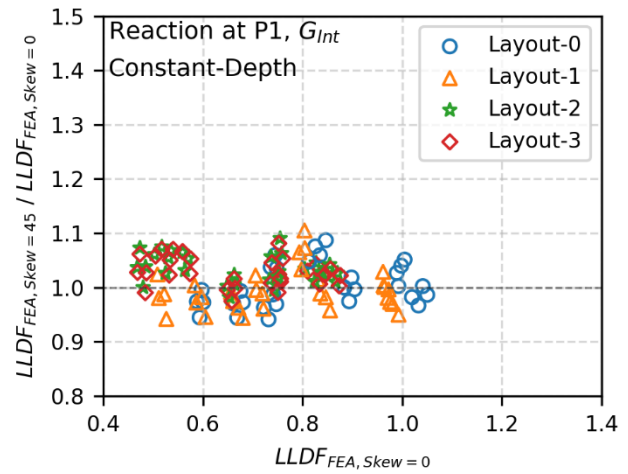


b) interior girder

Figure E.19. Normalized LLDFs for reactions at the obtuse end support A1 in constant-depth bridges with 45-degree skew



a) exterior girder



b) interior girder

Figure E.20. Normalized LLDFs for reactions at the obtuse interior support P1 in constant-depth bridges with 45-degree skew

Appendix F: Value of Research

F.1.1 Motivation and Significance

The stability of prestressed concrete I-girders have historically not been a major concern due to the relatively short span lengths and stocky nature of the standard sections. Current TxDOT standard shapes have been developed and employed for typical span ranges between 40 ft and 150 ft., where the bridges are simply supported with a continuous concrete deck. The maximum span lengths are often dictated by handling and shipping constraints. As a result of the maximum span lengths, conventional lateral-torsional buckling during erection and concrete deck slab placement has not been a primary concern. The primary stability limits were related to roll stability of the girders during lifting and deck construction. Recent advances in design and construction practices in Texas have made use of spliced-concrete girder bridges. The longer spans of the girders raise concerns about stability limits that have historically not been an issue. The focus of this study was to investigate the conventional lateral-torsional buckling behavior of spliced-concrete girders. Prior to this research project, clear guidance on the stability evaluation as well as the necessary bracing was not available. Additionally, the applicability of the current provisions given in AASHTO regarding live-load distribution factors were questionable, as the span lengths being built were outside the scope of the provisions.

The objective of this research project was to determine if spliced concrete I- and U-girders were susceptible to instability failure modes during erection and deck pour. Additionally, the research team (RT) was tasked with providing design guidance on the live load distribution of these systems, alongside the effect of diaphragms on stability and live load distribution.

Table 1 summarizes the Value of Research (VOR) with respect to the TxDOT Benefit Areas of the research, along with the nature of the benefit with respect to a qualitative or economic benefit, as well as whether that project benefit applies to TxDOT or the State as a whole. The significance of this research lies primarily in providing the necessary guidance to prevent the critical failure state of girder buckling while also improving the current level of knowledge. Stability failures can be sudden and catastrophic, leading to a major risk of life safety, substantial financial loss, and a reduction in confidence of bridge design and construction practices. This summary outlines the various benefit areas, with a discussion of the projected qualitative and economic benefits within each area.

Table 1: Summary of Benefit Areas

Benefit Area	Qualitative	Economic	Both	TxDOT	State	Both
Level of Knowledge	x			x		
System Reliability			x			x
Increased Service Life		x				x
Traffic and Congestion Reduction			x			x
Reduced User Cost		x			x	
Reduced Construction, Operations, and Maintenance Costs			x			x
Infrastructure Condition		x				x
Engineering Design Improvement			x	x		
Safety			x			x

F.1.2 Qualitative and Economic Assessments

The assessment of a given benefit area is based upon both qualitative and economic projections. Benefit areas may include assessments that are purely qualitative, purely economic, or both. Some qualitative assessments of a benefit area include very significant impacts – but those benefits may be deemed to possess non-monetary intangibles, or intangibles that are not readily evaluated monetarily. Qualitative assessments often include characteristics that may not be numerically represented. These values are often subjective and therefore described in terms of qualities, properties, or characteristics rather than establishing a numerical evaluation. Where possible, economic benefits are estimated and projected over the service life of Texas bridges.

F1.2.1 Level of Knowledge

A major contribution of this study is related to enhancing the level of knowledge since this is one of the first studies that is focused on establishing design guidance related to the stability behaviour of spliced-concrete I- and U-girders. The stability of these girder systems during erection and deck construction had not generally been evaluated prior to this study. In a similar fashion, the behavior of continuous spliced girder systems as a function of the range of likely geometries had not been previously investigated relative to the live-load distribution. The uncertainty of stability design could limit the adoption of the design and construction practices for spliced-concrete girder applications. The enhanced procedures outlined in this report improve the potential for extending the use of these girder systems throughout the state. While the improvements in the Level of Knowledge does have economic advantages to other benefit areas, those projected economic benefits are outlined within the given areas below. This benefit area primarily impacts TxDOT.

F1.2.2 System Reliability

The evaluation of the stability and live load distribution for the wide range of expected geometries of spliced concrete-girder applications enhances the reliability of future applications. Improvement in system reliability has both qualitative and economic benefits. While the evaluation of the stability and live load distribution in spliced-concrete girder applications are primarily targeted towards the design of the girders, the recommendations also impact the construction procedures as well as the long-term performance of these girder systems. The research team believes that the recommendations related to minimum diaphragm layouts improve safety during construction and will also lead to better long-term performance with regards to girder and deck cracking that will lead to less maintenance issues over the service life of the bridge. While these improvements are also projected to provide economic benefits, the RT has primarily assessed the economic benefits in the Safety benefit area below. Although design-related enhancements primarily benefit TxDOT, the impact on safety and long-term performance tend to benefit the entire state of Texas through improved safety for the travelling public and reduced long-term costs that benefit the state.

F1.2.3 Increased Service Life

The proposed design guidance will help to prevent potential partial or complete collapses during construction, which can entirely compromise the service life of the bridge system. Preventing these collapses and promoting an optimal layout of diaphragms, which improve the live load distribution between the girders, will enable these structures to meet their intended design service life. Accurate live load distributions alone will reduce the likelihood of crack initiation which leads to degradation of the concrete and steel structural components of the bridge. Apart from the mechanical effects of cracks that can compromise the flexural and torsional capacity of the concrete members, the introduced cracks also serve as pathways for accelerated intrusion of aggressive agents (Aliasghar-Mamaghani et al. 2025) – such as chloride ions entailed in deicing salts – as well as moisture, which can further compromise the mechanical properties (ductility and effective sectional area) of the reinforcing steel and prestressing strands, through electrochemical oxidation-reduction reactions (Aliasghar-Mamaghani et al. 2022; Aliasghar Mamaghani 2023). By increasing service life, TxDOT and the state of Texas stand to benefit by reducing long term construction and maintenance costs.

F1.2.4 Traffic and Congestion Reduction

Having confidence in the design practices of spliced concrete girder bridges will enable bridge designers in Texas to construct these bridges over locations where they have not previously been applied. In particular, the enhanced design methodology offers an effective method of increasing span lengths and structural efficiency, which is of prime importance, specifically in highly congested areas. Additionally, reductions in maintenance needs can further improve traffic flow as bridges will need to be closed less often. As travel time and congestion impact the economic output and quality of life of Texas citizens, minimizing both travel time and congestion is of relevance to both TxDOT and the state of Texas.

F1.2.5 Reduced User Cost

Due to confidence in their increased span lengths, spliced girder bridges may allow for the use of fewer piers in certain geographic settings (e.g., bridges crossing rivers, or those in highly congested urban areas). The reduction in the number of piers can significantly reduce the overall project cost.

For example, for a typical bridge unit of 600 ft between expansion joints, the current practice of 150 ft spans for prestressed concrete girders would require 5 piers. If a spliced concrete girder system was employed with a main span of 250 ft, it would result in the need of only 4 piers. Reducing the costs of bridge projects is important for the state of Texas as infrastructure spending is limited.

F1.2.6 Reduced Construction, Operations, and Maintenance Costs

Improved understanding of the live load distribution will result in better, more accurate designs. When inaccurate load distribution models are used in design, decks and girders may experience loading conditions that differ significantly from their projected designs, potentially leading to crack formation that requires subsequent repairs. If left untreated, cracks can lead to more extensive long-term deterioration and corrosion of the reinforcing steel. Repair and maintenance costs can be significant, as they may require additional related expenses related to mobilization, access, and lane control for an active roadway. The contribution of accurate live load distribution factors for these types of systems, which previously were not addressed by design standards, will improve the performance of both the girder systems and the supported deck. Keeping maintenance costs and requirements low is relevant to TxDOT and the state of Texas.

F1.2.7 Infrastructure Condition

The diaphragm layout recommendations provided in the report improve the distribution of live load-induced forces. By better distributing these forces, thus reducing concentrations of moment and forces on individual girders, the long-term performance of girders and decks will lead to the infrastructure condition being better preserved over the service life of the structure. Maintaining the condition of infrastructure is relevant to TxDOT and the state of Texas.

F1.2.8 Engineering Design Improvement

The RT has provided valuable research data on the behavior of concrete spliced girders, leading to a major improvement in understanding the fundamental behavior and the development of design requirements for these girders. The RT has also delivered benchmark cases that can be used to aid designers in properly calibrating analysis models and the methodology to ensure all pertinent aspects of stability design requirements are addressed. With these tools and knowledge, designers can be confident in the application of spliced concrete girders and will understand problematic behaviors and applicable design solutions. These tools help both design engineers and construction engineers throughout the course of a project. This is primarily a benefit to TxDOT, as TxDOT designers will be the ones to use or enforce these recommendations.

F1.2.9 Safety

Although there are not direct economic benefits as a result of this research project, as substantial changes to design and construction are unnecessary, the value of prevention of potential catastrophe in Texas bridges cannot be understated. As a collapse may lead to significant loss of life among workers and motorists, reputation of TxDOT, project funds, and financial damages. As mentioned in the report, instability of bridge girders may lead to sudden catastrophic failure.

A recent concrete bridge failure, which led to the death of 6 and injury of 10 others, was the Florida International University pedestrian bridge collapse (CBS Miami, 2019; NTSB, 2019). This bridge serves as a reference of potential economic loss, as the financial cost of the bridge, its replacement, and the settlements is public knowledge. The bridge cost \$14.2 million to construct, \$38 million to replace, and \$103 million in settlement damages to the victims and families (CBS Miami, 2019; Gonzalez, 2024; NBC6, 2024). In essence, the settlement and replacement alone cost nearly 10 times the initial construction cost, accounting for inflation. The losses due to damaged public confidence and to reputation are even harder to quantify. As such, research for design guidance which prevents such failures is an essential preventative expense.

F1.3 Summary

Project 0-7115 supplies recommendations that provide numerous benefits to both TxDOT and the state of Texas. These benefits include improvements to safety and design knowledge, as well as reductions in maintenance costs and traffic congestion. Above all, the RT emphasized the need to prevent collapses of concrete spliced girder due to instability, and the value of ensuring stable systems both economically and qualitatively.

F1.4 References

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