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Methodology for Load Rating Double-Tee Bridges

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ABSTRACT

The most common type of bridge on South Dakota local roads is a precast prestressed double-tee (DT) girder bridge. More than 700 DT bridges are currently in-service in South Dakota. Structural detailing, aging, traffic volume, and environmental conditions affect structural performance, integrity, and capacity of DT bridges. When a bridge is affected by one or more of the aforementioned parameter(s), estimation of the bridge safe live load is necessary to ensure safety of the traveling public and prevent excessive bridge damage and collapse. Load rating of damaged bridges is challenging because of a lack of information regarding the capacity and live load distribution of damaged components. In this study, quantitative definitions were first proposed to identify all damage types and condition states specific to double-tee girders. Subsequently, more than 370 inspection reports on South Dakota double-tee girder bridges were reviewed to determine the frequency of damage types and condition states, bridge span length, bridge number of spans, girder depth, and bridge skew conditions. The statistical database was then used to identify double-tee bridge candidates for field and strength testing. Using the database, 10 double-tee bridges were identified as suitable for field testing and inspected for further evaluation. Subsequently, two bridges were selected for field testing. Girder distribution factors (GDFs) and dynamic load allowance (IM) were measured. The field test results confirmed that AASHTO LRFD specifications can be used to estimate the moment and shear GDFs for South Dakota DT bridges with a longitudinal joint damage condition state 3 or less. For the calculation of moment and shear GDFs for a South Dakota DT bridge with a longitudinal joint damage condition state 4, GDFs were proposed to be the greater of: (a) the factor for the exterior girders, (b) the factor for the interior girders, and (c) 0.6. Furthermore, AASHTO LRFD specifications can be used for estimation of IM for damaged double-tee girders with no further modification. An accurate estimation of the capacity of damaged double-tee girders was critical in this project for a safe load rating. To verify the available moment and shear capacity estimation methods, two 45-year old double-tee girders, one 50 ft (15.24-m) long and another 30 ft (9.14-m) long, were extracted from a bridge located in Nemo Road, SD, and were strength tested at the Lohr Structures Laboratory at South Dakota State University. A four-point loading configuration was selected for the strength testing. Data was collected, and the methods were validated. Verified methods were then used to calculate the shear and moment capacities of 23 different double-tee sections, which have been used in South Dakota. Based on the statistical, experimental, and analytical studies, a load rating methodology was proposed for damaged double-tee girder bridges in which the rating may be performed similarly to the AASHTO Load and Resistance Factor Rating (LRFR) method that currently used in practice. Nevertheless, it was recommended to modify the capacity (C) and live load components (LL and IM) of the load rating equation accounting for different damage types and condition states.

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Acronym	Definition
AASHTO	American Association of State Highway and Transportation Officials
ADTT	Average Daily Truck Traffic
ASTM	American Society for Testing and Materials
BRM	Bridge Management Software
CS	Condition State
DOT	Department of Transportation
DT	Double-Tee Girder / Bridge
FHWA	Federal Highway Administration
ft	Foot/Feet
GDF	Girder Distribution Factor
hr	Hour/Hours
IM	Dynamic Load Allowance
in.	Inch/Inches
kip	1000 Pounds
ksi	kip per square inch
lb	Pound/Pounds
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
LVDT	Linear Variable Different Transformer
m	Meter
min	Minute/Minutes
MPC	Mountain Plains Consortium
MPa	Mega Pascal
mm	Millimeter, Meter / 1000
NCHRP	National Cooperative Highway Research Program
SD	State of South Dakota, USA
SDDOT	South Dakota Department of Transportation
sec	Second
SDSU	South Dakota State University

1. EXECUTIVE SUMMARY

1.1 Introduction

The most common type of bridge on South Dakota local roads is a precast prestressed double-tee girder bridge. More than 700 double-tee bridges are currently in-service in South Dakota. The local transportation system carries millions of dollars of agricultural products to market, connects people, and provides access to farms, state parks, and recreational sites.

Several types of damage with different condition states have been reported for double-tee bridges. When a bridge is damaged, the estimation of its safe live load is a challenge due to a lack of information on the capacity and live load transfer mechanism for the damaged components.

1.2 Problem Description

Structural detailing, aging, traffic volume, and environmental conditions affect the load carrying capacity of bridges. When a bridge is affected by one or more of these parameters, estimation of the bridge safe live loads is necessary to ensure safety of the traveling public and prevent excessive bridge damage and collapse. This process is usually referred to as “load rating”.

Load rating of a bridge requires accurate estimation of damaged member capacities and the knowledge of live load distribution and demands. The literature and current specifications are lacking a systematic method to include damage of bridge components in load rating equations. The same issue exists for double-tee bridges. The main goal of the present study was to develop a methodology for safe load rating of double-tee bridges when their girders are damaged.

1.3 Research Work

To achieve the project goal, quantitative definitions were proposed to identify all damage types and condition states specific to South Dakota double-tee bridges. Subsequently more than 370 inspection reports and the state Bridge Management (BrM) database were reviewed to determine frequency of damage types and condition states, bridge span length, bridge number of spans, girder depth, and bridge skew conditions. The statistical database was then used to identify double-tee bridge candidates for field and strength testing. Ten double-tee bridges were identified as suitable for field testing and were inspected for further evaluation. Subsequently two bridges were selected for field testing. Girder distribution factors (GDFs) and dynamic load allowance (IM) were measured during field testing of the two bridges.

To verify the available moment and shear capacity estimation methods, two 45-year old double-tee girders, one 50 ft (15.24-m) long and another 30 ft (9.14-m) long, were extracted from a bridge located on Nemo Road, SD, and were strength tested at the Lohr Structures Laboratory at South Dakota State University. A four-point loading configuration was selected for the strength testing. The measured data was used to validate the capacity estimation methods. Subsequently, verified methods were used to calculate the shear and moment capacities of all 23 different double-tee sections, which have been used in South Dakota.

1.4 Research Findings

Based on the review of the inspection reports, the most common damage type found on double-tee girders is the cover deterioration. Most double-tee bridges in the state are single span with a span length of 40–60 ft (12.19–18.3 m). Double-tee girders with a depth of 23 in. (584 mm) are more common than 30-in. (762-mm) deep girders. Furthermore, non-skewed double-tee bridges have been used more often than skewed bridges.

Field testing of the two double-tee bridges revealed that current AASHTO LRFD specifications are sufficient to determine the bridge live load parameters whether the girder-to-girder joint damage has a condition state of 3 or less. A conservative recommendation was proposed for the joints with damage condition state 4.

Strength testing of the two salvaged double-tee girders provided sufficient information to validate the shear and moment capacity estimation methods, which were used in an extensive analytical study to reduce girder capacity based on damage type and condition state.

Based on the statistical, experimental, and analytical studies, a methodology was proposed for damaged double-tee bridges in which the load rating can be performed similarly to the AASHTO Load and Resistance Factor Rating (LRFR) method currently used in practice. Nevertheless, it was recommended to modify the capacity (C) and live load components (LL and IM) of the load rating equation accounting for different damage types and condition states. Condition factors were proposed for all different double-tee sections that have been used in the state.

1.5 Recommendations

Based on the findings of this study, the research team offers the following recommendations.

1.5.1 Recommendation 1: General

The guidelines as detailed in Appendix C should be adopted for the load rating of damaged double-tee girder bridges.

In general, the load rating of damaged double-tee girder bridges is performed similarly to the LRFR method, but the capacity and live load parameters should be modified as recommended below.

1.5.2 Recommendation 2: Capacity Modification

The guidelines as detailed in Section C.2.2 of Appendix C should be adopted to modify the girder capacities accounting for different damage types and condition states.

The moment and shear capacities of a damaged double-tee girder at strength limit states should be reduced using the proposed condition factors (φ_c) for South Dakota double-tee sections. At service limit states, the bridge concrete and reinforcing steel mechanical properties as recommended should be used in the load rating equation.

1.5.3 Recommendation 3: Demand Modification

The guidelines as detailed in Section C.2.3 of Appendix C should be adopted to modify the live load parameters accounting for different girder-to-girder damage condition states.

If a double-tee bridge has a longitudinal joint damage condition state 3 or less, the AASHTO LRFD can be followed to determine the live load parameters. Recommendations were provided for longitudinal joint damage condition state 4.

2. INTRODUCTION

2.1 Problem Description

The most common types of bridge on South Dakota local roads are precast prestressed double-tee girder bridges with two typical girder depths of 23 in. (584 mm) and 30 in. (762 mm). More than 700 of these bridges are currently in-service in the state. The South Dakota local transportation system plays a significant role in the state economy and welfare by carrying millions of dollars of agricultural products to market, connecting people, and providing access to farms, state parks, and recreational sites.

Several types of damage with varying severity have been reported on South Dakota double-tee bridges. Figure 2.1 shows a few damage types for these bridges. It is critical to understand and quantify the effect of each damage type and its severity (condition state) on the capacity and live load transfer mechanism for double-tee bridges.

(a) Stem Cover Deterioration

(b) Stem Reinforcement Exposure

(c) Girder-to-Girder Damage

(d) Flange Cover Deterioration

Figure 2.1 Typical Damage of Double-tee Bridges

Structural detailing, aging, traffic volume, and environmental conditions such as a high number of freeze-thaw cycles and the use of de-icing agents may significantly affect the load carrying capacity of a bridge. These factors are specifically important for double-tee bridges located in South Dakota since: (1) recent research projects showed that conventional double-tee girder longitudinal joint detailing is not adequate for long-term performance (Wehbe et al., 2016; Tazarv et al., 2018), (2) more than 75% of these bridges are 20 years or older (Bohn et al., 2017), and (3) more than 100 freeze-thaw cycles are annually recorded in the state (Haley, 2011). These parameters expedite double-tee bridge deterioration.

When a bridge is affected by one or more of the aforementioned parameters, the evaluation of load carrying capacity of the bridge, commonly referred to as "load rating," is necessary to ensure the safety of the traveling public and to prevent excessive bridge damage and collapse. Load rating of a bridge requires an accurate estimation of the capacity of the damaged members and the knowledge of live load distribution and demands. Literature and current specifications are lacking a systematic method to include the damage of bridge components when performing a load rating. A methodology is needed to relate the double-tee bridge component damage to the load rating parameters.

2.2 Research Objectives

Following are the main research objectives.

2.2.1 Identify Methods of Load Rating

Review nationally recognized standards for visual and analytical techniques on load rating bridges.

An extensive review of the literature, guidelines, and specifications was performed to: identify methods of load rating, evaluate current capacity estimation methods for damaged concrete sections, categorize different damages and their condition states, and understand the live load distribution when the members are damaged.

2.2.2 Experimental Programs

Develop a testing plan to investigate the in-place structural integrity of double-tee bridges with varying amounts of visible distress.

Successful load rating of double-tee bridges requires accurate estimation of demands and capacities. The live load demand, distribution, and their analytical models can be established using field testing of double-tee bridges with different configurations (e.g. different span lengths, number of girders, girder geometry, and damage of girder-to-girder joints since it will affect the load distribution based on the study by Wehbe et al., 2016). Two double-tee bridges, one 34-year old and another 38-year old, were field tested using a 50-kip dump truck to determine their live load distribution factors and dynamic load allowance. Both bridges had a girder-to-girder joint damage with condition state 3.

The shear and moment capacities of double-tee bridges, however, cannot be determined through field testing. Furthermore, the inspection of in-service double-tee bridges has indicated different damage types and condition states, which may have significant adverse effects on the shear and moment capacities of the girders. It was critical to establish reliable methods for estimation of the double-tee girder capacities, including different damage types. Laboratory strength testing was performed on two 45-year old salvaged double-tee girders, one 30-ft (9.14-m) long and another 50-ft (15.24-m) long. These girders had severe damages such as exposure of tendons and loss of stem concrete. he collected information was used to verify the moment and shear capacity estimation methods for damaged double-tee girders.

2.2.3 Develop Load Rating Methodology for Damaged Double-Tee Bridges

Develop a methodology for engineers and highway superintendents in South Dakota to evaluate the structural integrity of double-tee bridges and estimate load limits through visual inspection.

Based on analytical and experimental studies performed in this project, a methodology was developed for load rating of damaged double-tee bridges. The method is generally the same as the AASHTO LRFR method currently used in practice, but the capacity and live load parameters should be modified accounting for different damage types and condition states.

3. LITERATURE REVIEW

The American Society of Civil Engineers (ASCE) rated the United States' 614,387 bridges with a "C+" grade meaning they are in a fair condition but require attention (ASCE Infrastructure Report Card, 2017). ASCE reported 40% of the nation's bridges are at least 50 years old, the average age of the U.S. bridges — currently 43 years — is increasing, and many are approaching the end of their design life. The Federal Highway Administration (FHWA) reported that 25% of the nation's bridges need repair, rehabilitation, or total replacement (FHWA-ABC, 2017), with 13% being structurally deficient and 12% obsolete. FHWA estimated that $12.8 billion is annually needed to maintain the U.S. bridges in service, while the backlog of rehabilitation projects is $123 billion. Our nation faced an historic period of bridge construction 50 years ago. Today, we face another historic period, but now the challenge is to repair and reconstruct those bridges.

Approximately 188 million trips are taken per day across the deficient bridges in the USA (ASCE Infrastructure Report Card, 2017). Because of lack of sufficient funding to fully restore all distressed bridges, proper measures should be devised to accurately estimate the safe service loads of bridges to prevent catastrophic events. One example is the collapse of the I-35W Mississippi River bridge in Minneapolis, Minnesota on August 1, 2007, which showed that deficient bridges can jeopardize the public safety, and their serviceability should be properly evaluated.

Bridges are required to be visually inspected every two years, according to the FHWA Bridge Inspector's Reference Manual (2012). Two methods are generally used to quantitatively evaluate the condition state of bridge components: (1) FHWA method with a scale of "0" to "9" in which "9" means the component is in an excellent condition and "0" means the component is significantly damaged and is out of service, and (2) method presented in the AASHTO Manual for Bridge Element Inspection (2013) in which four different condition states (good, fair, poor, and severe) are considered for bridge elements.

Bridge inspection is necessary to collect condition information on each bridge element. Accurate knowledge of bridge conditions helps to identify needed maintenance, repair, and replacement. Based on the inspection results, load rating might be needed. Load rating involves estimations of the safe live load capacity of a bridge based on existing structural conditions, material properties, and loads and traffic conditions at the bridge site. Load rating is usually carried out on aged or distressed bridges, or on those that encounter higher loads than design loads. Load rating improves the safety of a bridge by posting limitations.

Literature including national specifications, manuals, and guidelines was reviewed to identify inspection methods, load rating methods, bridge element damage types and condition states, and the capacity of aged and distressed elements. A summary of the findings is presented here.

3.1 Bridge and Bridge Element Inspection

Frequent bridge inspections are needed to monitor the condition of bridges and their elements for proper maintenance and possible repair or replacement. Several state Departments of Transportation (DOTs) including SDDOT (BSCM, 1998) have developed inspection manuals for bridges. In addition to state manuals, two nation-wide inspection manuals are available for bridge engineers to produce consistent reports across the nation: (1) the Bridge Inspector's Reference Manual (2012) by FHWA and the National Highway Institute (NHI), and (2) the Manual for Bridge Element Inspection (2013) by AASHTO. The former provides a 10-scale condition rating (Table 3.1) for bridge components including decks, superstructures, substructures, channels, and culverts. The latter provides four different condition states (good, fair, poor, and severe) for different bridge elements. One example of the AASHTO rating guide for prestressed girders is presented in Table 3.2.

Of the two manuals and rating methods discussed above, the AASHTO Manual for Bridge Element Inspection (2013) is better suited for double-tee bridges since: (1) element-level condition states are needed for successful evaluation of double-tee bridges, and (2) the damage of a double-tee bridge can be inclusively described with four condition states to be incorporated later as the input to AASHTO load rating methods (see Sec. 3.3). The FHWA 10-scale rating can be used for double-tee bridges but it is more involved and may not affect the outcome of the load rating.

Table 3.1 FHWA Component Condition Rating

Code	
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling, or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action may put bridge back in light service.
0	FAILED CONDITION - out of service; beyond corrective action.

Table 3.2 AASHTO Damage Types and Condition States for Prestressed Girders

Defect Types	Condition States CS-1 Good	CS-2 Fair	CS-3 Poor	CS-4 Severe
Delamination/Spall/Patched Area (1080)	None	Delaminated. Spall 1 in. or less deep or 6 in. or less in diameter. Patched area that is sound.	Spall greater than 1 in. deep or greater than 6 in. diameter. Patched area that is unsound or showing distress. Does not warrant structural review.	The condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects and the defects impact strength or serviceability of the element or bridge.
Exposed Rebar (1090)	None	Present without measurable section loss.	Present with measurable section loss but does not warrant structural review.	
Exposed Prestressing (1100)	None	Present without section loss.	Present with section loss but does not warrant structural review.	
Cracking (1110)	~~Width less than 0.004 in. or spacing greater than 3 ft.~~ Insignificant cracks or moderate-width cracks that have been sealed.	~~Width 0.004–0.009 in. or spacing 1.0–3.0 ft.~~ Unsealed moderate-width cracks or unsealed moderate pattern (map) cracking.	~~Width greater than 0.009 in. or spacing less than 1 ft.~~ Wide cracks or heavy pattern (map) cracking.	
Efflorescence/Rust Staining (1120)	None	Surface white without build-up or leaching without rust straining.	Heavy build-up with rust staining.	
Damage	Not applicable	The element has impact damage. The specific damage caused by the impact has been captured in condition state 2 under the appropriate material defect entry.	The element has impact damage. The specific damage caused by the impact has been captured in condition state 3 under the appropriate material defect entry.	The element has impact damage. The specific damage caused by the impact has been captured in condition state 4 under the appropriate material defect entry.

From: AASHTO Manual for Bridge Element Inspection (2011) – Section 3.3.1.6.
The crossed out text indicates the revision by AASHTO.

3.2 Load Rating

Load rating is performed to determine the safe live load capacity of bridges. Load rating depends on several factors including:

- existing structural conditions
- element material properties
- applied loads and traffic conditions

Load rating of bridges can be carried out through experimental or analytical methods according to the AASHTO Manual for Bridge Evaluation (2011). Experimental load rating is done by load testing a bridge but keeping the bridge in the linear-elastic range. Analytical methods include: (i) allowable stress rating (ASR), (ii) Load Factor Rating (LFR), and (iii) Load and Resistance Factor Rating (LRFR). ASR was the first generation of the analytical load rating using unfactored loads and allowable stresses. When design codes were upgraded with the Load Factor method, the load rating was also upgraded to LFR in which loads were factored. The current design method for bridges is based on Load and Resistance Factor Design (LRFD). LRFR is a load rating methodology based on LRFD.

The result of an analytical load rating method is a number. A number equal to or greater than unity means the bridge is safe and serviceable under the live load included in the rating. A number less than one indicates that the bridge is not safe and a load limit should be posted.

All three loading rating methods are currently allowed by the AASHTO Manual for Bridge Evaluation (2011). Of the three, LRFR was selected by the project technical panel to be used in this study since it conforms to current AASHTO design methods. A brief summary of LRFR is presented here.

3.2.1 Load and Resistance Factor Rating (LRFR)

LRFR is the current method for load rating of bridges consistent with current AASHTO LRFD bridge design specifications (2014). LRFR is performed in three load levels: (1) design live loads, (2) legal loads, and (3) permit loads.

3.2.1.1 Design Load Rating

Design load rating is the first level of the evaluation of bridges based on the HL-93 Loading and LRFD design specifications to check whether or not a bridge meets the current code requirements. If not, legal or permit load rating should be carried out.

3.2.1.2 Legal Load Rating

Legal load rating is the second level of the assessment of bridges. It provides a single safe live load capacity for a specific truck type according to AASHTO or state legal loads. The results of this load rating can be used for load posting or bridge strengthening.

3.2.1.3 Permit Load Rating

Permit load rating checks the safety and serviceability of bridges, which is the third level rating applied only to bridges having sufficient capacity for the AASHTO legal load. For example, the permit load rating is performed for overweight trucks.

3.2.1.4 LRFR Load-Rating Equation

Load rating of a bridge using the LRFR method is calculated through:

$$RF = \frac{C - (\gamma_{DC})DC - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \qquad \text{(Eq. 3.1)}$$

For the Strength Limit State:

$$C = \varphi_c \, \varphi_s \, \varphi \, R_n \qquad \text{(Eq. 3.2)}$$

$$\varphi_c \, \varphi_s \geq 0.85 \qquad \text{(Eq. 3.3)}$$

For the Service Limit State:

$$C = f_R \qquad \text{(Eq. 3.4)}$$

where,

RF = Rating factor,

C = Capacity,

f_R = Allowable stress specified in the LRFD code,

R_n = Nominal member resistance,

DC = Dead load effect due to structural components and attachments,

DW = Dead load effect due to wearing surface and utilities,

P = Permanent loads other than dead loads,

LL = Live load effect,

IM = Dynamic load allowance,

γ_{DC} = LRFD load factor for structural components and attachments,

γ_{DW} = LRFD load factor for wearing surface,

γ_P = LRFD Load factor for permanent loads other than dead loads = 1.0,

γ_{LL} = Evaluation live load factor,

φ_c = Condition factor,

φ_s = System factor,

φ = LRFD resistance factor.

Load rating is performed at each applicable limit state and load effect with the minimum value as the governing rating factor. Tables 3.3 to 3.7 present some of the LRFR parameters. Complete information can be found in the AASHTO Manual for Bridge Evaluation (2011).

Table 3.3 Limit States and Load Factors for Load Rating

Bridge Type	Limit State	Dead Load, γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Prestressed Concrete	Strength I	1.25	1.5	1.75	1.35	Table 2.4	-
	Strength II	1.25	1.5	-	-	-	Table 2.5
	Service III	1	1	0.8	-	1	-
	Service I	1	1	-	-	-	1

Table 3.4 Generalized Live Load Factors (γ_{LL})

Traffic Volume (One direction)	Load Factor for Routine Commercial Traffic: Type 3, Type 3S2, Type 3-3 and Lane Loads	Load Factor for Specialized Hauling Vehicles: NRL, SU4, SU5, SU6, and SU7
Unknown	1.8	1.6
ADTT ≥ 5000	1.8	1.6
ADTT = 1000	1.65	1.4
ADTT ≤ 100	1.4	1.15

Table 3.5 Permit Load Factors

Permit Type	Frequency	Loading Condition	DF(a)	ADTT (one direction)	Load Factor by Permit Weight(b) Up to 100 kips	Load Factor by Permit Weight(b) >=150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic(other vehicles may be on the bridge)	Two or more lanes	>5000	1.8	1.3
				1000	1.6	1.2
				<100	1.4	1.1
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One Lane	N/A	1.15	
	Single-Trip	Mix with traffic(other vehicles may be on the bridge)	One Lane	>5000	1.5	
				1000	1.4	
				<100	1.35	
	Multiple-Trips(less than 100 crossings)	Mix with traffic(other vehicles may be on the bridge)	One Lane	>5000	1.85	
				1000	1.75	
				<100	1.55	

(a) DF=LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.
(b) For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and ADTT value. Use only axle weights on the bridge.

Table 3.6 Condition Factor (φ_c)

Structural Condition of Member	φ_c
Good or Satisfactory	1
Fair	0.95
Poor	0.85

Table 3.7 System Factor (φ_c)

Superstructure Type	φ_s
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.9
Multiple Eyebar Members in Truss Bridges	0.9
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing <= 4 ft	0.95
All Other Girder Bridges and Slab Bridges	1
Floorbeams with Spacing > 12 ft and Noncontinuous Stringers	1
Redundant Stringer Subsystems between Floorbeams	0.85

Load factors are amplifying factors used in design equations to increase loads. Live load factors provide uniform and acceptable level of reliability for load rating. Live load factors in the AASHTO Manual for Bridge Evaluation (2011) are based on the traffic data available for the site. Dynamic load allowance (IM) is used to increase the applied static force effect to account for the dynamic interaction between the bridge and moving vehicles. Live load factor and dynamic load allowance vary in each level of load rating.

3.2.2 Material Mechanical Properties for Old Bridges

According to the AASHTO Manual for Bridge Evaluation (2011), Tables 3.8 to 3.10 can be used when properties of bridge materials are unknown. For prestressed concrete, the concrete compressive strength in Table 3.8 can be increased by 25%.

Table 3.8 Minimum Compressive Strength of Concrete by Year of Construction

Year of Construction	Compressive Strength, f'c, ksi
Prior to 1959	2.5
1959 and Later	3

Table 3.9 Yield Strength of Reinforcing Steel

Type of Reinforcing Steel	Yield Strength, f_y, ksi
Unknown steel constructed prior to 1954	33
Structural grade	36
Billet or intermediate grade, Grade 40, and unknown steel constructed during or after 1954	40
Rail or hard grade, Grade 50	50
Grade 60	60

Table 3.10 Tensile Strength of Prestressing Strand

Year of Construction	Tensile Strength, f_{pu}, ksi
Prior to 1963	232
1963 and Later	250

3.3 Field Testing of Bridges

The behavior of existing bridges can be investigated through two types of field testing: (1) long-term health monitoring, and (2) live load testing. Long-term health monitoring is used to record live load structural response (e.g. to random truck passage and wind gusts) and to monitor the bridge stiffness degradation to identify the deteriorating components. Live load (truck) testing is used to determine the live load response and the safe live load capacity of bridges. For load testing, loading may be static or dynamic by changing the speed of the test vehicle. Results of static and dynamic field testing for a bridge can be used to determine "load distribution factors" and "dynamic load allowance" specific to the test bridge (e.g. Seo et al., 2015).

3.3.1 Classification of Load Tests

Load testing is the observation of performance of a bridge under a controlled and predetermined load without affecting the bridge serviceability and performance. Generally, two types of load testing exist for bridges: (1) diagnostic test, and (2) proof test. Diagnostic tests are performed to evaluate the response of a bridge under the applied loads. The load transfer mechanism of the test bridge can be determined by installing strain and deflection sensors on structural members. Proof tests determine the maximum safe live load capacity of the test bridge. This is the only way to verify serviceability of distressed and aged bridges.

3.3.2 Type of Load Tests

Load testing can be further classified into either static or dynamic load testing. Static load testing is done using stationary or a slow-moving load (e.g. a truck passing the bridge with a speed of 5 mph), while a dynamic load test is performed using a time-varying load (e.g. a truck with a speed of 55 mph). Dynamic load allowance (IM) can be determined using these tests. Diagnostic load tests can be static or dynamic but proof load tests are usually performed with static loads.

3.3.3 Benefits of Load Tests

Load tests provide sufficient data to determine the safe live load capacity of old or distressed bridges. For some bridges, response of bridge members cannot be analytically determined because of a lack of sufficient information or detailing. Retrofitted or strengthened bridges also cannot be accurately load rated using analytical methods due to the unknown behavior of the various elements of the repaired bridge. In these cases, load testing can provide more realistic safe live load capacities than analytical methods.

3.3.4 Load Test Measurements

Various devices are usually used to measure strains, deflections, rotations, and dynamic characteristics of a bridge. Electrical resistance gauges, strain transducers or acoustic strain gauges can be used to measure strains of the test bridge. Linear Variable Differential Transformers (LVDT) can be used to measure relative deflections. Mechanical tilt meters installed on girder webs can measure rotations. Accelerometers can also be used in dynamic tests to determine dynamic characteristics of the test bridge such as modal frequencies, mode shapes, and damping ratios.

Before any field testing, a preliminary model can be developed to identify critical locations to place sensors. The use of strain transducers are required as the minimum for field testing; however, other devices can be installed to collect more information.

3.3.5 Bridge Load Testing in Literature

Several studies have performed bridge load testing: Nowak et al. (1996), Phares et al. (2005), Qiao (2012), Setty (2012), Schiff et al. (2006), Sanayei et al. (2015), Seo et al. (2015), and Hogan et al. (2016). Of these, the study by Setty (2012) was selected and summarized here to serve as an example.

Setty (2012) performed load testing on a 43-year old bridge with three 47.83-ft (14.58-m) equal-span prestressed concrete box beam bridge (Fig. 3.1) with a 15-degree skew. The bridge deck consisted of solid box girders with a height of 21 in. (584-mm) and a width of 36 in. (914-mm). Twenty-seven 3/8-in. (9.5-mm) diameter strands were used in each girder. Exterior beam concrete spalling, exposure of shear reinforcement and prestressing strands, and corrosion of exposed steel were reported in a pretest inspection.

Strain gauges and string potentiometers were installed at two sections of the bridge as shown in Fig. 3.2, which were selected to measure the maximum positive and negative moments in the west span. Thirty-six strain gauges and 16 string potentiometers were used. Four three-axle loaded trucks were placed over the bridge in eight different positions as shown in Fig. 3.3 as static testing. For dynamic testing, the heaviest truck available in the test was used with two speeds of 10 mph (16 kph) and 35 mph (56 kph).

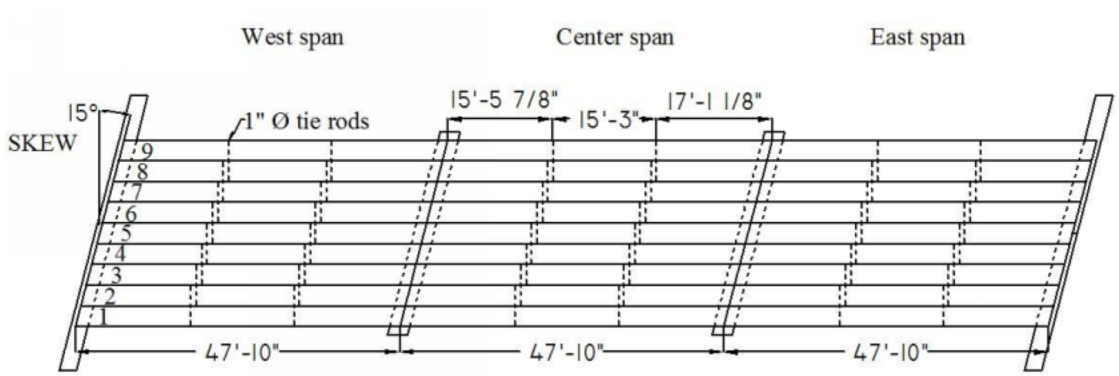


Figure 3.1 Plan View of Test Bridge (Setty, 2012)

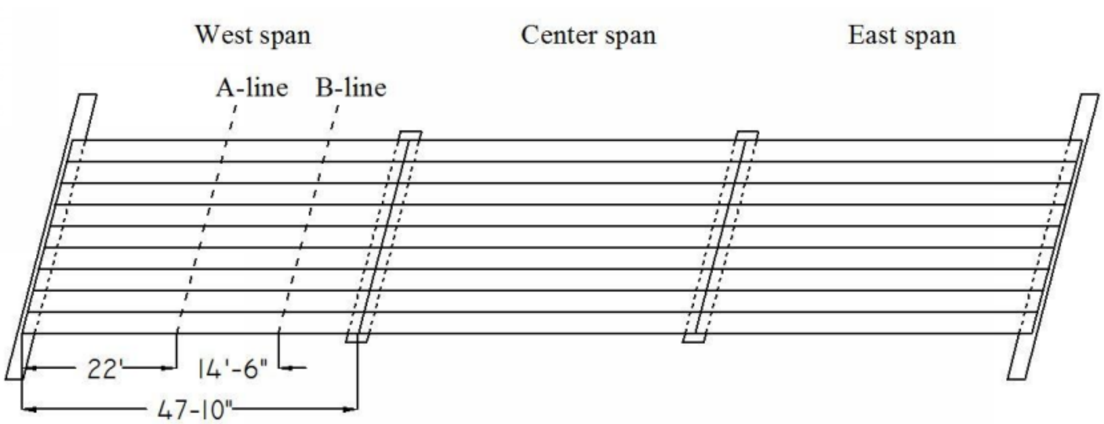
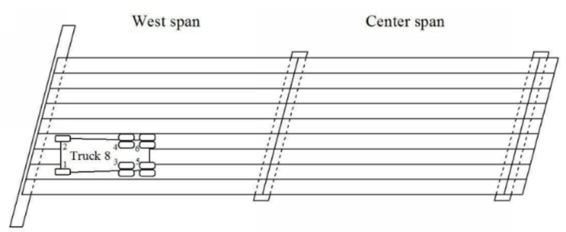
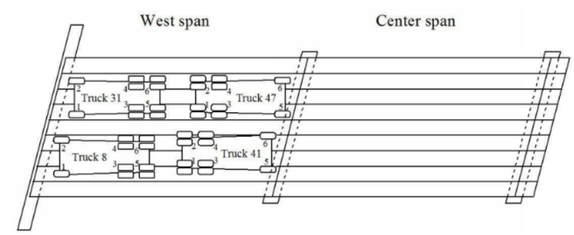


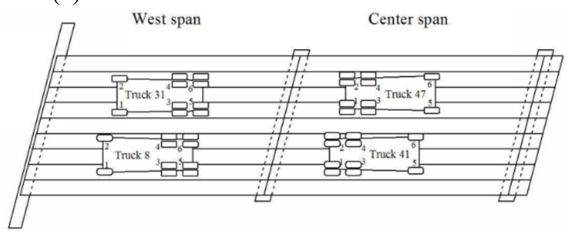
Figure 3.2 Sensor Locations (Setty, 2012)



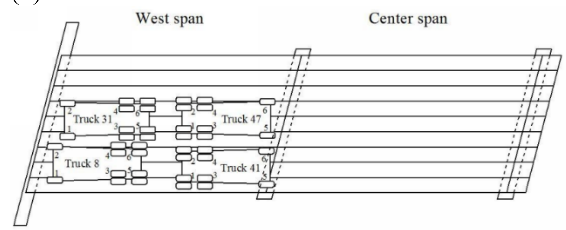
(a) Load Placement 1



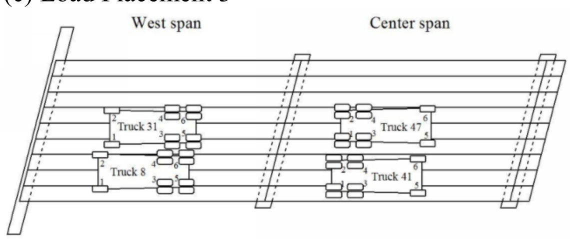
(b) Load Placement 2



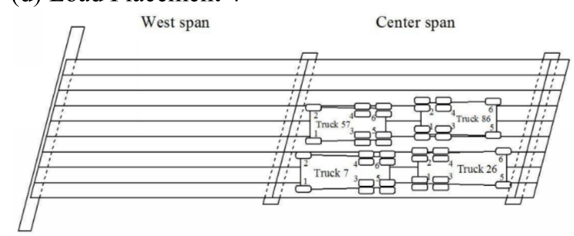
(c) Load Placement 3



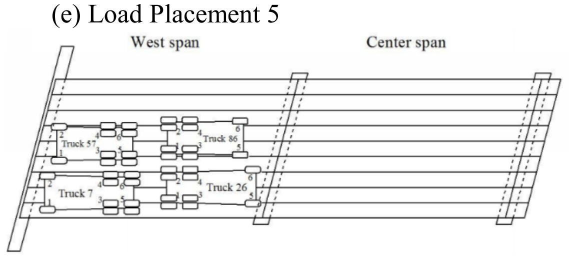
(d) Load Placement 4



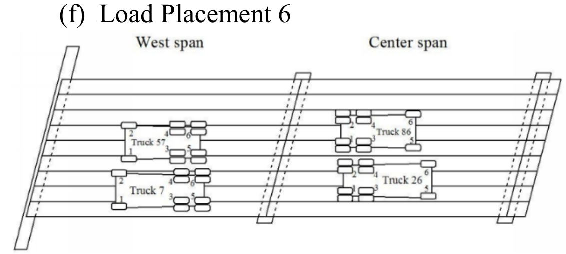
(e) Load Placement 5



(f) Load Placement 6



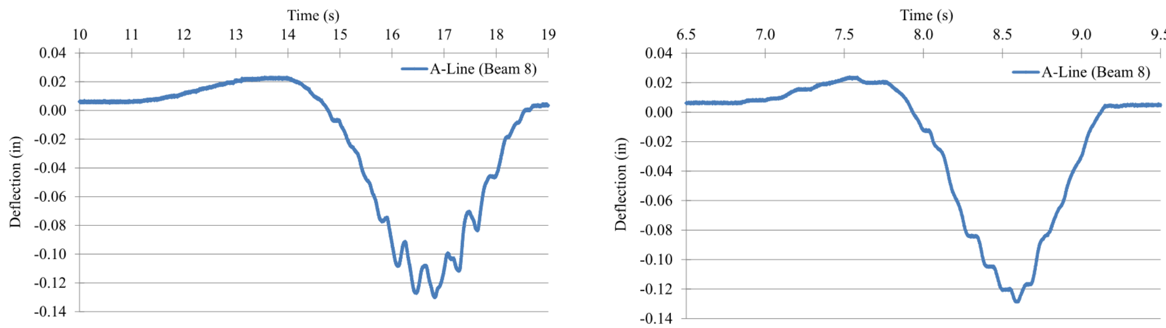
(g) Load Placement 7



(h) Load Placement 8

Figure 3.3 Placement of Trucks for Static Testing (Setty, 2012)

Dynamic load allowance (referred to as DLA in that study) was calculated using the maximum static and dynamic deflections. Figure 3.4 shows the A-Line dynamic response history for beam 8 (Fig. 3.1) with a speed of 10 mph (16 kph) and 35 mph (56 kph), respectively. It can be seen that the increase in the truck speed did not affect the maximum deflections. Figure 3.5 shows the maximum static and dynamic deflections for all beams. The dynamic load allowance calculated using the measured data was 1.10, which was less than the AASHTO LRFD value of 1.33 indicating that the AASHTO requirement was conservative.



(a) Dynamic Deflection (10 mph) (b) Dynamic Deflection (35 mph)

Figure 3.4 A-Line Dynamic Response for Beam 8 (Setty, 2012)

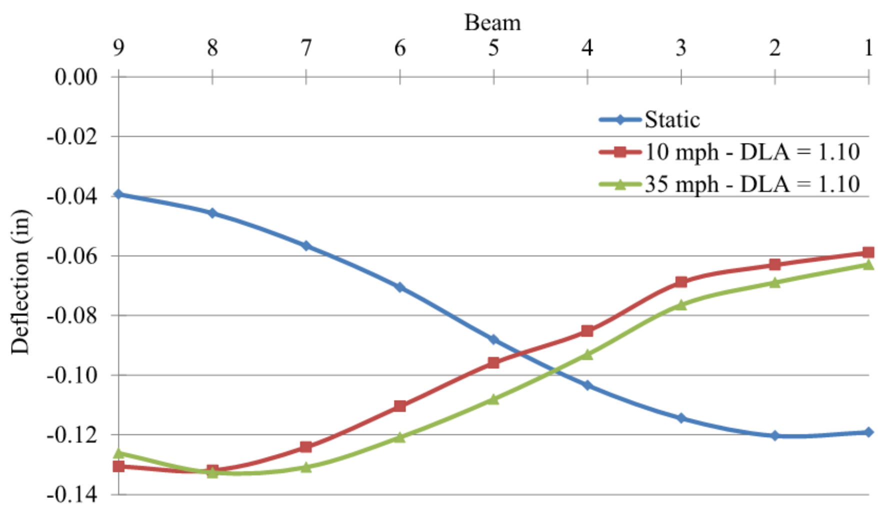


Figure 3.5 Dynamic and Static A-Line Deflections (Setty, 2012)

3.4 Damage Type and States for Bridge Elements

The AASHTO Manual for Bridge Element Inspection (2013) listed possible damages for different bridge elements. Each element and damage has a specific identification number in this manual. For example, the common damage seen in prestressed girders are:

- Delamination/Spall/Patched Area (1080)
- Exposed Rebar (1090)
- Exposed Prestressing (2200)
- Cracking (1110)
- Efflorescence/Rust staining (1120)
- Damage (7000)

This AASHTO manual also provides four damage states per damage type, which are usually defined using qualitative measures. For example, if the concrete spalling is less than 1-in. (25-mm) deep or 6 in. (150 mm) in diameter, the damage condition state is "Fair". When an exposed bar has measurable section loss without any warrant of structural review, which means a load rating is not required, the condition state is "Poor".

The AASHTO Manual for Bridge Element Inspection (2013) has been selected as the baseline to define damage types and condition states for double-tee bridges. However, the definitions were revised to be more quantitative rather than qualitative as discussed in next chapter.

3.5 Capacity of Aged Members

The nominal capacity of bridge members is calculated using the AASHTO LRFD (2014) methods. For example, the nominal flexural resistance for a prestressed flanged section is taken as:

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + 0.85f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \quad \text{(Eq. 3.5)}$$

where

M_n = The nominal moment capacity.

A_{ps} = The area of prestressing steel (in.^{2}).

f_{ps} = The average stress in prestressing steel at nominal bending resistance (ksi).

d_p = The distance from extreme compression fiber to the centroid of prestressing tendons (in.).

A_s = The area of nonprestressed tension reinforcement (in.^{2}).

f_s = The stress in the mild steel tension reinforcement at nominal flexural resistance (ksi).

d_s = The distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement.

A'_s = The area of compression reinforcement (in.^{2}).

d'_s = The distance from extreme compression fiber to the centroid of compression reinforcement (in.).

f'_c = The specified compressive strength of concrete at 28 days, unless another age is specified (ksi).

b = The width of the compression face of the member; for a flange section in compression.

b_w = The web width or diameter of a circular section (in.).

β_1 = The stress block factor specified in AASHTO LRFD Article 5.7.2.2.

h_f = The compression flange depth of an I or T member (in.).

a = cβ_1; The depth of equivalent stress block.

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(Eq. 3.6)}$$

where

f_{pu} = The specified tensile strength of prestressing strand (ksi).

$$k = 2(1.04 - \frac{f_{py}}{f_{pu}}) \quad \text{(Eq. 3.7)}$$

c = The distance between neutral axis and compression face as defined in Eq. 3.8.

f'_s = The stress in the mild steel compression reinforcement at nominal flexural resistance (ksi).

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{0.85 f_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}} \qquad \text{(Eq. 3.8)}$$

where, b is the width of compression flange.

AASHTO LRFD (2014) does not recognize any methods for the capacity estimation of damaged members. However, these methods might be valid for distressed members if sectional and material properties are modified to include the damage. To verify the AASHTO capacity estimation methods for salvaged girders, five studies were selected from the literature in which full-scale bridge girders — including one 48-year old 53-ft (16.15-m) long double-tee girder — were tested to failure (Table 3.11). It can be seen that using the measured material properties (with no sectional property modifications since the damage was not significant in these specimens), the calculated moment capacity was only 5.6% different than the measured moment capacity for all girders on average indicating that the current AASHTO methods are valid for aged girders. Nevertheless, full-scale strength testing of damaged double-tee girders is needed to verify these equations for girders with significant damages (Refer to Ch. 6).

Table 3.11 Measured and Calculated Flexural Capacities of Salvaged Bridge Girders

Reference	Section Type	Age (*yr*)	Span (*ft*)	Girder Damage Type	Width (*ft*)	Depth (*in.*)	f'_c (*ksi*)	f_y (*ksi*)	Measured Moment (*k ft*)	Calculated Moment (*k ft*)
Shenoy et al. (1991)	Box	27	54	Minor concrete cracking and spalling	36	27	7.1	150	936.9	987.21
Halsey et al. (1996)	Inverted Tee	40	29	Minor deterioration at the girder edges	12	12	11.79	260	353	339
Labia et al. (1997)	Box	20	70	No apparent damage	48	33	5.5	270	2520	2836
Eder et al. (2010)	I	50	45	Longitudinal cracks along post-tensioning tendons	16	40	9.8	150	1356	1500
Pettigrew et al. (2016)	Double-Tee	48	53	Deteriorated and exposure of rebar at some location	84	28	5.6	278	1134.6	1144

It is worth mentioning that the LRFR method in the AASHTO Manual for Bridge Evaluation (2011) uses three general condition factors to account for deterioration (Table 3.12). These factors are for whole superstructure not bridge elements. However, this method might be a viable technique to include the effect of damage types and condition states in the capacity calculation of double-tee girders. The modified capacity could then be used in load rating of damaged double-tee bridges.

Table 3.12 AASHTO LRFR Condition Factors

Good or Satisfactory	6 or higher	1.0
Fair	5	0.95
Poor	4 or lower	0.85

4. DAMAGE CATEGORIZATION FOR DOUBLE-TEE GIRDERS

Damage types and condition states for different bridge components were defined in the AASHTO Manual for Bridge Element Inspection (2015) and the South Dakota Bridge System Code Manual (BSCM, 1998). One example of damage type and condition states for prestressed girders according to the AASHTO manual was presented in Table 3.2. The AASHTO and South Dakota Department of Transportation (SDDOT) definition of condition states are general and mainly qualitative rather than quantitative. Nevertheless, more specific definition is needed to successfully relate visual distresses to load rating parameters.

4.1 Proposed Damage Types and Condition States for Double-Tee Girders

To minimize deviation from current codes, damage types and condition states for double-tee bridges were adopted from those presented in the AASHTO Manual for Bridge Element Inspection (2015) and the South Dakota Bridge System Code Manual (1998) for prestressed girders. Quantitative definitions were proposed for damage condition states specific to double-tee girders (Tables 4.1 and 4.2). One set of definitions was specific to the double-tee stem (Table 4.1) and another set of the definitions was for the double-tee flange (Table 4.2). This was done since the damage of the stem and flange may affect the shear and moment capacities in different ways.

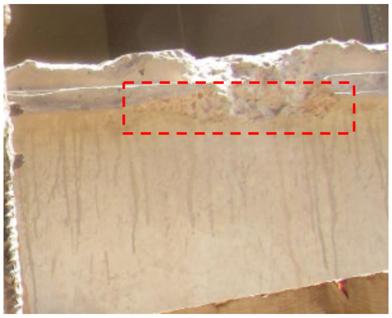
Figure 4.1 shows samples of damage types and condition states observed for double-tee bridges located in South Dakota. Identification of the damage types and condition states is expected to be straightforward with minimal variations when a bridge is inspected by different inspectors since the proposed definitions are mainly quantitative.

Table 4.1 Damage Types and Condition States for Prestressed Double-Tee Girder Stem

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Transverse Rebar	None	Minor corrosion of the reinforcement with minimal section loss.	Severe corrosion of only one leg of transverse reinforcement.	Severe corrosion of all legs of transverse reinforcement in a section.
Exposed Longitudinal Prestressing	None	50% section loss due to corrosion in the extreme tendon.	100% section loss due to corrosion in the extreme tendon.	Section loss due to corrosion in the two or more tendons.
Cracking	Insignificant cracks or moderate-width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.

Table 4.2 Damage Types and Condition States for Prestressed Double-Tee Girder Top Flange

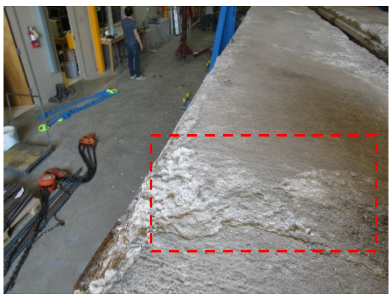
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area/Aberration	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Rebar	None	Minor corrosion of the outer layer of reinforcement with minimal section loss.	Severe corrosion of only the outer layer of reinforcement.	Severe corrosion of the outer and inner layers of reinforcement.
Cracking	Insignificant cracks or moderate width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.
Girder-to-Girder Longitudinal Joint Deterioration	None	Minimal deterioration, no sign of leakage.	Discrete signs of seepage along the joint, minor corrosion of steel plates.	Seepage along the joint, severe corrosion of steel plates.



(a) Stem Cover Deterioration (CS-2)

(b) Stem Cover Deterioration (CS-3)

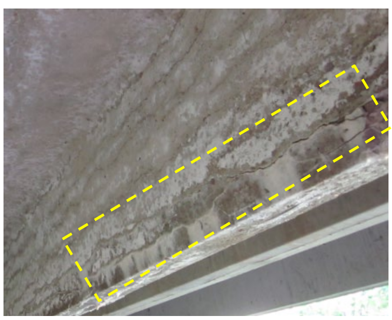
(c) Stem Cover Deterioration (CS-4)



(d) Flange Cover Deterioration (CS-2)

(e) Flange Cover Deterioration (CS-3)

(f) Flange Cover Deterioration (CS-4)

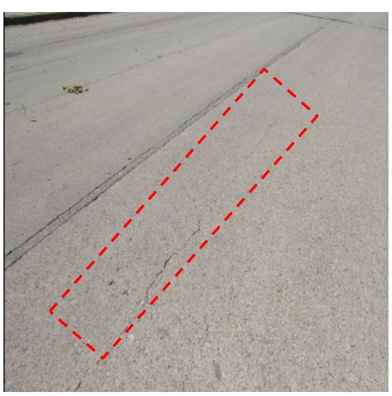


(g) Stem Cracking (CS-2)

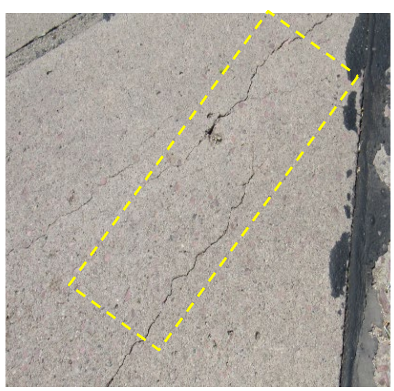
N/A

(h) Stem Cracking (CS-3)

(i) Stem Cracking (CS-4)



(j) Flange Cracking (CS-2)



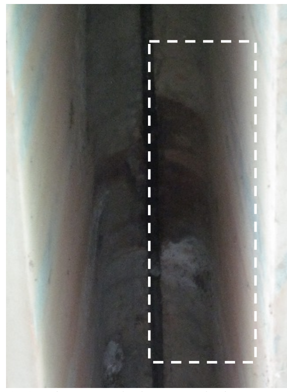
(k) Flange Cracking (CS-3)

N/A

(l) Flange Cracking (CS-4)

Figure 4.1 Sample Damage Types and Conditions States for Prestressed Double-Tee Girders

(m) Flange Girder to Girder Longitudinal Joint Deterioration (CS-2)



(n) Flange Girder to Girder Longitudinal Join Deterioration (CS-3)

N/A

(o) Flange Girder to Girder Longitudinal Joint Detrioration (CS-4)

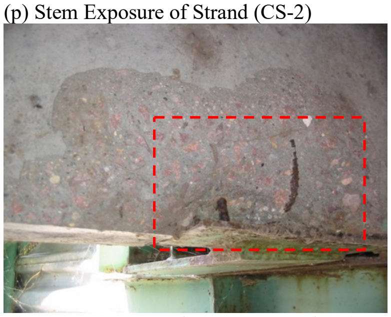
(p) Stem Exposure of Strand (CS-2)

N/A

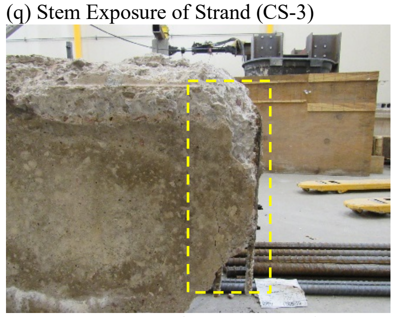
(q) Stem Exposure of Strand (CS-3)

N/A

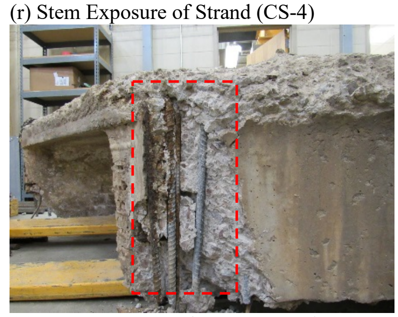
(r) Stem Exposure of Strand (CS-4)



(s) Stem Exposure of Transverse Rebar (CS-1)/Stem Cover Deterioration CS (4)



(t) Stem Exposure of Transverse Rebar (CS-2)



(u) Stem Exposure of Transvers Rebar (CS-3)

(v) Flange Exposure of Rebar (CS-1) & Flange Cover Deterioration (CS-4)

(w) Flange Exposure of Rebar (CS-2)

N/A

(x) Flange Exposure of Rebar (CS-3)

Figure 4.1 Continued

4.2 Damage Location

It is important to identify the location of each damage for successful load rating. Table 4.3 presents a matrix for double-tee bridge damages to be prepared by the field inspector for an accurate load rating.

Table 4.3 Damage Matrix for Prestressed Double-Tee Girder Bridges

Component	Damage Type	Damage Location	Condition State
Stem of Girder	Cover Damage	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.1)
Stem of Girder	Exposed Transverse Rebar	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.1)
Stem of Girder	Exposed Longitudinal Prestressing	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.1)
Stem of Girder	Cracking	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.1)
Flange of Girder	Cover Damage	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.2)
Flange of Girder	Exposed Rebar	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.2)
Flange of Girder	Cracking	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.2)
Girder to Girder Joint	Longitudinal Joint Deterioration	0, 0.25*L* or 0.5*L*	1, 2, 3, or 4 (Table 4.2)

Note: *L* is the bridge span length

4.3 Frequency of Damages for South Dakota Double-tee Bridges

SDDOT provided an extensive database of double-tee bridge inspection photographs and access to Bridge Management database (BrM). In addition, more than 375 inspection reports were collected from Brosz Engineering and Clark Engineering.

The inspection database was comprehensively reviewed to identify the frequency of each damage for South Dakota double-tee bridges using the proposed damage types and condition states (Tables 4.1 and 4.2). Tables 4.4 and 4.5 present a summary of the findings of the evaluation, which show that the most frequent double-tee stem damages are the cover deterioration and the cracking. Furthermore, the most common double-tee flange damages are the cover deterioration and girder-to-girder longitudinal joint deterioration.

Table 4.4 Frequency of Damage for South Dakota Double-Tee Girder Stem

Damage Type	Condition States				Total
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	
Cover Deterioration including Delamination/Spall/Patched Area	100	75	29	34	238
Exposed Transverse Rebar	3	1	0	0	4
Exposed Longitudinal Prestressing	4	2	1	1	8
Cracking	35	28	17	3	83

Table 4.5 Frequency of Damage for South Dakota Double-Tee Girder Top Flange

Damage Type	Condition States				Total
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	
Cover Deterioration including Delamination/Spall/Patched Area/Aberration	118	70	15	21	224
Exposed Rebar	1	1	0	0	2
Cracking	46	17	10	3	76
Girder-to-Girder Longitudinal Joint Deterioration	1	16	82	0	99

4.4 Frequency of Number of Spans and Span Length

Table 4.6 presents frequency of the span length and number of spans for South Dakota double-tee bridges, which their geometry was available in the inspection database. The most common double-tee bridges in South Dakota are single-span with a span length of 40 ft (12.19 m) to 60 ft (18.3 m).

Table 4.6 Frequency of Span Length and Number of Spans for South Dakota Double-Tee Bridges

Girder Span Length (ft)	Number of Spans				
	One	Two	Three	Four	Five
10 to 20	0	1	2	0	0
20 to 30	36	1	14	0	1
30 to 40	37	4	32	1	0
40 to 50	68	10	30	0	0
50 to 60	64	4	26	0	1
60 to 70	36	2	4	0	0
70 to 80	2	0	0	0	0
80 to 90	1	0	0	0	0
90 to 100	3	0	0	0	0
100 to 110	1	0	0	0	0

Note: 1 ft = 0.3048 m.

4.5 Frequency of Girder Depth

Table 4.7 presents frequency of the girder depth for South Dakota double-tee bridges for which data was available in the inspection database. It can be seen that the 23-in. (584-mm) deep double-tee girders have been used more often than 30-in. (762-mm) deep double-tee girders in this sample.

Table 4.7 Frequency of Girder Depth for South Dakota Double-Tee Bridges

Girder Depth, in. (mm)	Number of Bridges	Percentage
23 (584)	137	65%
30 (762)	74	35%

Note: The total number of double-tee bridges in which their depth was available in inspection reports was 211.

4.6 Frequency of Skewed Double-Tee Bridges

Table 4.8 presents the frequency of skewed double-tee bridges for which data was available. Non-skewed bridges have been used more frequently than skewed bridges in this sample.

Table 4.8 Frequency of Skewed Double-Tee Bridges in South Dakota

Girder End Geometry	Number of Bridges	Percentage
Non-Skewed	100	70%
Skewed	42	30%

Note: The total number of double-tee bridges in which their skew angle was available in inspection reports was 142.

Findings of the statistical analysis presented in this chapter were used to identify candidates for field and strength testing.

5. FIELD TESTING OF DOUBLE-TEE BRIDGES

Field testing is an important tool to evaluate the performance of old or deteriorated bridges. This is especially important because bridge live loads have been increasing in design codes (Nowak and Saraf, 1996). Furthermore, 25% of the nation's 600,000 bridges need rehabilitation, repair, or total replacement due to component deteriorations (FHWA-ABC, 2017). Field testing of old or distressed bridges provides insight on (1) how live loads are transferred through different elements, (2) whether a deficient bridge should be posted, repaired, or replaced, (3) what is the safe live load carrying capacity of a bridge, and (4) accuracy of analytical modeling methods.

The most common type of bridge on South Dakota local roads is a prestressed precast double-tee girder bridge. More than 700 of the bridges are currently in service in South Dakota. In this study, field testing was performed to determine the live load distribution factors and dynamic load allowance factors specific to South Dakota double-tee bridges.

5.1 Selection of Bridge Candidates for Field Testing

As was discussed in the previous chapter, SDDOT and two bridge engineering firms provided double-tee bridge inspection reports. The inspection database was reviewed to identify prevalence of damage, span length, and other parameters. The following criteria were used to identify bridge candidates for field testing:

1. The girder-to-girder longitudinal joints of bridge candidates should be deteriorated since this has the greatest effect on the live load distribution and demands in a double-tee bridge. More than 90 out of 375 double-tee bridges were identified exhibiting this type of damage (Table 4.5). The condition state for this damage type for 82 of these bridges were "poor" (or CS-3). No bridge was found with longitudinal joints that had a damage CS-4.
2. The bridge candidate should be single-span, and the span length should be between 40–60 ft (12.2–18.3 m) because this is the most common span length of the state double-tee bridges (Sec. 4.4).
3. The girder depth of bridge candidates can be either 23 in. (584 mm) or 30 in. (762 mm). However, at least one 23-in. (584-mm) deep girder bridge should be tested because they are more common than 30-in. (762-mm) deep girder bridges (65% versus 35%, Sec. 4.5).
4. The bridge candidate should be non-skewed, since 70% of the state double-tee bridges are non-skewed (Sec. 4.6).
5. Bridge candidates should be close to SDSU and a SDDOT facility.

Based on the above mentioned criteria, 10 double-tee bridges (Table 4.1) were identified as potential candidates for field testing. All 10 bridges (refer Appendix A for photographs) were inspected and a summary of the findings is presented in the table. Out of the 10 candidates, the SDDOT technical panel selected two bridges, Bridge 42-165-153 and 51-090-012, for field testing, which are highlighted in the table.

Table 5.1 Double-Tee Bridge Candidates for Field Testing

Bridge ID	County	Span Length and Depth	Damage Type and Condition State	Age, Yr.
31024230	Hanson, SD	40.8 ft (12.4 m) Seven 23-in (584-mm) Deep Girders	*Non-skewed,* Minor water leakage between deck units (with a condition state of Poor).	36
34075220	Hutchinson, SD	43 ft (13.1 m) Seven 23-in (584-mm) Deep Girders	*Non-skewed,* Light staining from leakage between longitudinal joints, spalling, and delamination. Only one longitudinal joint had water leakage after rain (with a condition state of poor).	37
34140033	Hutchinson, SD	100 ft (30.5 m) 3 span Eight 23-in (584-mm) Deep Girders	*Non-skewed,* Severe water leakage between all longitudinal joints after rain with minor corrosion of steel plates (with a condition state of poor).	39
42104110	Lincoln, SD	46 ft (14.02 m) Seven 30-in. (762-mm) Deep Girders	*Non-skewed, girders have transverse diaphragms,* Spalling of stem concrete cover (condition state not available), exposure of stem transverse reinforcement (with a condition state of severe), and leakage of girder-to-girder joints (with a condition state of poor).	35
42130065	Lincoln, SD	45.8 ft (13.9 m) Six 30-in. (762-mm) Deep Girders	*Non-skewed,* Spalling of both stem and flange concrete cover (with a condition state of fair and good, respectively), and leakage of girder-to-girder joints (with a condition state of poor).	40
42165153	Lincoln, SD	42 ft (12.8 m) Seven 30-in. (762-mm) Deep Girders	*Non-skewed,* Spalling of stem concrete cover (with a condition state of fair), and leakage of girder-to-girder joints (with a condition state of poor).	34
51008010	Moody, SD	50 ft (15.24 m) Six 23-in (584-mm) Deep Girders	*Non-skewed,* Spalling with exposed rebar, efflorescence and water staining between the deck units due to leaking of the joints.	40
51090012	Moody, SD	50 ft (15.24 m) Eight 23-in. (584-mm) Deep Girders	*Non-skewed,* Water leakage between all deck units, stains from minor corrosion of steel plates in longitudinal joints (with a condition state of poor), concrete spalling (with a condition state of fair).	38
51140067	Moody, SD	51.2 ft (15.6 m) Seven 23-in. (584-mm) Deep Girders	*Skewed bridge, girders have transverse diaphragms,* Minor water leakage between deck units but with no sign of corrosion of steel plates (with a condition state of poor).	8
51142060	Moody, SD	50 ft (15.24 m) Six 23-in. (584-mm) Deep Girders	*Posted bridge, non-skewed,* Staining and water leakage between the all deck units.	40

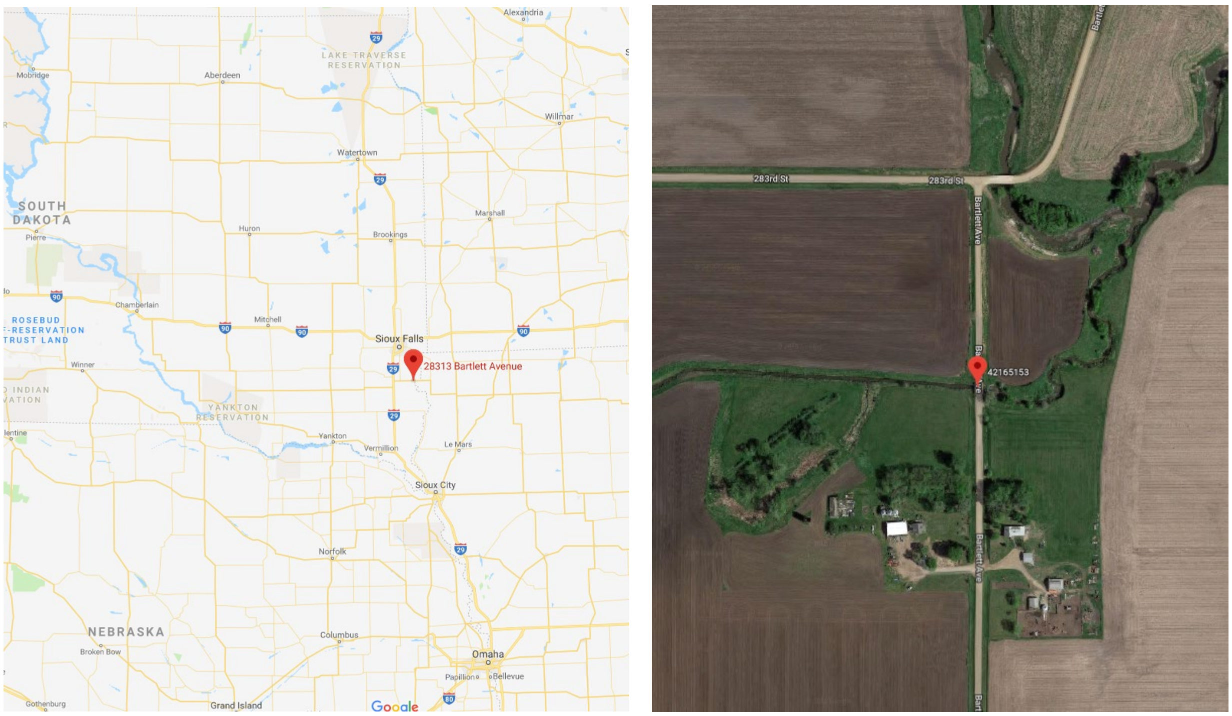
Note: The bridge age was by 2018.

5.2 Description of Double-Tee Field Test Bridges

This section presents the site location, geometry, and observed damage for each selected field test bridge.

5.2.1 Description of 30-in. (762-mm) Deep Double-Tee Girder Bridge

Bridge 42-165-153 is a single-span 34-year old structure with a span length of 42 ft (12.8 m) and a girder depth of 30 in. (762 mm). The bridge is located in Lincoln County, SD, on Barlett Avenue, 1.3 miles south of Canton, SD, (Fig. 5.1). Figures 5.2 and 5.3 show the photographs of the bridge, and Fig. 5.4 shows the observed damage of the bridge girder-to-girder joints in a plan view.



(a) Bridge Location in the State of South Dakota (b) Aerial View

Figure 5.1 Bridge 42-165-153 Located in Lincoln County, SD

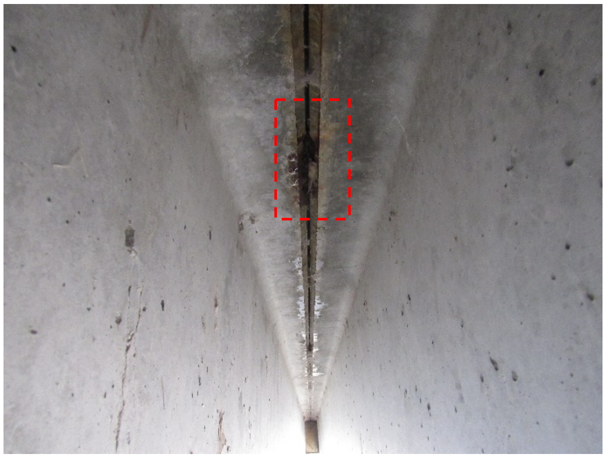
(a) Alignment Facing North

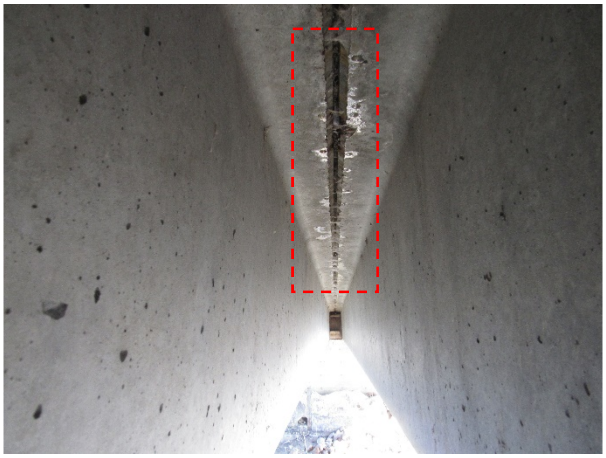

(b) Alignment Facing South

Figure 5.2 Top View of Bridge 42-165-153


(a) Efflorescence in Joint


(b) Spalling at bottom of Stem, G4


(c) Corrosion of Steel Plate


(d) Leakage in Joint


(e) Underneath of Bridge

Figure 5.3 Observed Damage of Field Test Bridge 42-165-153

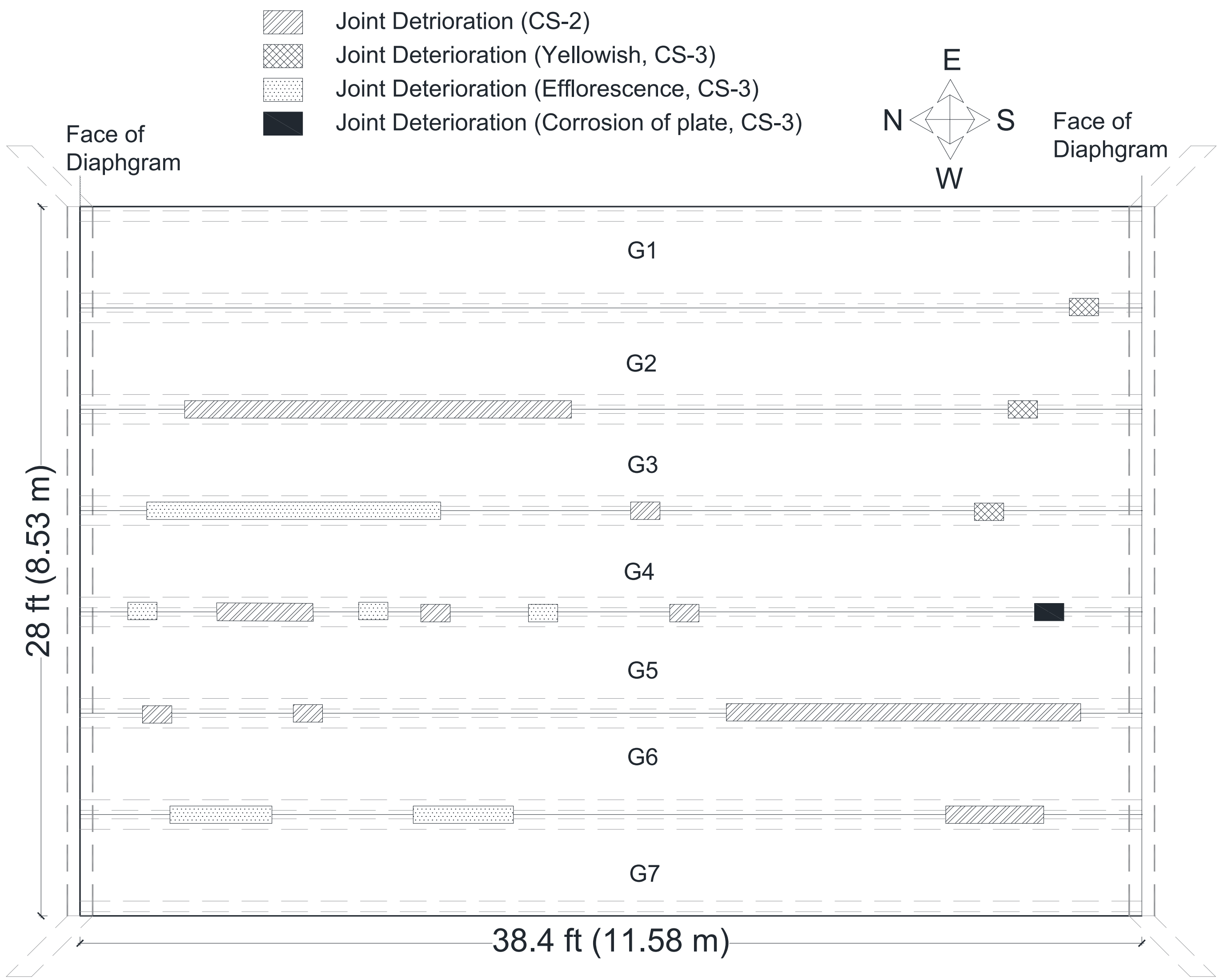


Figure 5.4 Observed Longitudinal Joint Damage of Field Test Bridge 42-165-153

5.2.2 Description of 23-in. (584-mm) Deep Double-Tee Girder Bridge

Bridge 51-090-012 is a single-span 38-year old structure with a span length of 50 ft (15.24 m) and a girder depth of 23 in. (584 mm). The bridge is located in Moody County, SD, on 475th Avenue, 1.8 miles north and 12 miles west of Ward, SD. Figures 5.5 through 5.7 show the photographs of the bridge and Fig. 5.8 shows the observed damage of the bridge girder-to-girder joints in a plan view.

(a) Bridge Location in the State of South Dakota (b) Aerial View

Figure 5.5 Bridge 51-090-012 Located in Moody County, SD

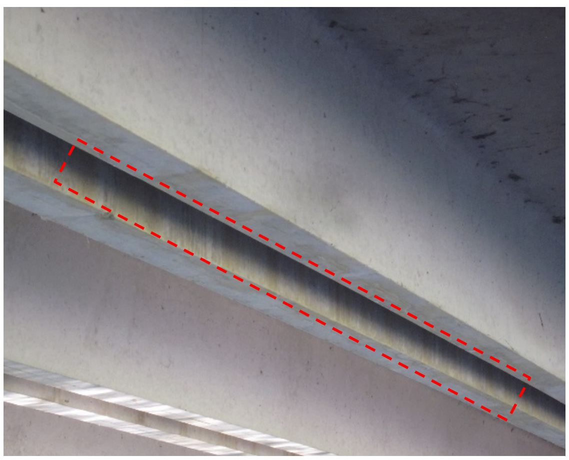
(a) Alignment Facing North


(b) Alignment Facing South

Figure 5.6 Top View of Bridge 51-090-012


(a) Underneath of Bridge


(b) Stains from Minor Corrosion of Steel Plates


(c) Sign of Water Leak b/w Deck Units


(d) Concrete Spalling at Railing

Figure 5.7 Observed Damage of Field Test Bridge 51-090-012

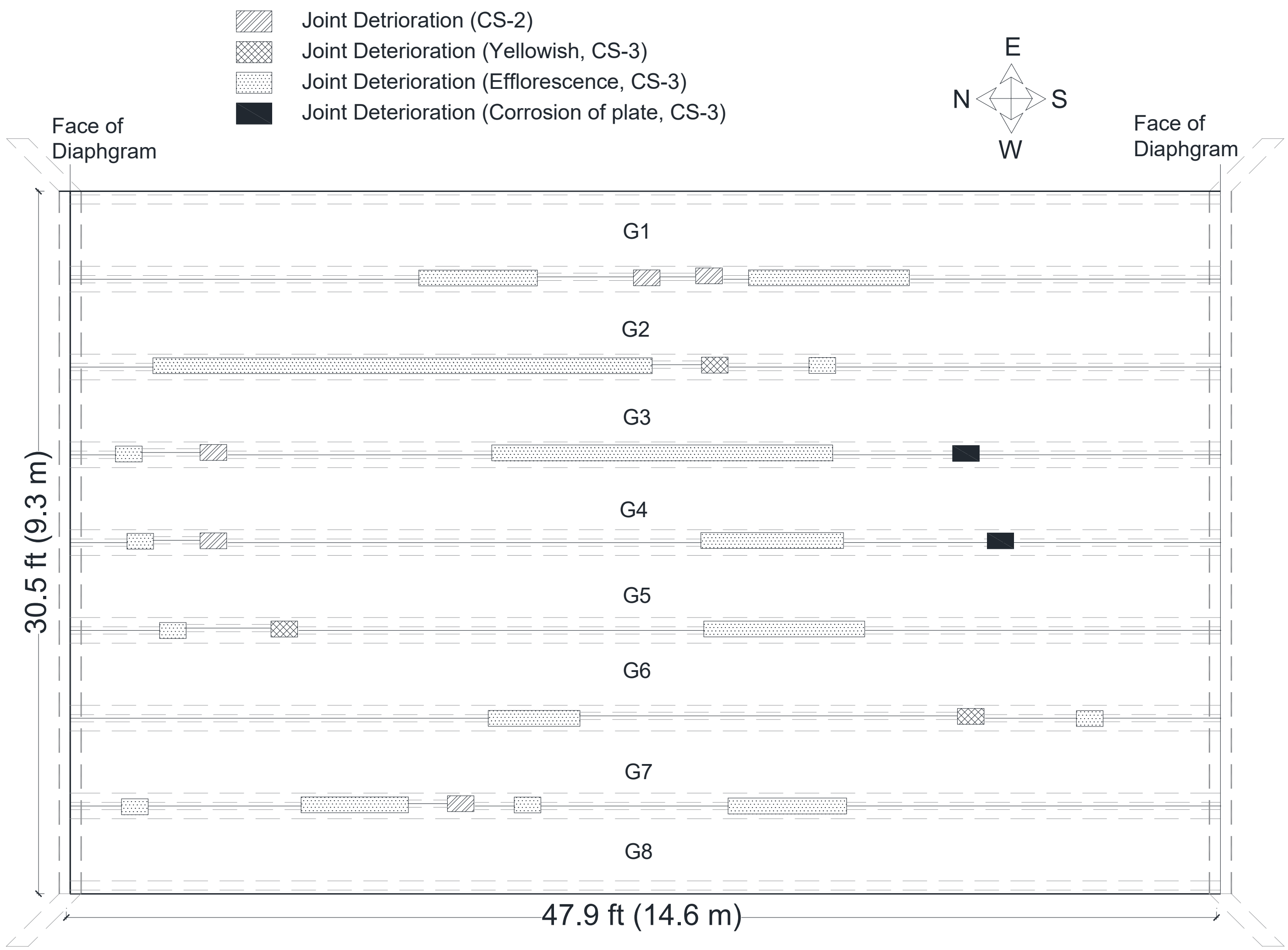


Figure 5.8 Observed Longitudinal Joint Damage of Field Test Bridge 51-090-012

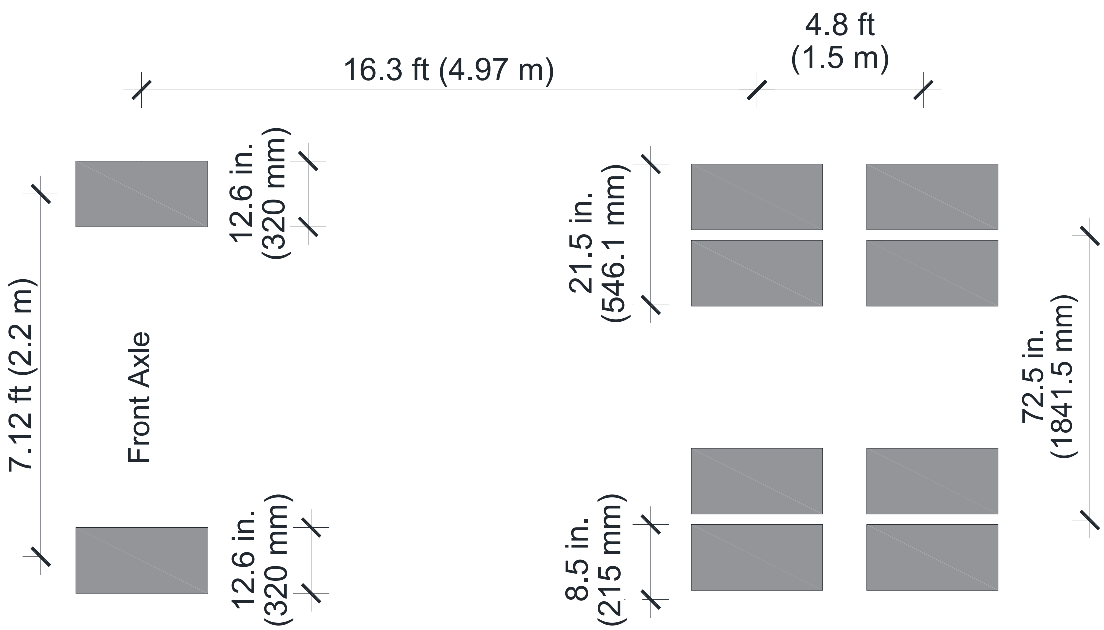
5.3 Field Testing Protocols for Double-Tee Bridges

This section presents field test loading protocols used to measure static and dynamic response of the test bridges, and determine the girder load distribution factors and dynamic load allowance factors. The test truck type and speed, loading paths, and the testing matrix were discussed herein.

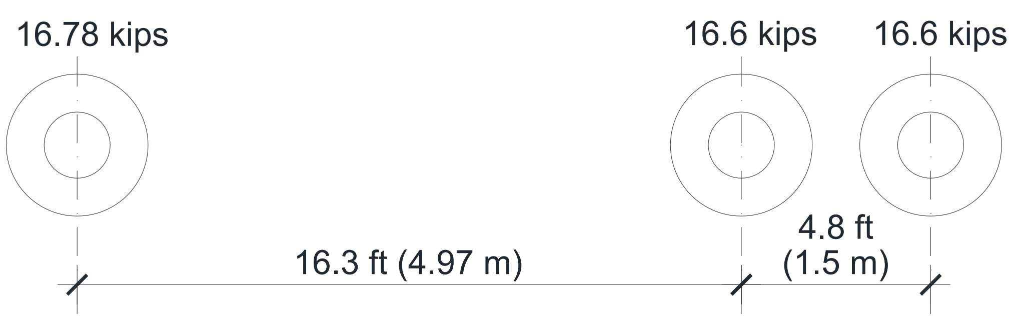
5.3.1 Field Test Truck

Both bridges were tested using a dump truck similar to SD Legal Truck Type 3 (Fig. 5.9). The test truck, which was loaded with dry sands, had a total weight of 49.98 kips (222.32 kN). The front axle weight was 16.78 kips (74.6 kN) and each rear axle weight was 16.6 kips (71.6 kN). The transverse axle spacing between the front tires was approximately 7 ft (2.1 m) and the transverse axle spacing between the centers of the rear tires was approximately 6 ft (1.8 m). The spacing between the front and the closest rear wheels was approximately 16 ft (4.9 m).

(a) Test Truck Used in Field Testing



(b) Test Truck Axle Spacing



(c) Test Truck Axle Weight Distribution

Figure 5.9 Field Test Truck

5.3.2 Test Truck Speed

For bridge field testing, a truck speed of 5 mph (miles per hour, or 8.05 kph) or less is usually considered a "static" test, and a truck speed of 55 mph (88.51 kph) is considered a "dynamic" test (Chajes et al., 2000). The same speed was initially adopted in the present study for the static and dynamic testing of the two bridges. After the dynamic testing of the 30-in. (762-mm) deep double-tee girder bridge for shear responses, the speed of dynamic tests for flexural responses was reduced to 35 mph (56.33 kph) due to the site conditions (gravel roads) and safety of the crew and bridge. The data collected from static tests was used to calculate the girder distribution factors and data obtained from the dynamic tests was used to calculate dynamic load allowance.

The truck driver was instructed to drive at the specified speed on specified load paths as discussed in the next section. The paths were marked on the bridge. Data was collected just before the test truck hit the bridge and ended when the truck had completely passed the bridge.

5.3.3 Field Testing Loading Paths

A proper selection of load paths is essential for successful field testing. The bridge geometry, such as the width and the number of girders, affects the selection of load paths. or field testing of double-tee bridges in the present study, five different paths were selected as shown in Fig. 5.10 and 5.11 to investigate the load transfer mechanism in both bridges. These paths were selected in a way that any girder of the test bridge was loaded at least once. All five paths were marked on the bridge with spray paint as shown in Fig. 5.12 and 5.13. Testing was repeated twice per path to minimize the measurement errors.

The exterior paths, Paths A and E, had a 2-ft (0.61-m) clearance from the railing per the AASHTO requirements for the calculation of live load distribution factors for the exterior girders. Due to a narrower width of the gravel road compared to the width of the test bridges, only static tests could be performed on the exterior paths.

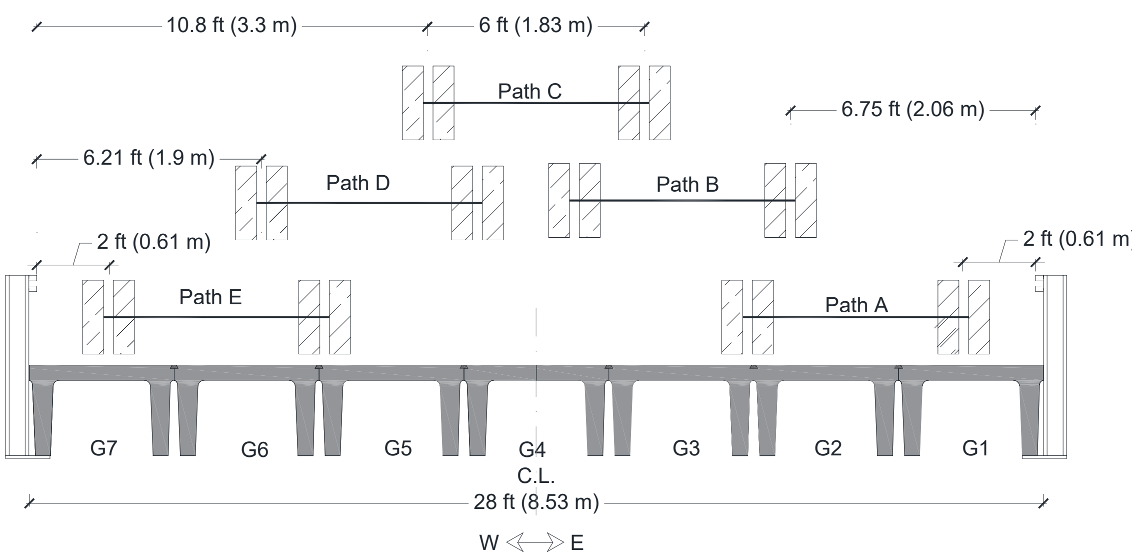


Figure 5.10 Field Test Truck Paths for 30-in. (762-mm) Deep Double-Tee Girder Bridge

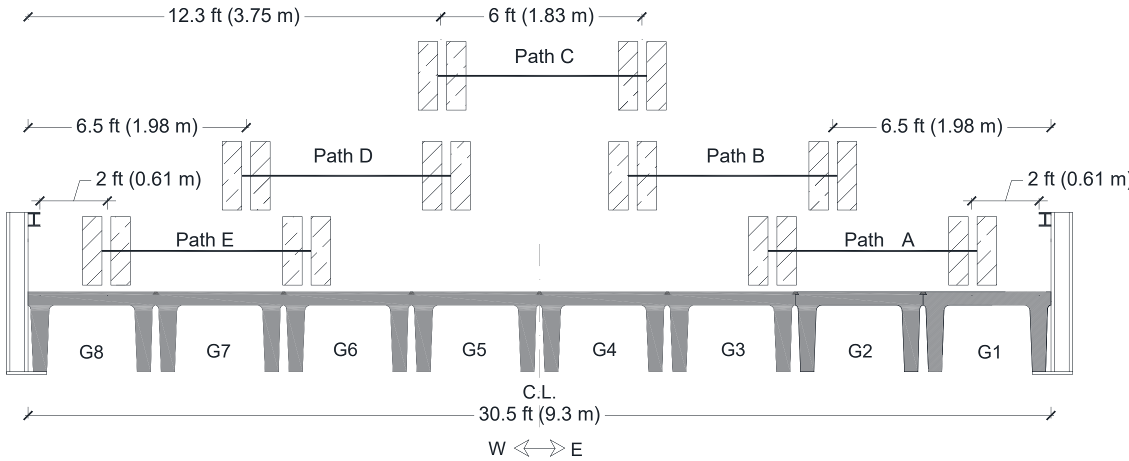


Figure 5.11 Field Test Truck Paths for 23-in. (584-mm) Deep Double-Tee Girder Bridge

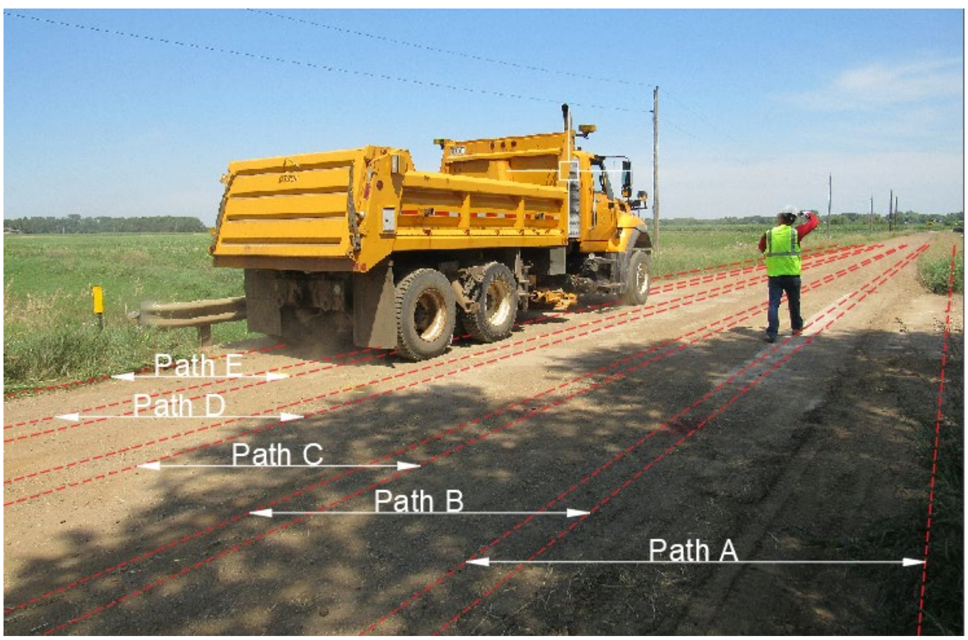


Figure 5.12 Photograph of Truck Paths for 30-in. (762-mm) Deep Double-Tee Girder Bridge

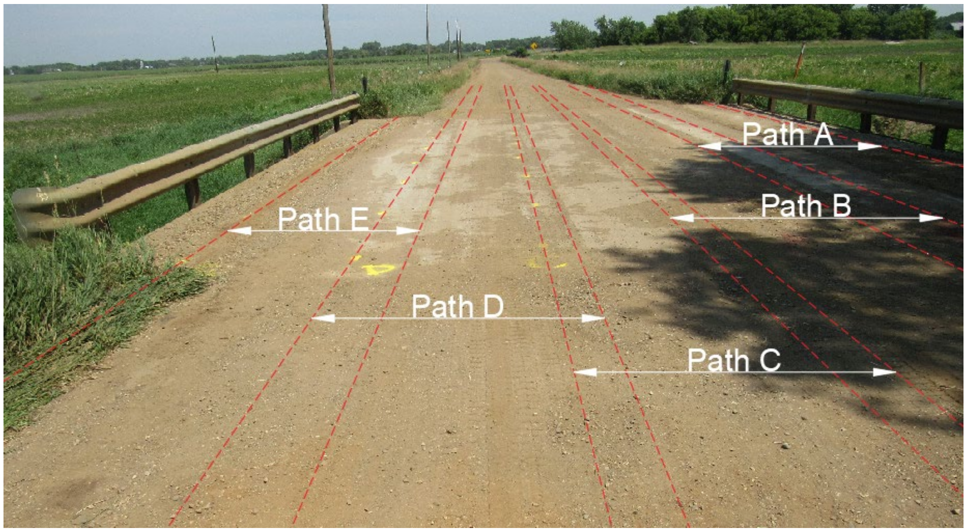


Figure 5.13 Photograph of Truck Paths for 23-in. (584-mm) Deep Double-Tee Girder Bridge

5.3.4 Bridge Field Testing Matrix

Tables 5.2 and 5.3 respectively present the field test matrices designed to obtain the flexural and shear response of the 30-in. (762-mm) deep double-tee girder bridge. Table 5.4 presents the field test matrix for measuring the flexural response of the 23-in. (584-mm) deep double-tee girder bridge. In these test matrices, letters "A, B, C, D and E" refer to the five different loading paths, and the term "St" refers to the static testing and the term "Dy" refers to the dynamic testing. For example, "A-St-1" under the "Test ID" column refers to the first run of the static test on Path A, while "B-Dy-2" refers to the second run of the dynamic test on Path B.

Due to instrumentation limitations, the 30-in. (762-mm) deep double-tee girder bridge was tested once for flexural response (gauges at the midspan) and another time for shear response (gauges close to one of the abutments). The 23-in. (584-mm) deep double-tee girder was tested only for flexural response. The next section discusses the field testing instrumentation plans.

Table 5.2 Field Test Matrix Measuring Flexural Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge

Test No.	Test ID	Path	Loading Type	Truck	Run No.	Measured Speed, mph (kph)
T1	A-St-1	A	Static	SD Type 3	1	5 (8)
T2	A-St-2	A	Static	SD Type 3	2	5 (8)
T3	B-St -1	B	Static	SD Type 3	1	5 (8)
T4	B-St -2	B	Static	SD Type 3	2	5 (8)
T5	B-Dy-1	B	Dynamic	SD Type 3	1	35 (56)
T6	B-Dy-2	B	Dynamic	SD Type 3	2	35 (56)
T7	C-St-1	C	Static	SD Type 3	1	5 (8)
T8	C-St-2	C	Static	SD Type 3	2	5 (8)
T9	C-Dy-1	C	Dynamic	SD Type 3	1	35 (56)
T10	C-Dy-2	C	Dynamic	SD Type 3	2	34.5 (55)
T11	D-St-1	D	Static	SD Type 3	1	5 (8)
T12	D-St-2	D	Static	SD Type 3	2	5 (8)
T13	D-Dy-1	D	Dynamic	SD Type 3	1	33.5 (54)
T14	D-Dy-2	D	Dynamic	SD Type 3	2	34.5 (55)
T15	E-St-1	E	Static	SD Type 3	1	5 (8)
T16	E-St-2	E	Static	SD Type 3	2	5 (8)

Note: This test was performed after to the shear response test (next table). No dynamic test was performed on Paths A & E due to site conditions. A speed gun was used to measure the test truck speed.

Table 5.3 Field Test Matrix Measuring Shear Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge

Test No.	Test ID	Path	Loading Type	Truck	Run No.	Measured Speed, mph (kph)
T21	A-St-1	A	Static	SD Type 3	1	5 (8)
T22	A-St-2	A	Static	SD Type 3	2	5 (8)
T23	B-St-1	B	Static	SD Type 3	1	5 (8)
T24	B-St-2	B	Static	SD Type 3	2	5 (8)
T25	B-Dy-1	B	Dynamic	SD Type 3	1	55 (88)
T26	B-Dy-2	B	Dynamic	SD Type 3	2	55 (88)
T27	C-St-1	C	Static	SD Type 3	1	5 (5)
T28	C-St-2	C	Static	SD Type 3	2	5 (5)
T29	C-Dy-1	C	Dynamic	SD Type 3	1	51 (82)
T30	C-Dy-2	C	Dynamic	SD Type 3	2	55 (88)
T31	D-St-1	D	Static	SD Type 3	1	5 (8)
T32	D-St-2	D	Static	SD Type 3	2	5 (8)
T33	D-Dy-1	D	Dynamic	SD Type 3	1	55 (88)
T34	D-Dy-2	D	Dynamic	SD Type 3	2	57 (92)
T35	E-St-1	E	Static	SD Type 3	1	5 (8)
T36	E-St-2	E	Static	SD Type 3	2	5 (8)

Note: This test was performed prior to the flexural response test (previous table). No dynamic test was performed on Paths A & E due to site conditions. A speed gun was used to measure the test truck speed.

Table 5.4 Field Test Matrix Measuring Flexural Response of 23-in. (584-mm) Deep Double-Tee Girder Bridge

Test No.	Test ID	Path	Loading Type	Truck	Run No.	Measured Speed, mph (kph)
T1	A-St-1	A	Static	SD Type 3	1	3 (4.8)
T2	A-St-2	A	Static	SD Type 3	2	3 (4.8)
T3	B-St -1	B	Static	SD Type 3	1	3 (4.8)
T4	B-St -2	B	Static	SD Type 3	2	3 (4.8)
T5	B-Dy-1	B	Dynamic	SD Type 3	1	36.7 (59)
T6	B-Dy-2	B	Dynamic	SD Type 3	2	35 (56)
T7	C-St-1	C	Static	SD Type 3	1	3 (4.8)
T8	C-St-2	C	Static	SD Type 3	2	3 (4.8)
T9	C-Dy-1	C	Dynamic	SD Type 3	1	36.5 (59)
T10	C-Dy-2	C	Dynamic	SD Type 3	2	35.6 (57)
T11	D-St-1	D	Static	SD Type 3	1	5 (8)
T12	D-St-2	D	Static	SD Type 3	2	5 (8)
T13	D-Dy-1	D	Dynamic	SD Type 3	1	29 (47)
T14	D-Dy-2	D	Dynamic	SD Type 3	2	29 (47)
T15	E-St-1	E	Static	SD Type 3	1	5 (8)
T16	E-St-2	E	Static	SD Type 3	2	5 (8)

Note: No dynamic test was performed on Paths A & E due to site conditions. A speed gun was used to measure the test truck speed.

5.4 Instrumentation Plans

This section presents instrumentation plans used for field testing of the two double-tee bridges. Only surface-mount strain transducers produced by Bridge Diagnostics, Inc. (BDI Model ST350) were used.

5.4.1 Instrumentation of 30-in. (762-mm) Deep Double-Tee Girder Bridge

For the 30-in. (762-mm) deep double-tee girder bridge, static and dynamic tests were performed to measure shear and flexural response of the bridge. For the shear response test, 24 strain gauges were installed 30 in. (762 mm) from the face of the south end diaphragm (Fig. 5.14 to 5.16). Pairs of strain gauges were installed at a 15.7-degree angle from the horizon 21 in. (533 mm) from the bottom of the stem (Fig. 5.16). To help with installation, a longitudinal line was drawn at a height of 21 in. (533 mm) from the bottom of stem and other two lines were drawn at 15.7 degrees from the longitudinal line. The two inclined lines met at a point 30 in. (762 mm) away from the south end diaphragm, as shown in Fig. 5.17.

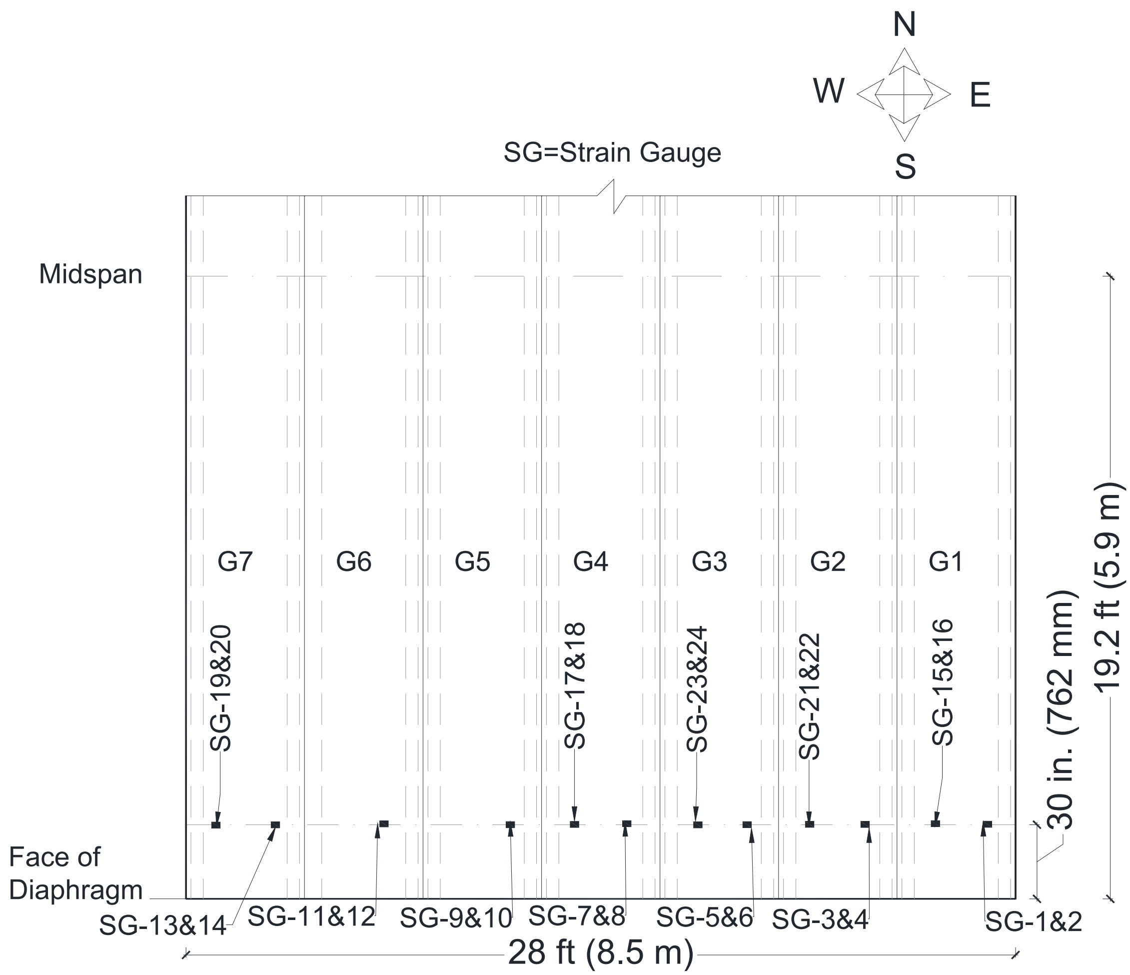


Figure 5.14 Strain Gauge Instrumentation Plan for Shear Response of 30-in. Deep Double-Tee Girder Bridge – Plan View

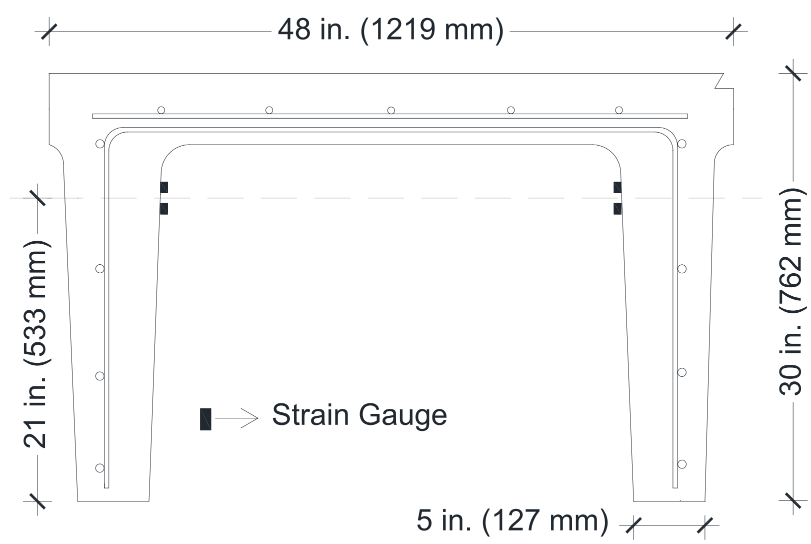


Figure 5.15 Girder Strain Gauge Instrumentation Plan for Shear Response of 30-in. Deep Double-Tee Girder Bridge – Section View

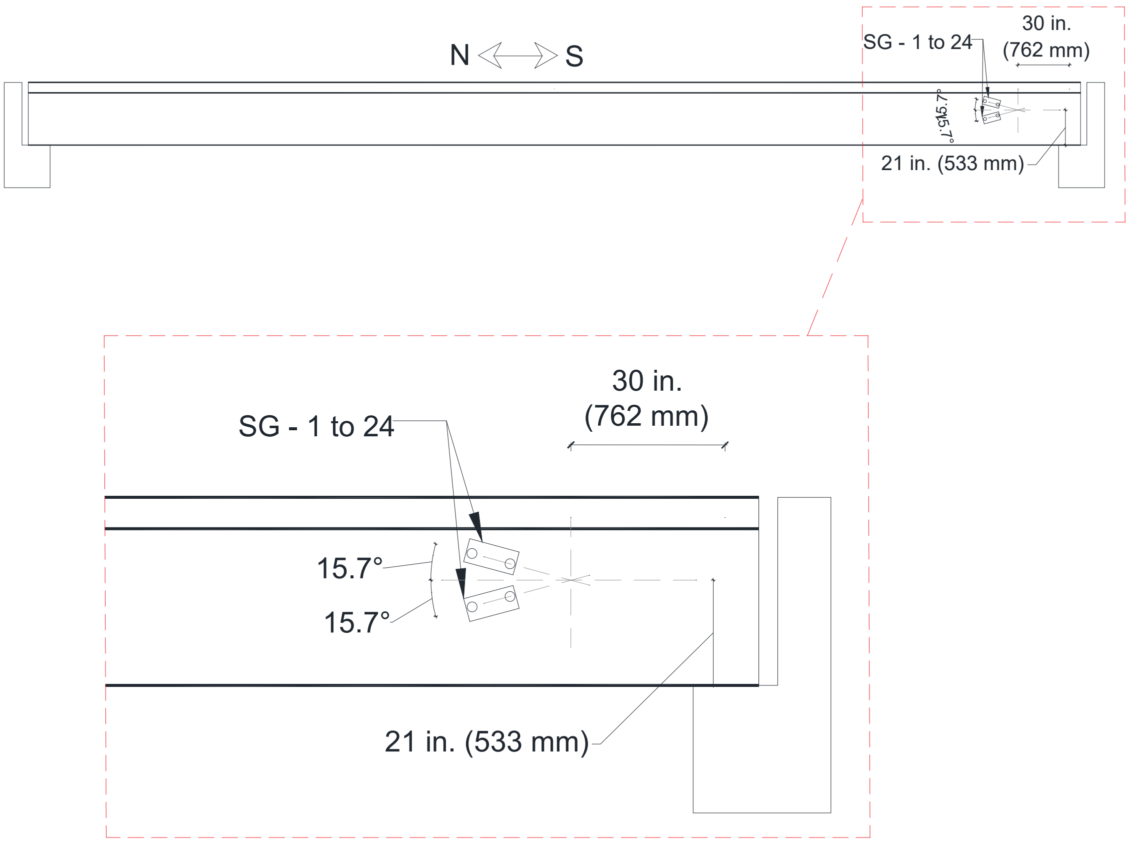
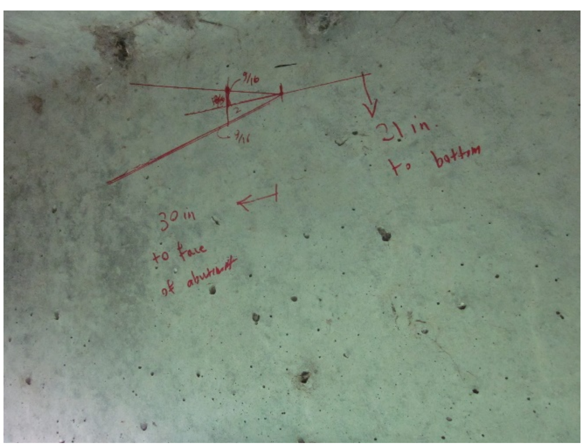
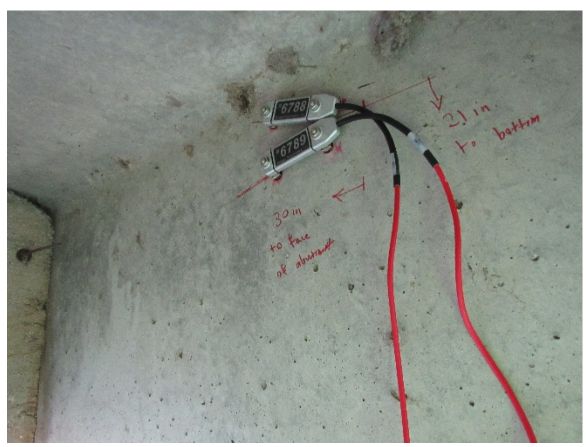


Figure 5.16 Girder Strain Gauge Instrumentation Plan for Shear Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge – Elevation View



(a) Lines for Gauge Installation



(b) Strain Gauge Installation

Figure 5.17 Strain Gauge Installation for Shear Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge

After completion of the shear tests, the strain gauges were removed. Subsequently, 14 strain gauges, each with a 12-in. (305-mm) extension, were installed at the bottom of all stems at the midspan as shown in Fig. 5.18 to 5.20. If the stem bottom face was damaged or the railing connection was at the midspan (Fig. 5.21), the strain gauge (SG-1, SG-8, and SG-14) was installed at the stem side at a distance of 1.25 in. (31 mm) from the bottom of the stem.

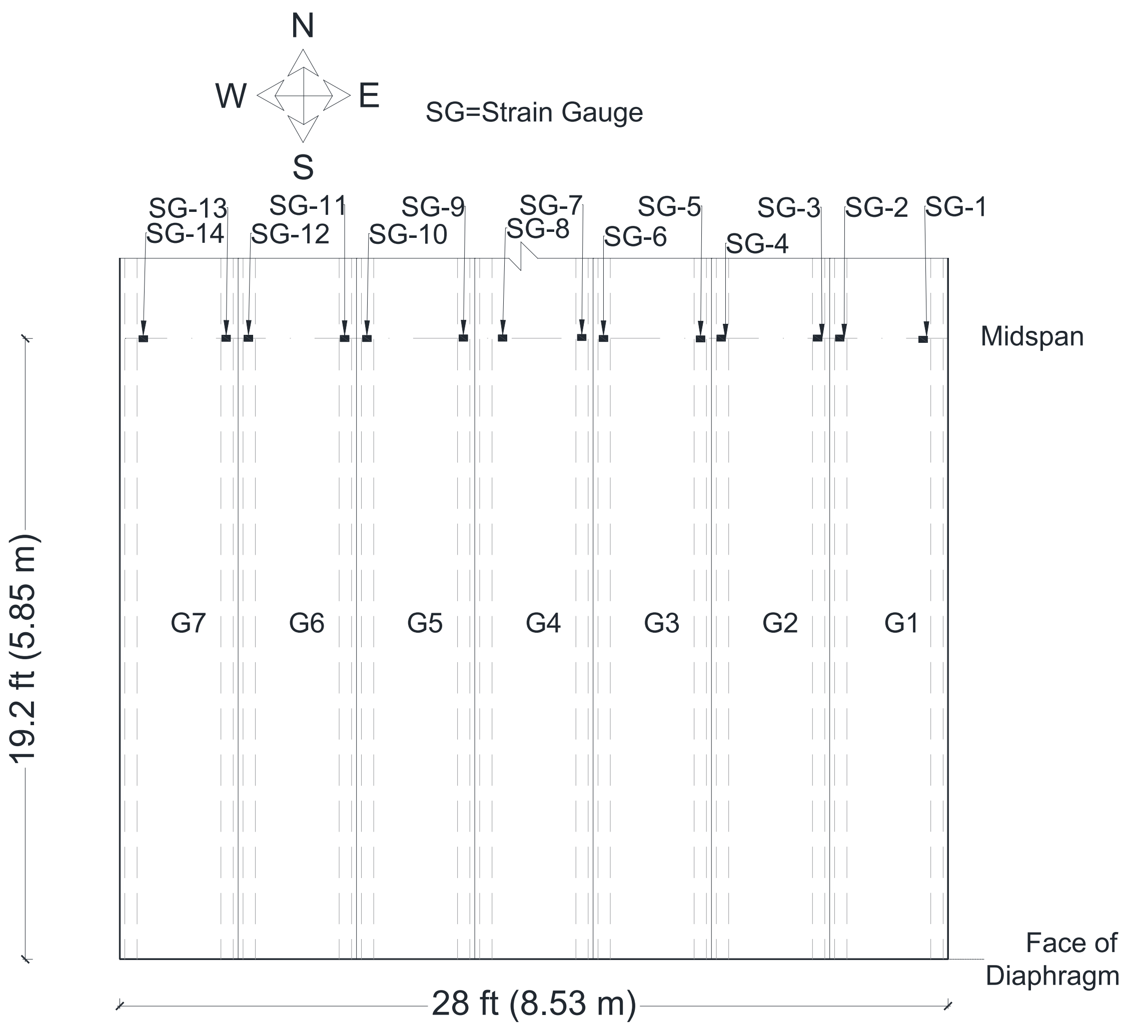


Figure 5.18 Strain Gauge Instrumentation Plan for Flexural Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge – Plan View

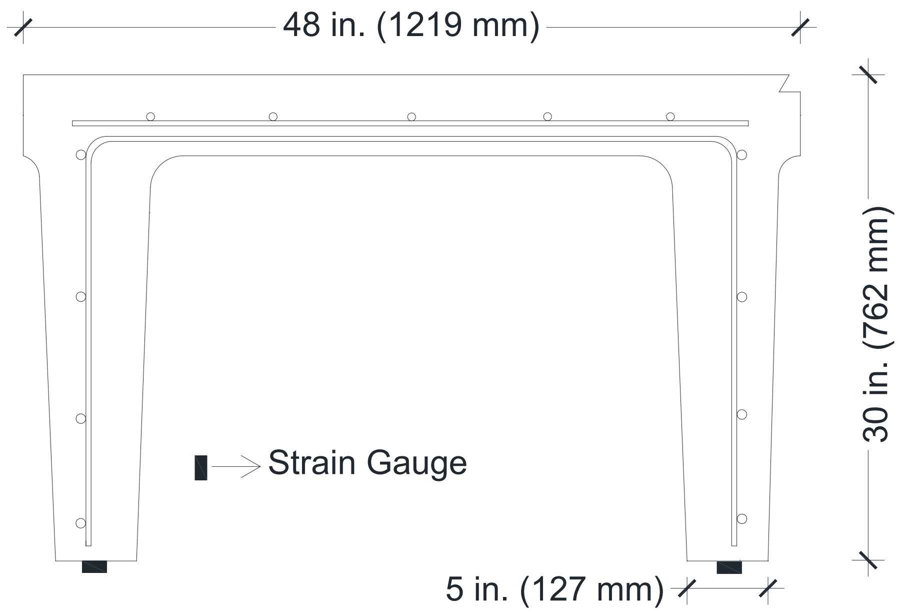


Figure 5.19 Girder Strain Gauge Instrumentation Plan for Flexural Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge – Section View

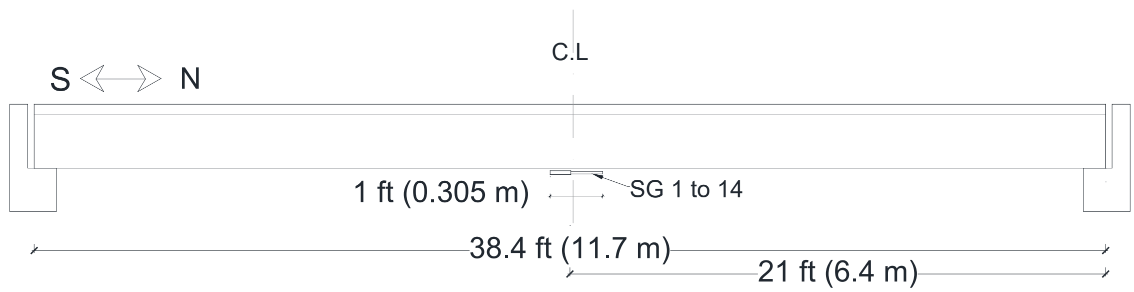


Figure 5.20 Girder Strain Gauge Instrumentation Plan for Flexural Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge – Elevation View

(a) Installation of Strain Gauges with Extension

(b) Strain Gauges at Side due to Railing

(c) Strain Gauges on Stem Side due to Damage

(d) Bridge Underneath View

Figure 5.21 Strain Gauge Installation for Flexural Response of 30-in. (762-mm) Deep Double-Tee Girder Bridge

5.4.2 Instrumentation of 23-in. (584-mm) Deep Double-Tee Girder Bridge

The instrumentation plan for the 23-in. (584-mm) deep double-tee girder bridge was initially the same as the 30-in (762-mm) deep double-tee girder bridge. However, after shear testing of the first bridge, the measured strains were close to or within the uncertainty range of the strain sensors. Furthermore, the shear girder distribution factors were significantly lower than those from the AASHTO (as discussed under the results). Therefore, the shear test was excluded and only the flexural response test was performed for the 23-in. (584-mm) deep double-tee girder bridge. Figures 5.22 to 5.24 show the instrumentation plans for the flexural response testing of the 23-in. (584-mm) deep double-tee girder bridge. Twenty-four strain gauges each with a 12-in. (305-mm) extension were installed at the midspan of the bridge (Fig. 5.25) to measure the flexural response. For some of the girders, additional strain sensors were installed at the inside of the stem at a distance of 15 in. (381 mm) from the stem bottom (Fig. 5.23 and 5.25) to obtain the strain profiles. As was discussed before, both static and dynamic tests were carried out to measure the girder distribution factors and dynamic load allowance.

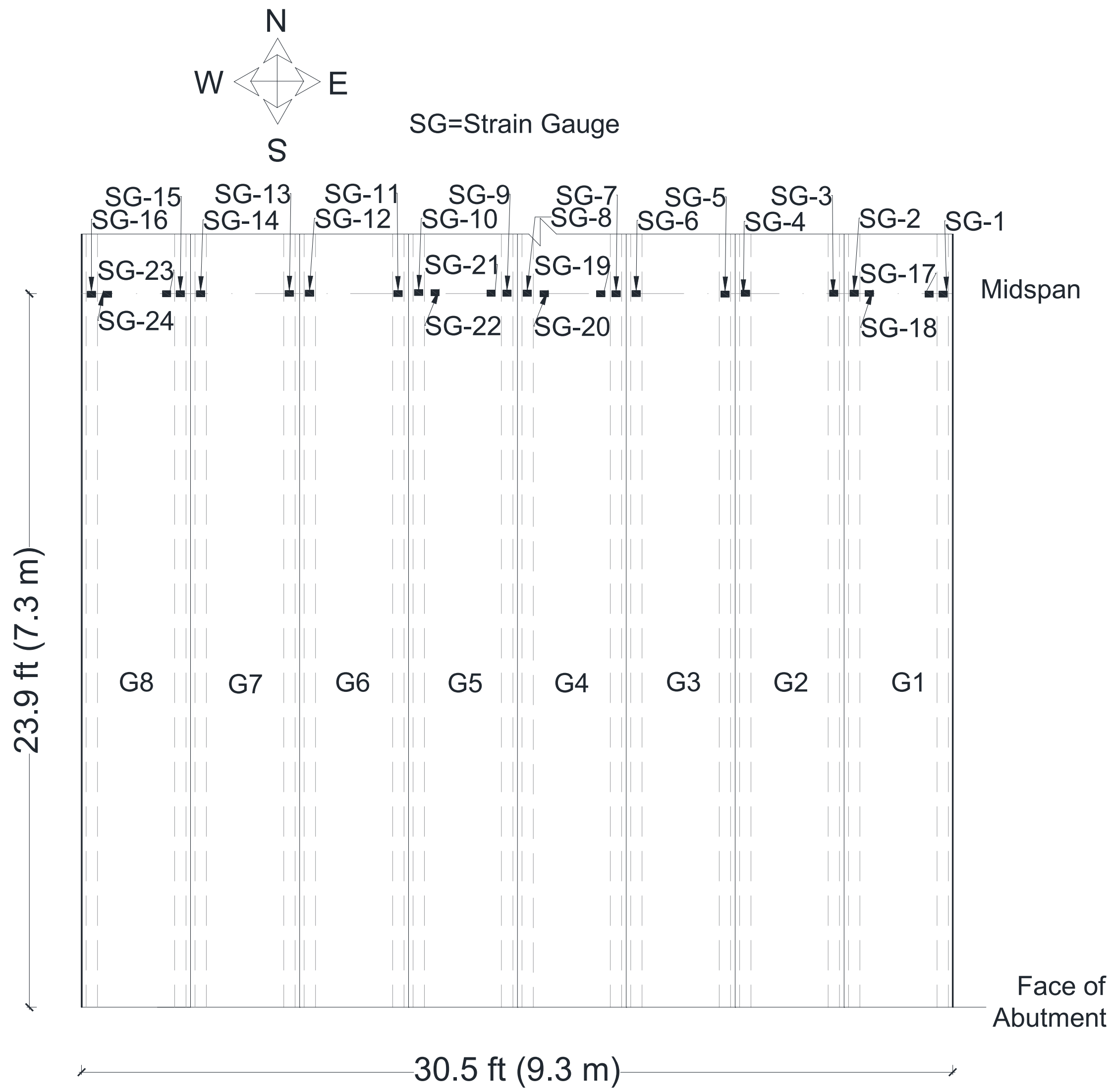


Figure 5.22 Strain Gauge Instrumentation Plan for Flexural Response of 23-in. (584-mm) Deep Double-Tee Girder Bridge – Plan View

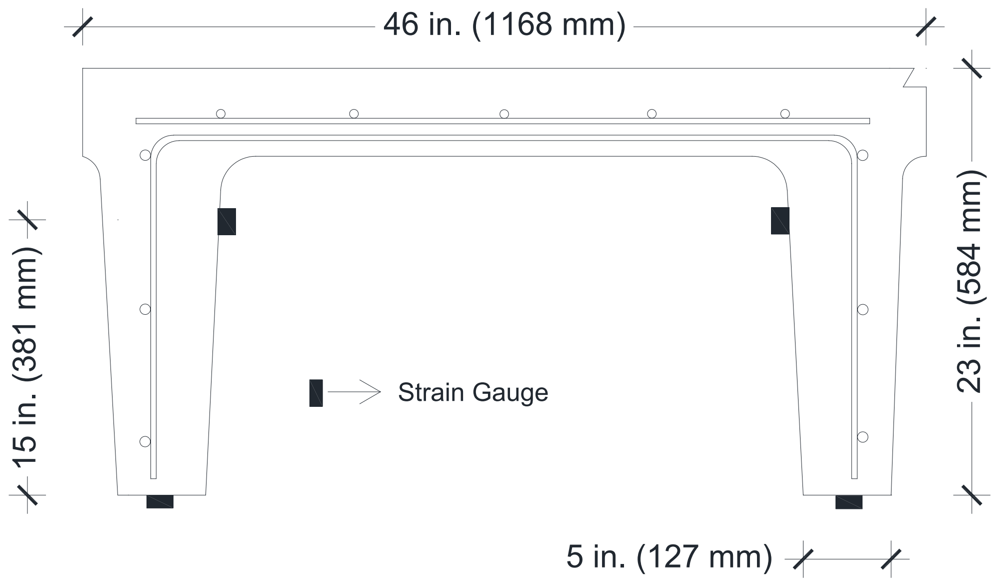


Figure 5.23 Girder Strain Gauge Instrumentation Plan for Flexural Response of 23-in. (584-mm) Deep Double-Tee Girder Bridge – Section View

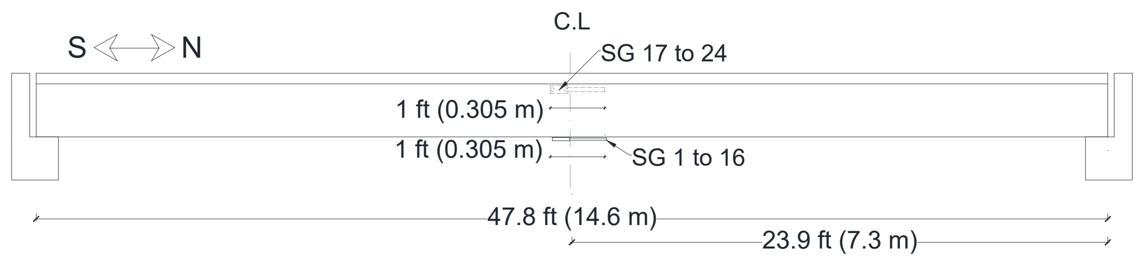


Figure 5.24 Girder Strain Gauge Instrumentation Plan for Flexural Response of 23-in. (584-mm) Deep Double-Tee Girder Bridge – Elevation View

(a) Installation of Strain Gauges with Extension

(b) Strain Gauges at Stem Bottom Face

(c) Strain Gauges at Top and Bottom of Stem

(d) Field Work Using Snooper Truck

Figure 5.25 Strain Gauge Installation for Flexural Response of 23-in. (584-mm) Deep Double-Tee Girder Bridge

5.5 Double-Tee Bridge Field Test Results

Strain data was recorded using a 128-channel data acquisition system with a reading rate of 256 points per second. The measured strains, live load distribution factors, and dynamic load allowance per bridge were processed and a summary of the results is presented herein.

5.5.1 Field Test Results for 30-in. (762-mm) Deep Double-Tee Girder Bridge

Shear and flexure tests were performed for the 30-in. (762-mm) deep double-tee girder bridge.

5.5.1.1 Shear Response Filed Test Results

For the shear tests, 24 strain gauges were installed at a distance of 30 in. (762 mm) from the south end diaphragm of the bridge. Static and dynamic tests were conducted. The test truck was driven across the bridge twice per path to minimize errors.

Measured Shear Strains. Figure 5.26 shows the maximum measured shear strains for each run of the field testing for the 30-in. (762-mm) deep double-tee girder bridge. Similarly, Fig. 5.27 shows the maximum measured shear strains but for each path, which were the average of the two runs. In both charts, the x-axis is the girder number and the y-axis is the strain in micro-strain (με). The maximum shear strains were calculated according to Eq. 5.1 using the uniaxial strains measured by the two shear strain sensors (Hughs et al., 2006). It can be seen that the loaded girders per run or path showed the highest shear strains compared to the not-loaded girders in that run or path. The peak measured strains per sensor were very small (less than 10 micro-strain) within the error range of the strain sensors used in the tests. Therefore, the shear strains thus the shear girder distribution factors may not be reliable. That is, the shear response test was not performed in field testing of the 23-in. (584-mm) deep double-tee girder bridge.

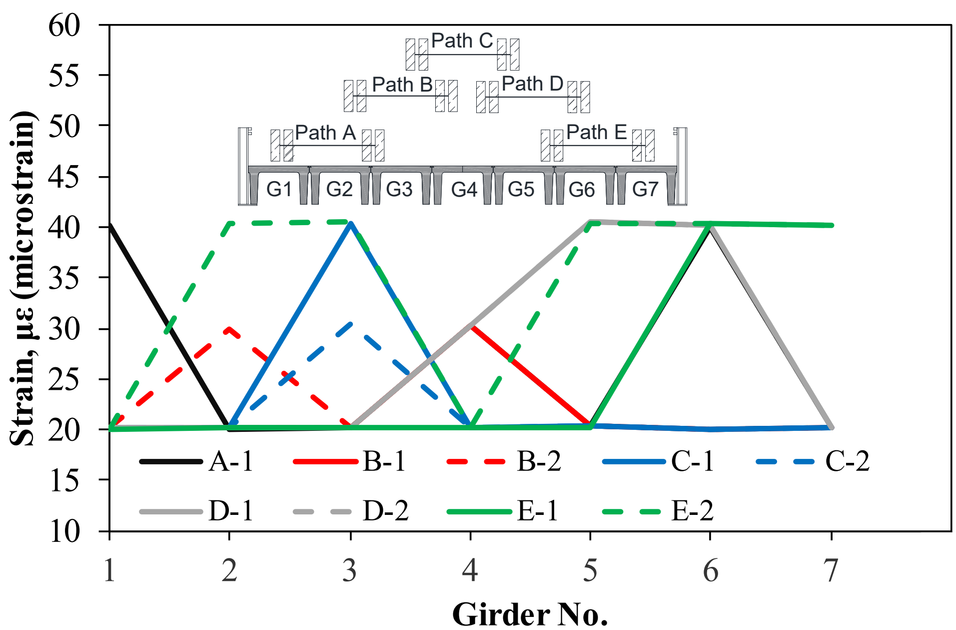


Figure 5.26 Maximum Measured Shear Strains for Each Girder in Each Run of 30-in. (762-mm) Deep Double-Tee Girder Bridge

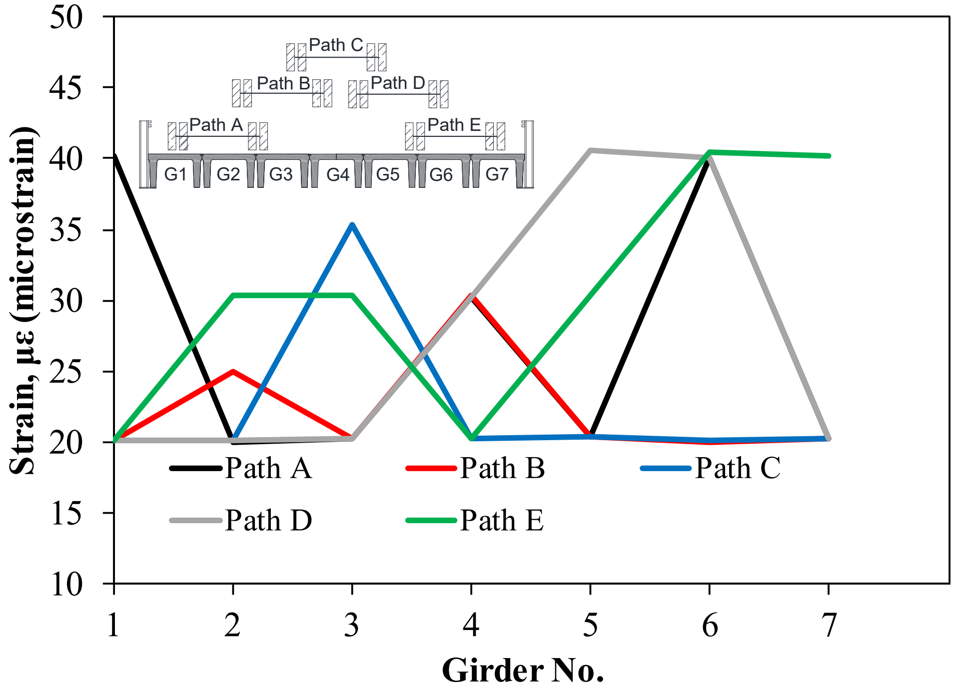


Figure 5.27 Maximum Measured Shear Strains for Each Girder in Each Path of 30-in. (762-mm) Deep Double-Tee Girder Bridge

$$\Upsilon = \frac{\varepsilon_1 - \varepsilon_2}{\sin(2\alpha)} \qquad \text{(Eq. 5.1)}$$

where,

Υ = The shear strain,

ε_1 = The measured uniaxial strain in one of the strain sensors,

ε_2 = The measured uniaxial strain in the second strain sensor,

α = The angle between the two strain sensors.

Measured Shear Girder Distribution Factors. The shear girder distribution factor (GDF) is the ratio of the girder maximum shear strain (Υ) to the sum of the maximum shear strains for all girders (Eq. 5.2 from Hughs et al., 2006).

$$GDF_i = \frac{\Upsilon_i}{\Sigma_{i=1}^{k} \Upsilon_i} \qquad \text{(Eq. 5.2)}$$

where the k is the total number of girders in the test bridge.

Table 5.5 presents the shear GDFs for the 30-in. (762-mm) deep double-tee girder bridge and Fig. 5.28 shows a graphical illustration of the values in the table. It can be seen that the measured shear GDFs are significantly lower than those calculated according to the AASHTO LRFD (2012) for this bridge. Therefore, the AASHTO shear GDFs can be used for 30-in. (762-mm) deep double-tee girder bridges in which their girder-to-girder joints are deteriorated with a condition state of 3 or less.

Table 5.5 Shear Girder Distribution Factors for 30-in. (762-mm) Deep Double-Tee Girder Bridge

Path A	0.13	0.15	0.15	0.17	0.15	0.18	0.07
Path B	0.13	0.16	0.13	0.19	0.13	0.13	0.13
Path C	0.13	0.13	0.23	0.13	0.13	0.13	0.13
Path D	0.11	0.10	0.11	0.16	0.21	0.21	0.11
Path E	0.09	0.14	0.14	0.10	0.14	0.19	0.19
Maximum GDF per Girder	0.13	0.16	0.23	0.19	0.21	0.21	0.19
AASHTO GDF per Girder	0.60	0.52	0.52	0.52	0.52	0.52	0.60

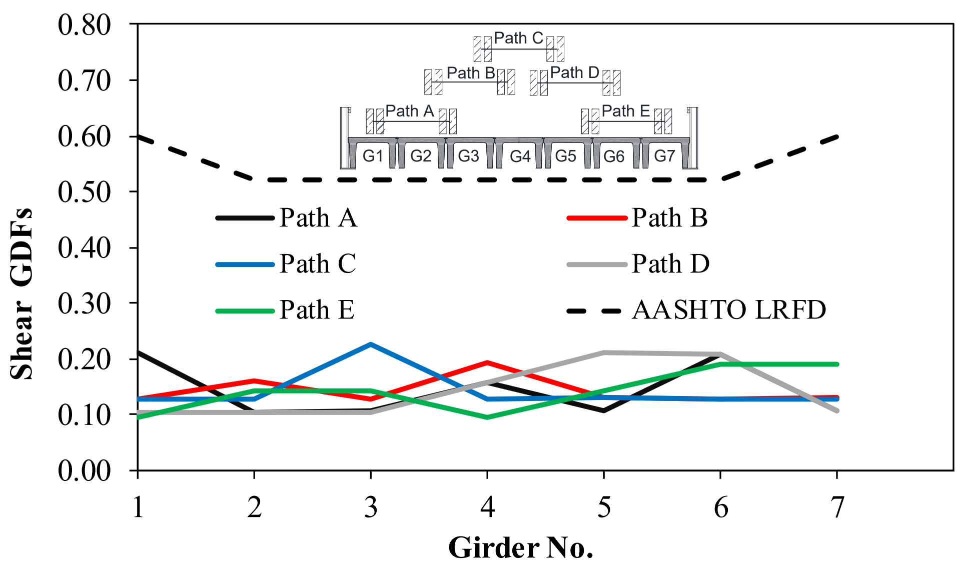


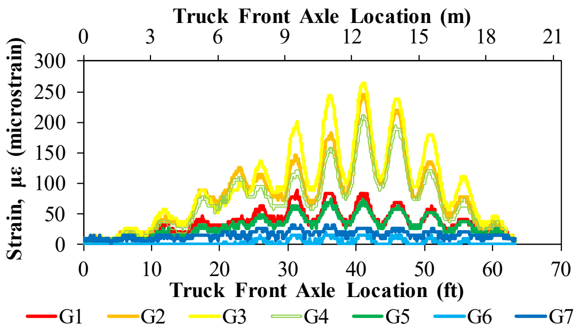
Figure 5.28 Shear Girder Distribution Factors for 30-in. (762-mm) Deep Double-Tee Girder Bridge

5.5.1.2 Flexural Response Field Test Results

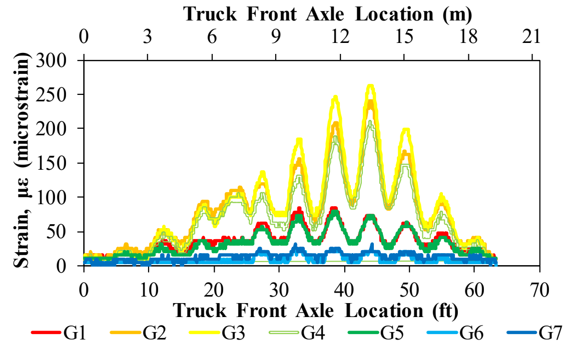
Since for a simply supported bridge under various live loads the maximum bending moment usually happens at the midspan, 14 strain sensors were installed at the bottom face of all stems for the 30-in. (762-mm) deep double-tee girder bridge as discussed in Sec. 5.4. A summary of the flexural test results is presented herein.

Measured Flexural Strains. Figure 5.29 shows the measured tensile strains for each girder of the 30-in. (762-mm) deep double-tee bridge. The x-axis shows the truck front tire position and y-axis is the average strains of the two stems per girder in micro-strain ($\mu\varepsilon$). The x-axis was limited to the sum of the bridge span length (42 ft, or 12.8 m) plus the truck length (21.2 ft, or 6.5 m) resulting in 63.2 ft (19.3 m). Due to a malfunctioning of the data acquisition system, the data for SG-1 to SG-8 during the Path A testing was lost, that is Path A was not included in the figure. Nevertheless, since the bridge is symmetric, the response of Path E might be valid for Path A. It can be seen that the loaded girders exhibited the largest stains, and the strains were maximum where the rear axles of the truck were close to the bridge midspan.

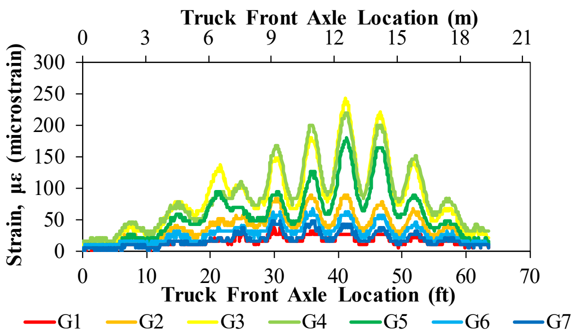
Figure 5.30 shows the measured flexural tensile strains for the 30-in. (762-mm) deep double-tee girder bridge in the bridge transverse direction. The flexural strain demands were highest for the exterior girders. Consistent results were observed in each run of each path.



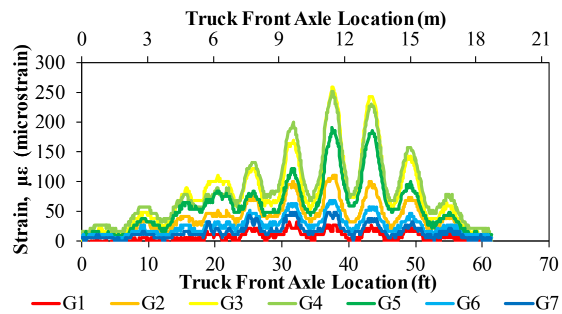
(a) Girder Midspan Strains in First Run of Path B



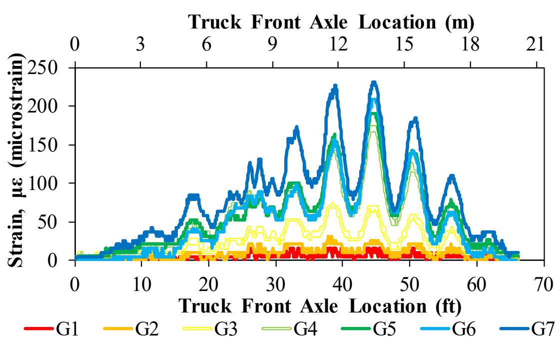
(b) Girder Midspan Strains in Second Run of Path B



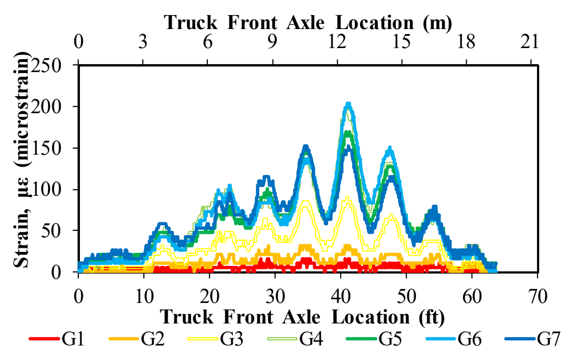
(c) Girder Midspan Strains in First Run of Path C



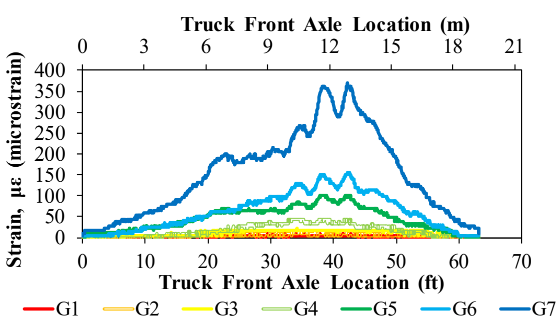
(d) Girder Midspan Strains in Second Run of Path C



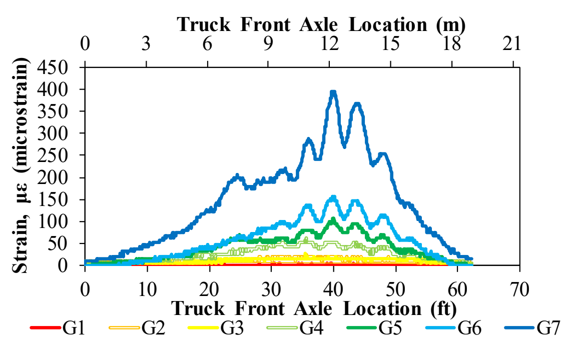
(e) Girder Midspan Strains in First Run of Path D



(f) Girder Midspan Strains in Second Run of Path D

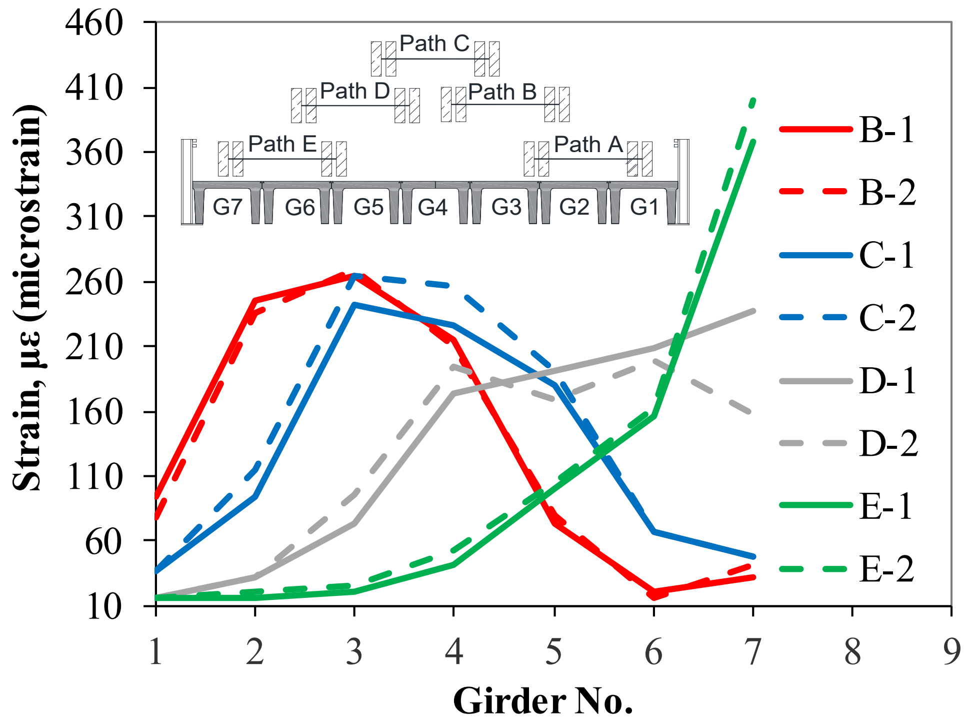


(g) Girder Midspan Strains in First Run of Path E

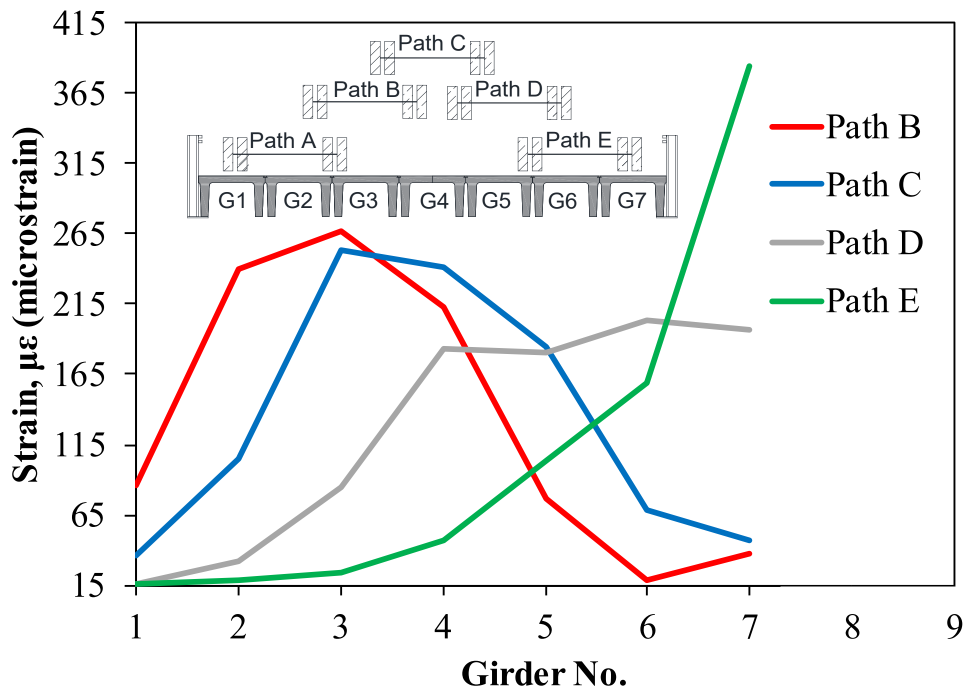


(h) Girder Midspan Strains in Second Run of Path E

Figure 5.29 Measured Flexural Tensile Strains for 30-in. (762-mm) Deep Double-Tee Girder Bridge in Longitudinal Direction



(a) Maximum Measured Flexural Tensile Strains for Each Girder in Each Run



(b) Maximum Measured Flexural Tensile Strains for Each Girder in Each Path

Figure 5.30 Measured Flexural Tensile Strains for 30-in. (762-mm) Deep Double-Tee Girder Bridge in Transverse Direction

Measured Moment Girder Distribution Factors. The moment girder distribution factor is defined as the ratio of the girder maximum flexural tensile strain ($\boldsymbol{\varepsilon}$) to the sum of the maximum flexural tensile strains for all girders (Hughs et al., 2006) as follows:

$$GDF_i = \frac{\varepsilon_i}{\Sigma_{i=1}^{k}\varepsilon_i} \qquad \text{(Eq. 5.3)}$$

where k is the total number of girders in the test bridge.

Table 5.6 presents the moment GDFs for the 30-in. (762-mm) deep double-tee girder bridge, and Fig. 5.31 is a graphical illustration of the values in the table. The calculated moment GDFs per the AASHTO LRFD requirements are also included. The loaded girders per path had the highest moment GDFs compared to the not-loaded girders in that path. The exterior girders showed the largest moment GDFs in this bridge. Furthermore, all measured moment GDFs were equal to or lower than those calculated using the AASHTO LRFD. Therefore, the AASHTO moment GDFs can be used for 30-in. (762-mm) deep double-tee girder bridges in which their girder-to-girder joints are deteriorated with a condition state 3 or less.

Table 5.6 Moment Girder Distribution Factors for 30-in. (762-mm) Deep Double-Tee Girder Bridge

Path A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Path B	0.09	0.26	0.28	0.23	0.08	0.02	0.04
Path C	0.04	0.11	0.27	0.26	0.20	0.07	0.05
Path D	0.02	0.04	0.09	0.20	0.20	0.23	0.22
Path E	0.02	0.02	0.03	0.06	0.14	0.21	0.51
Maximum GDF per Girder	0.09	0.26	0.28	0.26	0.20	0.23	0.51
AASHTO GDF per Girder	0.52	0.38	0.38	0.38	0.38	0.38	0.52

Note: Strain data for Path A was lost due to DAQ malfunctioning.

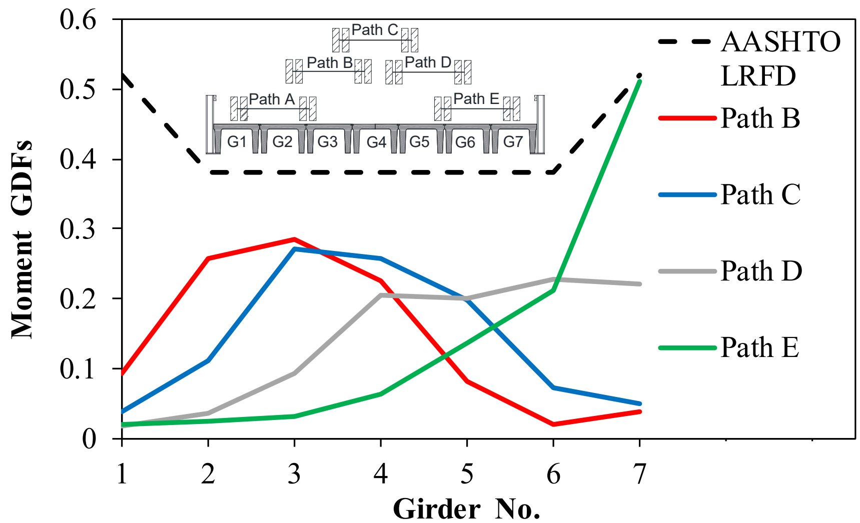


Figure 5.31 Moment Girder Distribution Factors for 30-in. (762-mm) Deep Double-Tee Girder Bridge

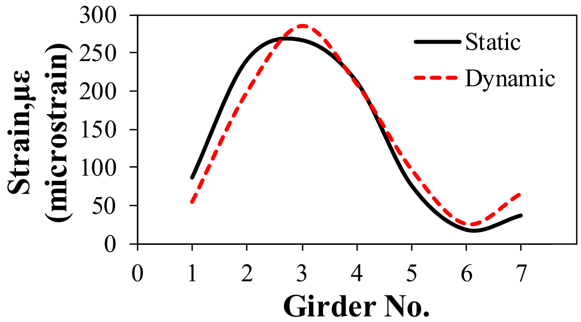
Measured Dynamic Load Allowance. The initial truck speed selected for dynamic testing was 55 mph (88.5 kph) but it was reduced to 35 mph (56.3 kph) for the safety of the crew and the bridge. The intention of the dynamic tests was to determine how the bridge would respond to a dynamic load and to evaluate the dynamic load allowance (IM) needed for load rating.

According to AASHTO MBE (2011), the dynamic load allowance is determined using the maximum dynamic strain and the corresponding maximum static strain for vehicles on the same path or transverse position on the bridge (Eq. 5.4). Table 5.7 presents the measured static and dynamic strains during flexural response testing of the 30-in. (762-mm) deep double-tee girder bridge. The measured IM is also included in the table. Figure 5.32 shows the measured static and dynamic strains in Paths B, C, and D in transverse and longitudinal directions of the bridge. Note that no dynamic test was performed on Paths A and E due to the bridge and road geometries. It can be seen that the maximum measured dynamic load was 7.2%, which is significantly lower than that required by the AASHTO LRFD for this bridge, which is 33%. Therefore, the AASHTO LRFD required dynamic load allowance can be used for 30-in. (762-mm) deep double-tee girder bridges in which their girder-to-girder joints are deteriorated with a condition state 3 or less.

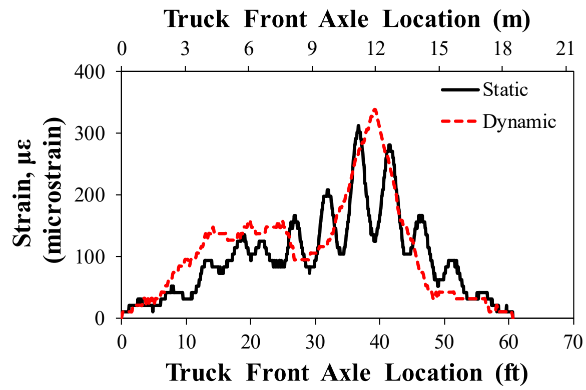
$$IM = \frac{Dynamic\ Strain - \ Static\ Strain}{Static\ Strain} \qquad \text{(Eq. 5.4)}$$

Table 5.7 Measured Static and Dynamic Strains and Dynamic Load Allowance (IM) for 30-in. (762-mm) Deep Double-Tee Girder Bridge

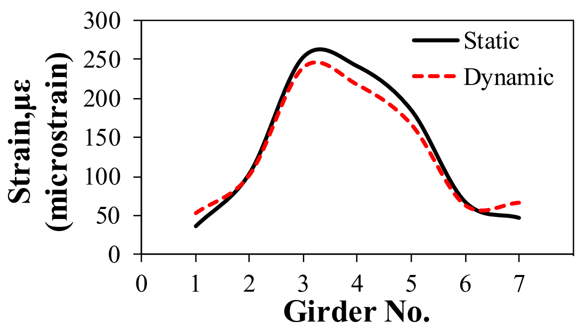
Girder Number	Path B		Path C		Path D	
	Static Strain (με)	Dynamic Strain (με)	Static Strain (με)	Dynamic Strain (με)	Static Strain (με)	Dynamic Strain (με)
1	87	55	37	52	16	29
2	241	199	105	102	31	42
3	267	285	253	240	84	87
4	212	210	241	218	184	197
5	77	98	185	167	180	185
6	18	26	68	63	204	214
7	37	66	47	66	197	218
Maximum Strain (με)	267	285	253	240	204	218
Dynamic Load Allowance	**6.9%**		**0%**		**7.2%**	
IM by AASHTO	**33%**					



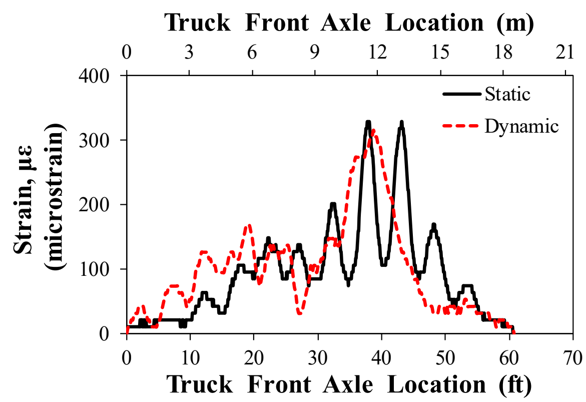
(a) Strains in Path B in Transverse Direction



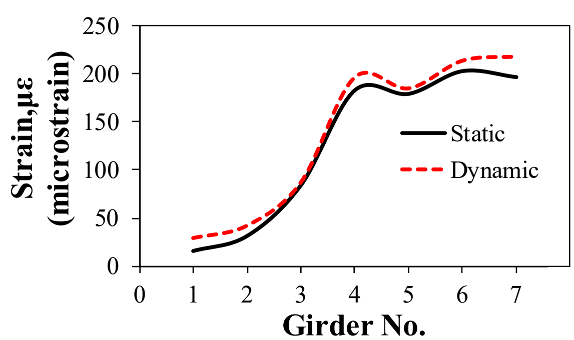
(b) Strains in Path B in Longitudinal Direction



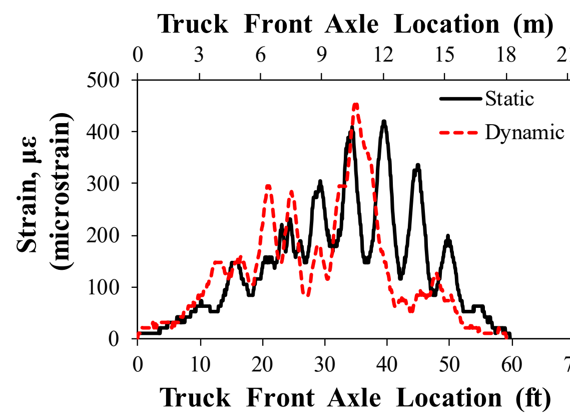
(c) Strains in Path C in Transverse Direction



(d) Strains in Path C in Longitudinal Direction



(e) Strains in Path D in Transverse Direction



(f) Strains Path D in Longitudinal Direction

Figure 5.32 Measured Static and Dynamic Strains for 30-in. (762-mm) Deep Double-Tee Girder Bridge

5.5.2 Field Test Results for 23-in. (584-mm) Deep Double-Tee Girder Bridge

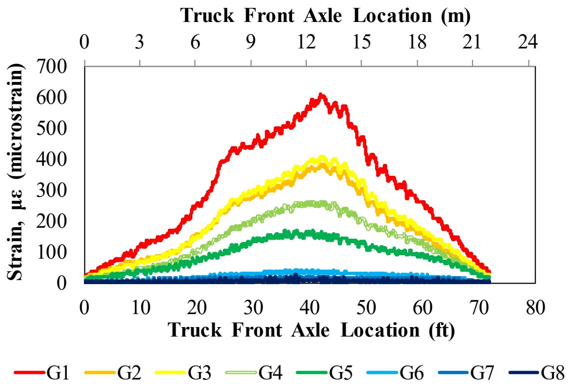
Only the flexural test was performed for the 23-in. (584-mm) deep double-tee girder bridge. This section presents a summary of the experimental findings.

5.5.2.1 Flexural Response Field Test Results

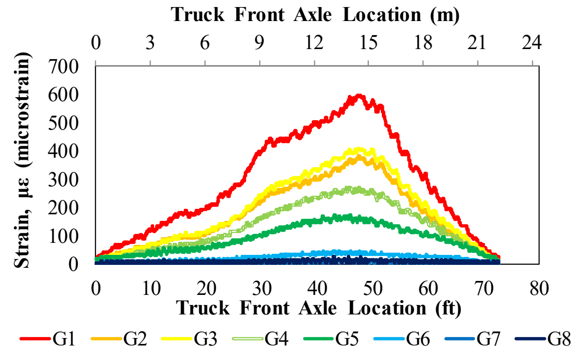
The 23-in. (584-mm) deep double-tee girder bridge had eight girders in which at least one strain sensor was installed on each stem. Refer to Sec. 5.4.2 for details of the instrumentation plan.

Measured Flexural Strains. Figure 5.33 shows the measured tensile strains for each girder of the 23-in. (584-mm) deep double-tee bridge. The x-axis shows the truck front tire position and y-axis is the average strains of the two stems per girder in micro-strain ($\mu\varepsilon$). The x-axis was limited to the sum of the bridge span length (50 ft, or 15.24 m) plus the truck length (21.2 ft, or 6.5 m) resulting in 71.2 ft (21.74 m). It can be seen that the loaded girders exhibited the largest stains, and the strains were maximum where the rear axles of the truck were close to the bridge midspan.

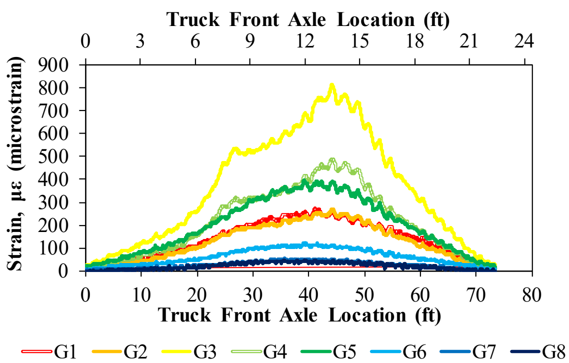
Figure 5.34 shows the measured flexural tensile strains for the 23-in. (584-mm) deep double-tee girder bridge in the bridge transverse direction. It can be seen that the flexural strain demands were highest for the exterior girders. Consistent results were observed in each run of each path.



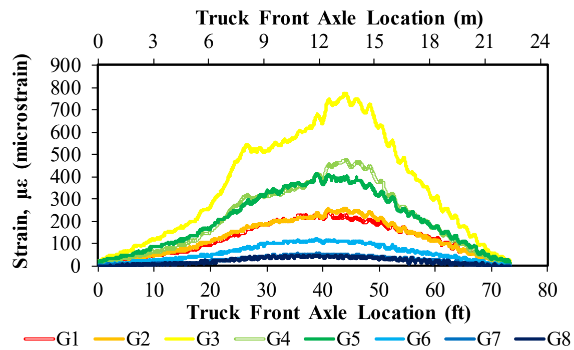
(a) Girder Midspan Strains in First Run of Path A



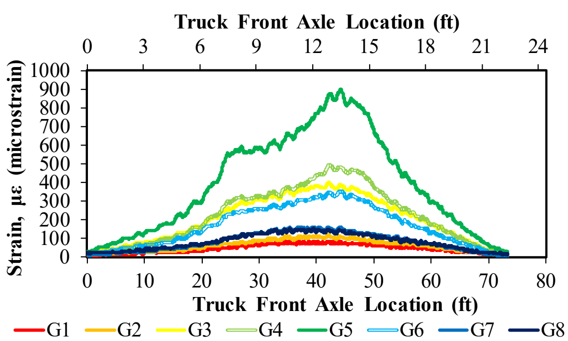
(b) Girder Midspan Strains in Second Run of Path A



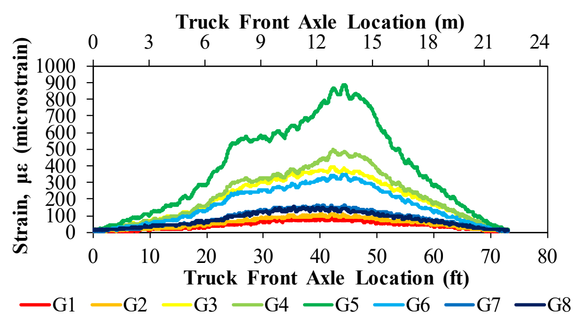
(c) Girder Midspan Strains in First Run of Path B



(d) Girder Midspan Strains in Second Run of Path B

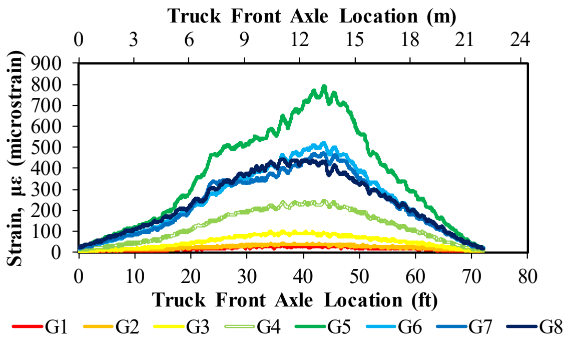


(e) Girder Midspan Strains in First Run of Path C

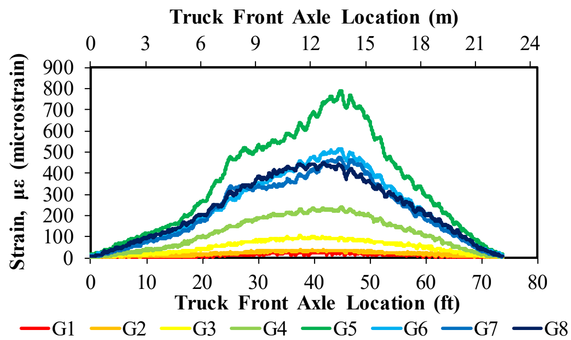


(f) Girder Midspan Strains in Second Run of Path C

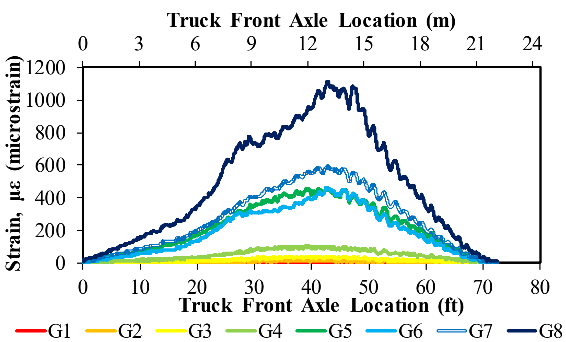
Figure 5.33 Measured Flexural Tensile Strains for 23-in. (584-mm) Deep Double-Tee Girder Bridge in Longitudinal Direction



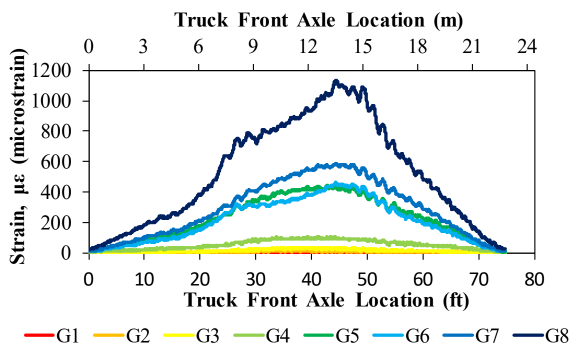
(g) Girder Midspan Strains in First Run of Path D



(h) Girder Midspan Strains in Second Run of Path D

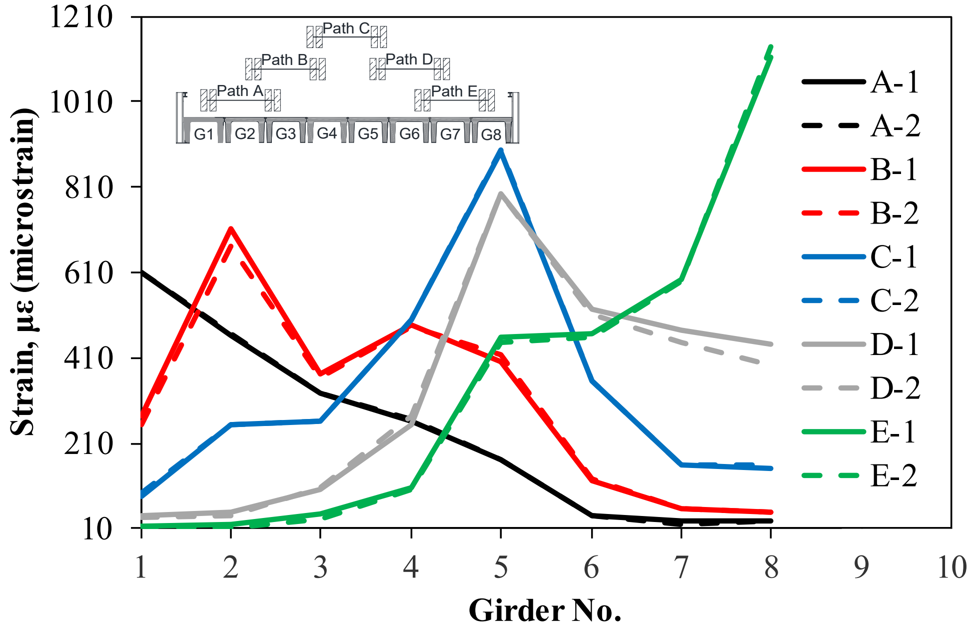


(i) Girder Midspan Strains in First Run of Path E

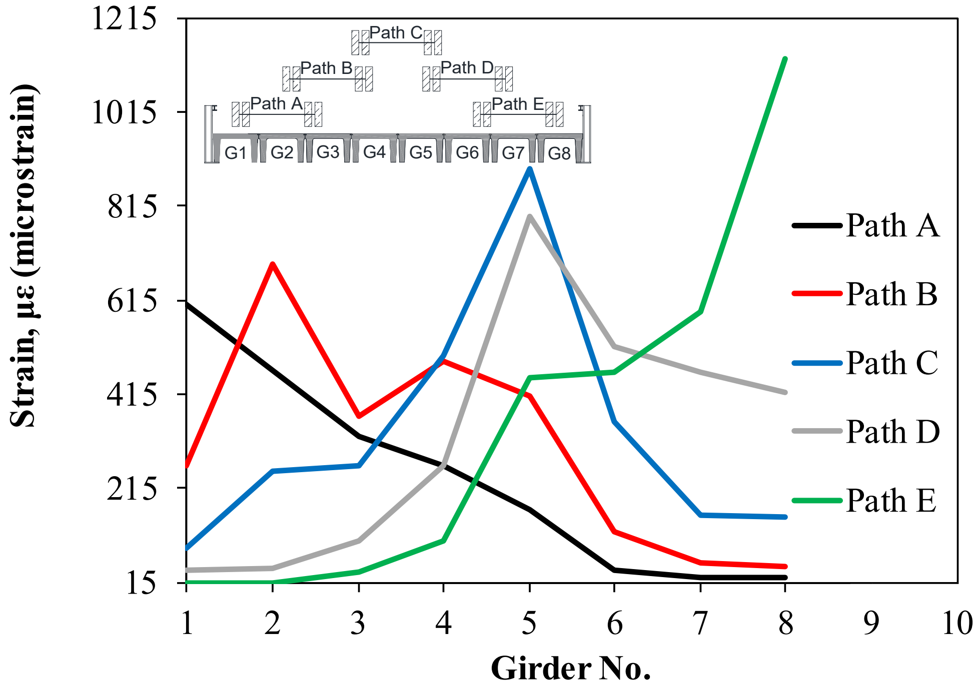


(j) Girder Midspan Strains in Second Run of Path E

Figure 5.33 Continued



(a) Maximum Measured Flexural Tensile Strains for Each Girder in Each Run



(b) Maximum Measured Flexural Tensile Strains for Each Girder in Each Path

Figure 5.34 Measured Flexural Tensile Strains for 23-in. (584-mm) Deep Double-Tee Girder Bridge in Transverse Direction

Measured Moment Girder Distribution Factors. The moment girder distribution factors were estimated using Eq. 5.3. Table 5.8 presents the measured moment GDFs for the 23-in. (584-mm) deep double-tee girder bridge, and Fig. 5.35 is a graphical illustration of the values in the table. The calculated moment GDFs per the AASHTO LRFD requirements are also included. It can be seen that the loaded girders per path had the highest moment GDFs compared to the not-loaded girders in that path. The exterior girders show the largest moment GDFs in this bridge. All measured moment GDFs were equal to or lower than those calculated using the AASHTO. Therefore, the AASHTO moment GDFs can be used for 23-in. (584-mm) deep double-tee girder bridges in which their girder-to-girder joints are deteriorated with a condition state 3 or less.

Table 5.8 Moment Girder Distribution Factors for 23-in. (584-mm) Deep Double-Tee Girder Bridge

Path A	0.32	0.24	0.17	0.14	0.09	0.02	0.01	0.01
Path B	0.11	0.28	0.15	0.20	0.17	0.05	0.02	0.02
Path C	0.03	0.09	0.10	0.19	0.34	0.13	0.06	0.06
Path D	0.01	0.02	0.04	0.10	0.30	0.20	0.17	0.16
Path E	0.01	0.01	0.01	0.04	0.16	0.16	0.21	0.40
Maximum GDF per Girder	0.32	0.28	0.17	0.20	0.34	0.20	0.21	0.40
AASHTO GDF per Girder	0.438	0.33	0.33	0.33	0.33	0.33	0.33	0.438

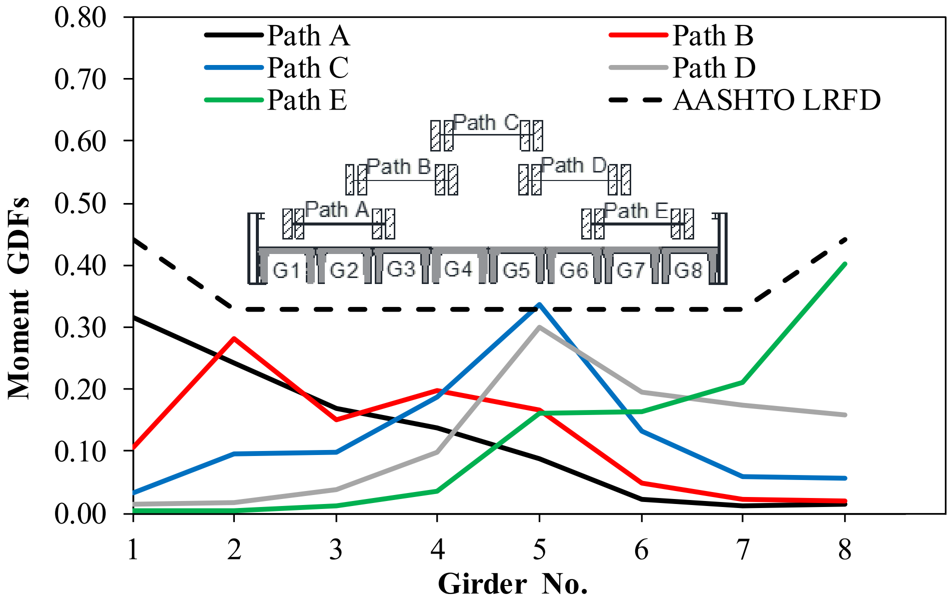
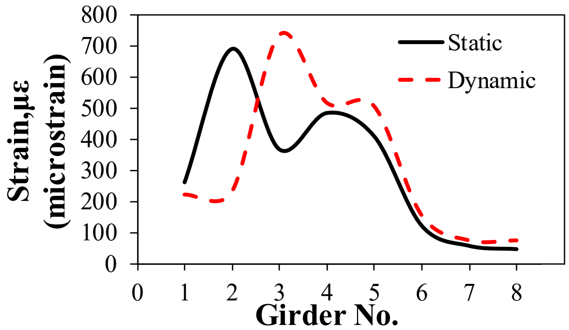


Figure 5.35 ment Girder Distribution Factors for 23-in. (584-mm) Deep Double-Tee Girder Bridge

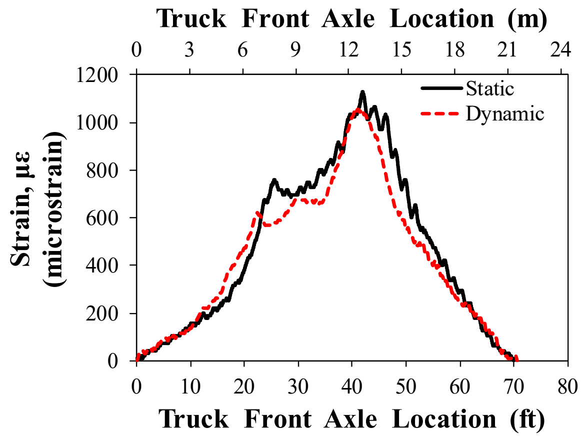
Measured Dynamic Load Allowance. Equation 7.4 was used to estimate the dynamic load allowance. Table 5.9 presents the measured static and dynamic strains during flexural response testing of the 23-in. (5842-mm) deep double-tee girder bridge. The measured IM is also included in the table. Figure 5.36 shows the measured static and dynamic strains in Paths B, C, and D in transverse and longitudinal directions of the bridge. No dynamic test was done on Paths A and E due to the bridge and road geometries. It can be seen that the maximum measured dynamic load was 6.2%, which is lower than that required by the AASHTO LRFD for this bridge, which was 33%. Therefore, the AASHTO LRFD required dynamic load allowance can be used for 23-in. (584-mm) deep double-tee girder bridges in which their girder-to-girder joints are deteriorated with a condition state 3 or less.

Table 5.9 Measured Static and Dynamic Strains and Dynamic Load Allowance (IM) for 23-in. (584-mm) Deep Double-Tee Girder Bridge

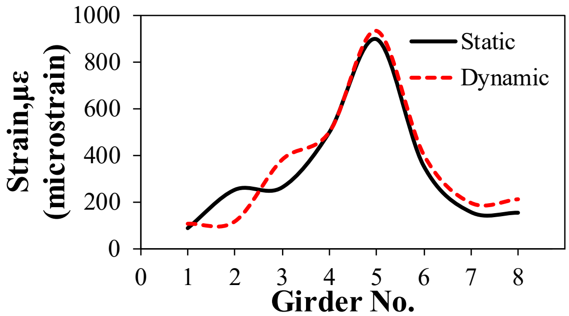
Girder Number	Path B		Path C		Path D	
	Static Strain (με)	**Dynamic Strain (με)**	**Static Strain (με)**	**Dynamic Strain (με)**	**Static Strain (με)**	**Dynamic Strain (με)**
1	262	223	89	105	39	45
2	691	233	253	115	45	42
3	368	734	263	380	103	103
4	485	517	498	498	262	254
5	410	505	896	933	793	817
6	123	157	355	402	517	509
7	58	76	158	195	461	458
8	47	76	155	210	418	444
Maximum Strain (με)	691	734	896	933	793	817
Dynamic Load Allowance	**6.2%**		**4.1%**		**3.0%**	
IM by AASHTO	**33%**					



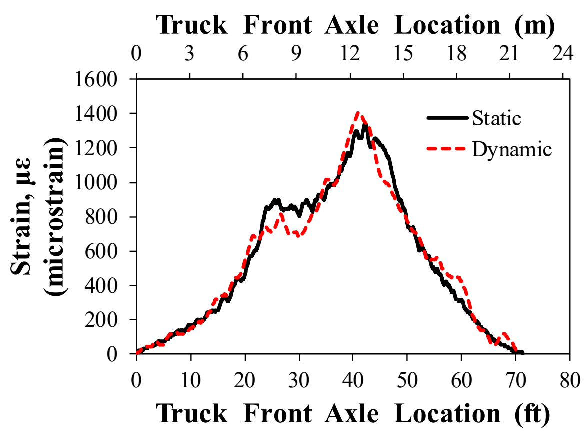
(a) Strains in Path B in Transverse Direction



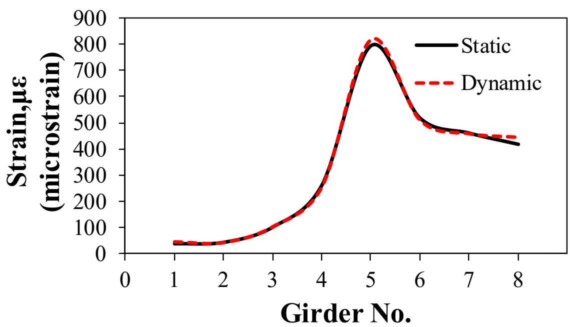
(b) Strains in Path B in Longitudinal Direction



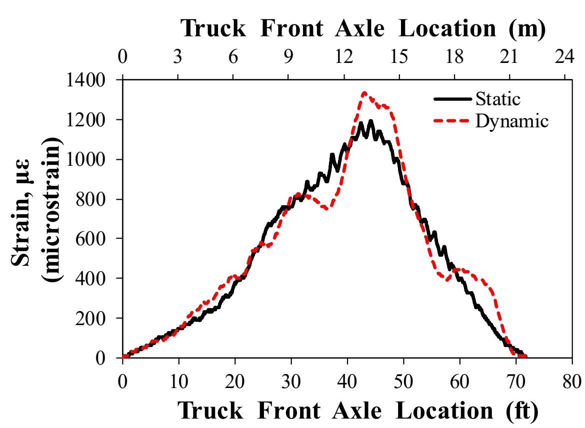
(c) Strains in Path C in Transverse Direction



(d) Strains in Path C in Longitudinal Direction



(e) Strains in Path D in Transverse Direction



(f) Strains in Path D in Longitudinal Direction

Figure 5.36 Measured Static and Dynamic Strains for 23-in. (584-mm) Deep Double-Tee Girder Bridge

5.6 Summary

Two double-tee bridges, one with 30-in. (762-mm) depth girders and another with 23-in. (584-mm) deep girders, were field tested to investigate their live load transfer mechanisms. Both bridges had deteriorated longitudinal joints with a damage condition state 3. Both bridges were tested for flexural response but only the bridge with the 30-in. (762-mm) deep girders was tested to obtain shear demands. The test data showed that the measured shear and moment girder distribution factors and the dynamic load allowance were equal to or lower than those calculated per the AASHTO LRFD requirements. Therefore, the AASHTO LRFD procedures can conservatively be used for the estimation of live loads for any South Dakota double-tee bridge with a girder-to-girder damage condition state 3 or less.

5.7 Recommendations for Live Load Estimation of Damaged Double-Tee Girder Bridges

Based on the field test findings and engineering judgment, the following guidelines are recommended for the live load estimation of double-tee girder bridges with deteriorated longitudinal joints. It is believed that other types of girder damage do not alter the live load distribution.

1. To calculate moment or shear GDFs for a SD double-tee girder bridge with a longitudinal joint damage condition state 3 or less, follow the AASHTO LRFD specifications.
2. To calculate moment or shear GDFs for a SD double-tee girder bridge with a longitudinal joint damage condition state 4, GDF is the greater of (a) the factor for the exterior girders, (b) the factor for the interior girders, and (c) 0.6.
3. To calculate the dynamic load allowance (IM), follow the AASHTO LRFD specifications.

During the time of the present project, no double-tee bridge was found in which its girder-to-girder joint was severely damaged (condition state 4). Therefore, no test was performed on such a bridge. Recommendation No. 2 is based on the fact that for a SD double-tee bridge with a typical girder width of 46 in. (1.17 m) to 48 in. (1.22 m) and a design truck with a transverse axle spacing of 6 ft (1.83 m), each girder can resist no more than 50% of the truck weight assuming that girders will act as individual members (completely unzipped) when the condition state of the longitudinal joints is 4. A 0.6 factor (10% more than 50%) was recommended for extra safety. Furthermore, in this case, any girder acts as an exterior girder because it is not connected to its adjacent girders. The recommendation ensures a conservative and safe live load estimation for the damaged double-tee bridges located in South Dakota.

6. STRENGTH TESTING OF 45-YR OLD SALVAGED DOUBLE-TEE GIRDERS

The South Dakota Department of Transportation (SDDOT) currently allows precast double-tee girder bridges on local roads since they are economical and fast in construction. The design service life of bridges is 75 years. However, many of double-tee bridges are deteriorating, need repair, or replacement after only 40 years of service (Mingo, 2016). Load rating of distressed bridges requires accurate estimation of capacities and demands. Using the test data from the literature, it was shown in Chapter 3 that the AASHTO LRFD methods of capacity estimation are accurate for aged girders with minor distresses. However, there was no test data on severely damaged aged girders (damage prior to testing) in the literature to verify the AASHTO capacity equations.

Strength testing of salvaged double-tee girders was needed to validate the capacity estimation methods available in the AASHTO or different references. Two 45-year double-tee girders extracted from the Nemo Road Bridge (Bridge 52-319-268) in Pennington County, SD, were selected for strength testing. This section presents a description of the salvaged girders, test setup, loading protocol, instrumentation plan, and strength test results of these girders.

6.1 Description of Salvaged Girders

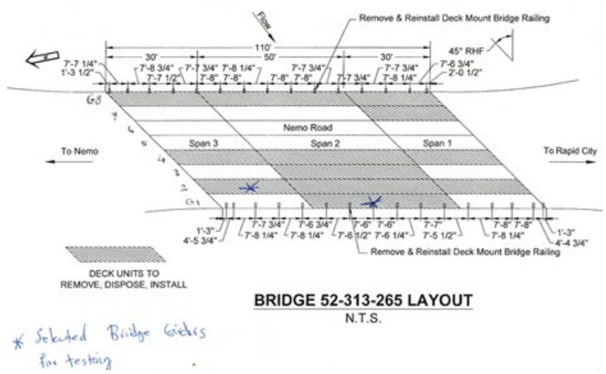
Two double-tee bridges (Fig. 6.1) close to Rapid City, SD, for which replacement funds became available, were inspected to select girders for lab test. Girders of the Nemo Road Bridge (ID 52-313-265, built in 1972) had more apparent damage compared to those of Norris Peak Road Bridge (ID 52-319-268, built in 1972). Therefore, one 30-ft (9.14-m) long double-tee girder and one 50-ft (15.24-m) long double-tee girder (Fig. 6.2), each 23-in. (584-mm) deep, were selected and extracted from this bridge (Fig. 6.3). The variation in the girder length was to investigate different failure modes. A short and damaged girder may fail in shear even though it was designed for a flexural failure. The two salvaged girders were delivered to the Lohr Structures Laboratory at South Dakota State University (SDSU).

(a) Bridge 52-313-265 (Nemo Road)

(b) Bridge 52-319-268 (Norris Peak Road)

Figure 6.1 Double-Tee Girder Bridges Inspected for Strength Testing

(a) Bridge 52-313-265 (Nemo Road)



(b) Selected Girders

(c) Selected 30-ft (9.14-m) Long Girder – Underneath View

(d) Selected 30-ft (9.14-m) Long Girder – Top View

(e) Selected 50-ft (15.24-m) Long Girder

(f) Midspan Close-up View of Selected 50-ft (15.24-m) Long Girder

Figure 6.2 Selected Salvaged Double-Tee Girders for Strength Testing

Figure 6.3 Extraction and Transportation of Salvaged Double-Tee Girders

Table 6.1 presents a summary of the girder damage according to the definitions presented in Chapter 4 and Fig. 6.4 shows damage of the salvaged girders. The 50-ft (15.24-m) long salvaged girder was 23-in. (584-mm) deep and 45-in. (1143-mm) wide with a 45-degree skew. It had concrete diaphragms at both ends. The flange was 5-in. (127-mm) thick and the stem was 18-in. (457-mm) deep. The prestressing strands for this girder were harped at a distance of $0.2L$ from each end, where L is the girder length. Seven 0.5-in. (12.7-mm) diameter uncoated low-relaxation ASTM A416 Grade 270 (1862 MPa) tendons were used in each stem of this girder. It is worth mentioning that in addition to the original damage (Table 6.1), this girder was further cracked at the midspan during unloading from the truck during transportation to SDSU (Fig. 6.4d).

The 30-ft (9.14-m) long salvaged girder had the same geometry as that in the 50-ft (15.24-m) long girder. However, it had concrete diaphragm at only one girder end. Furthermore, only four 0.5-in. (12.7-mm) diameter uncoated low relaxation ASTM A416 Grade 270 tendons were used per stem of this girder, all with a straight profile with no harp.

Table 6.1 45-Year Salvaged Double-Tee Girders Extracted from Bridge 52-319-268

Girder Depth, in. (mm)	Girder Length, ft (m)	As received Girder Damage Type and Condition State
23 (584)	30 (9.14)	Spalling of stem concrete cover (with a condition state of severe, Fig. 6.4a), exposure of stem transverse reinforcement (with a condition state of severe, Fig. 6.4a & c), and leakage of girder-to-girder joints (with a condition state of poor).
23 (584)	50 (15.24)	Deterioration of concrete cover (with condition state of severe, Fig. 6.4b), exposure of transverse rebar (with a condition state of severe, 8.4f), exposure of longitudinal prestressing (with a condition state of severe, Fig. 6.4f), and leakage of girder-to-girder joints (with a condition state of poor).

(a) Stem Cover Deterioration for 30-ft (9.14-m) Girder

(b) Stem Cover Deterioration for 50-ft (15.24-m) Girder

(c) Flange Cover Deterioration of 30-ft (9.14-m) Girder

(d) Damage of 50-ft (15.24-m) Girder during Unloading

(e) Reinforcement Exposure of 30-ft (9.14-m) Girder

(f) Exposure of Strands and Transverse Bars on Stem of 50-ft (15.24-m) Girder

Figure 6.4 As-received Damage of Salvaged Girders Selected for Strength Testing

6.2 Strength Test Setup for Salvaged Double-Tee Girders

Figures 6.5 to 6.9 show the strength test setup for the salvaged girders. Concrete reaction blocks were used as abutments, which were positioned in a skewed configuration to match with the girder skew angle and to balance the loads in the two stems. The height of the south end abutment was slightly shorter than the north end to accommodate load cells. A point load was applied to a spreader beam at the girder centerline at its midspan using a hydraulic actuator. The load was then split in two point loads equally spaced from the girder midspan to form a four-point loading configuration. The loading plates were 20-in. (508-mm) long and 10-in. (254-mm) wide simulating the AASHTO truck wheel areas.

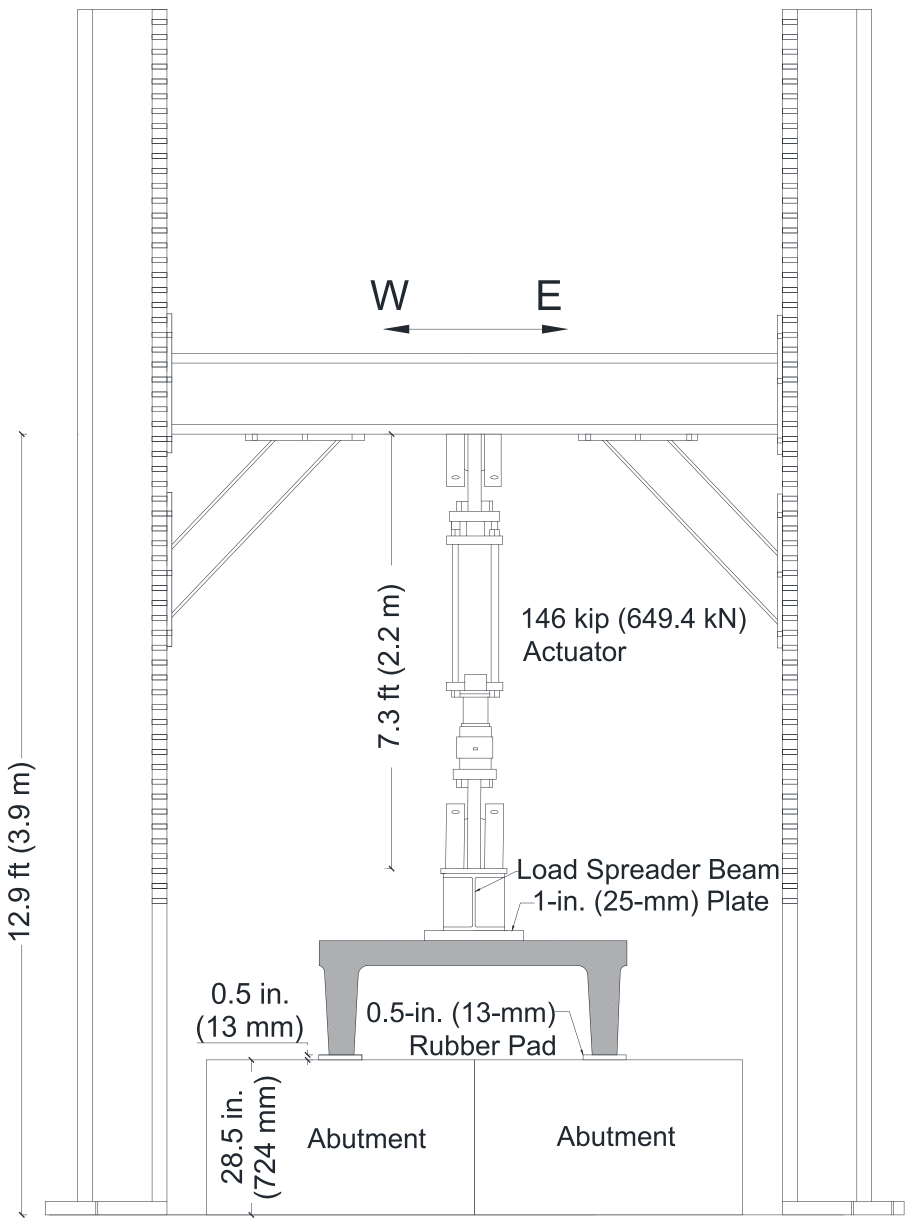


Figure 6.5 Strength Test Setup for Salvaged Girders – Section View

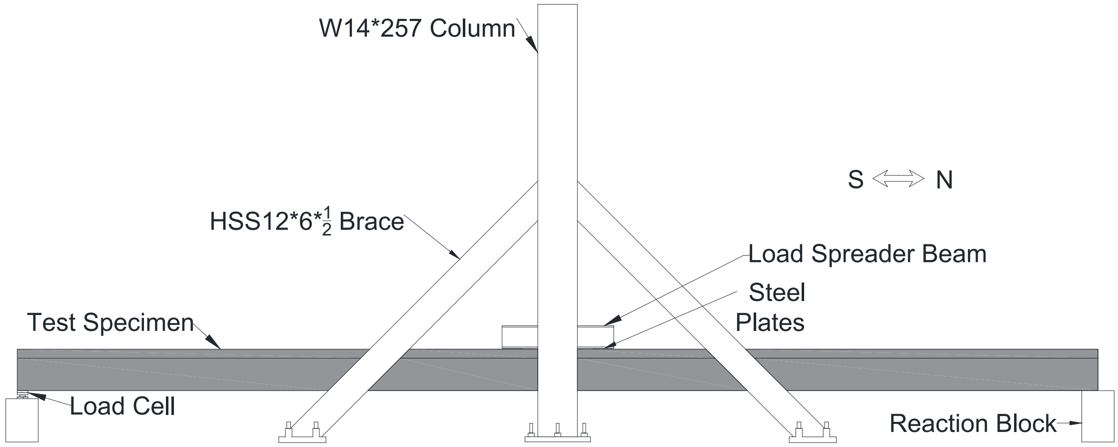


Figure 6.6 – **Strength Test Setup for Salvaged Girders – Elevation View**

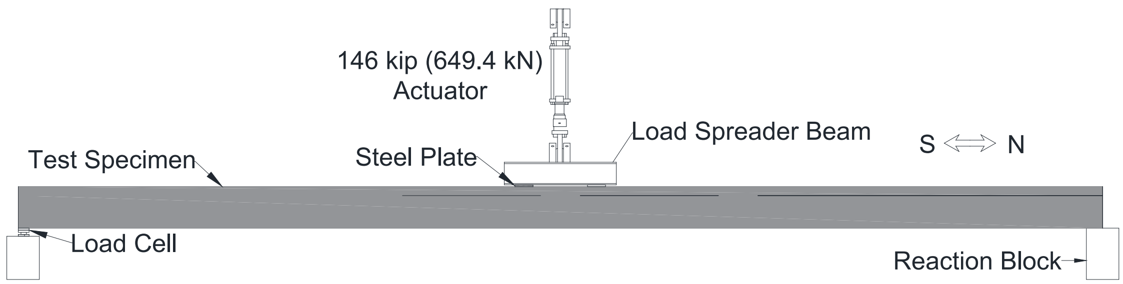


Figure 6.7 Strength Test Setup for Salvaged Girders – Elevation View without Test Frame

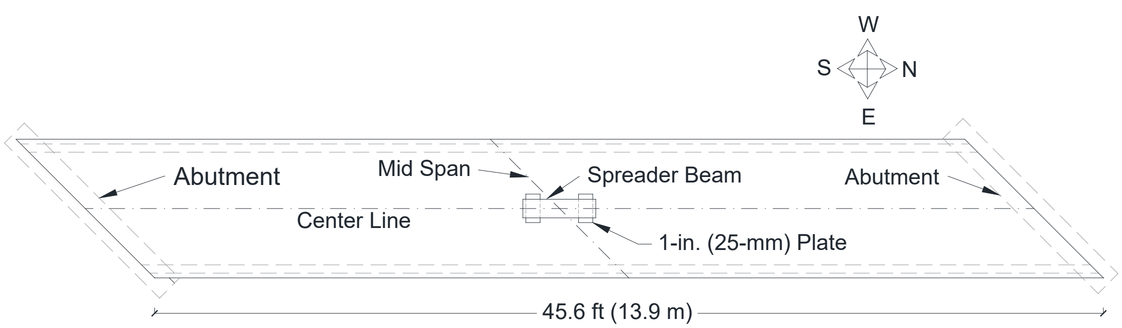


Figure 6.8 Point Loads in Plan View of 50-ft (15.24-m) Long Girder

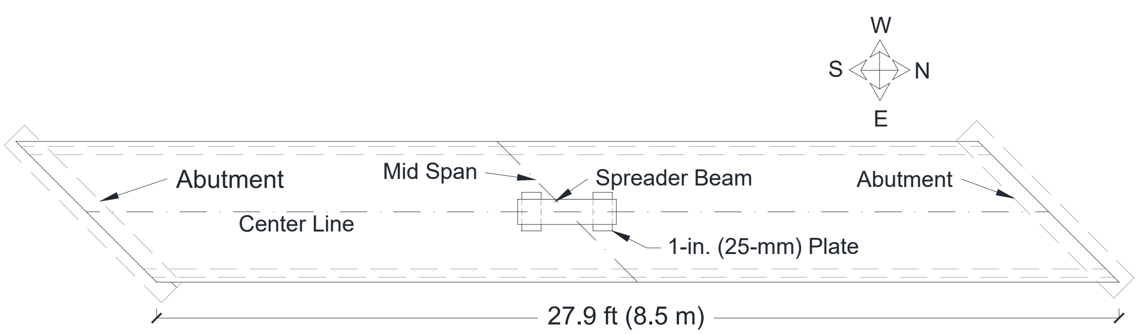


Figure 6.9 Point Loads in Plan View of 30-ft (9.14-m) Long Girder

6.3 Loading Protocol for Strength Testing of Salvaged Girders

Strength testing was performed on both girders to determine their capacities. The girders were tested under a monotonic loading using a 146-kip (649-kN) actuator with a displacement rate of 0.007 in/sec (0.178 mm/sec).

6.4 Instrumentation Plan

6.4.1 50-ft (15.24-m) Long Girder

Sensors used for strength testing of the 50-ft (15.24-m) long girder consisted of strain gauges, linear variable differential transformers (LVDTs), load cells, and string potentiometers (string pots). Table 6.2 presents a summary of the sensor types and locations. Details of the instrumentation plan are presented in the following sections.

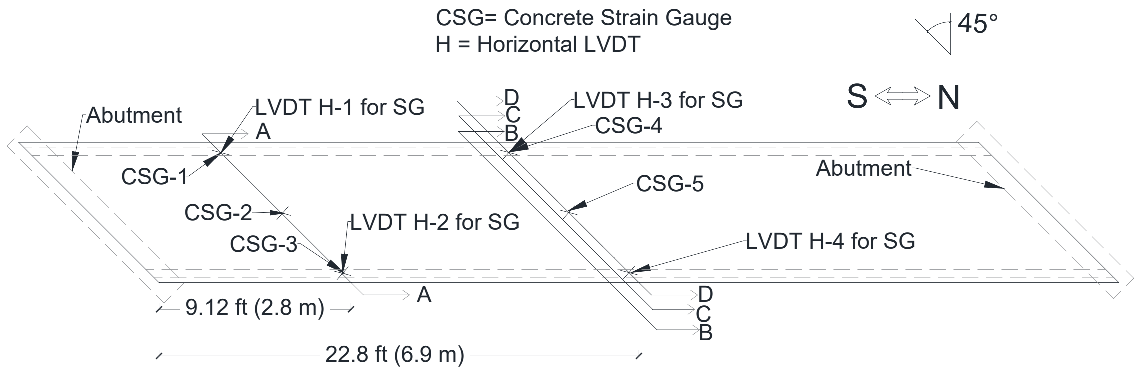
Table 6.2 Sensors Used in Strength Testing of 50-ft (15.24-m) Long Girder

Sensor Name		Identification	Location
Concrete Strain Gauge (CSG)	CSG-1	PMFLA-60-2LJRTA	Flange, 9.12 ft (2.8-m) away from the south end
	CSG-2	PMFLA-60-2LJRTA	
	CSG-3	PMFLA-60-2LJRTA	
	CSG-4	PMFLA-60-2LJRTA	Flange, midspan
	CSG-5	PMFLA-60-2LJRTA	
Steel Strain Gauge (SSG)	SSG-1	YFLA-2-5LJC	Stem, midspan exposed tendons
	SSG-2	YFLA-2-5LJC	
	SSG-3	YFLA-2-5LJC	
	SSG-4	YFLA-2-5LJC	
Horizontal LVDT (H)	H-1	LVDT 1.2	Stem, 9.12 ft (2.8-m) away from the south end
	H-2	LVDT 1.1	
	H-3	LVDT 1.3	West stem of girder, midspan
	H-4	LVDT 2.4	East side of flange, midspan
Vertical LVDT (V)	V1	LV-4	West stem of girder, near to the south end support
	V2	LV-3	East stem of girder, near to the south end support
Longitudinal Rotation LVDT (LR)	LR-1	LVDT 2.1	Underneath the flange, midspan
	LR-2	LVDT 1.4	Above the flange, midspan
String POT (SP)	SP-1	2	West stem, midspan
	SP-2	3	East stem, midspan
	SP-3	1	Between SP-1 & SP-2
Load Cell (LC)	LC-1	100 kips(444.8 kN)	West stem of girder, south end support
	LC-2	100 kips(444.8 kN)	East stem, south end support

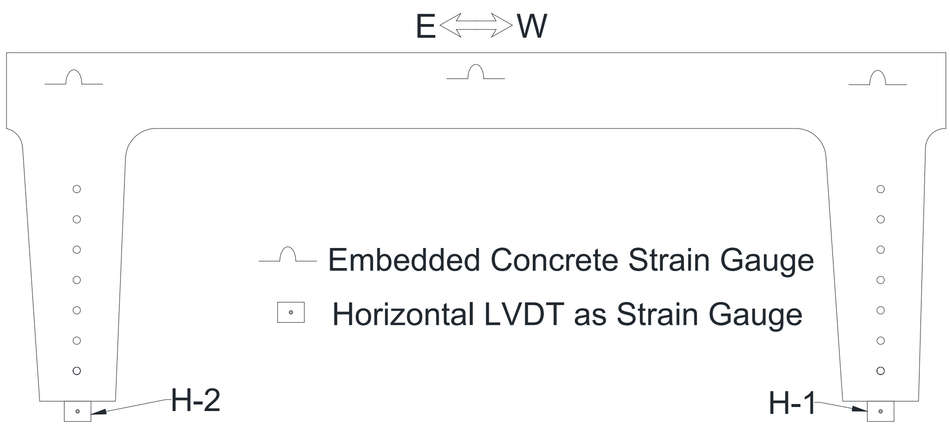
6.4.1.1 Strain Gauges

Figure 6.10 shows the strain gauge installation plan for the 50-ft (15.24-m) long girder. Five concrete strain gauges and four steel strain gauges were installed on the girder to measure strains in concrete and steel, respectively. Three concrete strain gauges were installed at 0.2*L* away from the south end of the girder (Fig. 6.10b) and two concrete strain gauges were used at the girder midspan (Fig. 6.10c). Furthermore, one LVDT was installed on the top of the girder flange to estimate the concrete strains. It was not possible to use concrete strain gauges in this location due to a severe damage of the flange concrete. Four steel strain gauges were installed in the exposed strands at the girder midspan (Fig. 6.10c to e and Fig. 6.11).

For installation of concrete strain gauges, 2-in. (50-mm) wide, 5-in. (127-mm) long, and 2-in. (50-mm) deep pockets were formed (Fig. 6.12a). One gauge was placed in each pocket in the longitudinal direction of the girder, the pockets were filled with a non-shrink grout, then the grout was cured for seven days.



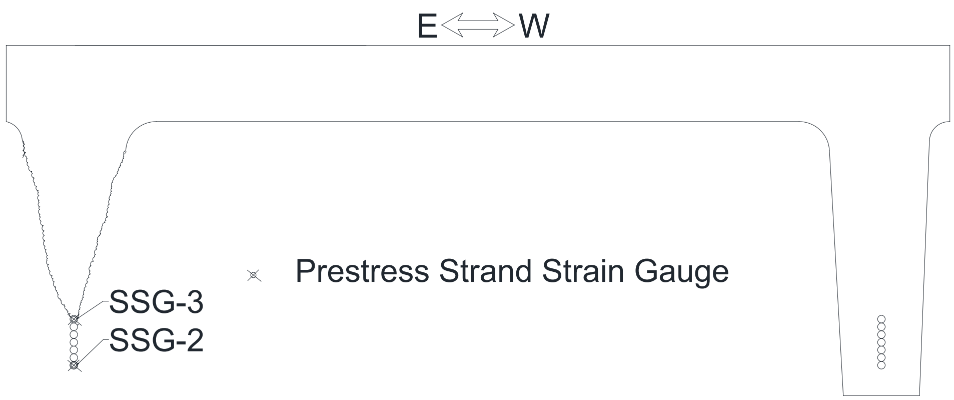
(a) Plan View



(b) Section A-A

Figure 6.10 Strain Gauge Instrumentation Plan for 50-ft (15.24-m) Long Girder

(c) Section B-B



(d) Section C-C

(e) Section D-D

Figure 6.10 Continued

Figure 6.11 Installation of Steel Tendon Strain Gauges

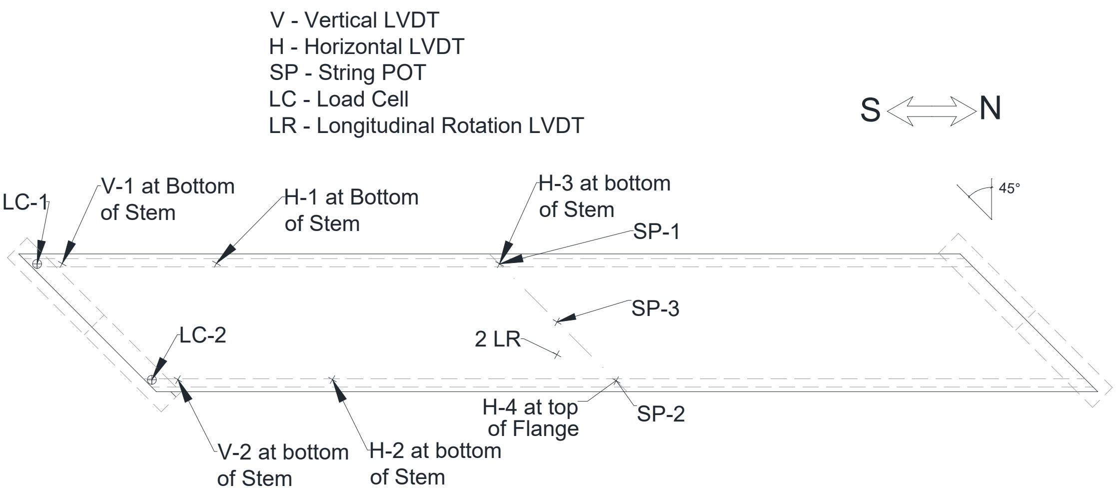
(a) Forming Pockets – Top Deck View

(b) After Pouring Grout

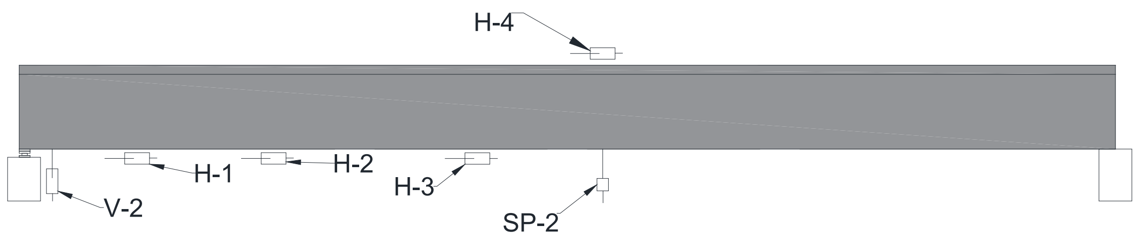
Figure 6.12 Installation of Concrete Strain Gauges

6.4.1.2 Linear Variable Differential Transformers (LVDTs)

Figure 6.13 shows the LVDT installation plan used in the strength testing of the 50-ft (15.24-m) long girder. Five LVDTs were installed to measure the horizontal displacements to be converted to the concrete strains (e.g. Fig. 6.14a). Two vertical LVDTs were used to measure the rubber bearing pad compressions and then to obtain the net midspan deflections (Fig. 6.14b). Furthermore, two horizontal LVDTs were installed at the midspan to measure the girder longitudinal rotations and curvatures (Fig. 6.14c & d).



(a) Plan View



(b) Elevation View Facing East Side

Figure 6.13 50-ft (15.24-m) Long Girder Instrumentation Plan including Displacement and Load Sensors

(a) LVDT as Concrete Strain Gauge

(b) Vertical LVDT

(c) LVDT underneath Flange for Rotations

(d) LVDT on top of Flange for Rotations

Figure 6.14 Installation of LVDTs on 50-ft (15.24-m) Long Girder

6.4.1.3 Load Cells

The end reactions of each stem were measured using a 100-kip (444.8-kN) load cell placed at the girder south end (Fig. 6.13 & 6.15). Load cells were placed between the steel plates for an adequate bearing. An elastomeric rubber bearing pad was placed between the top steel plate and girder to allow free rotations.

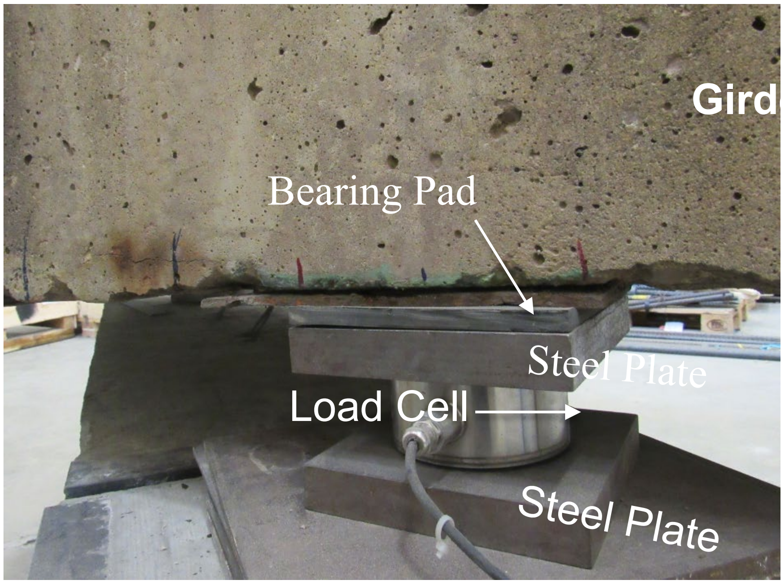


Figure 6.15 Load Cell Installation at Girder South End

6.4.1.4 String Pot

Three string pots were installed at the midspan to measure the girder deflections (Fig. 6.16). These sensors were placed in a configuration matching the girder skew angle (Fig. 6.13a).


Figure 6.16 – String Pot Installation at Midspan of 50-ft (15.24-m) Long Girder

6.4.1.5 Data Acquisition System

The sensor data was collected using a 128-channel data acquisition system (Vishay Precision Group, Model 7000, Fig. 6.17).

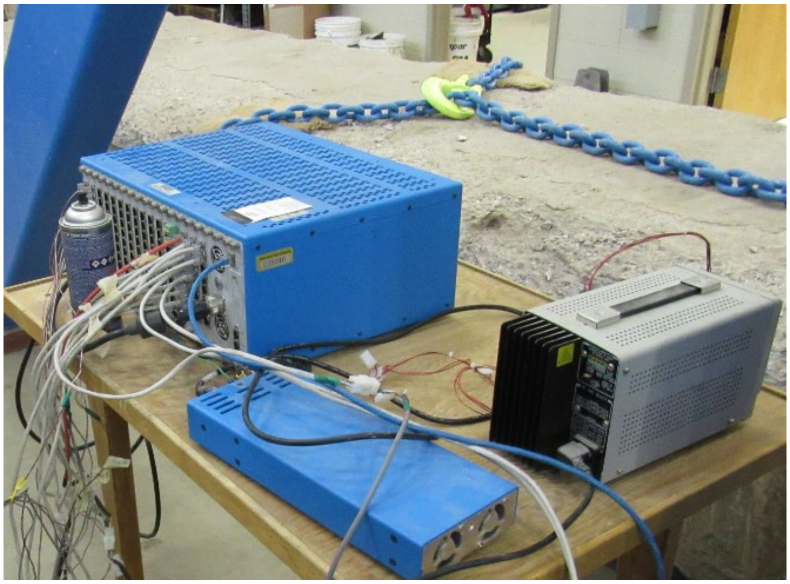


Figure 6.17 Data Acquisition System

6.4.2 30-ft (9.14-m) Long Girder

Sensors used for strength testing of the 30-ft (9.14-m) long girder consisted of strain gauges, LVDTs, load cells, and string pots. Table 6.3 presents a summary of the sensor types and locations. Details of the instrumentation plan are presented in the following sections.

Table 6.3 Sensors Used in Strength Testing of 30-ft (9.14-m) Long Girder

Sensor Name		Identification	Location
Concrete Strain Gauges (CSG)	CSG-1	PMFLA-60-2LJRTA	At 5.6 ft (1.7-m). from the south end (flange)
	CSG-2	PMFLA-60-2LJRTA	
	CSG-3	PMFLA-60-2LJRTA	
	CSG-4	PMFLA-60-2LJRTA	At mid span of girder(flange)
	CSG-5	PMFLA-60-2LJRTA	
	CSG-6	PMFLA-60-2LJRTA	
Horizontal Linear Variable Differential Transformer (H)	H-1	LVDT 2.1	At 5.6 ft (1.7-m) from the south end (stem)
	H-2	LVDT 1.4	
	H-3	LVDT 2.2	At mid span of west stem of girder (stem)
	H-4	LVDT 1.2	At mid span of east stem of girder (stem)
BDI Strain Transducer	BDI-1	6795	At 5.6 ft (1.7-m) from the south end (west stem)
	BDI-2	6792	At 5.6 ft (1.7-m) from the south end and above BDI-1 (west stem)
	BDI-3	6793	At 5.6 ft (1.7-m) from the south end (east stem)
	BDI-4	6781	At 5.6 ft (1.7-m) from the south end and above BDI-3 (east stem)
Vertical LVDT (V)	V1	LV-4	Near to the south end support (west stem)
	V2	LV-3	Near to the south end support (east stem)
Longitudinal Rotation LVDT (LR)	LR-1	LVDT 1.3	Underneath of the flange at midspan
	LR-2	LVDT 1.1	Over the flange at midspan
String POT (SP)	SP-1	3	At mid span (west stem)
	SP-2	2	At mid span (east stem)
	SP-3	1	Between SP-1 & SP-2
Load Cell (LC)	LC-1	100 kips (444.8 kN)	South end support (west stem)
	LC-2	100 kips (444.8 kN)	South end support (east stem)

6.4.2.1 Strain Gauges

Figure 6.18 shows the strain gauge installation plan for the 30-ft (9.14-m) long girder. Six concrete strain gauges were installed to measure the flange concrete strains. Three of which were installed at a distance of 0.2*L* from the south end and the remaining were installed at the girder midspan.

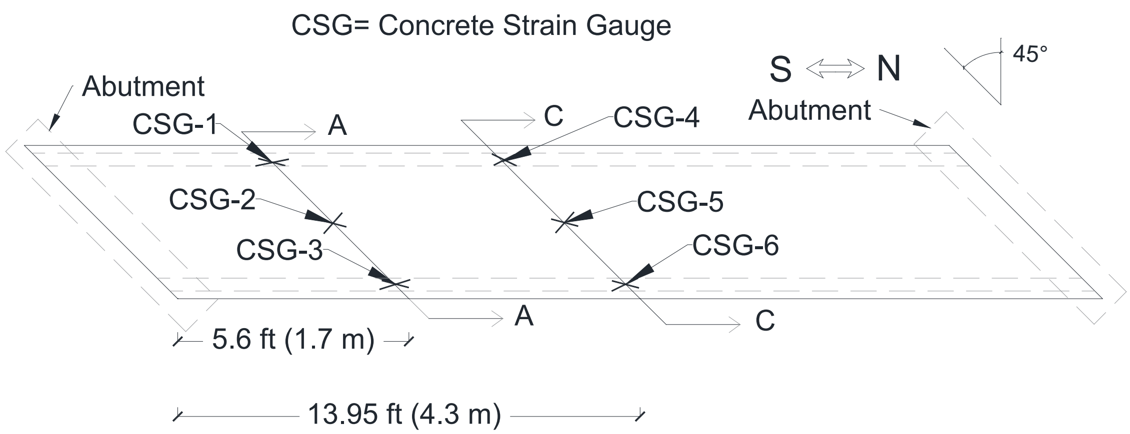


Figure 6.18 Strain Gauge Instrumentation Plan for 30-ft (9.14-m) Long Girder

6.4.2.2 Surface-Mount Strain Transducers

Four surface-mount strain transducers produced by Bridge Diagnostics, Inc. (BDI, Model ST350), two per stem, were installed at a distance of 0.2*L* from the girder south end (Fig. 6.19). Two sensors on the east stem had an extension of 12 in. (304 mm) (Fig. 6.19a) while the other two on the west stem has no extension measuring the strains over a 3-in. (76-mm) length (Fig. 6.19b). This was done to practice the sensor installation and to evaluate the performance of these sensors before field testing. Note the field testing (Chapter 5) was performed after the laboratory testing.

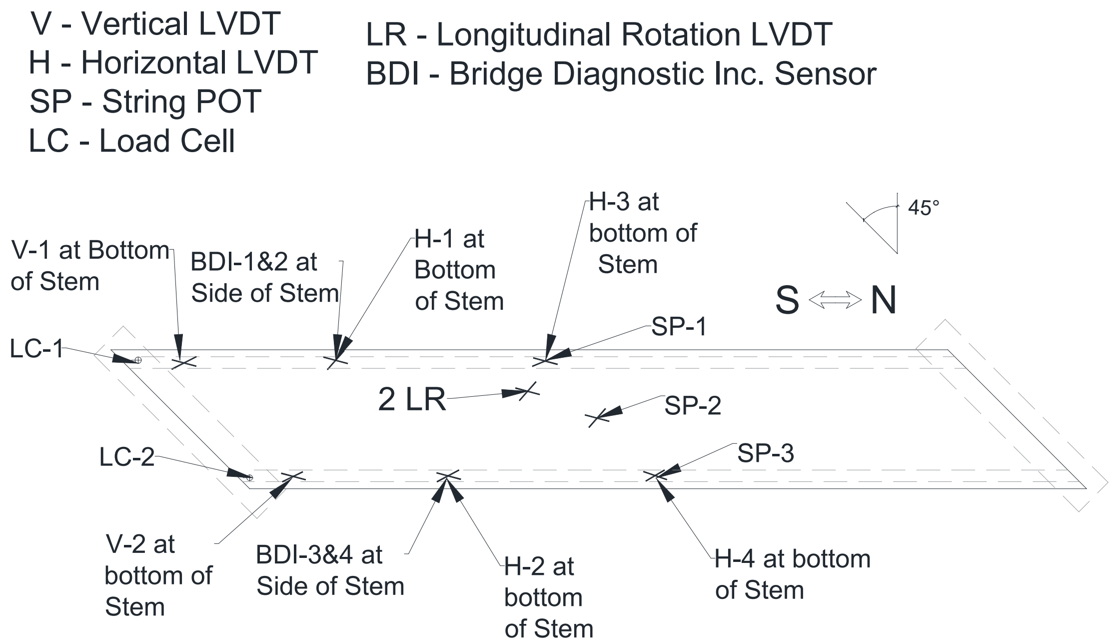
(a) BDI with Extension at East Stem.

(b) BDI without extension at West Stem.

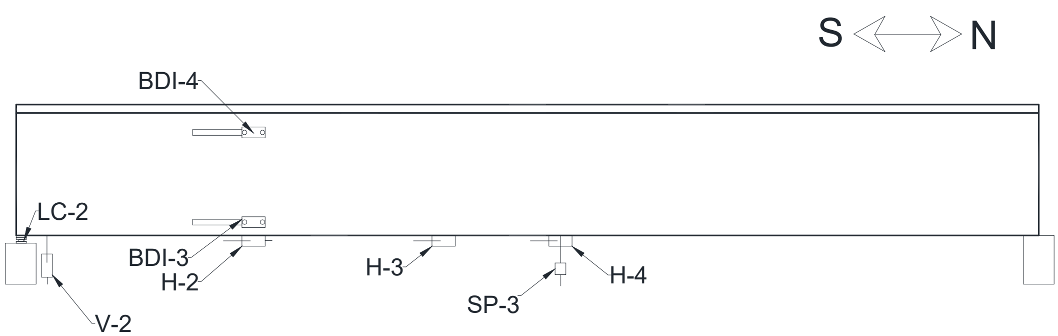
Figure 6.19 Installation of BDI Sensors on 30-ft (9.14-m) Long Girder

6.4.2.3 Linear Variable Differential Transformers (LVDTs)

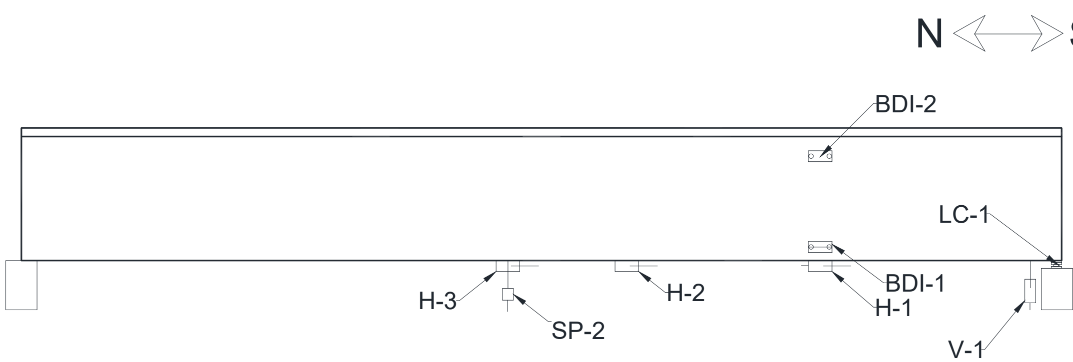
Figure 6.20 shows the LVDT installation plan used in strength testing of the 30-ft (9.14-m) long girder. Four LVDTs were installed to measure horizontal displacements to be converted to the concrete strains. Two vertical LVDTs were used to measure the rubber bearing pad compressions and then to obtain the net midspan deflections. Two horizontal LVDTs were installed at the midspan to measure girder longitudinal rotations and curvatures.



(a) Plan View



(b) Girder Elevation View on East Side



(c) Girder Elevation View on West Side

Figure 6.20 30-ft (9.14-m) Long Girder Instrumentation Plan including Displacement and Load Sensors

6.4.2.4 Load Cells

Two 100-kip (444.8-kN) load cells were installed under each stem at the south end to measure stem reactions (Fig. 6.20a).

6.4.2.5 String Pot

Three string pots were installed at the girder midspan, two at each stem and one at the flange, to measure girder deflections (Fig. 6.20a & Fig. 6.21). It was noticed that the middle string pot (SP-2) was not working properly. Therefore, its data was excluded in the post-processing.

Figure 6.21 String Pot Installation at Midspan of 30-ft (9.14-m) Long Girder

6.4.2.6 Data Acquisition System

The sensor data was obtained using a 128-channel data acquisition system (Fig. 6.17).

6.5 Material Properties

This section presents the material properties of concrete, reinforcing steel bars, and steel tendons used in the girders. The properties of the non-shrink grout used under the loading plates are also included.

6.5.1 Properties of Girder Concrete

Even though the design compressive strength for concrete was available in a shop drawing for similar girders (5000 psi, [34.5 MPa]), core samples were collected after strength testing to evaluate the actual concrete strength. Note both salvaged girders were severely damaged prior to the testing. The actual shop drawing for these salvaged girders could not be found.

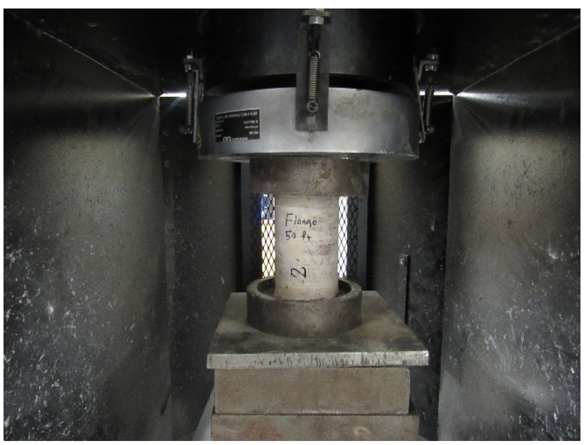
Figure 6.22 shows a sample core, which was obtained following ASTM C42 (2003), and the test setup, which was in accordance to ASTM C39 (2012). Table 6.4 presents a summary of the results. The concrete compressive strength for the 50-ft (15.24-m) long girder stem and flange was 3.15 ksi (21.7 MPa) and

1.92 ksi (13.2 MPa), respectively. Both strengths were significantly lower than those specified in shop drawings found for South Dakota double-tee girders.

Unfortunately, concrete in the 30-ft (9.14-m) long girder was severely deteriorated, so no samples could be obtained (all samples crushed during coring). The only core sample that was extracted from this girder had a short height that was not acceptable by ASTM C42 (2003). Due to a lack of test data, the compressive strength of the 30-ft (9.14-m) long girder is assumed to be the same as that in the 50-ft (15.24-m) long girder.

(a) Coring at Stem of 50-ft Long Girder

(b) Core Sample



(c) Sample in Compressive Machine

(d) Sample Failure

Figure 6.22 Concrete Coring and Testing for Salvaged Girders

Table 6.4 Concrete Compressive Strength Cored from 50-ft (15.24-m) Long Girder

Core Sample From	Core Diam. in. (mm)	Sample Length in. (mm)	Core Area in² (mm²)	Peak Force, lb (N)	Comp. Strength, psi (MPa)	Aspect Ratio	Correction Factor (ASTM C39)	Modified Strength, psi (MPa)	Average Strength, psi (MPa)
Stem	2.75 (70)	5.01 (127)	5.94 (3832)	19200 (85406)	3230 (22.3)	1.82	1	3230 (22.3)	3150 (21.7)
	2.75 (70)	5.02 (127)	5.94 (3832)	17930 (79757)	3020 (20.8)	1.83	1	3020 (20.8)	
	2.75 (70)	2.57 (65)	5.94 (3832)	21820 (97060)	3670 (25.3)	0.93	0.87	3190 (22)	
Flange	2.75 (70)	4.78 (121)	5.94 (3832)	13770 (61252)	2320 (16)	1.74	0.97	2250 (15.5)	1920 (13.2)
	2.75 (70)	4.8 (122)	5.94 (3832)	18790 (83582)	3160 (21.8)	1.75	0.98	3090 (21.3)	
	2.75 (70)	2.92 (74)	5.94 (3832)	10750 (47818)	1810 (12.5)	1.06	0.88	1590 (10.9)	

6.5.2 Properties of Prestressing Strands

The 50-ft (15.24-m) girder had seven tendons per stem, which were harped at a distance of 0.2L from each end of the girder while the 30-ft (9.14-m) girder had only four straight tendons per stem. The prestressing steel used in the two salvaged girders were uncoated seven-wire (A_{sp} = 0.196 in^2 [126 mm^2]) low-relaxation strands meeting the ASTM A416 requirements. Tendons were not tested in this study, but Table 6.5 presents the strand specified mechanical properties according to ASTM A416.

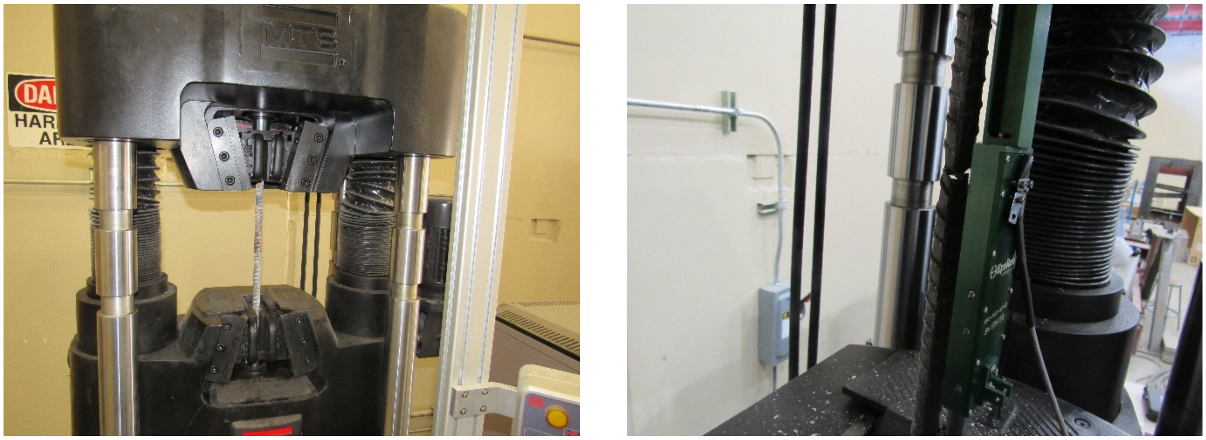
Table 6.5 Specified Mechanical Properties of Salvaged Girder Prestressing Strands

Properties	0.5 in. (12.7 mm) Strands (ASTM A416)
Yield Strength, f_y, ksi (MPa)	258 (1779)
Ultimate Strength, f_u, ksi (MPa)	285 (1965)
Strain at Break	7.4%
Modulus of Elasticity, E, ksi (MPa)	29000 (200000)

6.5.3 Properties of Reinforcing Steel Bars

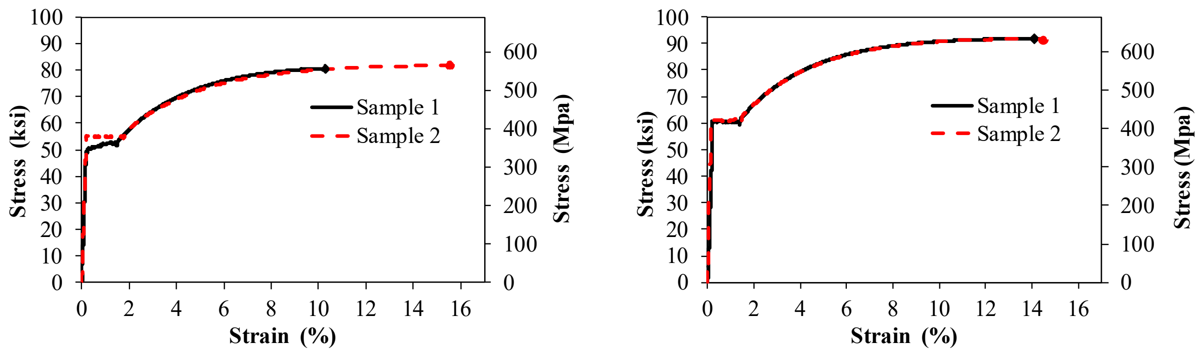
According to shop drawings of typical double-tee girders, transverse and longitudinal reinforcing steel bars used in the salvaged girders should conform to the requirements of ASTM A615 Grade 60. After girder testing, the reinforcement pattern was inspected, and sample bars were collected for tensile testing. The transverse reinforcement of the test girders was one size larger than that found in the shop drawing (No. 5 (16-mm) bars instead of No. 4 (13-mm) bars).

All extracted samples were tested according to the requirements of ASTM E8 (2016). Figure 6.23 shows one sample of the extracted bar test specimen, Fig. 6.24 shows the measured stress-strain relationships, and Table 6.6 presents a summary of the measured mechanical properties for the reinforcing steel bars used in the girders.



(a) Bar in Tensile Test Machine (b) Bar Failure

Figure 6.23 Tensile Testing of Steel Bars Extracted from Salvaged Girders



(a) Transverse No. 5 (16-mm) Bar (b) Longitudinal No. 5 (16-mm) Bar

Figure 6.24 Measured Stress-Strain Relationships for Steel Bars Extracted from Salvaged Girders

Table 6.6 Measured Mechanical Properties of Steel Bars Extracted from Salvaged Girders

		Longitudinal Bars	
Bar Size	No. 5 (16 mm)	Bar Size	No. 5 (16 mm)
Bar Spacing, in. (mm)	4 (101)	Bar Spacing, in. (mm)	8 (202)
Yield Strength, f_y, ksi (MPa)	52.5 (362)	Yield Strength, f_y, ksi (MPa)	60 (413.7)
Ultimate Strength, f_y, ksi (MPa)	81.3 (560)	Ultimate Strength, f_y, ksi (MPa)	92 (634)
Strain at Initiation of Strain Hardening, %	1.8	Strain at Initiation of Strain Hardening, %	1.4
Strain at Peak Stress, %	12.9	Strain at Peak Stress, %	14.5

Note: All values are the average of two tests.

6.5.4 Properties of Elastomeric Neoprene Bearing Pads

Mingo (2016) tested a 6-in. (152-mm) by 6-in. (152-mm) by 3/8-in. (9.5-mm) elastomeric neoprene bearing pad in compression to obtain its force-displacement relationship (Fig. 6.25). The same bearing pads were used in the present study. Stiffness of the linear region of the force-displacement relationship was 1128 kip/in (306.2 kN/mm).

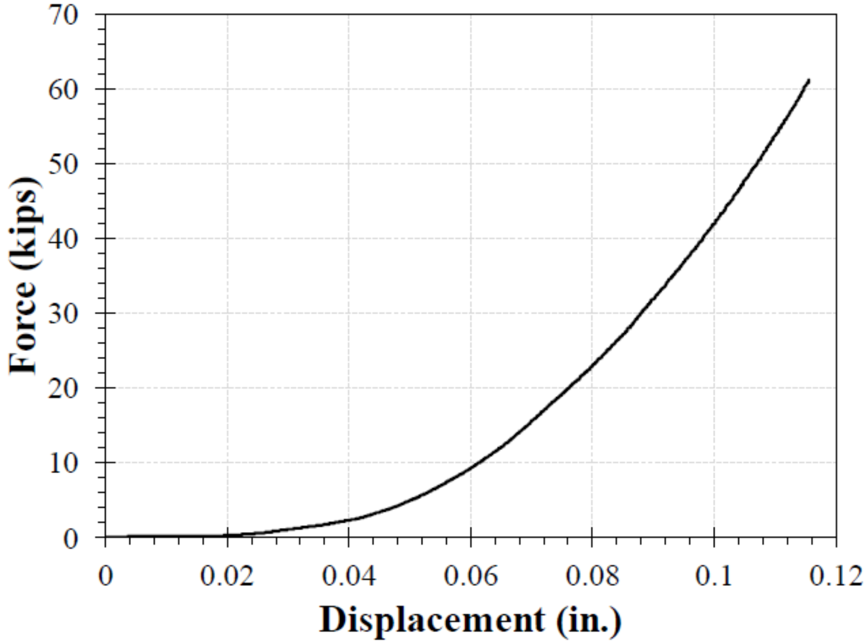


Figure 6.25 Measured Force-Displacement Relationship for Rubber Bearing Pad (Mingo, 2016)

6.6 Salvaged Girder Test Results

This section includes the experimental results of the two salvaged girders. The 50-ft (15.24-m) long girder was tested February 13, 2018, and the 30-ft (9.14-m) long girder was tested April 17, 2018.

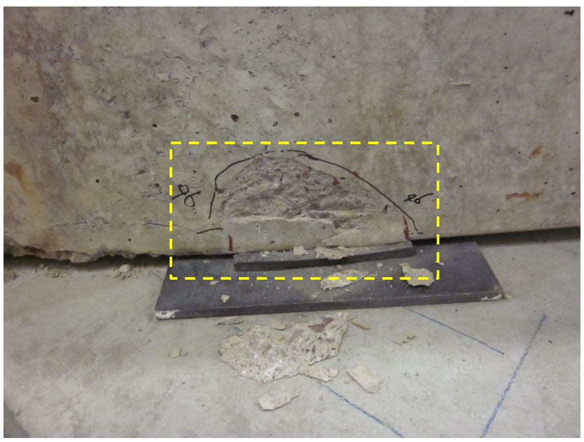
6.6.1 Strength Testing Results for 50-ft (15.24-m) Long Girder

6.6.1.1 Observed Damage

The 45-year old girder had several damages prior to testing (Fig. 6.4). As was mentioned in Sec. 6.1, this girder was cracked at the midspan during unloading from the delivery truck. The first flexural crack was observed at the midspan at a 24.9-kip (110.7-kN) load as shown in Fig. 6.26 (marked as Run No. 78). New flexural cracks developed at the midspan at higher loads (Fig. 6.26b) and the concrete spalled at the north support (Fig. 6.26c). Finally, the girder failed at the midspan in a brittle manner (Fig. 6.26d). It was concluded from the analytical study (Sec. 6.7) that the girder failure was due to failure of the flange concrete.

(a) First Flexural Crack at Midspan

(b) Extension of Flexural cracks



(c) Concrete Spalling at Support

(d) Brittle Failure at Midspan

Figure 6.26 Observed Damage of 50-ft (15.24-m) Long Girder during Strength Testing

6.6.1.2 Force-Deflection Relationship

Figure 6.27 shows the measured force-deflection relationship for the 50-ft (15.24-m) long double-tee girder. Loads equivalent to the AASHTO Service I Limit State and the AASHTO Strength I Limit State are also included in the figure. The first crack of the girder was at an actuator force of 24.9 kips (110.7 kN), which was 35% lower than the load equivalent to the AASHTO Service I Limit State. Failure mode of this girder was the compressive failure of the flange concrete at the midspan at a 5.4 in. (137 mm) of deflection. It was a brittle failure with no sign or warning while the girder was designed as a flexural member. It is clear that the girder did not meet the AASHTO requirements.

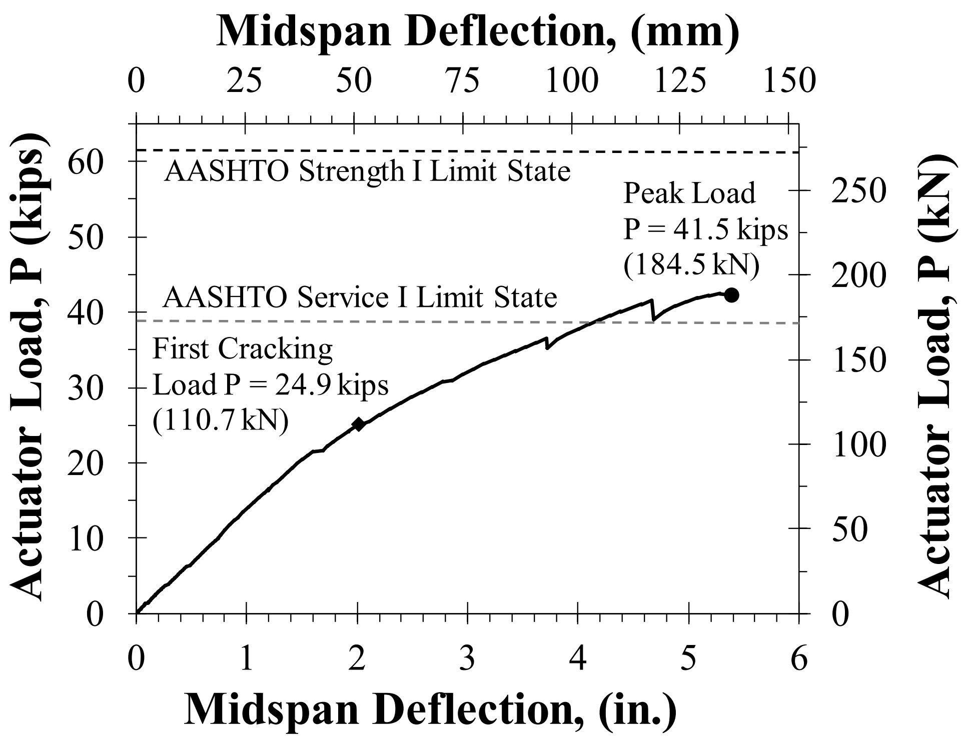


Figure 6.27 Measured Force-Deflection Relationship for 50-ft (15.24-m) Long Girder

6.6.1.3 Support Reactions

Figure 6.28 shows the measured south end stem reactions of the 50-ft (15.24-m) long girder at the peak load. It can be seen that the east stem resisted 81% more load than the west stem. The east stem was severely damaged (exposure of steel tendons) prior to delivery to the test lab.

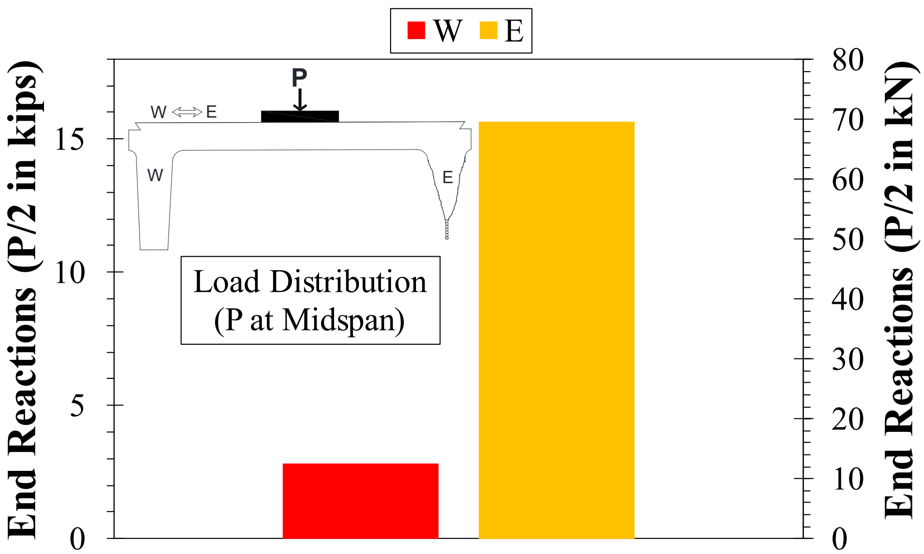


Figure 6.28 South End Support Reactions for 50-ft (15.24-m) Long Girder at Peak Load

6.6.1.4 Strain Profiles

Five concrete strain gauges (CSG) were installed on the girder flange as discussed in Sec. 6.4.1.1. Furthermore, one LVDT was installed as CSG-6 since a concrete strain gauge could not be installed due to the extent of damage at this location. Figure 6.29 shows the applied load versus the measured concrete strains. CSG-3 and CSG-6 show the highest strains compared to CSG-1 and CSG-4 because they were measuring the flange concrete strains of the east stem, which transferred higher loads to the supports.

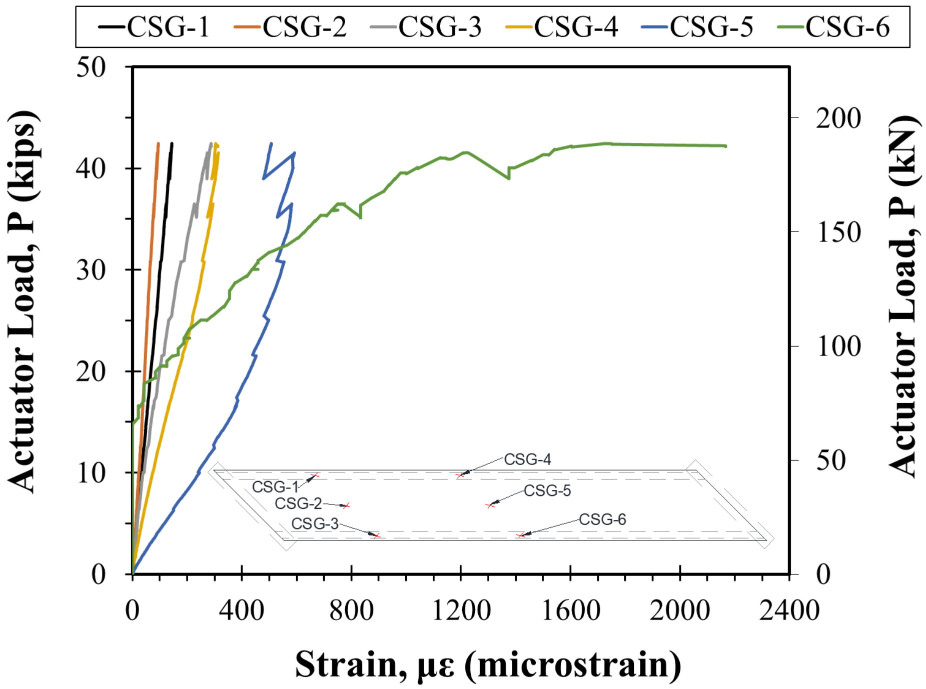
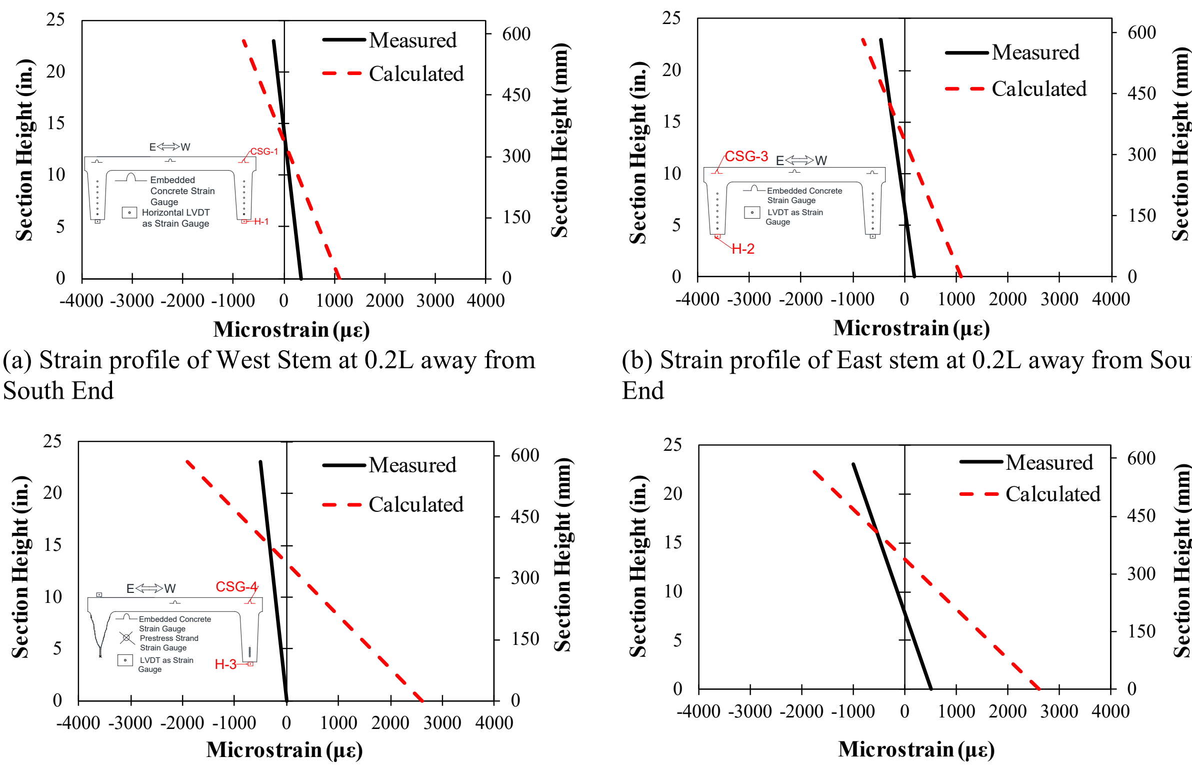


Figure 6.29 Measured Concrete Strains for 50-ft (15.24-m) Long Girder

LVDTs were also installed either at the bottom face of the stems right below the concrete strain gauges or at the top of the deck right above the steel tendon strain gauges to develop strain profiles. Figure 6.30 shows the measured and calculated (from Statics) strain profiles for the 50-ft (15.24-m) long girder at the actuator peak load. It can be seen that the calculated strains are not in good agreement with the measured strains, probably due to the extent of damage.



(a) Strain profile of West Stem at 0.2L away from South End

(b) Strain profile of East stem at 0.2L away from South End

(c) Strain Profile of West Stem at midspan

(d) Strain Profile of East Stem at midspan

Figure 6.30 Measured and Calculated Strain Profiles for 50-ft (15.24-m) Long Girder at Peak Load

6.6.1.5 Rotations

Figure 6.31 shows the rotations of the girder measured in the longitudinal direction at the midspan (LR in Fig. 6.13). The rotations were measured using two LVDTs, one was installed at the top of the deck (LR-2) and another was installed underneath the flange (LR-1). The maximum rotation was 0.19 degrees at the peak load of 41.9 kips (186.4 kN).

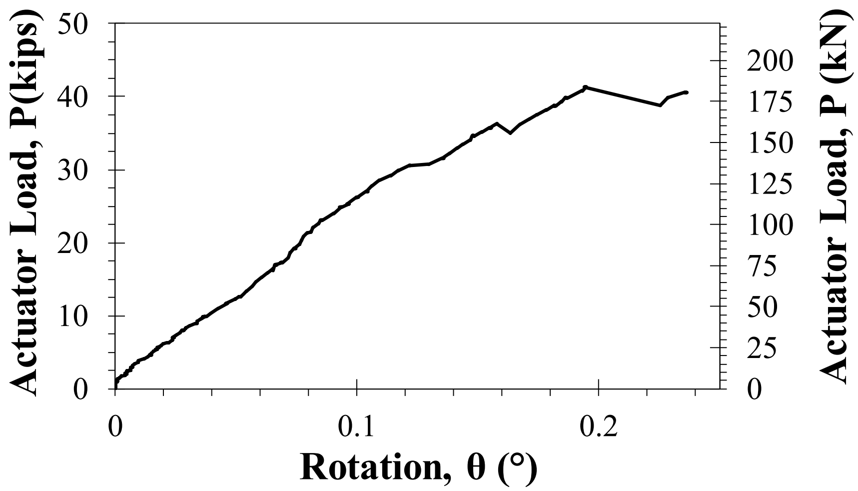


Figure 6.31 Measured Flange Longitudinal Rotations for 50-ft (15.24-m) Long Girder

6.6.2 Strength Testing Results for 30-ft (9.14-m) Long Girder

The results of strength testing on the 30-ft (9.14-m) girder is discussed here.

6.6.2.1 Observed Damage

The 45-year old girder had several damages prior to testing (Fig. 6.4). The north end of the girder had more prior-to-testing apparent damage than the south end. That is probably why the first crack occurred near the north end (Fig. 6.32a, marked as Run No. 34), 10 ft (3.05 m) away from the midspan at an actuator load of 15.3 kips (68.06 kN). The first flexural crack was observed at a distance of 5 ft (1.5 m) from the midspan at a 22.41-kip (99.68-kN) load as shown in Fig. 6.32b (marked as Run No. 47). The width of cracks extended, and new cracks formed, at higher loads (Fig. 6.32c & d). Finally, the girder failed in flexure at the midspan (a major flexural crack as marked in Fig. 6.32e), which was a ductile failure.

(a) First Shear Crack at North End

(b) Flexural Crack Near to Midspan

(c) Shear Crack Near North End

(d) Extension of Crack Width

(e) A Major Flexural Crack at Midspan– Stem inside View

Figure 6.32 Observed Damage of 30-ft (9.14-m) Long Girder during Strength Testing

6.6.2.2 Force-Deflection Relationship

Figure 6.33 shows the measured force-deflection relationship for the 30-ft (9.14-m) long double-tee girder. Loads equivalent to the AASHTO Service I and Strength I Limit States were also included in the figure. The first crack of the girder was at a force of 15.3 kips (68.1 kN), which was 44% lower than the load equivalent to the AASHTO Service I Limit State. This girder failed at a 2.3-in. (58-mm) deflection with a major flexural crack at the midspan. It is clear that the girder did not meet the AASHTO requirements.

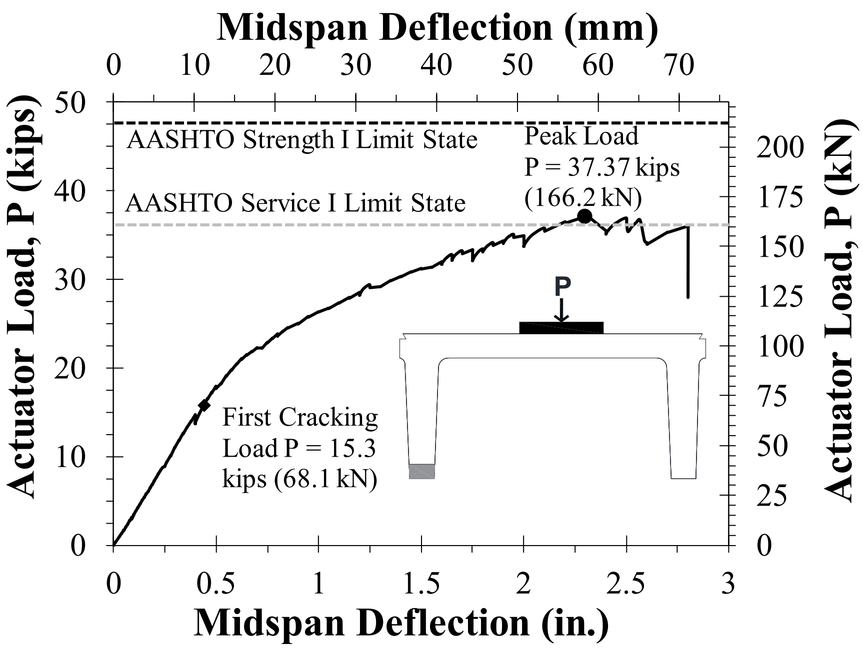


Figure 6.33 Measured Force-Deflection Relationship for 30-ft (9.14-m) Long Girder

6.6.2.3 Support Reactions

Figure 6.34 shows the measured south end stem reactions of the 30-ft (9.14-m) long girder at the peak load. The east stem resisted 37.8% more load than the west stem.

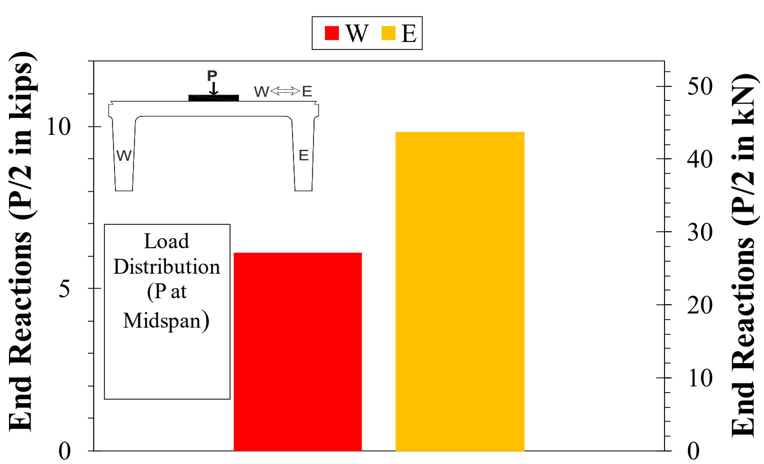


Figure 6.34 South End Support Reactions for 30-ft (9.14-m) Long Girder at Peak Load

6.6.2.4 Strain Profiles

Six concrete strain gauges (CSG) were installed at the girder flange as discussed in Sec. 6.4.2.1. Figure 6.35 shows the applied load versus measured concrete strains. An approximately linear behavior can be recognized for all gauges. The gauges at the girder midspan (CSG-4 to 6) exhibited the largest strains.

In addition to CSGs, LVDTs and surface-mount strain transducers were used at different depths of the stems to develop strain profiles. Refer to Sec. 6.4.2 for the instrumentation plan. Figure 6.36 shows the measured and calculated strain profiles for the 30-ft (9.14-m) long girder. Similar to the 50-ft (15.24-m) long girder, the calculated strains did not match well with the measured data probably due to the extent of the girder damage and the type of strain sensors used. Strain profiles are usually obtained using embedded concrete and steel strain gauges. Nevertheless, this could not be achieved in the present study to preserve the salvaged girders as received and to avoid further damage. Some strains were measured using LVDTs. This strain measuring method was found unreliable.

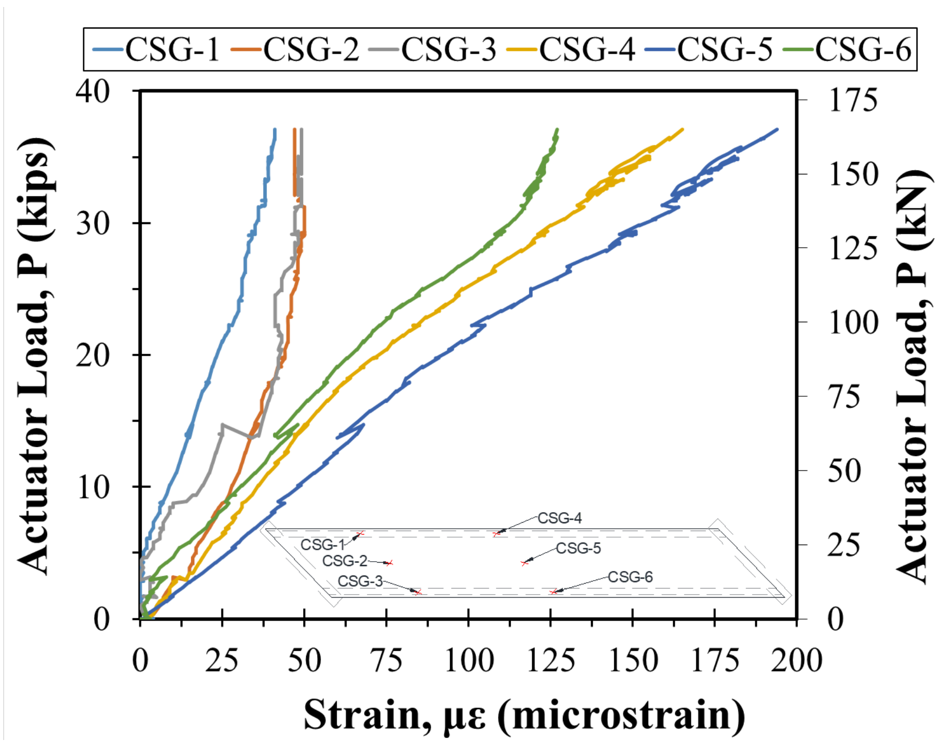
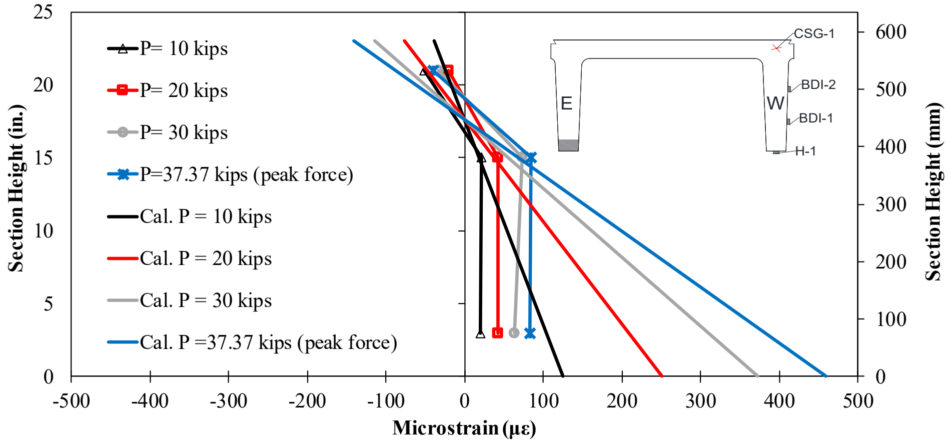
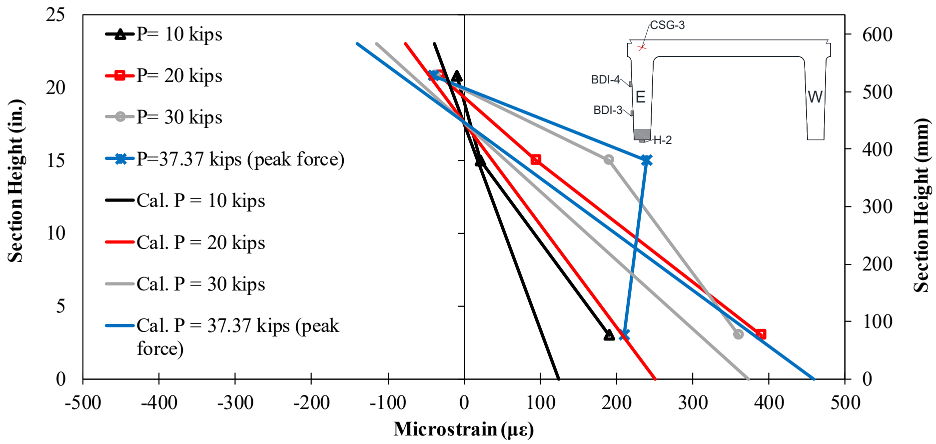


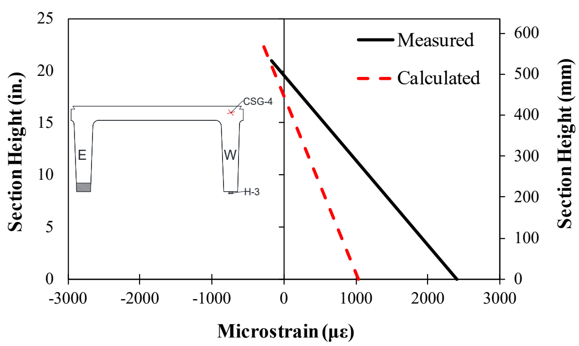
Figure 6.35 Measured Concrete Strains for 30-ft (9.14-m) Long Girder



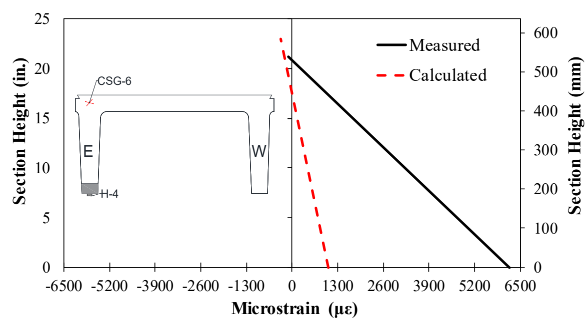
(a) Strain Profile of West Stem at 0.2L away from South End at Various Load



(b) Strain Profile of East Stem at 0.2L away from South End at Various Load



(c) Strain Profile of West Stem at Midspan at Peak Load



(d) Strain Profile of East Stem at Midspan at Peak Load

Figure 6.36 Measured and Calculated Strain Profiles for 30-ft (9.14-m) Long Girder

6.6.2.5 Rotations

Figure 6.37 shows rotations of the girder measured in the longitudinal direction at the midspan (LR in Fig. 6.16). The rotations were measured using two LVDTs. One was installed at the top of the deck (LR-2) and another was installed underneath the flange (LR-1). The maximum rotation was 0.03 degrees at the peak load of 37.37 kips (166.2 kN).

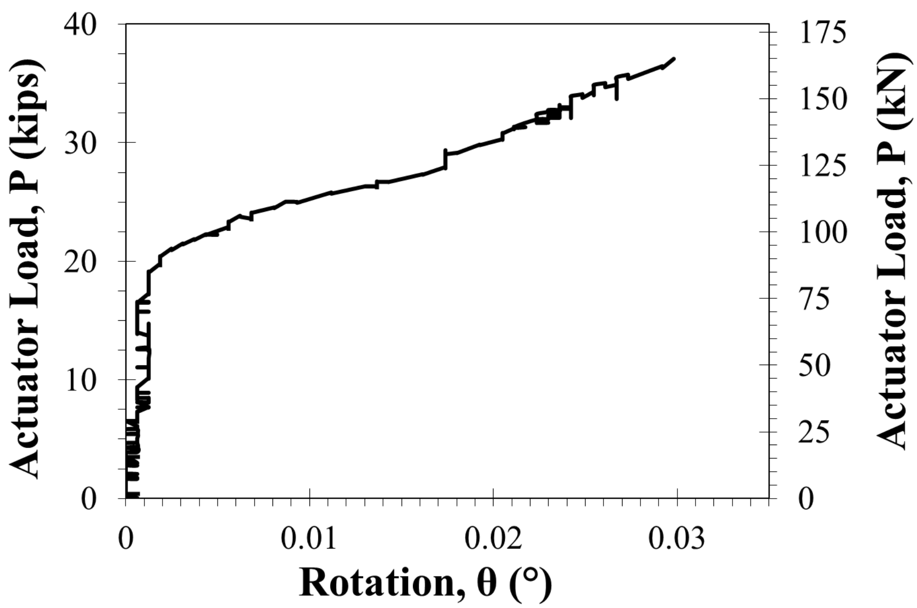


Figure 6.37 Measured Longitudinal Rotations for 30-ft (9.14-m) Long Girder

6.6.2.6 Decompression Test

Accurate estimation of prestressing losses is important for the analysis and design of prestressed or posttensioned concrete members. Decompression tests have been conducted in some studies (Pessiki et al., 1996; Osborn et al., 2012) to estimate tendon stress losses. The test is done by loading the member until the first flexural crack is developed, unloading the specimen to install a long strain gauge crossing the flexural crack at the extreme tensile face, then loading the specimen to reopen the crack. The measured load and strain data can be used to identify the cracking load (and also the cracking moment). Subsequently, Eq. 6.1 can be used to determine the actual (or effective) prestressing forces of the section.

$$f = -P\left(\frac{1}{A_c} + \frac{e.y_t}{I_g}\right) + \frac{M.y_t}{I_g} \qquad \text{(Eq. 6.1)}$$

where

f = The stress at the tensile face of the section (zero at the crack),

P = The section effective prestressing force,

A_c = The cross-sectional area at the crack location,

e = The eccentricity of the prestressing force at the crack location,

y_t = The distance between the neutral axis and the tensile face of the section,

I_g = The section moment of inertia at the crack location,

M = The moment in the member due to the cracking load.

This test was performed on the 30-ft (9.14-m) long girder to estimate the prestressing losses. The girder was monotonically loaded using a small displacement increment until a crack was observed on the stem of the girder. Subsequently, the girder was unloaded and reloaded until the initial crack redeveloped at the same location (5 ft [1.52 m] away from the girder centerline on the east side). Due to the time limitation, no strain gauge was installed at the cracked section, but a narrow displacement increment was used to determine the load before and after forming the crack. The load prior to cracking was 15.3 kips (68.1 kN) and after observing the crack was 22.41 kips (99.7 kN). For this girder, A_c was 353.72 in^2 (228206 mm^2), e was 6.27 in. (159 mm), y_t was 6.73 in. (171 mm), and I_g was 13964 in.^{4} (5812255627 mm^4). The estimated prestressing loss for the 45-yr old 30-ft (9.14-m) girder was estimated to be between 52.4% and 70.4%. This is a significant stress loss and must be included the shear and moment capacity calculation of damaged double-tee girders. Further discussion of the topic can be found in Sec. 7.3.

6.7 Capacity Calculation for Damaged Double-Tee Girders

This section presents methods to calculate capacity of the two salvaged girders tested in the present study. Experimental data from the literature is also included to further validate capacity calculation methods.

Table 6.7 presents a summary of parameters used for the capacity calculation of the two girders. One method to calculate the moment capacity of a reinforced concrete or a prestressed section is through a moment-curvature analysis. SAP2000 (2018) was used to perform this analysis in the present study. Moment capacity can also be calculated using Equations 6.2 through 6.5 presented below (from AASHTO LRFD, 2012).

Table 6.7 Parameters Used in Capacity Calculation of Salvaged Girders

Parameters	Notation	50 ft (15.24 m) Long Girder	30 ft (9.14 m) Girder
Area of Tendons	A_{ps}	2.75 in^2 (1774 mm^2)	1.57 in^2 (1013 mm^2)
Stress in Tendons	f_{ps}	246.6 kips (1096.9 kN)	238.9 kips (1062.7 kN)
Distance from extreme compression fiber to the centroid of tendons	d_p	18.45 in. (468 mm)	11 in. (279 mm)
Area of Tensile Steel	A_s	No tensile steel	No tensile steel
Stress in Tensile Steel	f_s	N/A	N/A
Stress in Compression Reinforcement	f'_s	60 ksi (413.7 MPa)	60 ksi (413.7 MPa)
Distance from extreme compression fiber to the centroid of tensile steel.	d_s	N/A	N/A
Area of Compression Reinforcement	A'_s	1.23 in.^{2} (793 mm^2)	1.23 in.^{2} (793 mm^2)
Distance from extreme compression fiber to the centroid of compression reinforcement	d'_s	4 in. (101 mm)	2.9 in. (73 mm)
Compressive strength of concrete	f'_c	1.92 ksi (13.24 MPa) for Flange	1.92 ksi (13.24 MPa) for Flange
Width of the section	b	60 in. (1524 mm)	61 in. (1529 mm)
Web width	b_w	10 in. (254 mm)	10 in. (254 mm)
Stress block factor	β	0.8	0.8
Compression flange depth	h_f	3.81 in. (97 mm)	4 in. (97 mm)
Effective web width	b_v	10 in. (254 mm)	10 in. (254 mm)
Effective shear depth	d_v	16.8 in. (427 mm)	9.77 in. (248 mm)
Area of shear reinforcement within a distance s.	A_v	0.61 in.^{2} (39355 mm^2)	0.61 in.^{2} (39355 mm^2)
Spacing of transverse reinforcement	s	5 in. (127 mm)	5 in. (127 mm)

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_sf_s\left(d_s - \frac{a}{2}\right) - A'_sf'_s\left(d'_s - \frac{a}{2}\right) + 0.85f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad \text{(Eq. 6.2)}$$

where

M_n = The nominal moment capacity,

A_{ps} = The area of prestressing steel (in.^{2}),

f_{ps} = The average stress in prestressing steel at nominal bending resistance (ksi),

d_p = The distance from extreme compression fiber to the centroid of prestressing tendons (in.),

A_s = The area of nonprestressed tension reinforcement (in.^{2}),

f_s = The stress in the mild steel tension reinforcement at nominal flexural resistance (ksi),

d_s = The distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement,

A'_s = The area of compression reinforcement (in.^{2}),

d'_s = The distance from extreme compression fiber to the centroid of compression reinforcement (in.),

f'_c = The specified compressive strength of concrete at 28 days, unless another age is specified (ksi),

b = The width of the compression face of the member; for a flange section in compression,

b_w = The web width or diameter of a circular section (in.),

β_1 = The stress block factor specified in AASHTO LRFD Article 5.7.2.2,

h_f = The compression flange depth of an I or T member (in.),

a = $c\beta_1$; The depth of equivalent stress block.

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(Eq. 6.3)}$$

where

f_{pu} = The specified tensile strength of prestressing strand (ksi).

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{(Eq. 6.4)}$$

c = The distance between neutral axis and compression face as defined in Eq. 3.5.

f'_s = The stress in the mild steel compression reinforcement at nominal flexural resistance (ksi).

$$c = \frac{A_{ps}f_{pu} + A_sf_s - A'_sf'_s}{0.85f_c\beta_1 b + kA_{ps}\frac{f_{pu}}{d_p}} \qquad \text{(Eq. 6.5)}$$

where,

b = The width of compression flange.

The shear capacity can also be calculated using the AASHTO methods:

$$V_n = V_c + V_s + V_p \qquad \text{(Eq. 6.6)}$$

$$V_c = 0.316\beta\sqrt{f'_c}\, b_v\, d_v \qquad \text{(Eq. 6.7)}$$

$$V_S = \frac{A_v f_y d_v \cot\theta}{s} \qquad \text{(Eq. 6.8)}$$

where

V_p = Component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear,

b_v = The effective web width,

d_v = The effective shear depth,

A_v = The area of shear reinforcement within a distance s (in.^{2}),

s = The spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.),

β = The factor indicating ability of diagonally cracked concrete to transmit tension and shear,

θ = The angle of inclination of diagonal compressive stresses.

6.7.1 Calculated Capacities of 50-ft (15.24-m) Long Girder

The 50-ft (15.24-m) long girder failed in a brittle manner at the midspan with a load of 41.9 kips (186.4 kN). Table 6.8 presents the calculated capacities of this girder including all damages. The calculated shear and moment capacities for this girder at the failure section were 65.4 kips (290.9 kN) and 846.41 kip.ft (1148 kN.m), respectively. The equivalent calculated load carrying capacities (a point load at the midspan, $P_{calculated}$) were respectively 130.8 kips (581.8 kN) and 82.74 kips (368.05 kN) based on the shear and moment capacities. Therefore, this girder did not fail under the shear or bending at the failure section.

To find the failure mode of the 50-ft (15.24-m) girder using analytical tools, it was assumed that the stems do not contribute to shear capacity of the girder due to the extent of the stem damage at the midspan (Fig. 6.4f). Therefore, shear capacity of this girder at the midspan consists of only the shear capacity of the flange concrete (as a slab). Using Eq. 6.7, the flange shear capacity was estimated as 19.9 kips (88.52 kN) equivalent to a calculated load carrying capacity (a point load at the midspan, $P_{calculated}$) of 39.93 kips (177.6 kN), which was only 4.7% lower than the measured peak load. Therefore, the 50-ft (15.24-m) long girder failed by the shear failure of the flange concrete, which is a brittle failure. It is worth mentioning that this finding was used in calculation of the capacity modification factors, when the stem had exposed tendons.

Table 6.8 Calculated Shear and Moment Capacities for Salvaged Double-Tee Girders

Salvaged Girder	Shear Capacity	Moment Capacity	Failure Load, P, kips (kN)
50-ft (15.24-m) Long	V_n = 65.4 kips (290.9 kN) for Section; Equivalent P = 130.8 kips (581.8 kN)	M_n = 685.13 k-ft (928.9 kN-m) using M-Φ Analysis; Equivalent P = 66.9 kips (297.6 kN)	$P_{calculated}$ = 39.93 (177.6)
	V_n = 19.9 kips (88.52 kN) for Flange Only; Equivalent P = 39.93 kips (177.6 kN)	M_n = 688.67 k-ft (933.72 kN-m) using AASHTO; Equivalent P = 67.3 kips (299.4 kN)	$P_{measured}$ = 41.9 (186.4) (4.7% difference)
30-ft (9.14-m) Long	V_n = 64.7 kips (287.8 kN) for Section; Equivalent P = 129.4 kips (575.6 kN)	M_n = 223.58 k-ft (303.5 kN-m) using M-Φ Analysis; Equivalent P = 35.89 kips (159.65 kN)	$P_{calculated}$ = 35.89 (159.6)
		M_n = 278 k-ft (377.3 kN-m) using AASHTO; Equivalent P = 44.6 kips (198.4 kN)	$P_{measured}$ =37.37 (166.2) (3.9% difference)

6.7.2 Calculated Capacities of 30-ft (9.14-m) Long Girder

The 30-ft (9.14-m) long girder failed in a ductile manner at the midspan with a load of 37.37 kips (166.23 kN). Table 6.8 also presents the calculated capacities of this girder including all damages. The calculated shear and moment capacities for this girder at the failure section were 64.7 kips (287.8 kN) and 223.58 kip.ft (303.5 kN.m), respectively. The equivalent calculated load carrying capacities (a point load at the midspan, $P_{calculated}$) were respectively 129.4 kips (575.6 kN) and 35.89 kips (159.65 kN), which is 3.9% higher than measured load. Therefore, the 30-ft (9.14-m) long girder failed by the flexural failure of the section, which is a ductile failure.

6.7.3 Summary of Capacity Calculation Methods for Salvaged and Damaged Girders

Based on experimental findings of the present study and other test data collected from the literature, the proposed capacity calculation methods for salvaged or damaged girders was further verified. Table 6.9 presents a summary of the analysis. It can be inferred that the available methods can estimate capacities of damaged girders with reasonable accuracy. The error between the calculated and measured peak loads was not more than 13% in all cases.

Overall, it is recommended to use a moment-curvature analysis in the calculation of moment capacity for damaged girders by including the damage in the analytical model. The shear capacity of a damaged girder may be calculated using current AASHTO method.

Table 6.9 Measured and Calculated Load Capacity for Different Salvaged Girders

Refer.	Sec. Type	Age (yr.)	Span, ft (m)	Girder Damage Type	Width in. (mm)	Depth in. (mm)	Concrete Strength, ksi (MPa)	Tendon Yield, f_y, ksi (MPa)	Measured Peak Load kips (kN)	Calculated Peak Load kips (kN)
Shenoy et al. (1991)	Box	27	54 (16.5)	Minor concrete cracking and spalling	36 (914)	27 (686)	7.1 (48.9)	150 (1034.2)	104.1 (463.1)	109.7 (487.9)
Halsey et al. (1996)	Inverted Tee	40	29 (8.8)	Minor deterioration at the girder edges	12 (305)	12 (305)	11.79 (81.3)	260 (1792.6)	46.9 (208.6)	50.2 (223.3)
Labia et al. (1997)	Box	20	70 (21.3)	No apparent damage	48 (1219)	33 (838)	5.5 (37.9)	270 (1861.6)	161 (716.2)	181.7 (808.2)
Eder et al. (2010)	I	50	45 (13.7)	Longitudinal cracks along post-tensioning tendons	16 (406)	40 (1016)	9.8 (67.6)	150 (1034.2)	146.6 (652.1)	162.2 (721.5)
Pettigrew et al. (2016)	Double Tee	48	53 (16.1)	Deteriorated and exposure of rebar at some location	84 (2133)	28 (711)	5.6 (38.6)	278 (1916.7)	105.5 (469.3)	106.41 (473.3)
50-ft Girder, Present Study	Double Tee	45	45.6 (13.9)	Table 6.1	40 (1016)	23 (584)	2.54 (17.5)	258 (1779)	41.9 (186.4)	39.93 (177.6)
30-ft Girder, Present Study	Double Tee	45	27.9 (8.5)	Table 6.1	44 (1117.6)	23 (584)	2.54 (17.5)	258 (1779)	37.37 (166.2)	35.89 (159.6)

7. CALCULATION OF DAMAGED DOUBLE-TEE GIRDER MOMENT AND SHEAR CAPACITIES

A successful load rating of distressed double-tee girder bridges should include the effect of damage on capacities of the girders. Results of full-scale strength testing on two salvaged double-tee girders were discussed in the previous chapter, and methods of estimation of shear and moment capacities for damaged double-tee girders were verified using these and other large-scale girder test data.

According to the AASHTO Manual for Bridge Evaluation (2011), "condition factors, φ_c" are used to include bridge superstructure damage in the load rating equation (refer to Sec. 3.2.1). However, specific condition factors should be developed for any possible damage of double-tee girders. Damage types specific to South Dakota double-tee girders were identified and categorized in Chapter 4. In an attempt to minimize variations from current codes, it was proposed to include the damage of a double-tee girder in the load rating equation through the use of the "condition factor," which is defined in the present study as the ratio of the damaged girder capacity to the undamaged girder capacity.

Review of available construction detailing and inspection reports revealed there are 23 different double-tee girder sections, which have been used in the state. Condition factors for moment and shear should be developed for each of these double-tee sections including different damage types and condition states. As discussed in the previous chapter, the moment capacity of a damaged prestressed girder can be calculated using a moment-curvature analysis and the shear capacity can be estimated using the AASHTO LRFD method.

In this chapter, the methods of calculation of moment and shear capacities for damaged double-tee sections were discussed including steps taken to develop the moment and shear condition factors for damaged double-tee girder stems and flanges. Finally, a summary of the findings for the 23 double-tee girder sections is presented in a tabulated format.

7.1 Stem Moment and Shear Capacities

Four damage types, each with four condition states, were defined for the stem of double-tee girders (Table 4.1). The steps and scenarios assumed to include such damages in the girder moment and shear capacities are discussed herein.

7.1.1 Stem Cover Deterioration including Delamination/Spall/Patched Area

The stem concrete cover may deteriorate from the girder inside, outside, or bottom face. Cover deterioration can be included in the capacity estimation method by removing the deteriorated concrete cover from the section. The stem concrete cover removal scenarios for the four condition states are discussed in this section.

7.1.1.1 Stem Cover Deterioration with Condition State 1 (CS-1)

No damage of the stem concrete cover is assumed under CS-1, therefore, capacity of the damaged girder in this state is the same as that for an undamaged girder (Fig. 7.1).

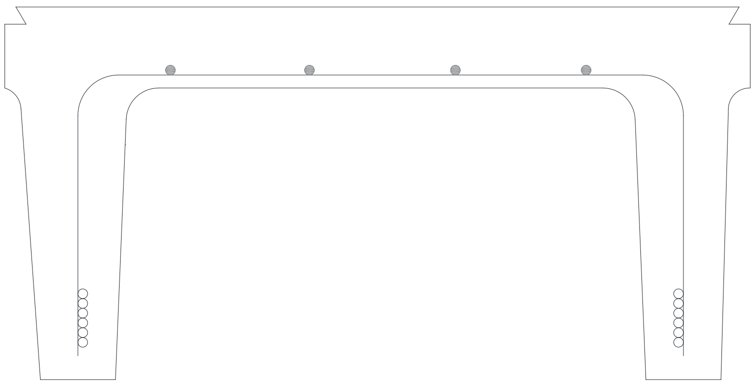
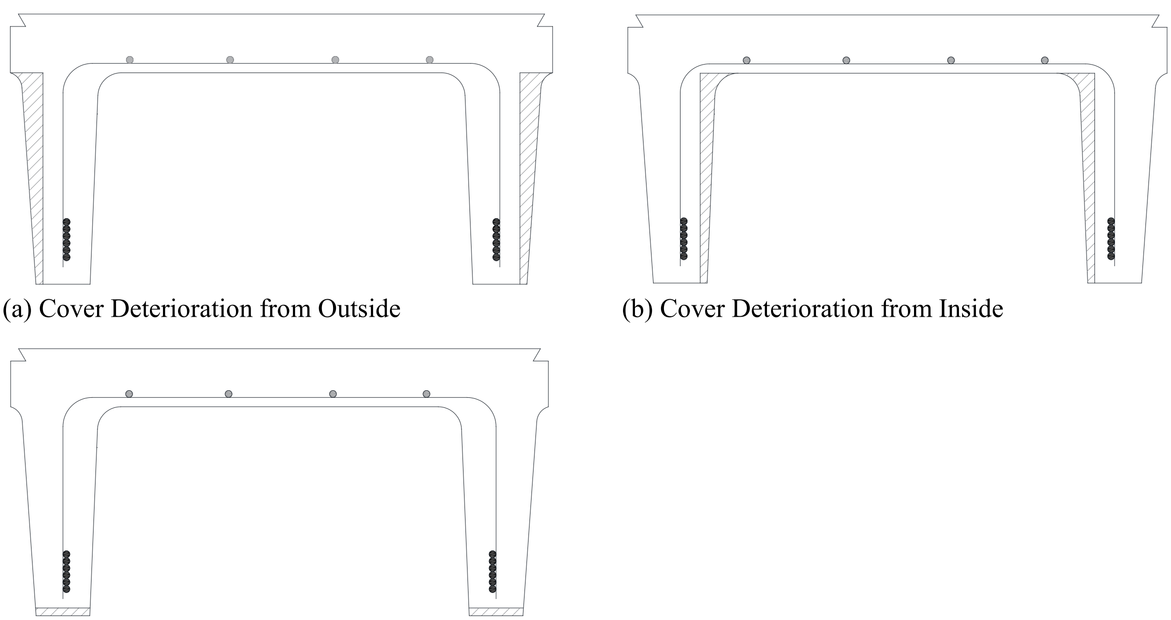


Figure 7.1 Stem Cover Deterioration with Condition State 1 equivalent to Undamaged Section

7.1.1.2 Stem Cover Deterioration with Condition State 2 (CS-2)

This damage condition state was defined as the "loss of one-third of the cover without exposure or corrosion of the reinforcement" (Table 4.1). To include this damage in the capacity calculation, one-third of the stem concrete cover was removed from the outside (Fig. 7.2a), inside (Fig. 7.2b), and bottom (Fig. 7.2c) face of the stem. Moment-curvature analyses were performed for these sections and the worst-case scenario (the lowest value) was reported as the condition factor. The same process was used to calculate the shear condition factors for the stem cover deterioration in which the web width, b_v, was reduced in the V_c component of the shear capacity equation (Eq. 6.7).



(a) Cover Deterioration from Outside

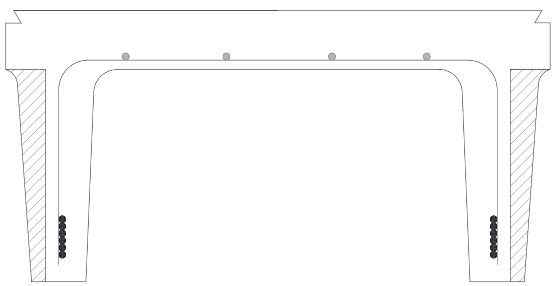
(b) Cover Deterioration from Inside

(c) Cover Deterioration from Bottom

Figure 7.2 Stem Cover Deterioration with Condition State 2

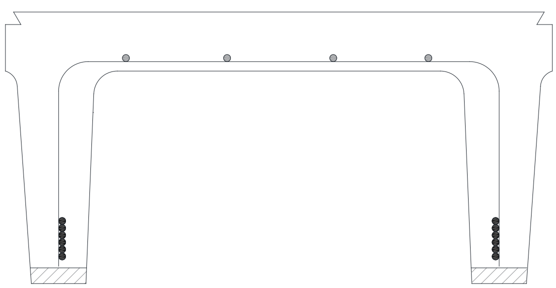
7.1.1.3 Stem Cover Deterioration with Condition State 3 (CS-3)

This damage condition state was defined as the "loss of 2/3 of the cover without exposure or corrosion of reinforcement" (Table 4.1). To include this damages in the capacity calculation, 2/3 of the stem concrete cover was removed from the outside (Fig. 7.3a), inside (Fig. 7.3b), and bottom (Fig 9.3c) face of the stem. Moment-curvature analyses were performed for these sections and the worst-case scenario (the lowest value) was reported as the condition factor. The same process was used to calculate the shear condition factors for the stem cover deterioration, in which the web width, b_v, was reduced in the V_c component of the shear capacity equation (Eq. 6.7).



(a) Cover Deterioration from Outside

(b) Cover Deterioration from Inside

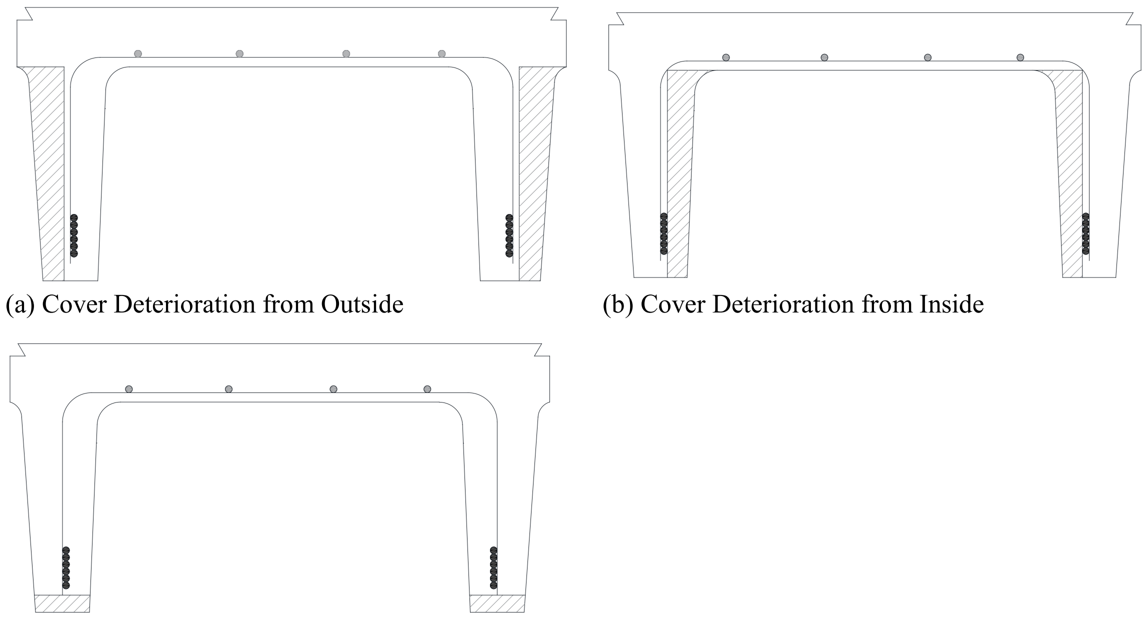


(c) Cover Deterioration from Bottom

Figure 7.3 Stem Cover Deterioration with Condition State 3

7.1.1.4 Stem Cover Deterioration with Condition State 4 (CS-4)

This damage condition state was defined as "exposure of reinforcement without any sign of corrosion." To include this damage in the capacity calculation, the stem concrete cover was completely removed from the outside (Fig. 7.4a), inside (Fig. 7.4b), and bottom (Fig. 7.4c) face of the stem. oment-curvature analyses were performed for these sections and the worst-case scenario (the lowest value) was reported as the condition factor. Refer to Sec. 7.1.4 regarding the effect of this damage type on the shear capacity.



(a) Cover Deterioration from Outside (b) Cover Deterioration from Inside

(c) Cover Deterioration from Bottom

Figure 7.4 Stem Cover Deterioration with Condition State 4 for Moment Capacity Calculation

7.1.2 Stem Exposed Transverse Bar

This damage type includes corrosion of the stem transverse bars in the moment and shear capacities. One may assume that the stem transverse bars will be exposed when the stem cover is fully lost. However, since this was addressed under the “stem cover deterioration” and the stem transverse bar may corrode without significant damage of the cover, only the transverse steel bar area was modified under this damage type to include the corrosion in the capacity calculations.

For double-tee girders, it was found that this damage has insignificant effect on the moment capacity since the girder neutral axis is usually inside the flange. However, this damage type will affect the shear capacity since the transverse bar area will be modified accounting for the bar corrosion.

7.1.2.1 Stem Exposed Transverse Bar Damage with Condition State 1 (CS-1)

This damage condition state was defined as “none” (Table 4.1). Therefore, the shear capacity remains the same as that for the undamaged section.

7.1.2.2 Stem Exposed Transverse Bar Damage with Condition State 2 (CS-2)

This damage condition state was defined as a “minor corrosion of the reinforcement with minimal section loss” (Table 4.1). To include this damage in the calculation of the shear capacity, the area of the stem transverse steel bars only for one leg (or stem) was reduced by 25%, which affects the V_s component of the shear capacity equation (Eq. 6.8).

7.1.2.3 Stem Exposed Transverse Bar Damage with Condition State 3 (CS-3)

This damage condition state was defined as a “severe corrosion of only one leg of transverse reinforcement” (Table 4.1). To include this damage in the calculation of the shear capacity, the area of the stem transverse steel bars only for one leg (or stem) was reduced by 50%, which affects the V_s component of the shear capacity equation (Eq. 6.8).

7.1.2.4 Stem Exposed Transverse Bar Damage with Condition State 4 (CS-4)

This damage condition state was defined as a “severe corrosion of all legs of transverse reinforcement” (Table 4.1). To include this damage in the calculation of the shear capacity, the area of the stem transverse steel bars for both legs (or both stems) was reduced by 50%, which affects the V_s component of the shear capacity equation (Eq. 6.8).

7.1.3 Stem Exposed Longitudinal Prestressing

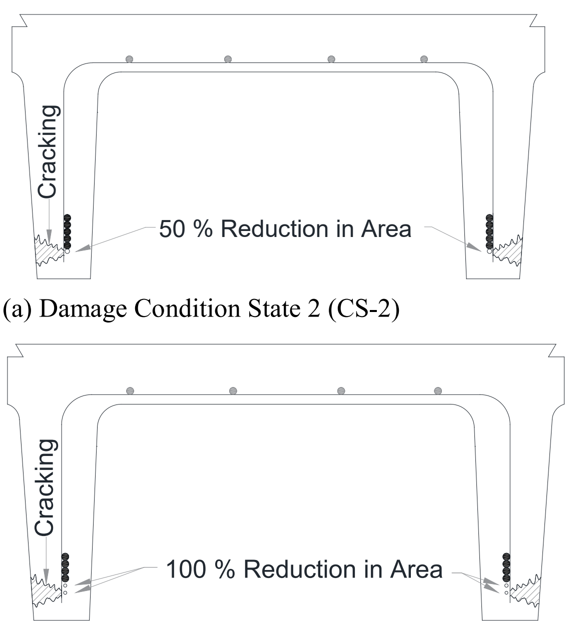
This damage type includes the effect of prestressing tendon corrosion in the calculation of shear and moment capacities, using a similar technique discussed for the “stem exposed transverse bar.” The area of stem tendons will be reduced to account for corrosion. This damage type will affect shear and moment capacities of double-tee girders.

7.1.3.1 Stem Exposed Longitudinal Prestressing Damage with Condition State 1 (CS-1)

This damage condition state was defined as “none” (Table 4.1). Therefore, the shear and moment capacities remain the same as those for the undamaged section.

7.1.3.2 Stem Exposed Longitudinal Prestressing Damage with Condition State 2 (CS-2)

This damage condition state was defined as a “50% section loss due to corrosion in the extreme tendon” (Table 4.1). To include this damage in analyses, the area of the extreme tendon for both stems was reduced by 50% (Fig. 7.5a). Moment-curvature analyses were performed to calculate the flexural capacity of the damaged sections. For the calculation of the shear capacity, a decrease in the area of extreme tendon shifts the tendon center of gravity up reducing d_v thus the V_c and V_s components of the shear strength equation (Eq. 6.7 & 6.8).



(a) Damage Condition State 2 (CS-2)

(b) Damage Condition State 3 (CS-3)

(c) Damage Condition State 4 (CS-4)

Figure 7.5 Stem Tendon Exposure

7.1.3.3 Stem Exposed Longitudinal Prestressing Damage with Condition State 3 (CS-3)

This damage condition state was defined as a "100% section loss due to corrosion in the extreme tendon" (Table 4.1). To include this damage in analyses, the area of the extreme tendon for both stems was reduced by 100% (Fig. 7.5b). Moment-curvature analyses were carried out to calculate the flexural capacity of the damaged section. For the calculation of the shear capacity, a decrease in the area of extreme tendon shifts the tendon center of gravity up reducing d_v thus the V_c and V_s components of the shear strength equation (Eq. 6.7 & 6.8).

7.1.3.4 Stem Exposed Longitudinal Prestressing Damage with Condition State 4 (CS-4)

This damage condition state was defined as a "section loss due to corrosion in the two or more tendons" (Table 4.1). To include this damage in analyses, the area of the two extreme tendons for both stems was reduced by 100% (Fig. 7.5c). Moment-curvature analyses were performed to calculate the flexural capacity of the damaged section. The same method discussed in the previous section was used for the calculation of the shear capacity.

7.1.4 Stem Cracking

Figure 7.6 shows three types of cracks which may be observed in the stem of a double-tee girder: (1) debonding cracks caused by the bond failure between a tendon and its surrounding concrete, (2) stem-to-flange longitudinal cracks possibly caused by an insufficient detailing, and (3) shear cracks. Each will happen at a different location and depth of a girder making this damage type a challenge to include in the shear and moment capacity calculations.

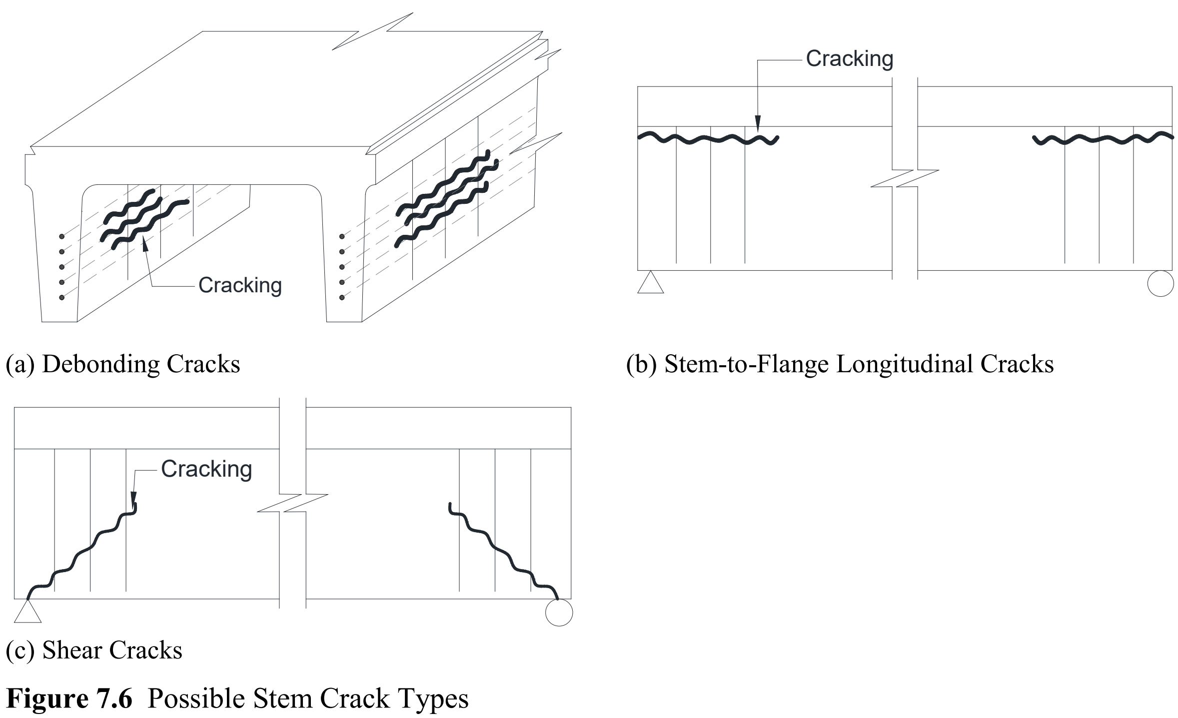


Figure 7.6 Possible Stem Crack Types

Since the neutral axis of all 23 double-tee sections is inside the flange or close to the flange, it can be assumed that the stem cracks have a minimal effect on the moment capacity. However, the shear capacity of the section will change if any of these damage types (or stem cover deterioration with CS 4) are seen.

To include the effect of debonding cracks on the shear capacity, it was assumed that the stem concrete below the crack was fully lost and then the V_c component of the shear capacity equation (Eq. 6.7) was modified using the reduced effective shear depth, d_v. Furthermore, a portion of the transverse reinforcing bars is exposed in this case and does not contribute to the shear capacity. To include this condition in analyses, the maximum bar stress that can be developed excluding the exposed portion of the transverse bar was estimated using Eq. 7.1. Subsequently, the V_s component of the shear capacity equation (Eq. 6.8) was modified using the reduced bar strength and the reduced effective shear depth, d_v. Furthermore, the V_p is zero in this case.

$$f_s = \frac{development\ length - length\ of\ stem\ bottom\ face\ cover\ deterioration}{development\ length} * f_y \quad \text{(Eq. 7.1)}$$

where,

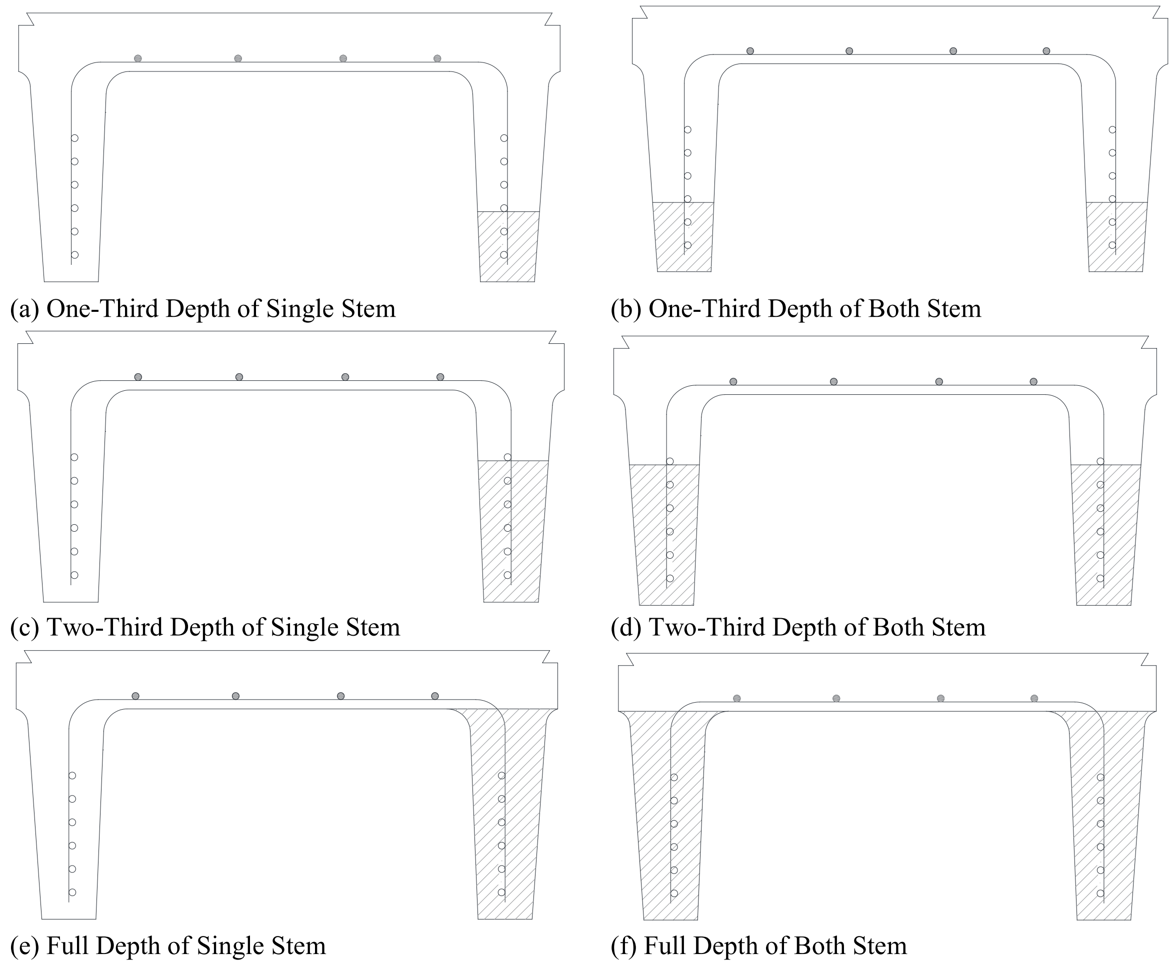
f_s = The bar maximum stress that can be developed using the available embedment length,

f_y = The yield strength of the transverse bar.

To include the effect of flange-to-stem cracks on the shear capacity, the stem concrete below the flange-to-stem interface can be fully removed. In this case, shear capacity of the girder is similar to that of a one-way slab (as was seen in the strength testing of the 50-ft (15.24-m) long salvaged double-tee girder, Ch. 6). Finally, to include the effect of shear cracks on the shear capacity, the V_c component of the shear capacity equation can be assumed to be zero when there is a diagonal crack.

Because there are different stem crack types (or stem cover deterioration with CS-4) and they may happen at a different depth of the girder, several combinations are feasible. However, for practical purposes, only three stem cracking (or stem cover deterioration) scenarios were assumed: (i) if the crack (or stem cover deterioration with CS-4), regardless of the type, is reported at the bottom 1/3 of the stem, remove the bottom 1/3 of the stem concrete (Fig. 7.7a & b) and then calculate the shear capacity as discussed above, (ii) if there is a crack (or stem cover deterioration with CS-4) between the bottom 1/3 to 2/3 stem depth, repeat (i) but remove the bottom 2/3 of the stem concrete (Fig. 7.7c & d), and (iii) if there is a crack (or stem cover deterioration with CS-4) between the bottom 2/3 to 1.0 stem depth, repeat (i) but fully remove the stem concrete (Fig. 7.7e & f). In case (iii), shear capacity was the minimum of the girder shear capacity, as discussed above, and the one-way slab (flange only) shear capacity based on the findings of the salvaged double-tee girder strength testing.

These conservative assumptions were made because the shear failure is brittle and must be avoided. Furthermore, regardless of the condition state, the same shear capacity condition factors were proposed for stem cracking to avoid shear failure.



(a) One-Third Depth of Single Stem (b) One-Third Depth of Both Stem

(c) Two-Third Depth of Single Stem (d) Two-Third Depth of Both Stem

(e) Full Depth of Single Stem (f) Full Depth of Both Stem

Figure 7.7 Scenarios to Include Double-Tee Stem Cracking (or Stem Cover Deterioration with CS-4) in Shear Capacity

7.2 Flange Moment and Shear Capacities

Four damage types, each with four condition states, were defined for the flange of double-tee girders (Table 4.2). The steps and scenarios assumed to include such damage in the girder moment and shear capacities are discussed herein.

7.2.1 Flange Cover Deterioration including Delamination/Spall/Patched Area/Aberration

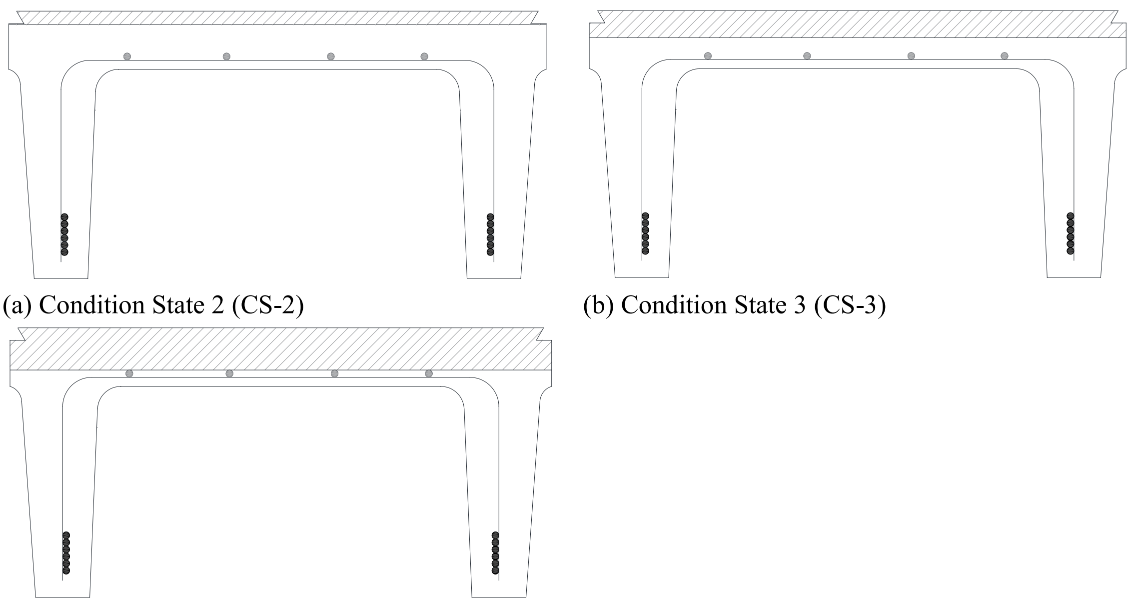
Flange cover deterioration in a form of delamination, spalling, patched area, or aberration can be included in the capacity estimation method by removing the deteriorated concrete cover from the section. The flange concrete cover removal scenarios for the four condition states are discussed in this section.

7.2.1.1 Flange Cover Deterioration with Condition State 1 (CS-1)

This damage condition state was defined as “none” (Table 4.2). Therefore, capacity of the damaged girder in this condition state is the same as that for an undamaged girder.

7.2.1.2 Flange Cover Deterioration with Condition State 2 (CS-2)

This damage condition state was defined as the "loss of 1/3 of the flange cover without exposure or corrosion of the reinforcement" (Table 4.2). To include this damage in the calculation of moment and shear capacities, 1/3 of the flange concrete cover was removed (Fig. 7.8a). Moment-curvature analyses were performed to calculate the moment capacity of damaged girders. For the shear capacity calculation, the depth of section is reduced when the concrete cover is removed from the top of the flange by which the section effective shear depth, d_v, is reduced thus the V_c (Eq. 6.7) and V_s (Eq. 6.8) components of the shear capacity equation are reduced.



(a) Condition State 2 (CS-2)

(b) Condition State 3 (CS-3)

(c) Condition State 4 (CS-4)

Figure 7.8 Flange Cover Deterioration

7.2.1.3 Flange Cover Deterioration with Condition State 3 (CS-3)

This damage condition state was defined as the "loss of 2/3 of the flange cover without exposure or corrosion of the reinforcement" (Table 4.2). To include this damage in the calculation of moment and shear capacities, 2/3 of the flange concrete cover was removed (Fig. 7.8b). The same procedures discussed above were used for calculation of the moment and shear capacities.

7.2.1.4 Flange Cover Deterioration with Condition State 4 (CS-4)

This damage condition state was defined as the "exposure of reinforcement without any sign of corrosion" (Table 4.2). To include this damage in the calculation of moment and shear capacities, all the flange concrete cover was removed (Fig. 7.8c) and the same methods discussed above were used to calculate the capacities.

7.2.2 Flange Exposed Bar

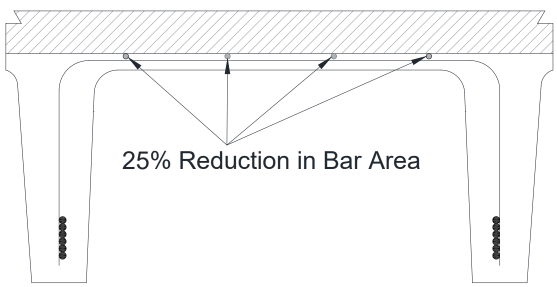
This damage type includes corrosion of the flange longitudinal and transverse bars in the moment and shear capacities. It is assumed that the flange bars will be exposed (complete loss of the concrete cover), then corroded. This was assumed because the flange concrete cover for South Dakota double-tee girders is deeper than 3 in. (83 mm, or 68% of the flange thickness). The flange concrete cover was fully removed, and the flange reinforcement area was reduced to include this damage type in the shear and moment capacities of double-tee girders.

7.2.2.1 Flange Exposed Bar with Damage Condition State 1 (CS-1)

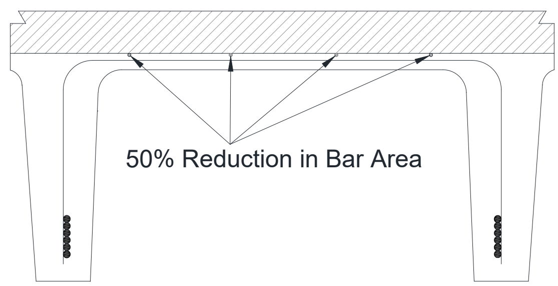
This damage condition state was defined as "none" (Table 4.2) indicating that there was no corrosion of the flange reinforcement. However, the full cover was removed. Therefore, this condition state is the same as the "Flange Cover Deterioration with Damage Condition State 4" discussed in the previous sections.

7.2.2.2 Flange Exposed Bar with Damage Condition State 2 (CS-2)

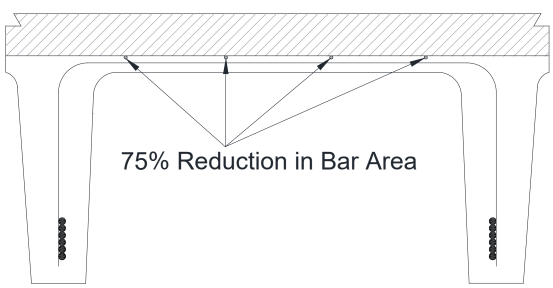
This damage condition state was defined as a "minor corrosion of the outer layer of reinforcement with minimal section loss" (Table 4.2). To include this damage in the moment and shear capacities, the flange concrete cover was fully removed, and the area of both flange longitudinal and transverse bars was conservatively reduced by 25% (Fig. 7.9a). Moment-curvature analyses were carried out to calculate the moment capacity of the damaged section. Furthermore, this damage type reduces the effective shear depth (d_v) and thus the V_c and V_s components of the shear capacity equation (Eq. 6.5).



(a) Condition State 2 (CS-2)



(b) Condition State 3 (CS-3)



(c) Condition State 4 (CS-4)

Figure 7.9 Flange Exposed Longitudinal and Transverse Bars

7.2.2.3 Flange Exposed Bar with Damage Condition State 3 (CS-3)

This damage condition state was defined as a "severe corrosion of only the outer layer of reinforcement" (Table 4.2). To include this damage in the moment and shear capacities, the flange concrete cover was fully removed, and the area of flange longitudinal and transverse bars was conservatively reduced by 50% (Fig. 7.9b). The same methods discussed above were used to calculate the moment and shear capacities.

7.2.2.4 Flange Exposed Bar with Damage Condition State 4 (CS-4)

This damage condition state was defined as a "severe corrosion of the outer and inner layers of reinforcement" (Table 4.2). To include this damage in the moment and shear capacities, the flange concrete cover was fully removed, and the area of flange longitudinal and transverse bars was conservatively reduced by 75% (Fig. 7.9c). The same methods discussed above were used to calculate the moment and shear capacities.

7.2.3 Flange Cracking

Since the flange cracking would have at most the same effect as the flange cover deterioration discussed in the previous section, the effect of the flange cracking was not separately investigated. The cover deterioration will govern.

7.2.4 Girder-to-Girder Longitudinal Joint Deterioration

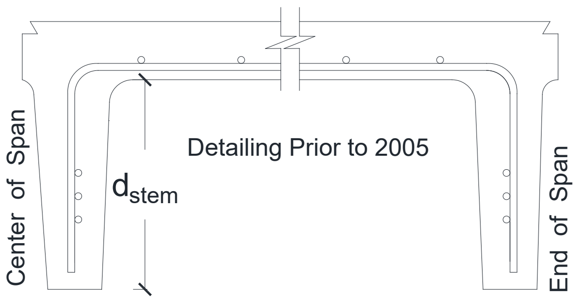
The moment and shear capacities are calculated for a single girder at a time. Therefore, the girder-to-girder longitudinal joint deterioration has no effect on the girder capacities.

7.3 Loss of Tendon Stresses

Prestressing loss has minimal effect on the moment capacity of concrete sections. Furthermore, the V_p component of the shear capacity equation has less than 3% contribution to the shear capacity for South Dakota double-tee sections, and it is zero when tendons are straight. Nevertheless, a 20% prestressing loss was assumed in all analyses based on the findings of the literature review on damaged or old girders (Dasar et al., 2016; Pessiki et al., 1996). It is worth mentioning that the decompression test carried out on the 30-ft (9.14-m) long girder (Chapter 8) showed approximately 50% loss.

7.4 Proposed Condition Factors for Different Double-Tee Girder Sections

Twenty-three different double-tee sections, which have been used in South Dakota, were identified. Moment and shear condition factors for each section were developed and summarized in Fig. 7.10 to 7.32. The girder properties were also reported, which were extracted from the available shop drawings. Appendix E of this report presents the details of the available double-tee sections.



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 3 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 6-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.80	0.65	0.30
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 6-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70
Exposed Rebar	0.70	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 6-Tendon Double-Tee Girders

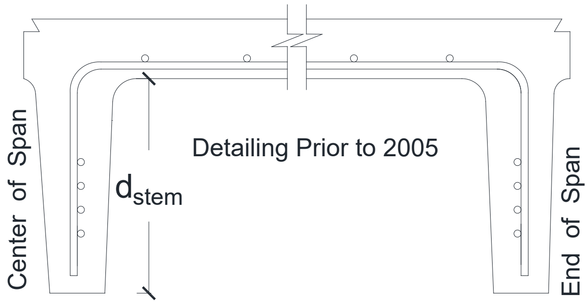
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking(a)
Exposed Transverse Rebar(b)	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing(b)	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stems	1	1/3 bottom stem depth (1/3 d_{stem})		0.45
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 6-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.75
Exposed Rebar	0.75	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.10 Condition Factors for 23-in. Deep 6-Straight Tendon Double-Tee Girders (Before 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 4 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.85	0.70	0.45
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.85	0.75	0.65
Exposed Rebar	0.65	0.65	0.65	0.65
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

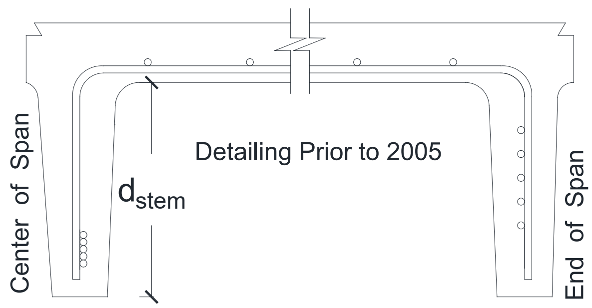
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stems	1	1/3 bottom stem depth (1/3 d_{stem})		0.45
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.75
Exposed Rebar	0.75	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.11 Condition Factors for 23-in. Deep 8-Straight Tendon Double-Tee Girders (Before 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 5 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.80	0.60
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.75
Exposed Rebar	0.75	0.75	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

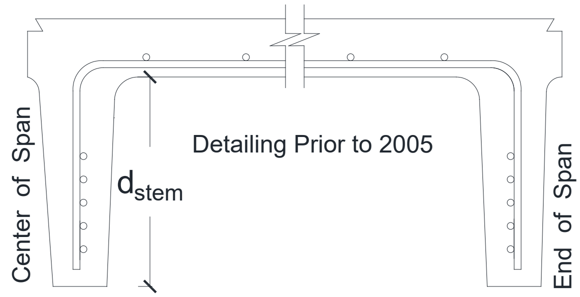
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.40
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.75
Exposed Rebar	0.75	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.12 Condition Factors for 23-in. Deep 10-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 5 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.85	0.75	0.55
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70
Exposed Rebar	0.70	0.65	0.65	0.65
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

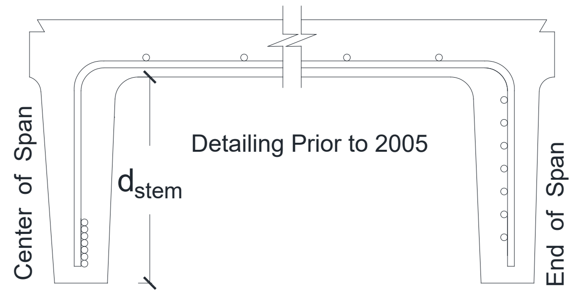
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.40
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 10-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.75
Exposed Rebar	0.75	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.13 Condition Factors for 23-in. Deep 10-Straight Tendon Double-Tee Girders (Before 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70
Exposed Rebar	0.70	0.70	0.65	0.65
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

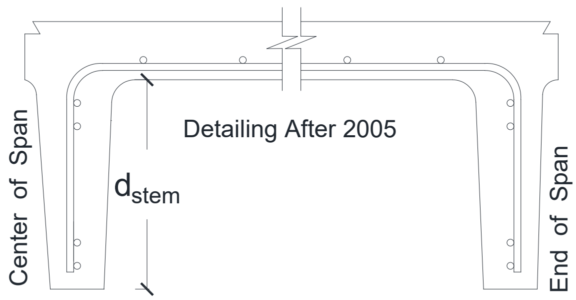
Damage Type	Condition States			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})	0.70	
		2/3 bottom stem depth (2/3 d_{stem})	0.45	
		1/3 top stem depth (1/3 d_{stem})	0.40	
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})	0.45	
		2/3 bottom stem depth (2/3 d_{stem})	0.0	
		1/3 top stem depth (1/3 d_{stem})	0.0	

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70
Exposed Rebar	0.70	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.14 Condition Factors for 23-in. Deep 14-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 4 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.80	0.65	0.30
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

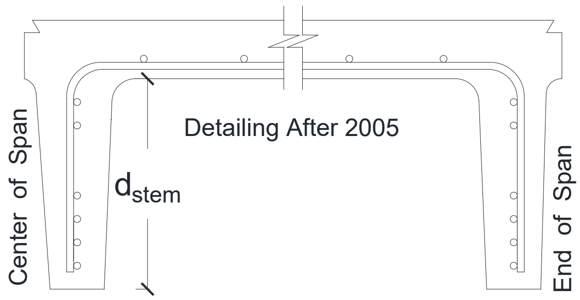
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	0.95	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.90	0.85	0.55
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.15 Condition Factors for 23-in. Deep 8-Straight Tendon Double-Tee Girders (After 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 6 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 23-in. Deep 12-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.85	0.75	0.55
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 12-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 12-Tendon Double-Tee Girders

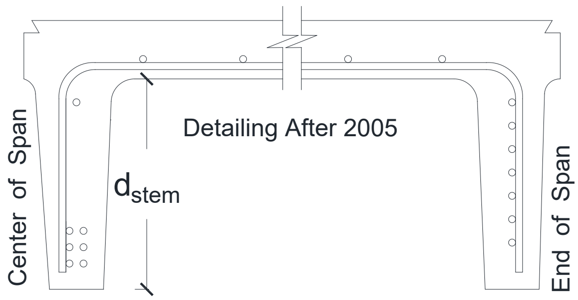
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.80
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.65
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.35
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 12-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.16 Condition Factors for 23-in. Deep 12-Straight Tendon Double-Tee Girders (After 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.41L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.60
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

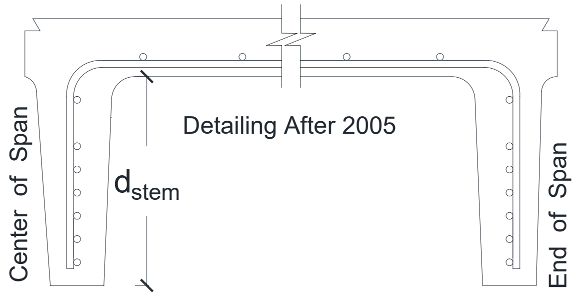
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.80
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.40
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.40
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.17 Condition Factors for 23-in. Deep 14-Straight Tendon Double-Tee Girders (After 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.80	0.60
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

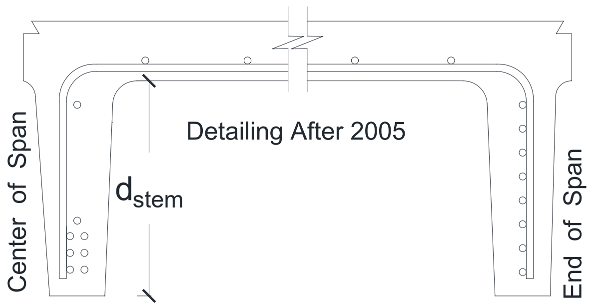
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.65
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.40
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.18 Condition Factors for 23-in. Deep 14-Straight Tendon Double-Tee Girders (After 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 8 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.37L
Transverse Bar size = No. 4 (13 mm)
f'_c = 7.25 ksi (50 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 23-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 16-Tendon Double-Tee Girders

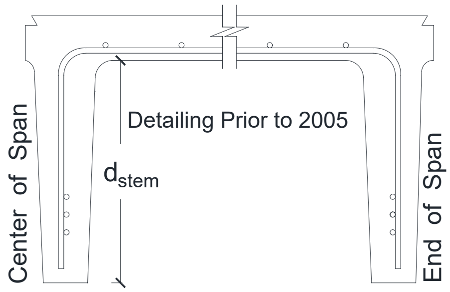
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	0.95	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.70	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.85	0.85	0.75
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.65
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.35
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.19 Condition Factors for 23-in. Deep 16-Harped Tendon Double-Tee Girders (After 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 3 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 6-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.80	0.65	0.30
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 6-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 6-Tendon Double-Tee Girders

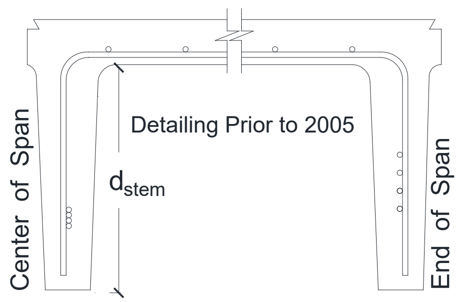
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.45
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 6-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.20 Condition Factors for 30-in. Deep 6-Straight Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 4 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.85	0.75	0.50
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.9	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

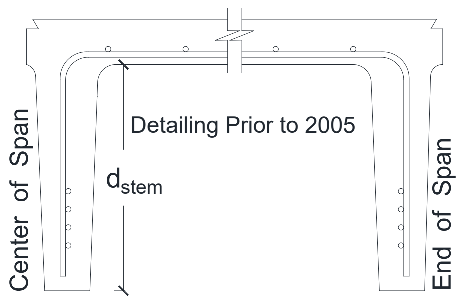
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.25
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.21 Condition Factors for 30-in. Deep 8-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 4 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.85	0.70	0.45
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

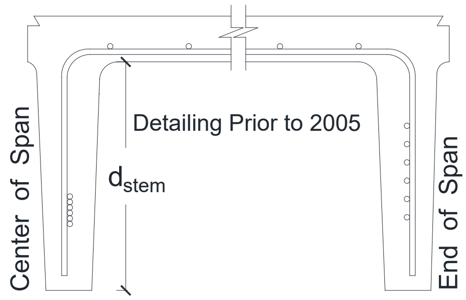
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.99	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.50
		1/3 top stem depth (1/3 d_{stem})		0.20
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.22 Condition Factors for 30-in. Deep 8-Straight Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 6 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 12-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.80	0.65
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 12-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.80
Exposed Rebar	0.80	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 12-Tendon Double-Tee Girders

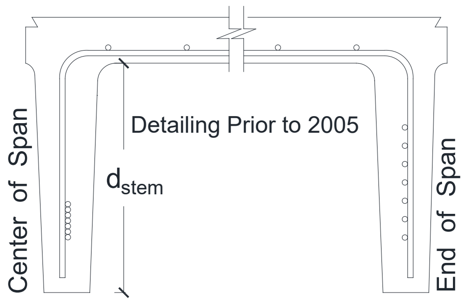
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 12-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.23 Condition Factors for 30-in. Deep 12-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.75
Exposed Rebar	0.75	0.75	0.75	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 14-Tendon Double-Tee Girders

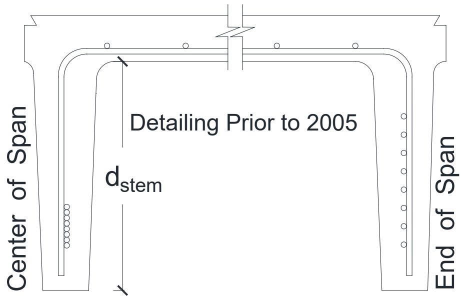
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in “Cracking on Single Stem”).
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.24 Condition Factors for 30-in. Deep 14-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 8 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.75
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.75
Exposed Rebar	0.75	0.75	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

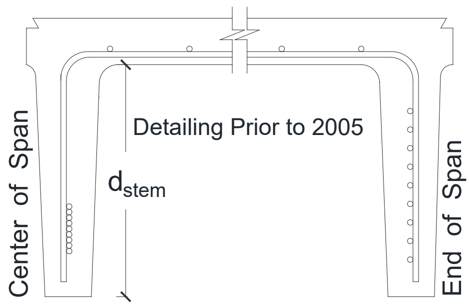
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.25 Condition Factors for 30-in. Deep 16-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 9 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.95	0.85	0.75
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70
Exposed Rebar	0.70	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

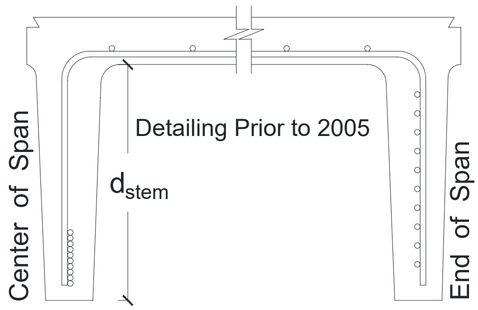
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.20
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.26 Condition Factors for 30-in. Deep 18-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 10 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.9 kips (128.6 kN)

Stem Moment Condition Factors for 30-in. Deep 20-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.95	0.90	0.80
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 20-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.85	0.75
Exposed Rebar	0.75	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 20-Tendon Double-Tee Girders

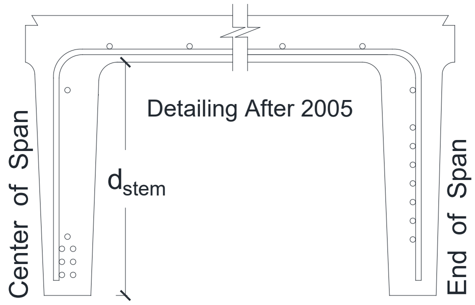
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.55
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.40
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.15
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 20-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.75
Exposed Rebar	0.75	0.75	0.75	0.75
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.27 Condition Factors for 30-in. Deep 20-Harped Tendon Double-Tee Girders (Before 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 8 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.39
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

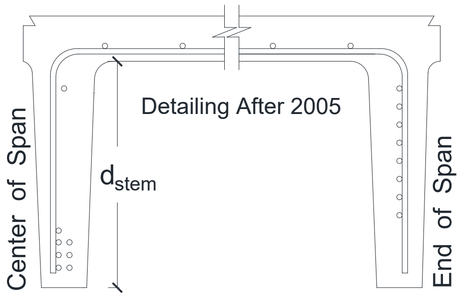
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.28 Condition Factors for 30-in. Deep 16-Harped Tendon Double-Tee Girders (After 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 8 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.4L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

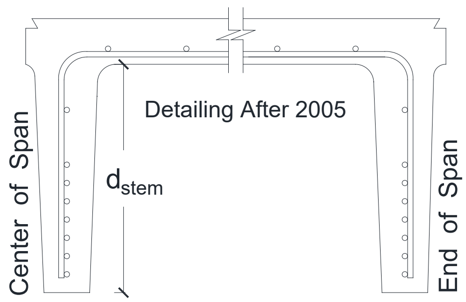
Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0..35
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	**Condition States**			
	CS-1	**CS-2**	**CS-3**	**CS-4**
	Good	**Fair**	**Poor**	**Severe**
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.29 Condition Factors for 30-in. Deep 16-Harped Tendon Double-Tee Girders (After 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 8 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.80	0.65
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.84
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

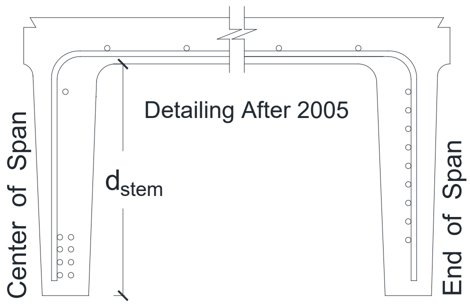
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking(a)
Exposed Transverse Rebar(b)	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing(b)	1	0.95	0.95	0.90
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.25
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.20
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 16-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.30 Condition Factors for 30-in. Deep 16-Straight Tendon Double-Tee Girders (After 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 9 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.4L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.75
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

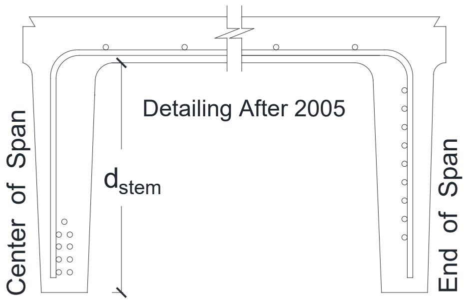
Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.55
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.31 Condition Factors for 30-in. Deep 18-Harped Tendon Double-Tee Girders (After 2005)



Depth = 30 in. (762 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 9 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.34L
Transverse Bar size = No. 4 (13 mm)
f'_c = 6 ksi (41.4 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 30.98 kips (137.8 kN)

Stem Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.95	0.85	0.75
Cracking	1	1	1	1

Flange Moment Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.95	0.90	0.85
Exposed Rebar	0.85	0.85	0.85	0.85
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.60
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.30
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.25
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 30-in. Deep 18-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.80
Exposed Rebar	0.80	0.80	0.80	0.80
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure 7.32 Condition Factors for 30-in. Deep 18-Harped Tendon Double-Tee Girders (After 2005)

7.5 Modification of Condition Factors Accounting for Lower Concrete Compressive Strength

The girder properties shown in the above figures were extracted from available shop drawings. It is possible that the actual load rating bridge had a lower concrete compressive strength than that specified. Through an analytical study, it was found that a change in the concrete compressive strength only affects flange moment condition factors, specifically those pertaining to the cover deterioration and the exposed bars. These condition factors should be reduced when the concrete compressive strength of the load rating bridge is lower than that specified in the table as:

$$\varphi_c{}^{f'_c} = \varphi_c{}^{Table} - 0.06\left(\Delta f'_c\right) \qquad \text{(Eq. 7.2)}$$

where

$\varphi_c{}^{f'_c} =$ A reduced condition factor with a lower concrete compressive strength,

$\varphi_c{}^{Table} =$ The condition factor from the flange moment condition factor tables,

$\Delta f'_c =$ The difference in the concrete compressive strength defined as

$f'_c{}^{Table} - f'_c{}^{Actual}$ (ksi)

$f'_c{}^{Table} =$ The concrete compressive strength specified for the girder (shown the figure),

$f'_c{}^{Actual} =$ The actual concrete compressive strength for the girder to be load rated.

7.6 Verification of Proposed Capacity Estimation Method

Appendix B of this report presents the verification of the method discussed above. The data collected from the strength testing of the 30-ft long girder was used. The measured girder capacity was compared with the calculated capacity of the girder using the method proposed in this chapter (Sec. 7.4). Calculated capacity of the girder was 16% lower than the measured capacity, which is safe.

Overall, the proposed condition factor method was found to be relatively simple and safe for the capacity calculation of damaged double-tee girders.

8. RECOMMENDATIONS

Based on the findings of this study, the research team offers the following recommendations.

8.1 Recommendation 1: General

The guidelines as detailed in Appendix C should be adopted for the load rating of damaged double-tee girder bridges.

In general, the load rating of damaged double-tee girder bridges is performed similarly to the AASHTO LRFR method, but the capacity and the live load parameters should be modified as recommended below.

8.2 Recommendation 2: Capacity Modification

The guidelines as detailed in Section C.2.2 of Appendix C should be adopted to modify the girder capacities accounting for different damage types and condition states.

The moment and shear capacities of a damaged double-tee girder at strength limit states should be reduced using the proposed condition factors (φ_c) for South Dakota double-tee sections. At service limit states, the bridge concrete and reinforcing steel mechanical properties as recommended should be used in the load rating equation.

8.3 Recommendation 3: Demand Modification

The guidelines as detailed in Section C.2.3 of Appendix C should be adopted to modify the live load parameters accounting for different girder-to-girder damage condition states.

If double-tee bridge has a longitudinal joint damage condition state 3 or less, the AASHTO LRFD can be followed to determine live load parameters. Recommendations were provided for longitudinal joint damage condition state 4.

9. SUMMARY AND CONCLUSIONS

Precast prestressed double-tee girder bridges, which are the most common type of bridge on South Dakota local roads, are deteriorating and may need replacement only after 40 years of service. The estimation of the bridge safe live load, especially when the bridge elements are deteriorated, is challenging. The present project was conducted to propose a methodology for load rating of double-tee girder bridges accounting for different damage types and condition states for the girder.

9.1 Summary

The equation for bridge load rating consists of the bridge member capacity, member dead load, and member live load. One way to include the effect of different damage types and condition states on the load rating equation is through the modification of the capacity and live load components of the equation.

The literature was lacking quantitative definition of bridge element damage types and condition states. This gap was addressed by proposing systematic and quantitative definitions for double-tee bridge damage types and condition states. More than 370 inspection reports specific to the state double-tee bridges and the Bridge Management database (BrM) were reviewed to determine the frequency of each damage type and its condition state, number of bridge spans, span length, girder depth, and number of skewed double bridges. The statistical database was used to identify double-tee bridge candidates suited for the field and strength testing.

Using the inspection reports and frequency of double-tee bridge damage types and other aforementioned parameters, 10 bridges were identified as suitable field testing candidates to determine the bridge live load transfer mechanisms. All 10 bridges were inspected and two double-tee bridges, one with 30-in. (762-mm) deep girders and another with 23-in. (584-mm) deep girders, were selected for field testing. Both bridges had girder-to-girder longitudinal joint deterioration with a damage condition state 3. Only girder-to-girder damage will affect the live load distributions in double-tee bridges since they are statically determinate (simply supported bridges). Both bridges were tested for flexural response but only the first bridge with 30-in. (762-mm) deep girders was tested to obtain shear demands. Strain transducers were installed at the bridge midspan in flexural response tests, and the strain transducers were installed at a distance equal to the girder depth from the face of end diaphragm in the shear response test. Both static and dynamic tests were performed for these bridges to determine the girder distribution factors and dynamic allowance.

Accurate estimation of the capacity of a damaged double-tee girder is crucial in this project for a safe load rating. To verify the available moment and shear capacity estimation methods, two 45-year old double-tee girders, one 50-ft (15.24-m) long and another 30-ft (9.14-m) long, were extracted from a bridge located in Nemo Road, SD, and were strength tested at the Lohr Structures Laboratory at South Dakota State University. A four-point loading configuration was selected for the strength testing. The verified methods were then used to calculate the shear and moment capacities of 23 different double-tee sections, which have been used in the state.

9.2 Conclusions

Based on the review of inspection reports for double-tee bridges, the most common damage type found for double-tee girders was the cover deterioration. The most common double-tee bridges in the state have single span with a span length of 40 ft (12.19 m) to 60 ft (18.3 m). Double-tee girders with a depth of 23 in. (584 mm) are more common than 30-in. (762-mm) deep girders. Furthermore, non-skewed double-tee bridges have been used more often than skewed bridges.

Based on findings of the two bridge field testing, the following conclusions can be drawn:

- The measured interior girder moment and shear distribution factors were lower than those specified in the AASHTO LRFD.
- Measured exterior girder distribution factors are less than or equal to calculated exterior girder distribution factor using the AASHTO methods.
- The measured dynamic load allowance was lower than that specified in the AASHT LRFD.

Based on strength testing of two salvaged double-tee girders, the following conclusions were drawn:

- The first flexural crack in the stem of the 50-ft (15.24-m) girder was observed at 24.9 kips (110.7 kN), which was 35% lower than the AASHTO Service I limit state. Furthermore, the 50-ft (15.24-m) girder load carrying capacity of 41.5 kips (184.5 kN) was 32% lower than the AASHTO Strength I Limit State. This girder failed in a brittle manner by the compressive failure of the flange concrete. All indicated this girder was unsafe for service.
- The first flexural crack in the stem of the 30-ft (9.14-m) girder was observed at 15.3 kips (68.1 kN), which was 44% lower than the AASHTO Service I limit state. Furthermore, the 30-ft (9.14-m) girder load carrying capacity of 37.37 kips (166.2 kN) was 21% lower than the AASHTO Strength I Limit State. This girder failed in a ductile manner; however, it did not meet the AASHTO limit state requirements and was not safe for service.

Based on the statistical, experimental, and analytical studies, a methodology is proposed for damaged double-tee bridges (Appendix C). In this method, the load rating can be performed similarly to the AASHTO LRFR method that is currently used in practice. Nevertheless, it is recommended to modify the capacity (C) and live load components (LL and IM) of the load rating equation accounting for different damage types and condition states. Condition factors were proposed for all different double-tee sections that have been used in the state in the previous chapter.

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APPENDIX A. PHOTOGRAPHS OF INSPECTED BRIDGES

Based on the double-tee bridge selection criteria (refer to Sec. 5.1), ten bridges were identified suitable for field testing. Each bridge was inspected by the research team and two bridges were selected for the testing. This appendix presents a summary of the 10-bridge inspection findings.

Table A.1 Double-Tee Bridge Candidates for Field Testing

Bridge ID	County	Span Length and Depth	Damage Type and Condition State	Age, Yr.
31024230	Hanson, SD	40.8 ft (12.4 m) Seven 23-in (584-mm) Deep Girders	*Non-skewed,* Minor water leakage between deck units (with a condition state of Poor).	36
34075220	Hutchinson, SD	43 ft (13.1 m) Seven 23-in (584-mm) Deep Girders	*Non-skewed,* Light staining from leakage between longitudinal joints, spalling, and delamination. Only one longitudinal joint had water leakage after rain (with a condition state of Poor).	37
34140033	Hutchinson, SD	100 ft (30.5 m) 3 span Eight 23-in (584-mm) Deep Girders	*Non-skewed,* Severe water leakage between all longitudinal joints after rain with minor corrosion of steel plates (with a condition state of Poor).	39
42104110	Lincoln, SD	46 ft (14.02 m) Seven 30-in. (762-mm) Deep Girders	*Non-skewed, girders have transverse diaphragms,* Spalling of stem concrete cover (condition state not available), exposure of stem transverse reinforcement (with a condition state of Severe), and leakage of girder-to-girder joints (with a condition state of Poor).	35
42130065	Lincoln, SD	45.8 ft (13.9 m) Six 30-in. (762-mm) Deep Girders	*Non-skewed,* Spalling of both stem and flange concrete cover (with a condition state of Fair and Good, respectively), and leakage of girder-to-girder joints (with a condition state of Poor).	40
42165153	Lincoln, SD	42 ft (12.8 m) Seven 30-in. (762-mm) Deep Girders	*Non-skewed,* Spalling of stem concrete cover (with a condition state of Fair), and leakage of girder-to-girder joints (with a condition state of Poor).	34
51008010	Moody, SD	50 ft (15.24 m) Six 23-in (584-mm) Deep Girders	*Non-skewed,* Spalling with exposed rebar, efflorescence and water staining between the deck units due to leaking of the joints.	40
51090012	Moody, SD	50 ft (15.24 m) Eight 23-in. (584-mm) Deep Girders	*Non-skewed,* Water leakage between all deck units, stains from minor corrosion of steel plates in longitudinal joints (with a condition state of Poor), concrete spalling (with a condition state of Fair).	38
51140067	Moody, SD	51.2 ft (15.6 m) Seven 23-in. (584-mm) Deep Girders	*Skewed bridge, girders have transverse diaphragms,* Minor water leakage between deck units but with no sign of corrosion of steel plates (with a condition state of Poor).	8
51142060	Moody, SD	50 ft (15.24 m) Six 23-in. (584-mm) Deep Girders	*Posted bridge, non-skewed,* Staining and water leakage between the all deck units.	40

Note: The bridge age was by 2018.

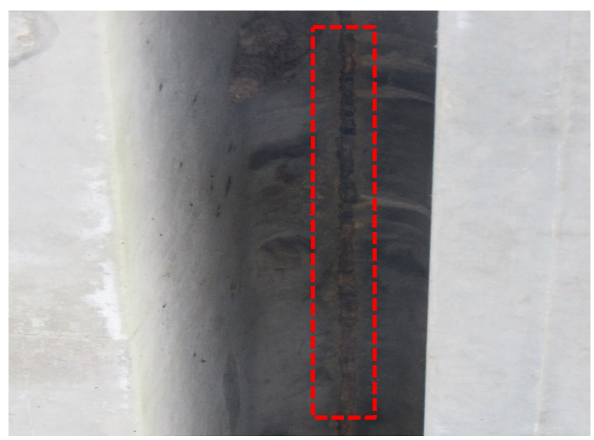
(a) Top view of bridge

(b) Diaphragm at the exterior girder

(c) Leakage from joint

(d) Underneath of bridge

(e) Efflorescence in joint



(f) Joint gap

Figure A.1 Photographs of Bridge 31-024-230


(a) Top view of bridge


(b) Underneath of bridge


(c) Efflorescence in joint


(d) Deterioration at bottom of stem


(e) Scouring from bottom of abutment

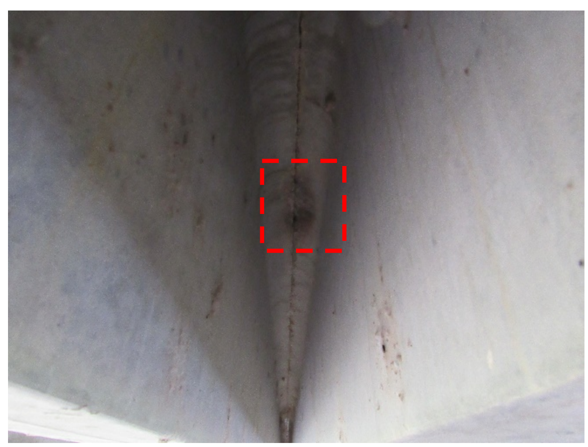

(f) Reddish color, sign of corrosion

Figure A.2 Photographs of Bridge 34-075-220


(a) Side view of bridge


(b) Cracking on pavement over the bridge


(c) Underneath of bridge


(d) Leakage from joint


(e) Efflorescence


(f) Sign of corrosion

Figure A.3 Photographs of Bridge 34-140-033


(a) Side view of bridge


(b) Cracking at bottom of exterior girder


(c) Joint deterioration


(d) Efflorescence in joint


(e) Leakage from joint


(f) Underneath of bridge

Figure A.4 Photographs of Bridge 42-104-110


(a) Side view of bridge


(b) Corrosion in joint


(c) Efflorescence in joint

(d) Cracking in diaphragm


(e) Deterioration in Joint


(f) Underneath of bridge

Figure A.5 Photographs of Bridge 42-130-065


(a) Side view of bridge


(b) Spalling at stem of bridge


(c) Corrosion in the joint


(d) Scouring at abutment of bridge


(e) Underneath of bridge


(f) Leakage from joint

Figure A.6 Photographs of Bridge 42-165-153

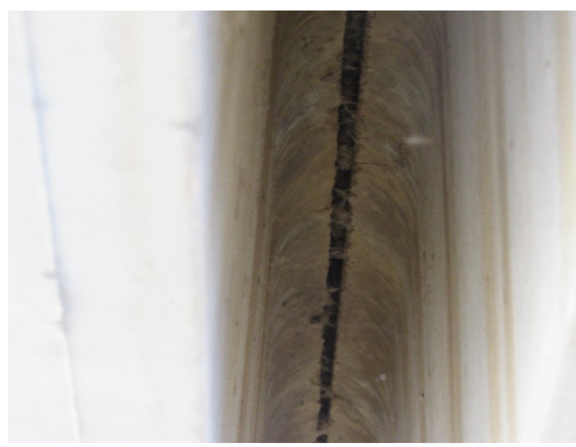
(a) Top of bridge

(b) Deterioration at side of bridge

(c) Underneath of bridge

(d) Deterioration at bottom of stem

(e) Efflorescence at joint



(f) Wide gap in joint

Figure A.7 Photographs of bridge 51-008-010

(a) Top view of bridge

(b) Gap of joint

(c) Underneath of bridge

(d) Leakage from joint

(e) Deterioration at joint

(f) Deterioration at bottom of stem

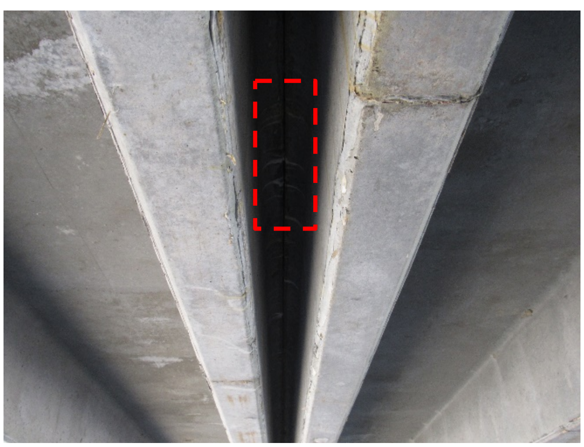
Figure A.8 Photographs of Bridge 51-090-012

(a) Top of bridge

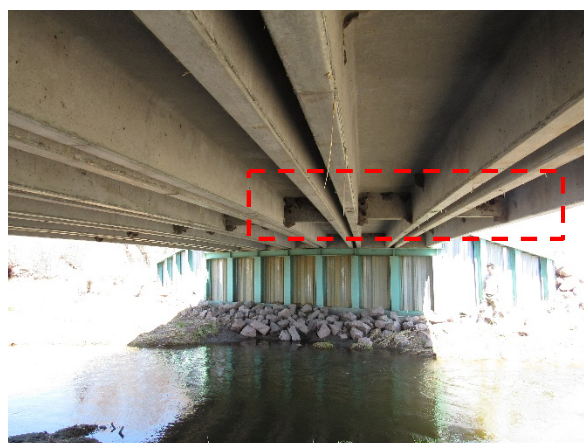
(b) Corrosion of plate of joint

(c) Sign of leakage

(d) Underneath of bridge



(e) Efflorescence at joint



(f) Diaphragm in the girder

Figure A.9 Photographs of Bridge 51-140-067


(a) Top view of bridge


(b) Underneath of bridge


(c) Wooden abutment


(d) Efflorescence in joint

(e) Posting of bridge


(f) Wooden diaphragm at end of girder

Figure A.10 Photographs of Bridge 51-142-060

APPENDIX B. VERIFICATION OF PROPOSED CONDITION FACTORS

Appendix B presents a verification of the damaged double-tee girder capacity estimation using proposed damage condition factors. A 30-ft (9.14-m) damaged double-tee girder was tested to failure as part of this project. Table B.1 presents the description of the girder and Fig. B.1 shows the girder damage. Furthermore, the girder observed damage types and condition states were marked in Tables B.2 and B.3 using "golden stars". The measured force capacity of the girder in a four-point loading configuration (Fig. B.1b) was 37.37 kips (166.2 kN).

(a) Underneath View of Girder

(b) Stem Cover Deterioration

(c) Reinforcement Exposure

(d) Flange Cover Deterioration

Figure B.1 Damage of 30-ft (9.14-m) Salvaged Girder

Table B.1 Description of 30-ft Girder

Girder Depth, in. (mm)	Girder Length, ft (m)	Damage Type and Condition State
23 (584)	30 (9.14)	Spalling of stem concrete cover (CS-4), exposure of stem transverse reinforcement (CS-3), flange cover deterioration (CS-4), exposure of flange rebar (CS-2) and cracking at stem and flange joint (CS-1).

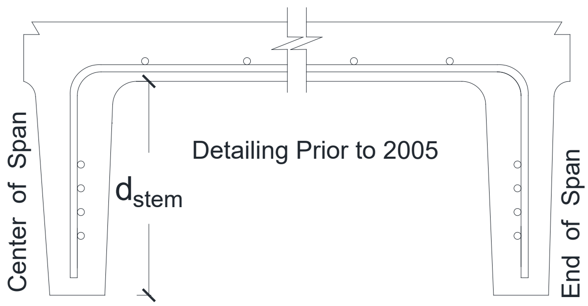
Table B.2 Damage Types and Condition States for Prestressed Double-Tee Girder Stem

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Transverse Rebar	None	Minor corrosion of the reinforcement with minimal section loss.	Severe corrosion of only one leg of transverse reinforcement.	Severe corrosion of all legs of transverse reinforcement in a section.
Exposed Longitudinal Prestressing	None	50% section loss due to corrosion in the extreme tendon.	100% section loss due to corrosion in the extreme tendon.	Section loss due to corrosion in the two or more tendons.
Cracking	Insignificant cracks or moderate-width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.

Table B.3 Damage Types and Condition States for Prestressed Double-Tee Girder Top Flange

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area/Aberration	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Rebar	None	Minor corrosion of the outer layer of reinforcement with minimal section loss.	Severe corrosion of only the outer layer of reinforcement.	Severe corrosion of the outer and inner layers of reinforcement.
Cracking	Insignificant cracks or moderate width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.
Girder-to-Girder Longitudinal Joint Deterioration	None	Minimal deterioration, no sign of leakage.	Discrete signs of seepage along the joint, minor corrosion of steel plates.	Seepage along the joint, severe corrosion of steel plates.

This 23-in. (584-mm) deep girder was built before 2005 and it had four straight tendons per stem. The damage condition factors for this double-tee section are presented in Fig. B.2 (the same as those presented in Fig. 7.11 of Ch. 7).



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 4 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Straight
Transverse Bar size = No. 4 (13 mm)
f'_c = 5 ksi (34.5 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity	
	CS-1	CS-2	CS-3	CS-4		
	Good	Fair	Poor	Severe	M (k.ft)	P (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1	422.5	67.81
Exposed Transverse Rebar	1	1	1	1	422.5	67.81
Exposed Longitudinal Prestressing	1	0.85	0.70	0.45	N/A	N/A
Cracking	1	1	1	1	422.5	67.81

Flange Moment Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity	
	CS-1	CS-2	CS-3	CS-4		
	Good	Fair	Poor	Severe	M (k.ft)	P (k.ft)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.66	0.56	0.46	194.35	31.19
Exposed Rebar	0.46	0.46	0. 46	0.46	194.35	31.19
Cracking	1	1	1	1	N/A	N/A
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1	N/A	N/A

Stem Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity	
	CS-1	CS-2	CS-3	CS-4		
	Good	Fair	Poor	Severe	*V* (kips)	*P* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]	24. 66	49.32
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50	52.84	105.68
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.95	0.90	N/A	N/A
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70	N/A	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.45	N/A	N/A
		1/3 top stem depth (1/3 d_{stem})		0.35	24.66	49.32
Cracking on Both Stems	1	1/3 bottom stem depth (1/3 d_{stem})		0.45	N/A	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.0	N/A	N/A
		1/3 bottom stem depth (1/3 d_{stem})		0.0	N/A	N/A

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 8-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity	
	CS-1	CS-2	CS-3	CS-4		
	Good	Fair	Poor	Severe	*V* (kips)	*P* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.75	52.84	105.6
Exposed Rebar	0.75	0.75	0.75	0.75	52.84	105.6
Cracking	1	1	1	1	N/A	N/A
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1	N/A	N/A

Figure B.2 Condition Factors for 23-in. Deep 8-Straight Tendon Double-Tee Girders

The design concrete compressive strength for this girder extracted from the shop drawing was 5 ksi (34.5 Mpa). The measured concrete compressive strength for the girder flange was 1.92 ksi (13.24 MPa). Based on Section 9.5, the flange moment condition factors were modified and reported in Fig. B.2.

The undamaged moment and shear capacities for this girder were 422.5 kip-ft (572.83 kN-m) and 70.46 kips (313.4 kN), respectively. The damaged girder moment or shear capacity presented in Fig. B.2 was calculated by multiplying the undamaged capacity by its corresponding condition factor. An applied load (*P*) equivalent to the moment or shear capacity was calculated using equations B.2 or B.3. Figure B.3 shows the test girder load configuration.

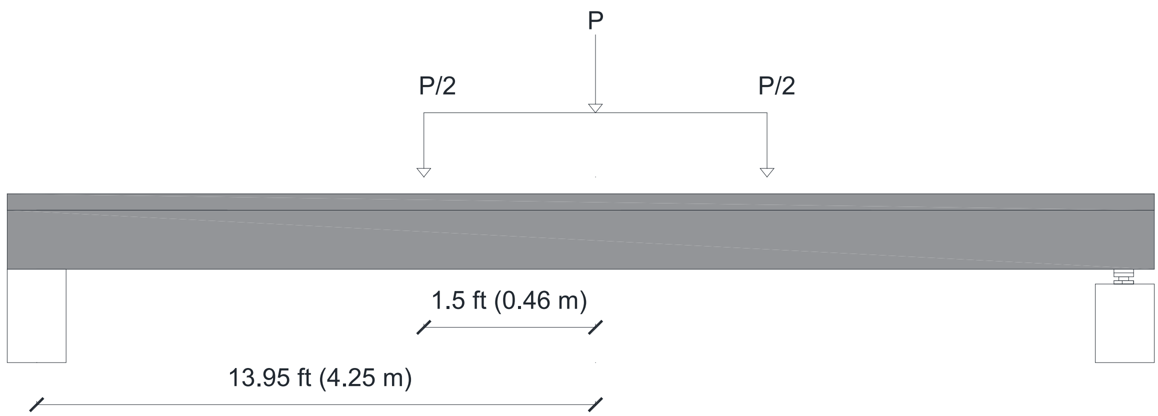


Figure B.3 Applied Load Configuration for 30-ft Long Girder in Strength Test

The equivalent *P* where the moment is maximum is:

$$M = \frac{P}{2} \times 13.95 - \frac{P}{2} \times 1.5 \quad (k.ft) \qquad \text{(Eq. B.1)}$$

by rearranging the equation:

$$P = \frac{M}{6.23} \quad (kips) \qquad \text{(Eq. B.2)}$$

The equivalent *P* where the shear is maximum is:

$$V = \frac{P}{2} \qquad \text{(Eq. B.3)}$$

where,

P = The applied load
M = The calculated moment capacity
V = The calculated shear capacity

The minimum calculated *P* is 31.19 kips (138.7 kN), which is 16% lower than the measured *P* of 37.37 kips (166.2 kN). The proposed method indicates the girder will fail in flexure. The actual girder also failed in flexure. Overall, it can be inferred that the proposed condition factor method is simple and safe to estimate damaged girder capacities.

APPENDIX C. PROPOSED METHODOLOGY FOR LOAD RATING DAMAGED DOUBLE-TEE GIRDER BRIDGES

C.1 Current Load Rating Methods

The AASHTO Manual for Bridge Evaluation (2011) presents load rating, field testing, and posting methods for existing bridges. This manual allows three load rating methods: (1) Load and Resistance Factor Rating (LRFR), (2) Load Factor Rating (LFR), and (3) Allowable Stress. All three methods are currently used to comment whether an existing bridge will be safe and serviceable under a specific live load. Since LRFR is consistent with the current AASHTO LRFD Bridge Design Specifications (2016), the research team proposed to use only LRFR in this project, which was approved by the project technical panel.

LRFR is carried out for three levels of live load: (i) design live load (HL-93), (ii) legal live load (for a given truck allowed by AASHTO or a state DOT), and (iii) permit loads, which are higher than legal loads. In addition to live loads, knowledge of dead loads, wearing surface loads, permanent loads, and dynamic loads are needed in LRFR. A bridge "rating factor (RF)" based on the LRFR method can be calculated as:

$$RF = \frac{C - (\gamma_{DC})DC - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \qquad \text{(Eq. C.1)}$$

where *C* is the member capacity (e.g. shear and flexural capacities for Service and Strength Limit States), γ_{DC} is the LRFD load factor for structural components and attachments, *DC* is the dead load effect due to structural components and attachments, γ_{DW} is the LRFD load factor for wearing surfaces and utilities, DW is the dead load effect due to wearing surfaces and utilities, γ_P is the LRFD load factor for permanent loads other than dead loads, *P* is the permanent load effect other than dead loads, γ_{LL} is the evaluation live load factor, *LL* is the live load effect, and *IM* is the dynamic load allowance. The AASHTO Manual for Bridge Evaluation (2015) provides load factors for different limit states for the three live load levels discussed above.

The member capacity (*C*) is calculated based on the ultimate capacities under Strength Limit State as

$$C = \varphi_c \,.\, \varphi_s \,.\, \varphi \,.\, R_n \qquad \text{(Eq. C.2)}$$

where φ_c is the condition factor, φ_s is the system factor, φ is the LRFD resistance factor, and R_n is the nominal member resistance. For Service Limit State,

$$C = f_R \qquad \text{(Eq. C.3)}$$

where f_R is the allowable stresses.

Load rating of a bridge is done using the rating factor equation (Eq. C.1). If *RF* is greater than 1.0, no restrictive posting is necessary but if it is less than 1.0, posting for that bridge is required.

C.2 Proposed Load Rating Methodology for Damaged Double-Tee Girder Bridges Located in South Dakota

Load rating of damaged double-tee girder bridges may be performed similarly to the LRFR method, which currently is used in practice. Nevertheless, it is recommended to modify the capacity (*C*) and live load components (*LL* and *IM*) of the load rating equation (Eq. C.1) accounting for different damage types and condition states.

C.2.1 Data Needed for Successful Load Rating Damaged Double-Tee Bridges

Before performing the load rating, the inspector or bridge engineer should identify all damage types, their condition states, and the damage location, and should determine the sectional properties (girder length, girder depth, girder width, number of tendons per stem, number and size of transverse reinforcement, and material properties) of girders of the bridge to be load rated.

Review of available drawings and reports revealed that 23 different double-tee sections have been incorporated in South Dakota bridges. The sectional and material properties for these girders can be found in Fig. 7.10 to 7.32. In a case where the load rating bridge girder sectional properties do not match with those in any of the 23 sections, use the condition factors for a section with the same girder depth and the closest number of tendons per stem.

C.2.2 Modification of Damaged Girder Capacities (C)

The moment and shear capacities of a damaged double-tee girder at strength limit states should be reduced using the proposed condition factors (φ_c in Fig. 7.10 to 7.32) for South Dakota double-tee sections as:

$$C_{damaged} = \varphi_c \, . \, C_{undamaged} \qquad \text{(Eq. C.4)}$$

where

$$C_{undamaged} = \varphi_s \, . \, \varphi \, . \, R_n \qquad \text{(Eq. C.5)}$$

All other parameters and methods remain the same as those specified in the AASHTO Manual for Bridge Evaluation (2011 or succeeding).

If the mechanical properties of the load rating bridge constitutive materials are unknown, use the values and methods specified in the AASHTO Manual for Bridge Evaluation (Sec. 6A.5, 2011 or succeeding).

The condition factors should be reduced per Sec. 7.5 of the present document when the concrete compressive strength for the load rating bridge is lower than that specified by the manufacturer for the girders (indicated in Fig. 7.10 to 7.32).

At service limit states, the bridge concrete and reinforcing steel mechanical properties as discussed above should be used in the load rating equation.

C.2.3 Modification of Damaged Girder Live Load Parameters

The live load parameters of the load rating equation should be modified for a damaged double-tee girder as:

1. To calculate moment or shear girder distribution factors (GDFs) for a South Dakota double-tee girder bridge with a longitudinal joint damage condition state 3 or less, follow the AASHTO LRFD specifications.
2. To calculate moment or shear GDFs for a South Dakota double-tee girder bridge with a longitudinal joint damage condition state 4, GDF is the greater of (a) the factor for the exterior girders, (b) the factor for the interior girders, and (c) 0.6.
3. To calculate the dynamic load allowance (IM), follow the AASHTO LRFD specifications.

APPENDIX D. LOAD RATING EXAMPLE FOR DAMAGE DOUBLE-TEE GIRDER BRIDGE

D.1 Introduction

A damaged double-tee girder bridge (Fig. D.1) was considered for load rating. Since some of the bridge damage types, condition states, and damage locations were assumed but not actual, the bridge identification number and location were not reported herein to avoid posting. The load rating example presented herein should be treated as a sample only.

The bridge is a single-span 46-year old structure with a span length of 50 ft (15.24 m). The bridge is non-skewed, and all girders have a depth of 23 in. (584 mm). Figure D.2 shows the photographs of the existing bridge, and Fig D.3 shows the damage of the bridge. he inspection report and photographs of the actual bridge were provided by Brosz Engineering, Inc.

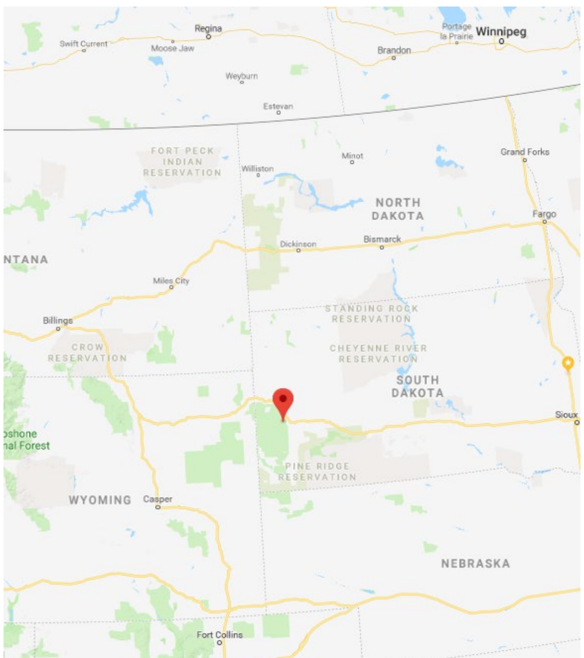


Figure D.1 Load Rating Example - Bridge Location

(a) South Approach

(b) North Approach

(c) Upstream Looking East

(d) Downstream Looking West

Figure D.2 Load Rating Example – Bridge Photographs

(a) Deterioration of Concrete on Top of East Deck Unit

(b) Deterioration of Concrete on Top of West Deck Unit

(c) Deterioration of West Side of West Deck Unit

(d) Longitudinal Cracking and Severe Efflorescence-North End of Deck Unit 1

(d) Longitudinal Cracking and Severe Efflorescence-North End of Deck Unit 1

(e) Spall with Prestressing Strand Exposed – South End of East Stem of Deck Unit 3

Figure D.3 Load Rating Example – Bridge Damage

(f) Spall with Prestressing Strand Exposed – South End of East Stem of Deck Unit 6

(g) Spall with Prestressing Strand Exposed – South End of East Stem of Deck Unit 7

(h) Spall with Exposed Rebar Near Post 4 – Deck Unit 7

(i) Spall Near Rail Post 5 – Deck Unit 7

(k) Spall in North Backwall with Exposed Rebar Beneath Deck Unit 7

Figure D.3 Continued

D.2 Bridge Geometry and Component Properties

The bridge girder detailing was not available. However, using the 23 sections found for double-tee girders and based on the year of construction and span length (Appendix E), it was determined that the girders should have seven tendons per stem, which were harped at a distance of 0.2L from each end of the girder. The prestressing steel for these girder were assumed to be uncoated seven-wire (A_{sp} = 0.196 in^2 [126 mm^2]) low-relaxation strands meeting the ASTM A416 requirements. Table D.1 presents the strand specified mechanical properties according to ASTM A416.

Table D.1 Specified Mechanical Properties for Prestressing Strands

Properties	0.5 in. (12.7) Strands (ASTM A416)
Yield Strength, f_y, ksi (MPa)	258 (1779)
Ultimate Strength, f_u, ksi (MPa)	285 (1965)
Strain at Break	7.4%
Modulus of Elasticity, *E*, ksi (MPa)	29000 (200000)

According to the shop drawing of a similar double-tee girder, transverse and longitudinal reinforcing steel bars should meet the requirements of ASTM A615 Grade 60. Similarly, the concrete compressive strength should be 5.5 ksi (34.5 MPa).

D.3 Undamaged Double-Tee Girders

The girder sectional properties, dead loads, live loads, and the capacity of the undamaged girder are discussed here.

D.3.1 Section Properties

46 in. × 23 in. double-tee girder
A = 377 in^2
I_x = 16084 in.^{4}
S_{bot} = 933.49 in.^{3}
S_{top} = 2787.5 in.^{3}

D.3.2 Dead Load Analysis

Density of concrete = 0.15 kip/ft^3
Beam self-weight = 0.39 kip/ft
Railing = 0.003 kip/ft
Total DC = 0.393 kip/ft
M_{DC} = 122.8 kip-ft
V_{DC} = 9.825 kips
Wearing surface = 0.09 kip/ft
M_{DW} = 28.125 kip-ft
V_{DW} = 2.25 kips

D.3.3 Live Load Analysis

This section presents the shear and moment girder distribution factors for external and internal bridge girders. Calculations are based on Table 4.6.2.2.2b-1 of the AASHTO LRFD Bridge Design Specifications, 6th Edition.

Table D.2 Girder Distribution Factor (GDF)

Lane	Shear GDF		Moment GDF	
	Exterior Girder	Interior Girder	Exterior Girder	Interior Girder
One Lane Loaded	0.49	0.49	0.49	0.35
Two or More Lanes Loaded	0.39	0.49	0.37	0.38
Governing GDF	**0.39**	**0.49**	**0.37**	**0.38**

D.3.4 Capacity of Undamaged Girders

The moment capacity of the undamaged girder calculated using Eq. 6.2 at the midspan is 1052.3 kip-ft and the shear capacity calculated using Eq. 6.6 at the support is 58.11 kips.

D.4 Damaged Double-Tee Girders

This section presents the type and location of the damage per girder, condition factors and girder capacities, and other factors that are needed to complete the load rating for the damaged bridge.

D.4.1 Condition Factors for Damaged Girders

Tables D.3 and D.4 present a summary of damage type and location for an external and internal girder, respectively. The damage of the girders was shown in Fig. D.3. The selected external and internal girders had more damage than other girders. The girder damage types and condition states were identified using the proposed definitions for double-tee girders as marked in Tables D.5 through D.8 with golden stars.

Table D.3 Damage of 50-ft External Girder

Component	Damage Type	Damage Location	Condition State
Stem of Girder	Cover Damage	0.2L	4 (Table D.5)
Stem of Girder	Exposed Transverse Rebar	0.2L	2 (Table D.5)
Stem of Girder	Exposed Longitudinal Prestressing	0.0L	2 (Table D.5)
Stem of Girder	Cracking (Both Stem)	0.5L	2 (Table D.5)
Flange of Girder	Cracking	0.25L	2 (Table D.6)
Flange of Girder	Cover Damage	0.5L	4 (Table D.6)
Girder to Girder Joint	Longitudinal Joint Deterioration	0.0L	2 (Table D.6)

Note: L is the bride span length measured from the south end support toward north.

Table D.4 Damage of 50-ft Internal Girder

Component	Damage Type	Damage Location	Condition State
Stem of Girder	Cover Damage	0.0L	4 (Table D.7)
Stem of Girder	Exposed Longitudinal Prestressing	0.0L	2 (Table D.7)
Stem of Girder	Cracking (Both Stem)	0.6L	2 (Table D.7)
Girder to Girder Joint	Longitudinal Joint Deterioration	0.0L	2 (Table D.8)

Note: L is the bride span length measured from the south end support toward north

Table D.5 Damage Types and Condition States for External Prestressed Double-Tee Girder Stem

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Transverse Rebar	None	Minor corrosion of the reinforcement with minimal section loss.	Severe corrosion of only one leg of transverse reinforcement.	Severe corrosion of all legs of transverse reinforcement in a section.
Exposed Longitudinal Prestressing	None	50% section loss due to corrosion in the extreme tendon.	100% section loss due to corrosion in the extreme tendon.	Section loss due to corrosion in the two or more tendons.
Cracking	Insignificant cracks or moderate-width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.

Table D.6 Damage Types and Condition States for External Prestressed Double-Tee Girder Top Flange

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area/Aberration	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Rebar	None	Minor corrosion of the outer layer of reinforcement with minimal section loss.	Severe corrosion of only the outer layer of reinforcement.	Severe corrosion of the outer and inner layers of reinforcement.
Cracking	Insignificant cracks or moderate width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.
Girder-to-Girder Longitudinal Joint Deterioration	None	Minimal deterioration, no sign of leakage.	Discrete signs of seepage along the joint, minor corrosion of steel plates.	Seepage along the joint, severe corrosion of steel plates.

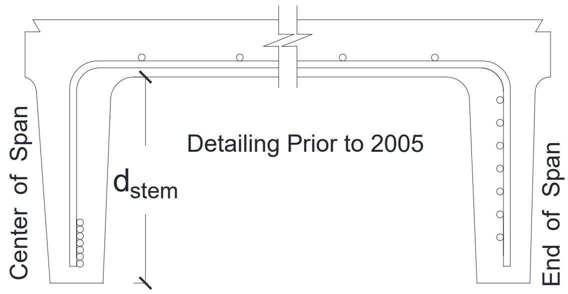
Table D.7 Damage Types and Condition States for Internal Prestressed Double-Tee Girder Stem

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Transverse Rebar	None	Minor corrosion of the reinforcement with minimal section loss.	Severe corrosion of only one leg of transverse reinforcement.	Severe corrosion of all legs of transverse reinforcement in a section.
Exposed Longitudinal Prestressing	None	50% section loss due to corrosion in the extreme tendon.	100% section loss due to corrosion in the extreme tendon.	Section loss due to corrosion in the two or more tendons.
Cracking	Insignificant cracks or moderate-width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.

Table D.8 Damage Types and Condition States for Internal Prestressed Double-Tee Girder Top Flange

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/Spall/Patched Area/Aberration	None	Loss of 1/3 of the cover without exposure or corrosion of reinforcement.	Loss of 2/3 of the cover without exposure or corrosion of reinforcement.	Exposure of reinforcement without any sign of corrosion.
Exposed Rebar	None	Minor corrosion of the outer layer of reinforcement with minimal section loss.	Severe corrosion of only the outer layer of reinforcement.	Severe corrosion of the outer and inner layers of reinforcement.
Cracking	Insignificant cracks or moderate width cracks that have been sealed.	Unsealed moderate width cracks or unsealed moderate pattern (map) cracking. Cracks from 0.004 to 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking. Cracks greater than 0.009 inches wide.	Wide cracks or heavy pattern (map) cracking that crosses multiple shear reinforcement.
Girder-to-Girder Longitudinal Joint Deterioration	None	Minimal deterioration, no sign of leakage.	Discrete signs of seepage along the joint, minor corrosion of steel plates.	Seepage along the joint, severe corrosion of steel plates.

These 23-in. (584-mm) deep girders were built before 2005. As discussed in the previous section, these girders most likely had seven tendons per stem based on the year of construction and span length. The moment and shear damage condition factors for the external and internal double-tee girders are presented in Fig. D.4 and D.5, respectively.



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1 Good	CS-2 Fair	CS-3 Poor	CS-4 Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1 Good	CS-2 Fair	CS-3 Poor	CS-4 Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70
Exposed Rebar	0.70	0.70	0.65	0.65
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

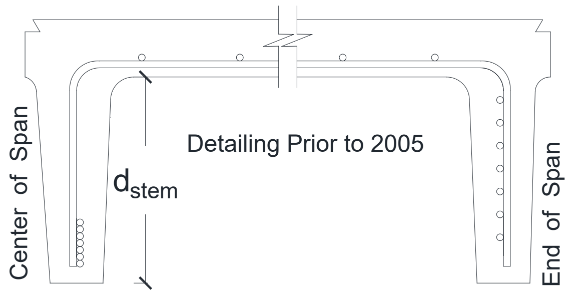
Damage Type	Condition States			
	CS-1 Good	CS-2 Fair	CS-3 Poor	CS-4 Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.40
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.45
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1 Good	CS-2 Fair	CS-3 Poor	CS-4 Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70
Exposed Rebar	0.70	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure D.4 Condition Factors for 23-in. Deep 14-Harped Tendon External Double-Tee Girder (Pre 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1
Exposed Transverse Rebar	1	1	1	1
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70
Cracking	1	1	1	1

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70
Exposed Rebar	0.70	0.70	0.65	0.65
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)]
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50
Exposed Longitudinal Prestressing[(b)]	1	0.95	0.90	0.85
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70
		2/3 bottom stem depth (2/3 d_{stem})		0.45
		1/3 top stem depth (1/3 d_{stem})		0.40
Cracking on Both Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.45
		2/3 bottom stem depth (2/3 d_{stem})		0.0
		1/3 top stem depth (1/3 d_{stem})		0.0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States			
	CS-1	CS-2	CS-3	CS-4
	Good	Fair	Poor	Severe
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70
Exposed Rebar	0.70	0.70	0.70	0.70
Cracking	1	1	1	1
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1

Figure D.5 Condition Factors for 23-in. Deep 14-Harped Tendon Internal Double-Tee Girder (Pre 2005)

D.4.2 Dead Load Analysis

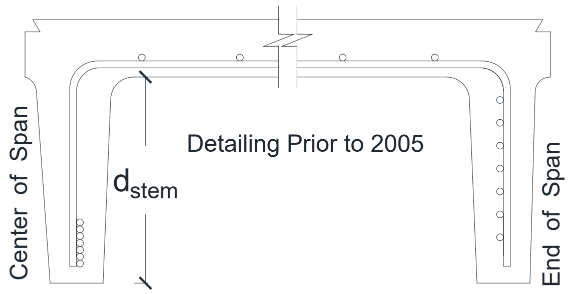
Dead load of damaged and undamaged girders are the same.

D.4.3 Live Load Analysis

The condition state for the girder-to-girder longitudinal joint damage is less than 4. Therefore, the live load distribution factors for the damaged girders remain the same as those for undamaged girders.

D.4.4 Capacity of Damaged Girders

The moment and shear capacities of the damaged girders are calculated by multiplying the capacity of the undamaged girder by the corresponding condition factors as shown in Fig. D.6 and D.7.



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	M (k.ft)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1	1052.3
Exposed Transverse Rebar	1	1	1	1	1052.3
Exposed Longitudinal Prestressing	1	0.90	0.85	0.70	947.07
Cracking	1	1	1	1	1052.3

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	M (k.ft)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70	0.7×1052.3=736.61
Exposed Rebar	0.70	0.70	0.65	0.65	N/A
Cracking	1	1	1	1	1052.3
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1	1052.3

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

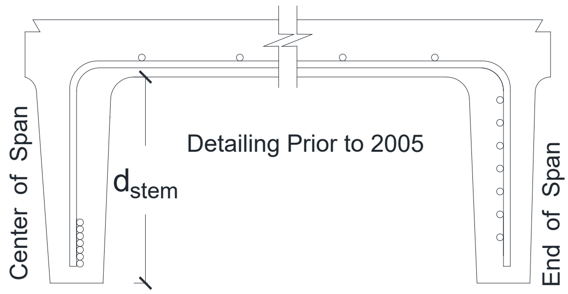
Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	*V* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[a]	See Cracking
Exposed Transverse Rebar[b]	1	0.85	0.75	0.50	0.85×58.11=49.39
Exposed Longitudinal Prestressing[b]	1	0.95	0.90	0.85	0.95×58.11=55.2
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.45	N/A
		1/3 top stem depth (1/3 d_{stem})		0.40	N/A
Cracking on Both Stems	1	1/3 bottom stem depth (1/3 d_{stem})		0.45	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.0	N/A
		1/3 top stem depth (1/3 d_{stem})		0.0	0.×58.11=0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	*V* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70	0.7×58.11=40.68
Exposed Rebar	0.70	0.70	0.70	0.70	N/A
Cracking	1	1	1	1	58.11
Girder-to-Girder Longitudinal Joint Deterioration	1	1	1	1	58.11

Figure D.6 Capacity of 23-in. Deep 14-Harped Tendon External Double-Tee Girder (Pre 2005)



Depth = 23 in. (584 mm)
Width = 46 in. (1168 mm)
No. of Tendons = 7 per stem
Tendon Diameter = 0.5 in. (13 mm)
Tendon Profile = Harped at 0.2L
Transverse Bar size = No. 4 (13 mm)
f'_c = 5.5 ksi (37.9 Mpa)
f'_y = 60 ksi (413.7 Mpa)
Initial Tendon Force = 28.91 kips (128.6 kN)

Stem Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	M (k.ft)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	1	1 ☆	1052.3
Exposed Transverse Rebar	1	1	1	1	N/A
Exposed Longitudinal Prestressing	1	0.90 ☆	0.85	0.70	0.9×1052.3=894.46
Cracking	1	1 ☆	1	1	1052.3

Flange Moment Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	M (k.ft)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.85	0.70	N/A
Exposed Rebar	0.70	0.70	0.65	0.65	N/A
Cracking	1	1	1	1	N/A
Girder-to-Girder Longitudinal Joint Deterioration	1	1 ☆	1	1	1052.3

Stem Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	*V* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area	1	1	0.95	Use C.F. for Cracking[(a)] ☆	See Cracking
Exposed Transverse Rebar[(b)]	1	0.85	0.75	0.50	N/A
Exposed Longitudinal Prestressing[(b)]	1	0.95 ☆	0.90	0.85	0.95×58.11=55.2
Cracking on Single Stem	1	1/3 bottom stem depth (1/3 d_{stem})		0.70	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.45	N/A
		1/3 top stem depth (1/3 d_{stem})		0.40	N/A
Cracking on Both Stems	1	1/3 bottom stem depth (1/3 d_{stem})		0.45	N/A
		2/3 bottom stem depth (2/3 d_{stem})		0.0	N/A
		1/3 top stem depth (1/3 d_{stem})		0.0 ☆	0.×58.11=0

Note: (a) This is the same as cracking (e.g., if cover deteriorates at the bottom 1/3 of one stem, use the first row in "Cracking on Single Stem").
(b) Assuming the cover deterioration is minimal (CS-1). Otherwise, cover deterioration will automatically govern.

Flange Shear Condition Factors for 23-in. Deep 14-Tendon Double-Tee Girders

Damage Type	Condition States				Calculated Capacity
	CS-1	CS-2	CS-3	CS-4	
	Good	Fair	Poor	Severe	*V* (kips)
Cover Deterioration including Delamination/ Spall/ Patched Area/ Aberration	1	0.90	0.80	0.70	N/A
Exposed Rebar	0.70	0.70	0.70	0.70	N/A
Cracking	1	1	1	1	N/A
Girder-to-Girder Longitudinal Joint Deterioration	1	1 ☆	1	1	58.11

Figure D.7 Capacity of 23-in. Deep 14-Harped Tendon Internal Double-Tee Girder (Pre 2005)

D.5 Load Rating of Damaged Double-Tee Bridge

The bridge "rating factor (RF)" based on the LRFR method can be calculated as:

$$RF = \frac{C - (\gamma_{DC})DC - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \qquad \text{(Eq. D.1)}$$

where *C* is the member capacity (e.g. shear and flexural capacities for Service and Strength Limit States), γ_{DC} is the LRFD load factor for structural components and attachments, *DC* is the dead load effect due to structural components and attachments, γ_{DW} is the LRFD load factor for wearing surfaces and utilities, *DW* is the dead load effect due to wearing surfaces and utilities, γ_P is the LRFD load factor for permanent loads other than dead loads, *P* is the permanent load effect other than dead loads, γ_{LL} is the evaluation live load factor, *LL* is the live load effect, and *IM* is the dynamic load allowance. The AASHTO Manual for Bridge Evaluation (2015) provides load factors for different limit states for the three live load levels discussed above.

Based on the proposed load rating method (Appendix C), the moment and shear capacities of a damaged double-tee girder at strength limit states should be reduced using the proposed condition factors (φ_c) for South Dakota double-tee sections as:

$$C_{damaged} = \varphi_c \,.\, C_{undamaged} \qquad \text{(Eq. D.1)}$$

where

$$C_{undamaged} = \varphi_s \,.\, \varphi \,.\, R_n \qquad \text{(Eq. D.2)}$$

All other parameters and methods remain the same as those specified in the AASHTO Manual for Bridge Evaluation (2011 or succeeding).

For Service Limit State,

$$C = f_R \qquad \text{(Eq. D.3)}$$

where f_R is the allowable stresses.

D.5.1 Evaluation Factors for Strength Limit States

<u>Resistance factors:</u>
Ø = 1.0 for flexure
<u>Condition factors</u>
$Ø_c =$ Figures 4 & 5
<u>System factor</u>
$Ø_s$ = 1.0

Table D.9 Strength I limit state

	Inventory	Operating
DC	1.25	1.25
DW	1.5	1.5
LL+IM	1.75	1.35

Inventory equation for Strength I Limit State

$$RF = \frac{(\emptyset_c)(\emptyset_s)(\emptyset)R_n - (\Upsilon_{DC})(DC) - (\Upsilon_{DW})(DW)}{(\Upsilon_L)(LL + IM)} \quad \text{(Eq. D.4)}$$

Operating Equation

$$RF = (Inventory\ RF) * \frac{1.75}{1.35} \quad \text{(Eq. D.5)}$$

Service III Limit State for Inventory Level

$$RF = \frac{f_R - \Upsilon_d f_D}{\Upsilon_L(f_{LL+IM})} \quad \text{(Eq. D.6)}$$

D.6 Summary of Load Rating

Table D.10 presents a summary of the input parameter used in the calculation of rating factors. Rating factors for the moment capacity under strength I and service III limit state and the shear capacity under strength I limit state were calculated using Eq. D.4 to D.6 and were summarized in Tables D.11 and D.12.

Table D.10 Input Parameters for Calculation of Rating Factors

Parameters	Limit State				
	Strength I				Service III
	Flexure		Shear		Flexure
	Inventory	Operating	Inventory	Operating	Inventory
$Ø_c$	0.7	N/A	0	N/A	N/A
$Ø_s$	1	N/A	1	N/A	N/A
$Ø$	1	N/A	1	N/A	N/A
R_n	1052.3 kip.ft	N/A	58.1 kips	N/A	N/A
Υ_{DC}	1.25	N/A	1.25	N/A	N/A
DC	122.8 kip.ft	N/A	9.825 kips	N/A	N/A
Υ_{DW}	1.25	N/A		N/A	N/A
DW	28.125 kip.ft	N/A	2.25 kips	N/A	N/A
Υ_L	1.75	N/A	1.75	N/A	N/A
$IM + LL$	393.36 kip.ft	N/A	28.73 kips	N/A	N/A
f_R	N/A	N/A	N/A	N/A	6.63 kips
Υ_d	N/A	N/A	N/A	N/A	1
f_D	N/A	N/A	N/A	N/A	1.93 ksi
f_{LL+IM}	N/A	N/A	N/A	N/A	5.05 ksi

Table D.11 – Summary of Rating Factors – Exterior Girder

Limit State		Design Load Rating	
		Inventory	Operating
Strength I	Flexure	0.78	1.01
	Shear	0.	0.
Service II	Flexure	1.16	N/A

Table D.12 Summary of Rating Factors – Internal Girder

Limit State		Design Load Rating	
		Inventory	Operating
Strength I	Flexure	1.02	1.32
	Shear	0.	0.
Service II	Flexure	1.16	N/A

APPENDIX E. DOUBLE-TEE GIRDER BRIDGE DETAILING

E.1 Introduction

Twenty-three different double-tee sections, which have been used in South Dakota, were identified. Figure E.1 shows the breakdown of the identified sections categorized based on the year built (prior to 2005 and after 2005) and the girder depth (23 in. and 30 in). The following sections present these girder shop drawing.

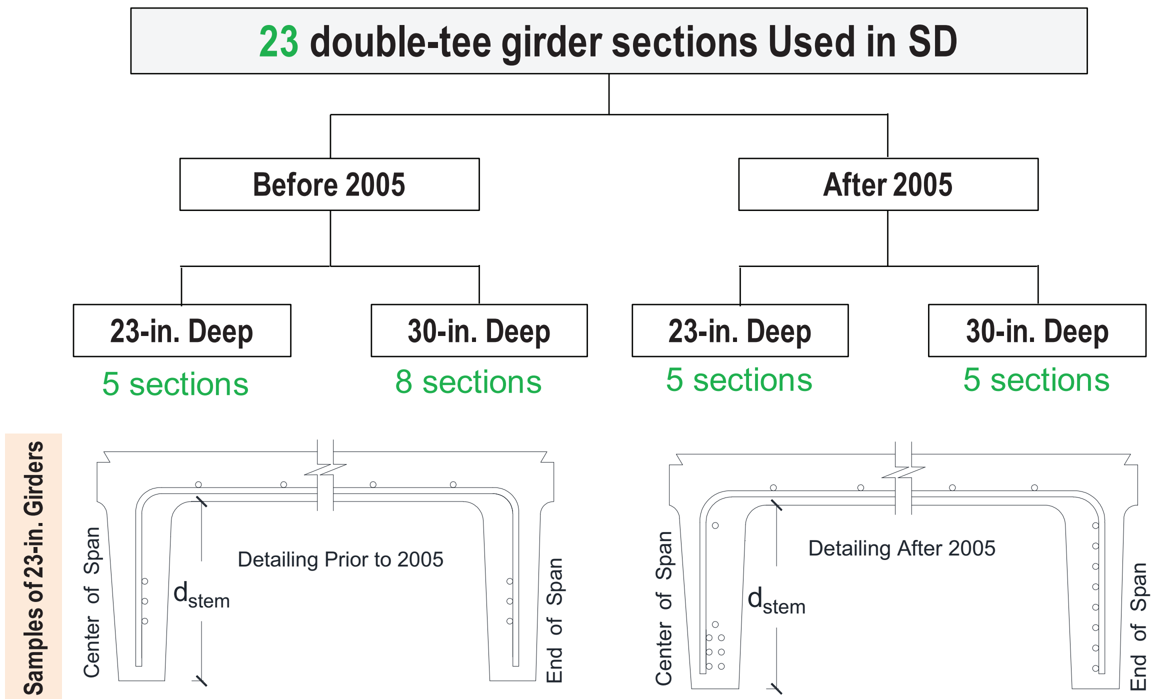
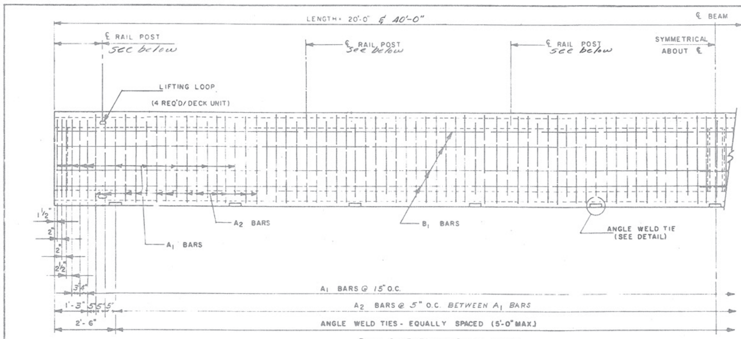
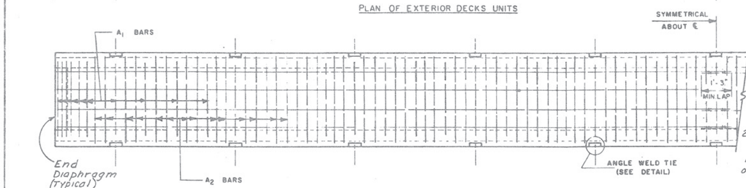


Figure E.1 – Double-Tee Sections Used in South Dakota

E.2 – 23-in. Double-Tee Sections Built before 2005



PLAN OF EXTERIOR DECKS UNITS



PLAN OF INTERIOR DECK UNITS

DECK UNIT LEGEND										
No. Req'd	Section Length	Weight (LBS)	Ø° Skew	Number of Rail Posts & Rail Post Spacing	C.G.S. End of Span	C.G.S. Center of Span	No. of Strds	f'ci P.S.I.	f'c P.S.I.	Strand Profile	AASHTO Loading
20	20'-0"	10100	20° RHF	See Below	8.00	8.00	6	4000	5000	STGT	HS20
	~~25'-0"~~	~~12400~~		~~4 RAIL POSTS @ 2'-0", 3 SPS @ 7'-0", 2'-0"~~	~~8.00"~~	~~8.00"~~	~~6~~	~~4000~~	~~5000~~	~~STGT~~	~~HS20~~
	~~30'-0"~~	~~14700~~		~~4 RAIL POSTS @ 2'-6", 3 SPS @ 8'-4", 2'-6"~~	~~8.00"~~	~~8.00"~~	~~8~~	~~4000~~	~~5000~~	~~STGT~~	~~HS20~~
	~~35'-0"~~	~~17000~~		~~5 RAIL POSTS @ 2'-0", 4 SPS @ 7'-9", 2'-0"~~	~~7.20~~	~~7.20~~	~~10~~	~~4000~~	~~5000~~	~~STGT~~	~~HS20~~
10	40'-0"	19300	20° RHF	See Below	8.00	4.00	10	4000	5000	DRAPED	HS20
	~~45'-0"~~	~~21900~~		~~6 RAIL POSTS @ 2'-6", 5 SPS @ 8'-0", 2'-6"~~	~~10.00"~~	~~4.50~~	~~14~~	~~4600~~	~~5000~~	~~DRAPED~~	~~HS20~~
	~~50'-0"~~	~~24200~~		~~7 RAIL POSTS @ 2'-6", 6 SPS @ 7'-6", 2'-6"~~	~~10.00"~~	~~3.50~~	~~14~~	~~4600~~	~~5500~~	~~DRAPED*~~	~~HS20~~

*CENTER .2L OF STRAND ARE HORIZONTAL (2 DRAPE HOLDDOWNS)

LENGTH	WIDTH OF BRIDGE: 6-UNITS 23'-0"	7-UNITS 26'-10"	8-UNITS 30'-8"	9-UNITS 34'-6"
20'-0"	HS-20	HS-20	HS-20	HS-20
25'-0"	↕	↕	↕	↕
30'-0"				
35'-0"	HS-20			
40'-0"	HS-20†	HS-20		
45'-0"	H-20	HS-20†	HS-20	HS-20
50'-0"	H-20†	H-20	HS-20†	HS-20†

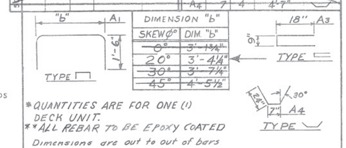
†-NO PROVISION FOR OVERLAY

ALLOWABLE LOADING

{16-20'-0"
{8-40'-0"
{4-20'-0"
{2-40'-0"

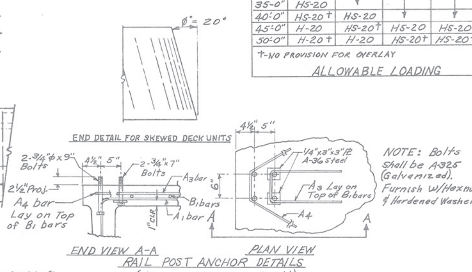
REINFORCING STEEL SCHEDULE **											
	INTERIOR DECK UNITS					EXTERIOR DECK UNITS					
Span	Mark	No.*	Size	Length	Type	Mark	No.*	Size	Length	Type
20'-0" SPAN	A_1	25	4	"b"+(3'-0")	⊓	A_1	25	4	"b"+(3'-0")	⊓
	A_2	28	4	"b"+3"	STR.	A_2	28	4	"b"+3"	STR.
						A_3	3	4	3'-9"	═
	B_1	4	4	19'-10"	STR.	B_1	4	4	19'-10"	STR.
						A_4	3	4	4'-7"	⌴
~~25'-0" SPAN~~	~~A_1~~	~~29~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~	~~A_1~~	~~29~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~
	~~A_2~~	~~36~~	~~4~~	~~"b"+3"~~	~~STR.~~	~~A_2~~	~~36~~	~~4~~	~~"b"+3"~~	~~STR.~~
						~~A_3~~	~~4~~	~~4~~	~~3'-9"~~	~~═~~
	~~B_1~~	~~4~~	~~4~~	~~24'-10"~~	~~STR.~~	~~B_1~~	~~4~~	~~4~~	~~24'-10"~~	~~STR.~~
						~~A_4~~	~~4~~	~~4~~	~~4'-7"~~	~~⌴~~
~~30'-0" SPAN~~	~~A_1~~	~~33~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~	~~A_1~~	~~33~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~
	~~A_2~~	~~44~~	~~4~~	~~"b"+3"~~	~~STR.~~	~~A_2~~	~~44~~	~~4~~	~~"b"+3"~~	~~STR.~~
						~~A_3~~	~~4~~	~~4~~	~~3'-9"~~	~~═~~
	~~B_1~~	~~4~~	~~4~~	~~29'-10"~~	~~STR.~~	~~B_1~~	~~4~~	~~4~~	~~29'-10"~~	~~STR.~~
						~~A_4~~	~~4~~	~~4~~	~~4'-7"~~	~~⌴~~
~~35'-0" SPAN~~	~~A_1~~	~~37~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~	~~A_1~~	~~37~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~
	~~A_2~~	~~52~~	~~4~~	~~"b"+3"~~	~~STR.~~	~~A_2~~	~~52~~	~~4~~	~~"b"+3"~~	~~STR.~~
						~~A_3~~	~~5~~	~~4~~	~~3'-9"~~	~~═~~
	~~B_1~~	~~8~~	~~4~~	~~18'-1"~~	~~STR.~~	~~B_1~~	~~8~~	~~4~~	~~18'-1"~~	~~STR.~~
						~~A_4~~	~~5~~	~~4~~	~~4'-7"~~	~~⌴~~
40'-0" SPAN	A_1	41	4	"b"+(3'-0")	⊓	A_1	41	4	"b"+(3'-0")	⊓
	A_2	60	4	"b"+3"	STR.	A_2	60	4	"b"+3"	STR.
						A_3	6	4	3'-9"	═
	B_1	8	4	20'-7"	STR.	B_1	8	4	20'-7"	STR.
						A_4	6	4	4'-7"	⌴
~~45'-0" SPAN~~	~~A_1~~	~~45~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~	~~A_1~~	~~45~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~
	~~A_2~~	~~68~~	~~4~~	~~"b"+3"~~	~~STR.~~	~~A_2~~	~~68~~	~~4~~	~~"b"+3"~~	~~STR.~~
						~~A_3~~	~~6~~	~~4~~	~~3'-9"~~	~~═~~
	~~B_1~~	~~8~~	~~4~~	~~23'-1"~~	~~STR.~~	~~B_1~~	~~8~~	~~4~~	~~23'-1"~~	~~STR.~~
						~~A_4~~	~~6~~	~~4~~	~~4'-7"~~	~~⌴~~
~~50'-0" SPAN~~	~~A_1~~	~~49~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~	~~A_1~~	~~49~~	~~4~~	~~"b"+(3'-0")~~	~~⊓~~
	~~A_2~~	~~76~~	~~4~~	~~"b"+3"~~	~~STR.~~	~~A_2~~	~~76~~	~~4~~	~~"b"+3"~~	~~STR.~~
						~~A_3~~	~~7~~	~~4~~	~~3'-9"~~	~~═~~
	~~B_1~~	~~8~~	~~4~~	~~25'-7"~~	~~STR.~~	~~B_1~~	~~8~~	~~4~~	~~25'-7"~~	~~STR.~~
						~~A_4~~	~~7~~	~~4~~	~~4'-7"~~	~~⌴~~

DIMENSION "b"	
SKEW Ø°	DIM. "b"
~~0°~~	~~3'-1¾"~~
20°	3'-4¼"
~~30°~~	~~3'-7¼"~~
~~45°~~	~~4'-5½"~~



*QUANTITIES ARE FOR ONE (1) DECK UNIT.
**ALL REBAR TO BE EPOXY COATED
Dimensions are out to out of bars

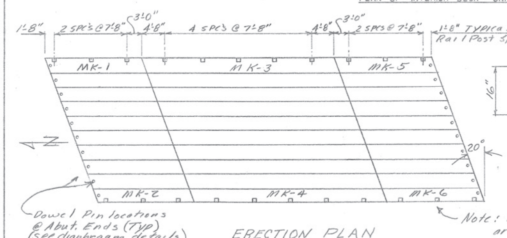
MARK	NO.	SIZE	LENGTH
A5	240	4	2'-7"



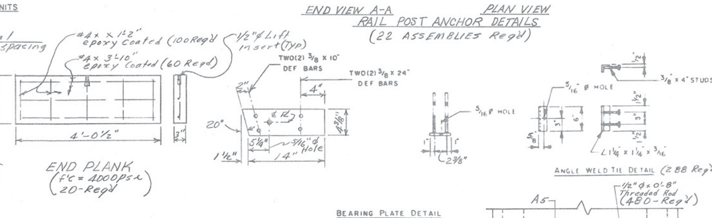
END VIEW A-A

PLAN VIEW

RAIL POST ANCHOR DETAILS (22 ASSEMBLIES Req'd)



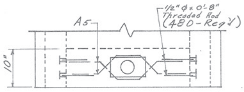
ERECTION PLAN



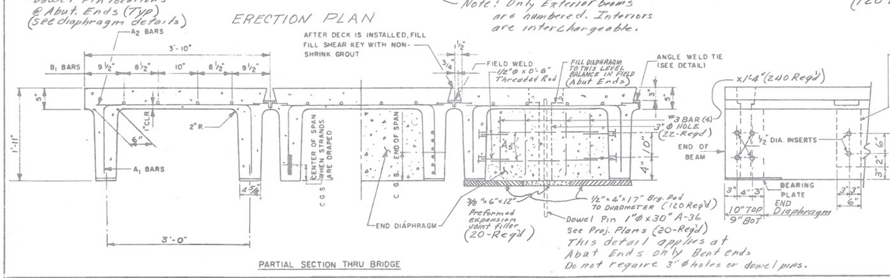
END PLANK (f'c = 4000 psi) (20-Req'd)

BEARING PLATE DETAIL (120 Req'd)

ANGLE WELD TIE DETAIL (288 Req'd)



END DIAPHRAGM-PLAN (Skewed with End of Beam)



PARTIAL SECTION THRU BRIDGE

GENERAL NOTES:

1. DESIGN SPECIFICATIONS: A.A.S.H.T.O. STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1983, WITH INTERIM SPECIFICATIONS THRU 1984
2. LIVE LOAD: A.A.S.H.T.O. HS 20-44.
3. CONCRETE: USING TYPE II CEMENT
4. REINFORCING STEEL: A.S.T.M. SPEC. A-615 GRADE 60
5. PRESTRESSING STEEL: STRANDS SHALL CONSIST OF SEVEN (7) WIRE UNCOATED, LOW RELAXATION, A.S.T.M. DESIGNATION A-416, ½" DIAMETER, 270K GRADE, MIN ULTIMATE STRENGTH OF 41,300 POUNDS PER STRAND. AN INITIAL TENSILE FORCE OF 28,910 POUNDS SHALL BE APPLIED TO EACH STRAND

SHOP DRAWINGS FOR PRESTRESSED CONCRETE BRIDGE DECKS

PROJECT NO. BRS 6491 (5)

Pennington County

Contractor: Heavy Constructors, Inc.

Plans By: Banner Associates Inc.

Manufacture at Rapid City Plant

Nov. 28-88

Sheet 1 of 3

PRESTRESSED CONCRETE BRIDGE DECK UNITS AS MANUFACTURED BY SOUTH DAKOTA CONCRETE PRODUCTS COMPANY			
23" DEEP			R/S 408090
DESIGNED BY	DRAWN BY WWL	CHECKED BY	APPROVED BRIDGE ENGINEER

3/18/88
1/14/79
4/3/78
1/11/78
REV: 10/4/76

E.3 – 23-in. Double-Tee Sections Built after 2005

FORTERRA™

Brian Anderson
PO Box 1620
Rapid City, SD 57709-1620
605-737-5208 (TEL)
605-737-5208 (FAX)
Brian.Anderson@forterrabp.com

To: **Grant County**

Attn: Kerwin

Date: **9/14/2017**

Project: **23"x35' DTEE - Str. # 360-200**
Grant County - Milbank, SD

Project#

Owner: **Kerwin - Grant County**

R/S # : **9017200BR1**

1	Set of	**Approved Shop Drawings**	sheets	1 - 11

____ For your approval. Please return 1 set to: **FORTERRA PIPE AND PRECAST**
PO BOX 1620, RAPID CITY, SD 57709-1620

____ ***PRODUCTION CANNOT BE SCHEDULED OR BEGIN UNTIL APPROVALS ARE RECEIVED.***

X For production as noted   X For jobsite use   X For your files

____ Per your request   ____ For your information   ____ Other

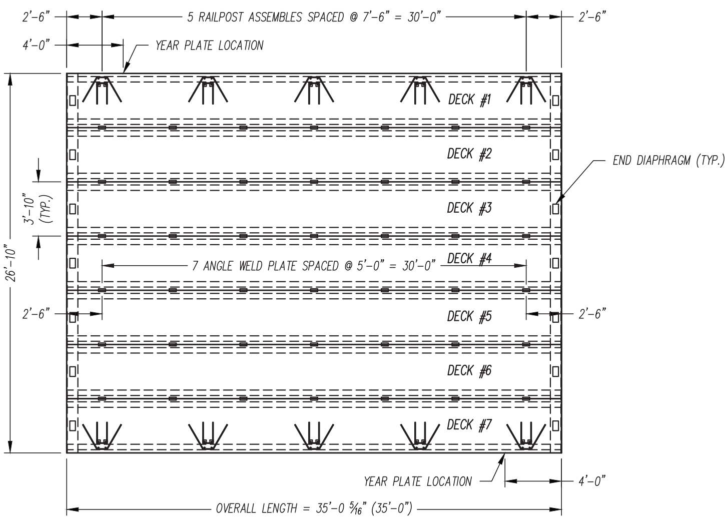
Copy:
1 Mitchell Plant, Proj. File
1 Mike Pardy

Sincerely,
FORTERRA PIPE AND PRECAST

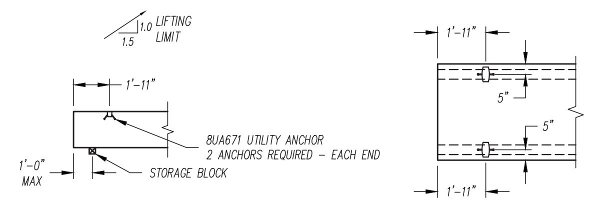
Brian Anderson

Brian Anderson, Project Engineer

By: JWB



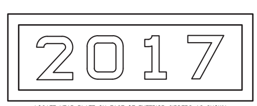
23" X 3'-10" X 35'-0" DTEE (0° SKEW)
17,000 LBS EACH



LIFTING & STORAGE DETAIL

PREPOUR LENGTH (POSTPOUR LENGTH)

DIMENSION KEY



LOCATE YEAR PLATE ON FACE OF EXTERIOR GIRDERS AS SHOWN. CENTER THE PLATE VERTICALLY ON THE EDGE OF THE SLAB.

MISCELLANEOUS ITEMS

28	4" X 15" X ½" LOOSE BEARING PADS
42	¼" X 1" X 5" WELD PLATES
1	POLYSEAL SILL PLATE SEALER 50' ROLL
14	1" X 29" DOWEL PIN
14	⅜" X 6" X 10" EXP. JOINT FILLER

PROJECT INFORMATION

PLACE OF FABRICATION	MITCHELL, SD
PROJECT #	N/A
STRUCTURE #	360-200
COUNTY	GRANT COUNTY
FORTERRA PROJECT #	9017200BR1
ENGINEER	N/A
CONTRACTOR	KERWIN - GRANT COUNTY
STATE INSPECTION REQ'D	NO

SECTION INFORMATION

BEAM TYPE	23" DTEE - NO SKEW
CONCRETE	TYPE I-II LA
f'ci (DETENTION)	5,000 psi
f'c (28 DAY)	6,000 psi
PS STRAND	1/2" DIA. X 270K LOW RELAXATION STRAND PER ASTM A-416
INITIAL TENSION	30,980 lbs
RE-BAR	ASTM A-615 GRADE 60
RE-MESH	ASTM A-497

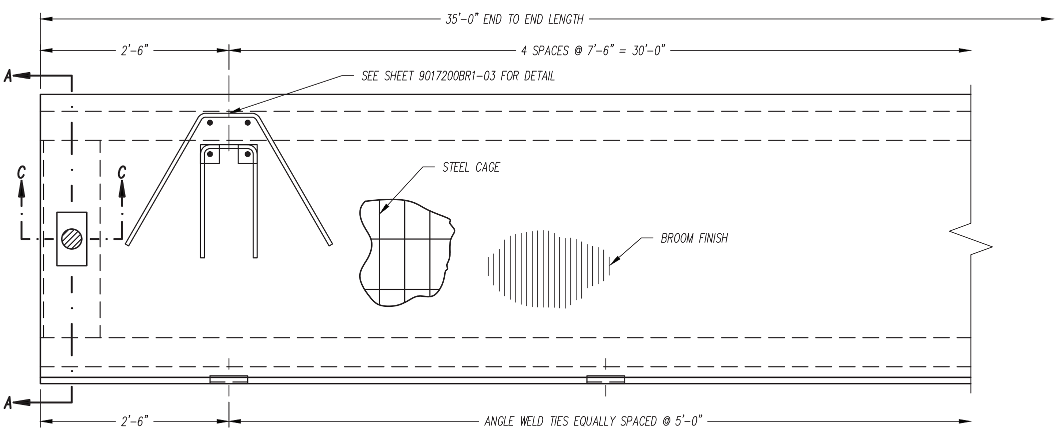
1. STENCIL EACH DECK WITH THE INFORMATION LISTED BELOW. PLACE STENCIL ON EXTERIOR FACE OF INTERIOR BEAMS - INSIDE FACE OF EXTERIOR BEAMS. CENTER STENCIL VERTICALLY ON LEG 4'-0" FROM END OF BEAM.

DATE OF MANUFACTURE
FORTERRA™
MITCHELL, SD
HL-93 LOADING
STR. # 360-200
GRANT COUNTY, SD
DECK # 1 2 3 4 5 6 7

2. THE TOP SURFACE SHALL BE ROUGHED BY BROOMING IN THE TRANSVERSE DIRECTION. THE AREA UNDER EACH RAILPOST SHALL BE FINISHED TO A SMOOTH AND UNIFORM SURFACE TO ASSURE PROPER FIT.
3. CONTRACTOR TO GROUT DEPRESSION AROUND LIFT ANCHORS IN FIELD (TYP.) - GROUT NOT PROVIDED.

FORTERRA™
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: 23" X 35'-0" DTEE (0° SKEW) STR. # 360-200 GRANT COUNTY - MILBANK, SD
DATE: 08/02/17	
OR#: 9017200BR1	
DR'N BY: JWB	CUSTOMER: KERWIN - GRANT COUNTY
CHK'D BY: _	DWG NAME: 9017200BR1-01

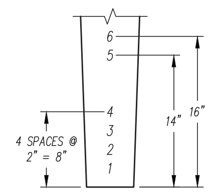


TOP VIEW

STRESSING INFORMATION

STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



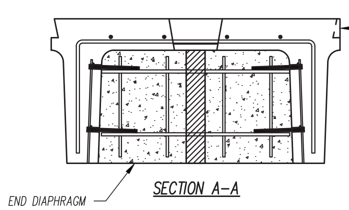
END

CONCRETE STRENGTH = 5,000 AT DETENTION
6,000 AT 28 DAYS

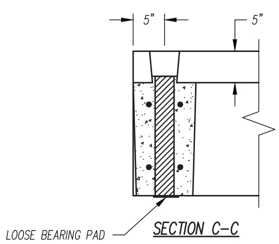
PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)

VERTICAL FACE FOR DECK UNITS WITH RAIL POST CONNECTIONS OR KEYWAY AND ANGLE WELD TIES FOR DECK UNITS WITHOUT RAIL POST CONNECTIONS

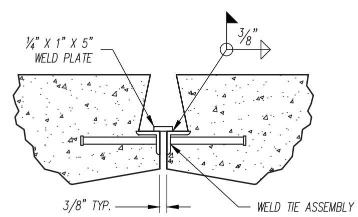
SEE DIAPHRAGM DETAILS ON FOLLOWING PAGES



SECTION A-A



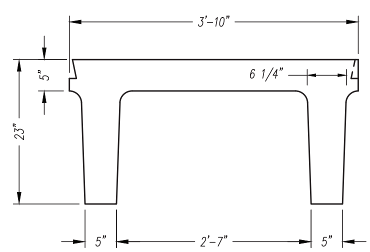
SECTION C-C



ANGLE WELD TIE CONNECTION DETAIL

FORTERRA
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

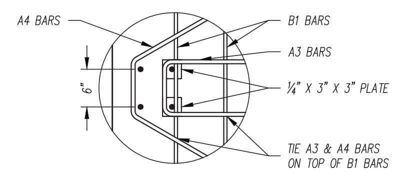
SCALE: NONE	PROJECT: CASTING DETAILS 23" X 35'-0" DTEE (0° SKEW) GRANT COUNTY - MILBANK, SD
DATE: 08/02/17	
OR#: 9017200BR1	
DR'N BY: JWB	CUSTOMER: KERWIN - GRANT COUNTY
CHK'D BY: _	DWG NAME: 9017200BR1-02



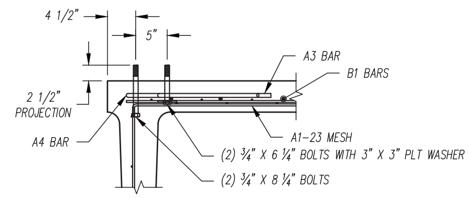
TYPICAL SECTION VIEW

23" DEEP PRESTRESSED CONCRETE DTEE

FURNISH WITH GALVANIZED HEX NUTS AND HARDENED WASHERS

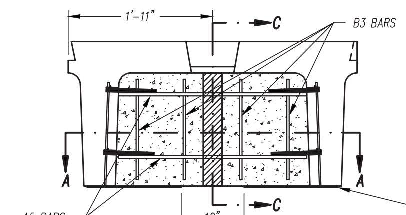


TOP SECTION

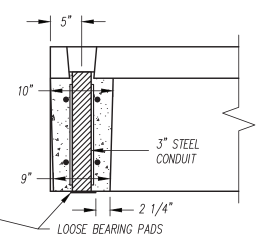


END SECTION

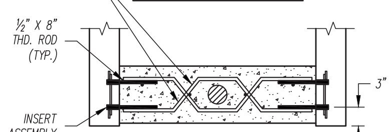
T-101 RAIL POST



SECTION THRU DIAPHRAGM



SECTION C-C



SECTION A-A

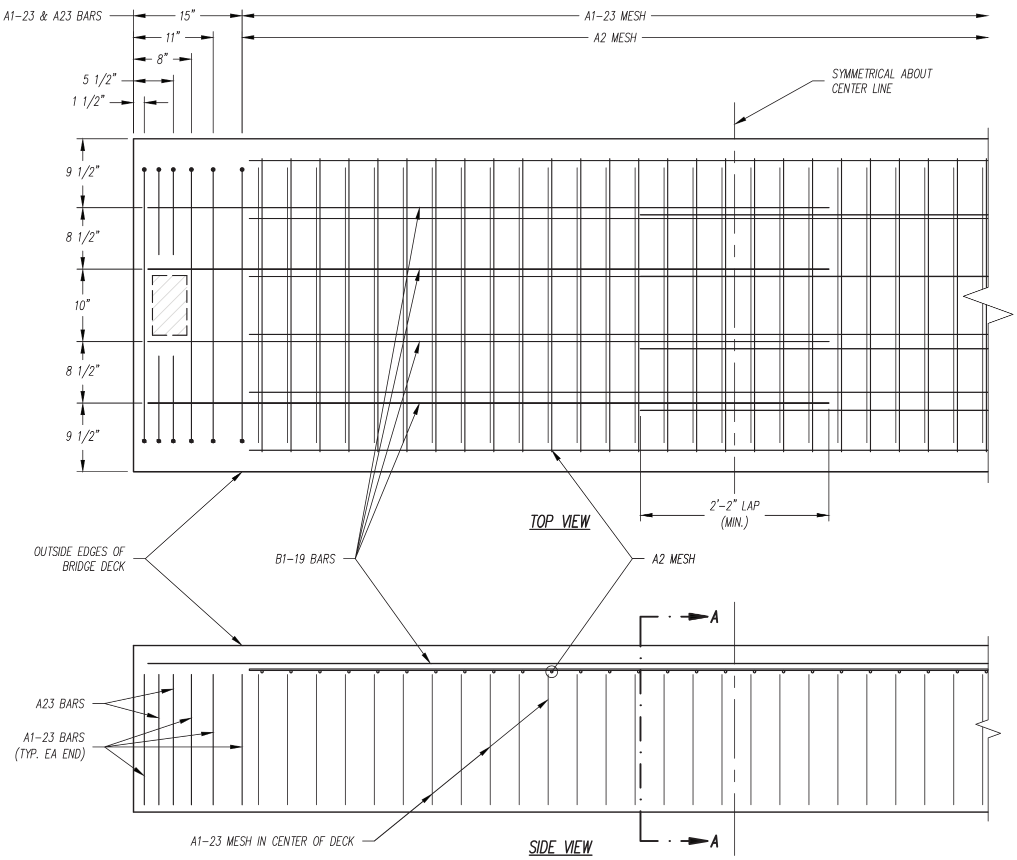
END DIAPHRAGM

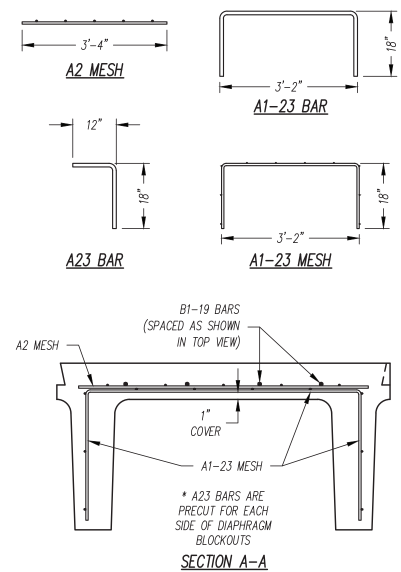
FORTERRA™

Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: RAIL POST & DIAPHRAGM DETAILS 23" X 35'-0" DTEE (0° SKEW) GRANT COUNTY – MILBANK, SD
DATE: 08/02/17	
OR#: 9017200BR1	
DR'N BY: JWB	CUSTOMER: KERWIN – GRANT COUNTY
CHK'D BY: –	DWG NAME: 9017200BR1-03

MARK	# PER DECK	SIZE	LENGTH	SHAPE	TOTAL
A1-23 BAR	8	4	6'-2"		72
A1-23 MESH	5	4X8 MESH D8.0XD4.0	6'-2"		45
A2 MESH	5	4X8 MESH D8.0XD4.0	3'-4"	STR.	45
B1-19	8	4	19-0"	STR.	72
* A23	8	4	2'-6"		72

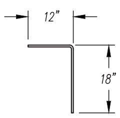




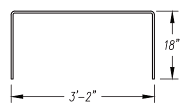
FORTERRA

Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

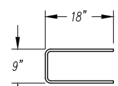
SCALE: NONE	PROJECT: STEEL CAGE LAYOUT 23" X 35'-0" DTEE (0° SKEW) GRANT COUNTY - MILBANK, SD
DATE: 08/02/17	
OR#: 9017200BR1	
DR'N BY: JWB	CUSTOMER: KERWIN - GRANT COUNTY
CHK'D BY: –	DWG NAME: 9017200BR1-04



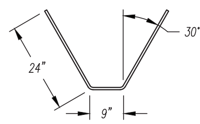
A23 BAR



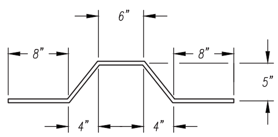
A1-23 BAR



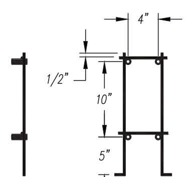
A3



A4



A5



INSERT ASSEMBLY

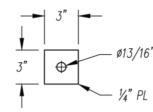
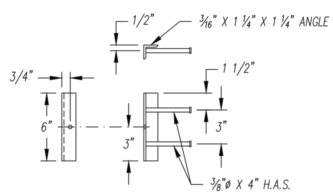
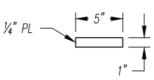


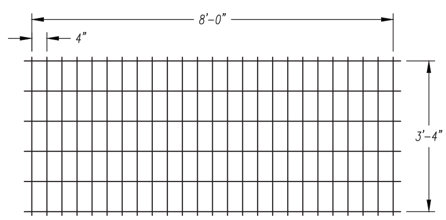
PLATE WASHER



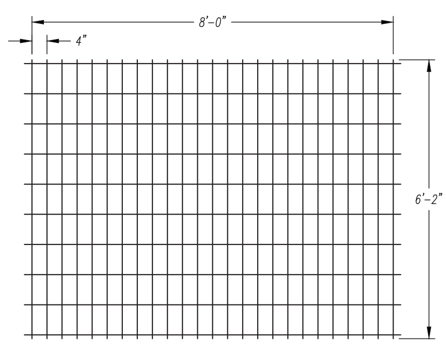
ANGLE WELD TIE



WELD PLATE



A2 MESH



A1-23 MESH

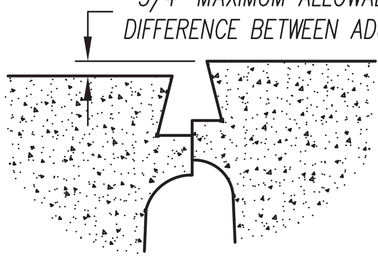
NOTE:
1) REINFORCING STEEL DIMENSIONS ARE OUT TO OUT
2) REINFORCING STEEL AS PER ASTM 1-615 GRADE 60
3) STRUCTURAL STEEL AS PER ASTM A-36

MARK	# / DECK INT/EXT	SHAPE	SIZE	DIMENSION	TOTAL
A1-23 BAR	8		4	6'-2"	56
A1-23 MESH	5		8.0/4.0	6'-2" X 8'-0"	35
A2 MESH	5	STR.	8.0/4.0	3'-4" X 8'-0"	35
A3	0/5		5	3'-9"	10
A4	0/5		5	4'-9"	10
A5	8		5	4'-9"	56
A23	8		4	2'-6"	56
B1-19	8	STR.	4	19'-0"	56
B3	8	STR.	3	1'-4"	56
B4	3	STR.	3	1'-1"	21
B5	2	STR.	4	2'-5"	14
ANGLE WELD TIE	14/7				84
INSERT ASMBLY	4				28
* GALV BOLT	0/10	GALV	¾" X 6 ¼"		20
* GALV BOLT	0/10	GALV	¾" X 8 ¼"		20
PLT WASHER	0/10				20
THD. ROD	16		½" X 8"		112
CONDUIT	2		3" DIA.	19"	14
LOOSE ITEMS					
WELD PLATE	7		¼" PL	1" X 5"	42
BRG. PAD	4	70 DURO	½" X 4"	15"	28
DOWEL PINS	2	STR.	1"	29"	14
EXP. FILLER	2		⅜"	6" X 10"	14
POLY SEAL BOND BREAKER				50'	1 ROLL

* GALVANIZED PER ASTM A153
* BOLT PER ASTM A449

FORTERRA
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

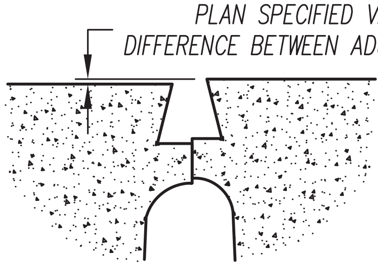
SCALE:	NONE	PROJECT:	MATERIALS LIST 23" X 35'-0" DTEE (0° SKEW) GRANT COUNTY - MILBANK, SD
DATE:	08/02/17		
OR#:	9017200BR1		
DR'N BY:	JWB	CUSTOMER:	KERWIN - GRANT COUNTY
CHK'D BY:	_	DWG NAME:	9017200BR1-05



FABRICATION SPECIFICATION

Producer Responsibility

A. Maximum Allowable camber difference as specified in PCI Quality Contro Manual Section 6.4.4 is limited to 1/4" per 10 ft. (3/4" max). This is a production tolerance only. This tolerance may not apply to beams that have a span-to-depth ratio approaching or exceeding 30.



INSTALLATION SPECIFICATION

Contractor Responsibility

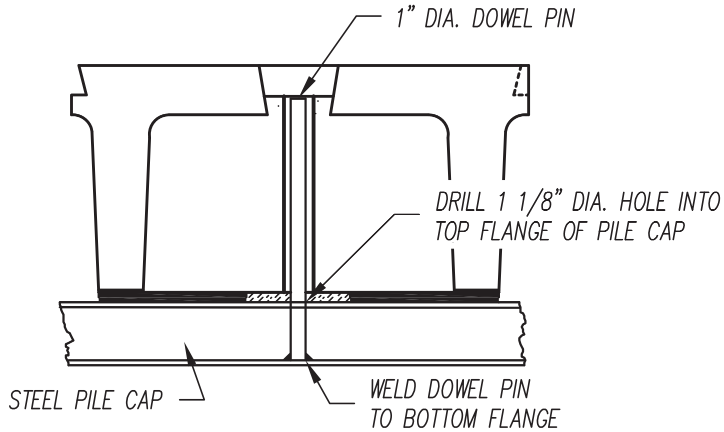
B. Installation tolerances are specified in the project plans. If the differential is more than allowed by plan notes, this tolerance can be achieved by the contractor by several methods including but not limited to the following: lifting the end of a deck and stitch welding at points along the deck to meet specifications, loading a deck unit to bring it into tolerance prior to welding, or in some situations shimming the end of the deck unit.

Application of any method to install deck units to meet this specification is completely the Contractor's responsibility and needs to be approved by the manufacturer and the Project Engineer prior to performing the work.

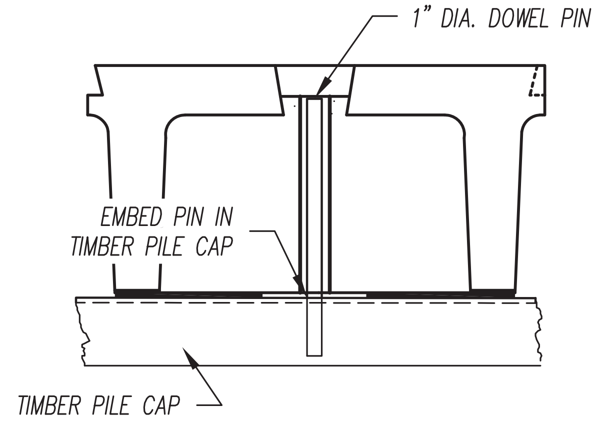
NOTE: These methods apply for both Double Tee Deck Units and Wide Flange Bulb Tee Deck Units.

FORTERRA™
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

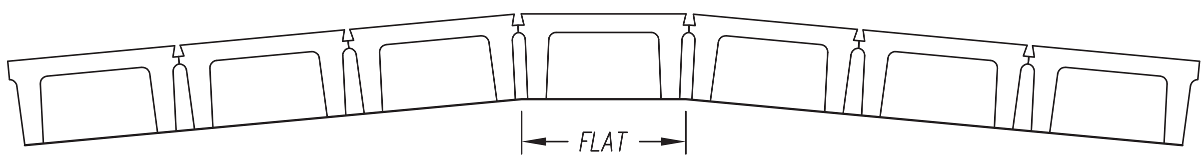
SCALE: NONE	PROJECT: CONCRETE DECK UNIT INSTALLATION
DATE: 06/03/04	
DR'N BY: RTF	
REV: 01/14/16 JWB	DWG NAME: 2010-35



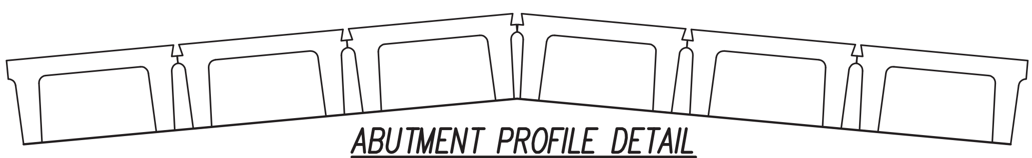
PRESTRESS DECK UNIT TO STEEL ABUTMENT CONNECTION



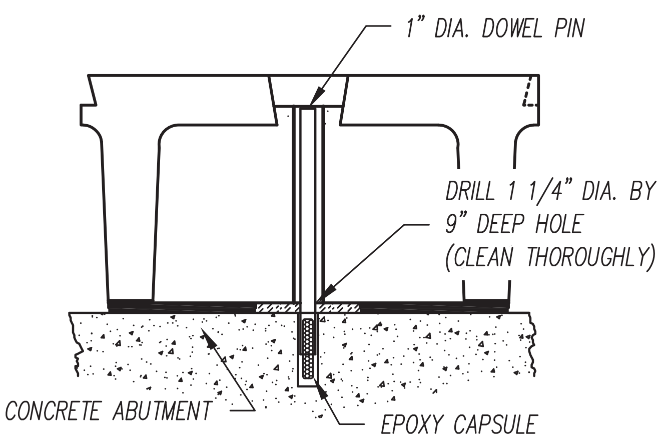
PRESTRESS DECK UNIT TO TIMBER ABUTMENT CONNECTION



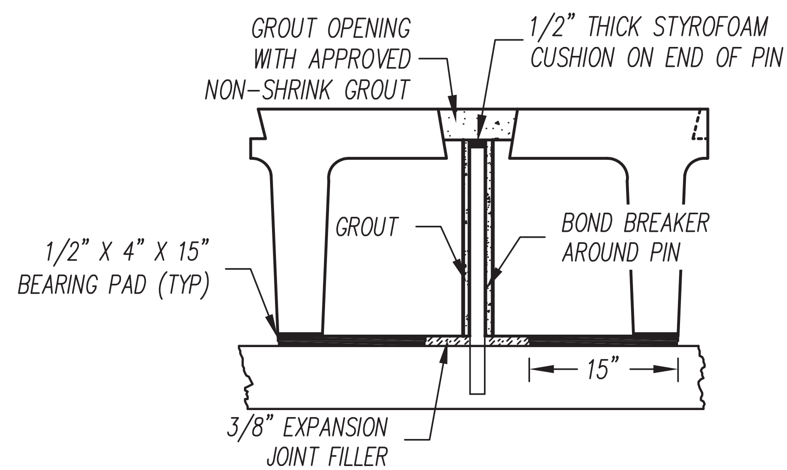
ABUTMENT PROFILE DETAIL
ODD#-UNITS WIDE - CROWNED SLOPE



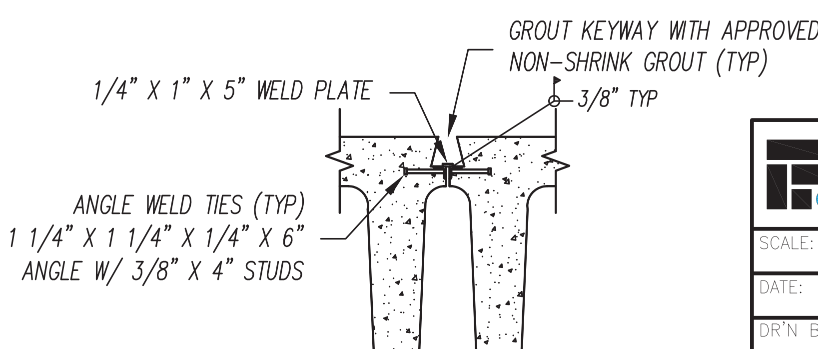
ABUTMENT PROFILE DETAIL
EVEN#-UNITS WIDE - CROWNED SLOPE



PRESTRESS DECK UNIT TO CONCRETE ABUTMENT CONNECTION



BEARING AND GROUTING DETAILS



DTEE CONNECTION DETAIL

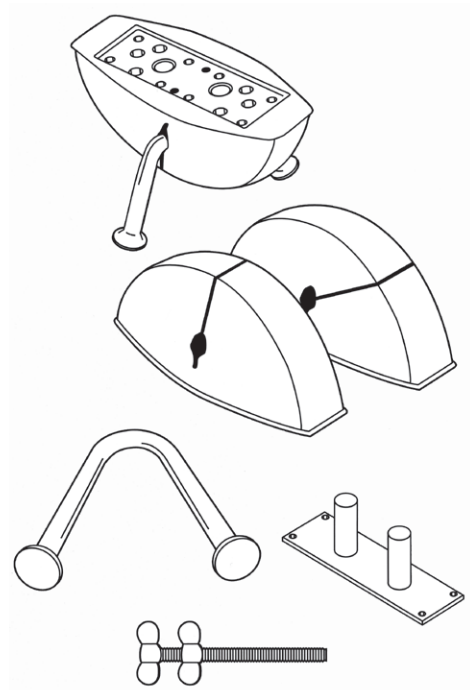
FORTERRA™

Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: DOUBLE TEE DECK UNIT ABUTMENT CONNECTION DETAILS
DATE: 06/03/04	
DR'N BY: RTF	
REV: 01/14/16 JWB	DWG NAME: 2020-10

DAYTON® SUPERIOR®

Utility Anchor System

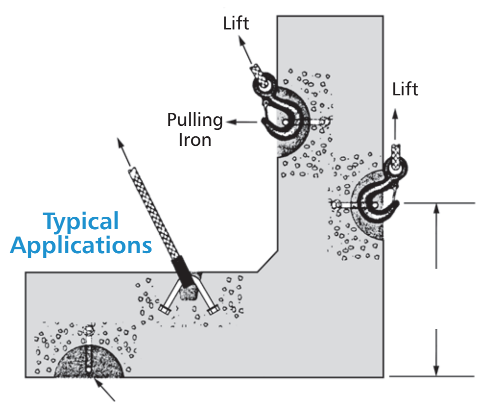


Key Advantages

Added Benefit

Utilize the Utility Anchor System to:

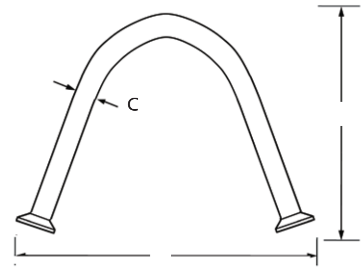
Anchor Placement



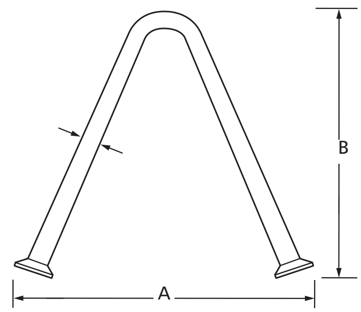
DAYTON® SUPERIOR®

P75 and P75H Utility Anchor®

P75 and P75H Utility Anchor						
	4UA444	121877	5-1/4"	3-1/8"	0.444"	Swift Lift
	5UA444	123442	6"	3-3/4"	0.444"	Swift Lift
P75	5UA671	123441	6-7/16"	3-3/4"	0.671"	Swift Lift
	6UA671	121889	7-3/8"	4-3/4"	0.671"	Swift Lift
	8UA671	121891	9-3/4"	6-3/4"	0.671"	Swift Lift



P75 Utility Anchor



P75-H Utility Anchor

Anchor	Type	Product Code No.	Minimum Panel Thickness	Safe Working Load Tension 90	Safe Working Load Shear 90	Safe Working Load Tension/ Shear 45	Minimum Edge Distance
	5US444	123442	5"	3,860	7,710	2,730	10"
	5UA671	123441	5"	4,560	8,430	3,220	10"
	8UA671	121801	7 5/8"	10,830	18,850	7,660	16"

Note:

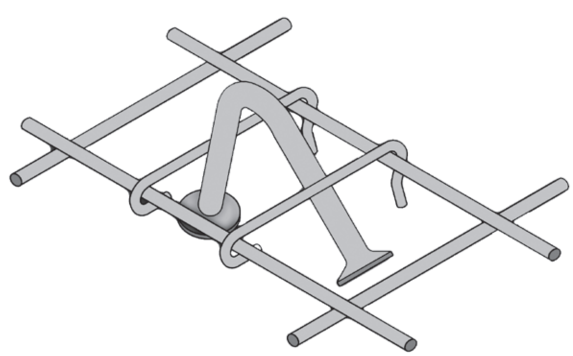
To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P75 Utility Anchors, 5UA444.

P75C Utility Anchor® with Clip

To Order:
Specify: (1) quantity, (2) name, (3) product code (4) anchor size, (5) wire spacing (6) wall thickness.

Example:
200, P75C, #121443, 5UA444anchor, 9" wire spacing, 5" wall.



Product Code	Utility Anchor	Wire Clip Lengths	Wall Thickness
121890	5UA671	9"	5"
121893	6UA671	9"	6"

Utility Anchor Lifting System

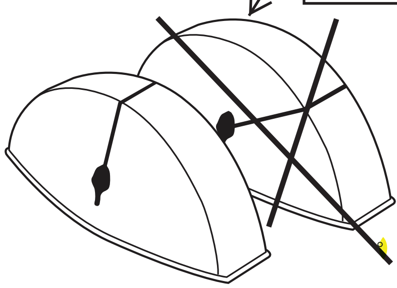
P76 Utility Anchor® Setting Plugs

NOT USED

P76 Utility Anchor Setting Plug					
Type	Product Code No.	Length	Width	Depth	Color
~~45P444~~	~~123176~~	~~8.00"~~	~~3.25"~~	~~3"~~	~~Blue~~
90P671	127786	9.00"	4.58"	3.35"	Orange
90P875	124685	15.00"	6.13"	5"	Blue

NOT USED

45° NOT USED



P76 Utility Anchor Setting Plugs

To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P76 Utility Anchor Setting Plugs, 90P444.

BLUE PLUG USED FOR UA444
ORANGE PLUG USED FOR UA671
LARGE BLUE PLUG USED FOR UA875

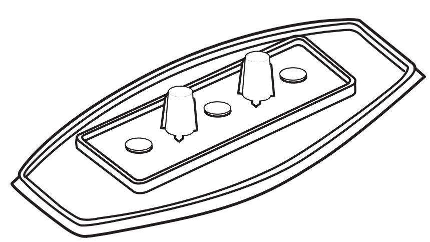
P76D Disposable Setting Plugs

P76D Disposable Utility Anchor Setting Plugs 0.671

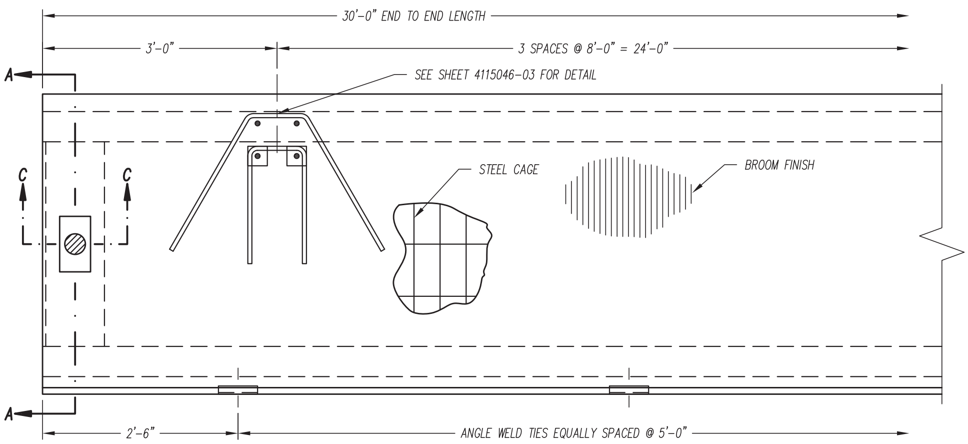
To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P76D, #126214.

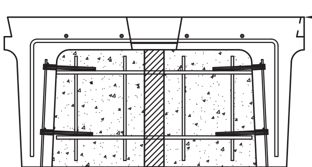
P76C Utility Anchor Cover/Patch



P76C Utility Anchor Cover/Patch



TOP VIEW

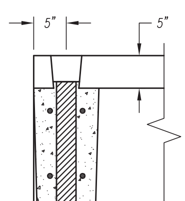


VERTICAL FACE FOR DECK UNITS WITH RAIL POST CONNECTIONS OR KEYWAY AND ANGLE WELD TIES FOR DECK UNITS WITHOUT RAIL POST CONNECTIONS

SEE DIAPHRAGM DETAILS ON FOLLOWING PAGES

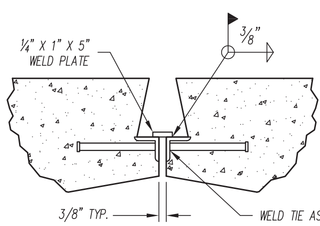
END DIAPHRAGM

SECTION A-A



LOOSE BEARING PAD

SECTION C-C

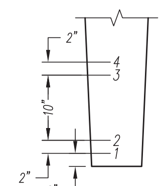


ANGLE WELD TIE CONNECTION DETAIL

STRESSING INFORMATION

STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



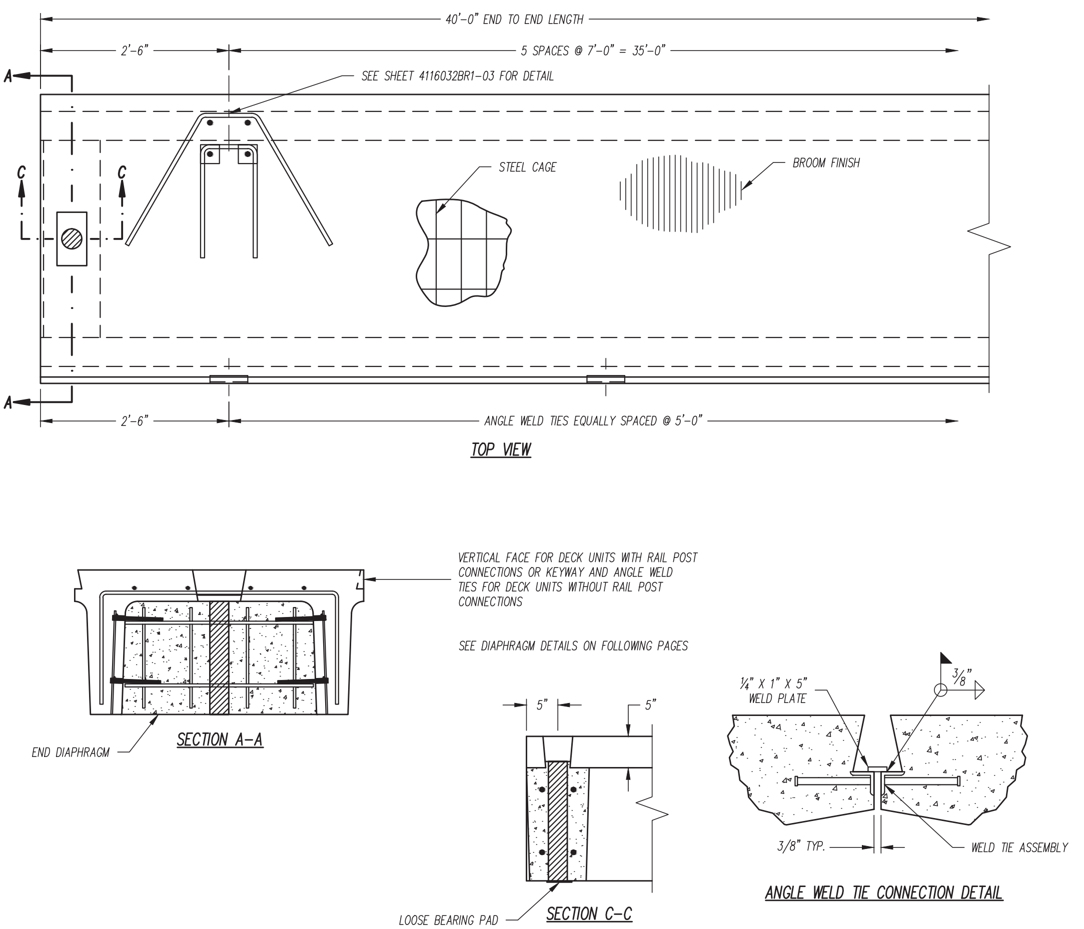
END

CONCRETE STRENGTH = 5000 AT DETENTION
6000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHIO M203, SUPPLEMENT 1 (LOW LAX STRAND)

Cretex Concrete Products
OFFICES IN: BISMARCK HELENA RAPID CITY

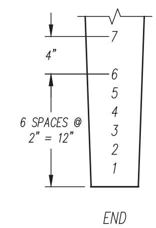
SCALE NONE
DATE 04/13/15
DR'N BY JWB
TITLE CASTING DETAILS
RS# 4115046
CUSTOMER GRANT COUNTY - KERWIN
REV DATE –
DWG NAME 4115046-02



STRESSING INFORMATION

STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



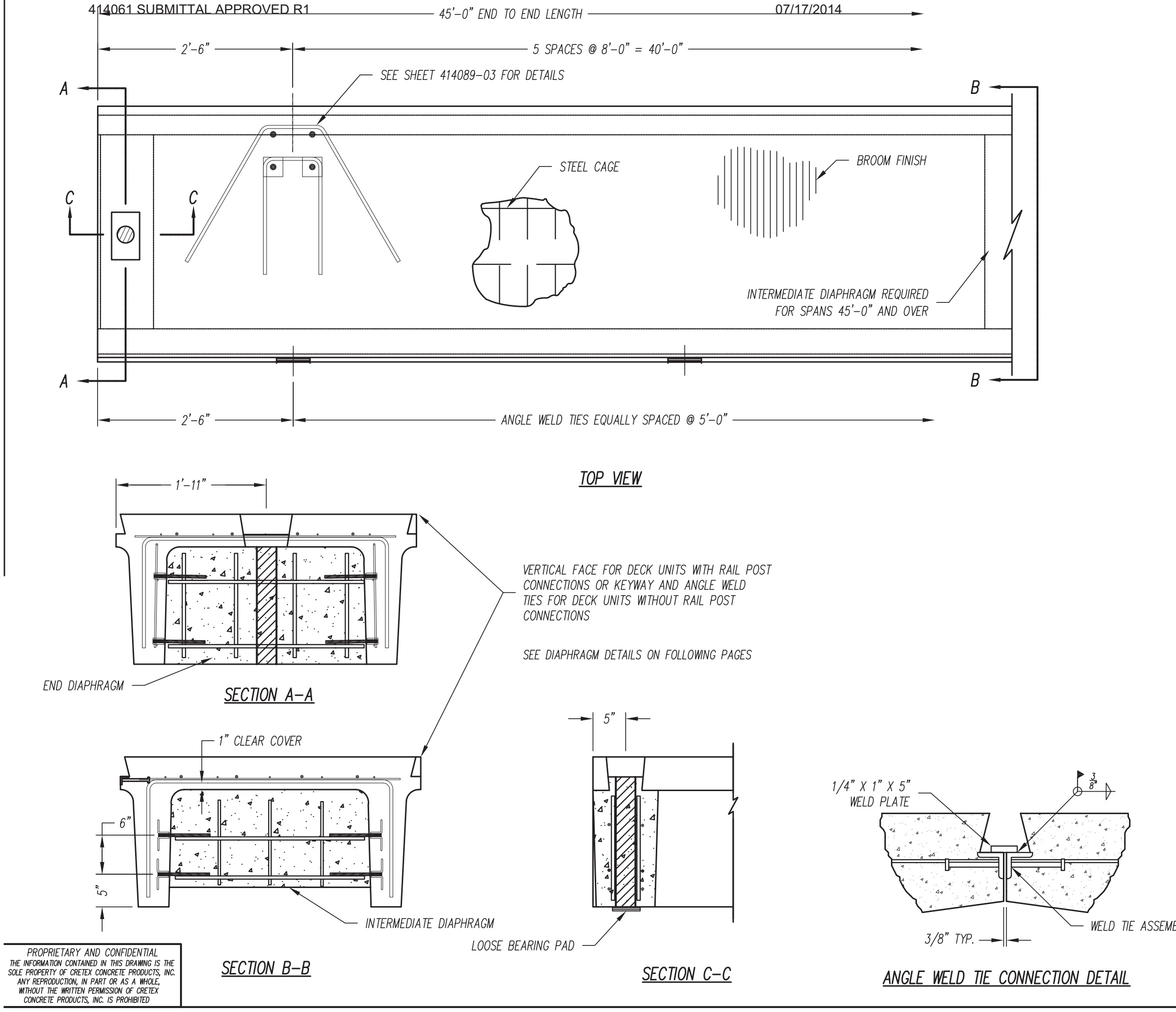
CONCRETE STRENGTH = 5,500 AT DETENTION
6,000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)

FORTERRA
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: CASTING DETAILS 23" X 40'-0" DTEE (0° SKEW) CLAREMONT, SD
DATE: 04/05/16	
OR#: 4116032BR1	
DR'N BY: JWB	CUSTOMER: BROWN COUNTY HWY DEPT
CHK'D BY: –	DWG NAME: 4116032BR1-02

REV:	DESCRIPTION:	DATE:	BY:
1	CHANGED RELEASE STRENGTH	04/11/16	JWB



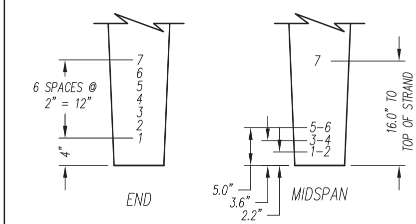
STRESSING INFORMATION

DRAPED STRAND

GAUGE PRESSURE = 3094 ELONGATION = 10.50

STRAIGHT STRAND

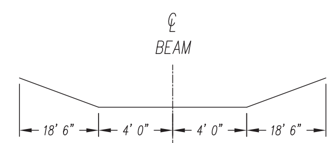
GAUGE PRESSURE = 3165 ELONGATION = 10.77



STRAND PATTERN

CONCRETE STRENGTH = 5500 AT DETENSION
6000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)

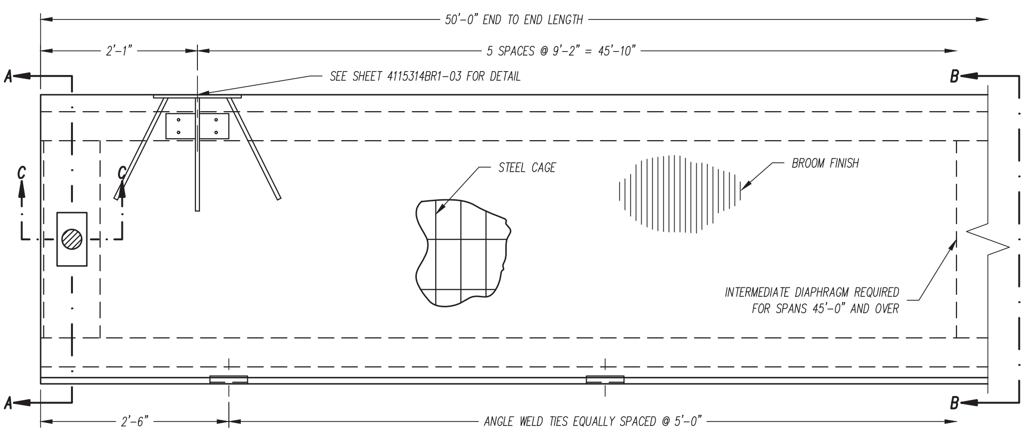


HOLD-DOWN DETAIL

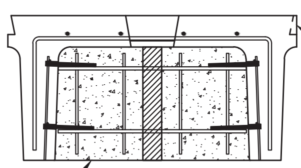
Cretex Concrete Products

OFFICES IN: BISMARCK HELENA RAPID CITY

SCALE none	TITLE CASTING DETAILS MARSHALL COUNTY
DATE 6/10/14	
DR'N BY NJC	
RS# 414061	CUSTOMER MARSHALL COUNTY HWY DEPT.
REV DATE 7/17/14 NJC	DWG NAME 414061-02



TOP VIEW

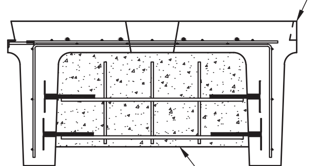


SECTION A-A

END DIAPHRAGM

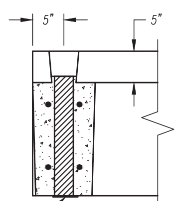
VERTICAL FACE FOR DECK UNITS WITH RAIL POST CONNECTIONS OR KEYWAY AND ANGLE WELD TIES FOR DECK UNITS WITHOUT RAIL POST CONNECTIONS

SEE DIAPHRAGM DETAILS ON FOLLOWING PAGES



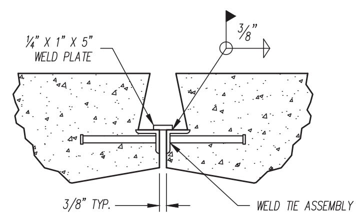
SECTION B-B

INTERMEDIATE DIAPHRAGM



LOOSE BEARING PAD

SECTION C-C

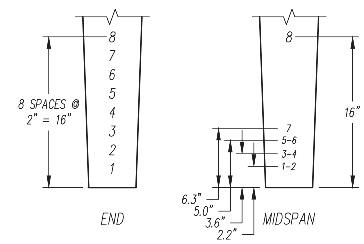


ANGLE WELD TIE CONNECTION DETAIL

STRESSING INFORMATION

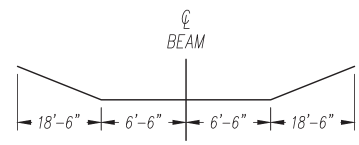
STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



CONCRETE STRENGTH = 6,000 AT DETENTION
7,250 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)



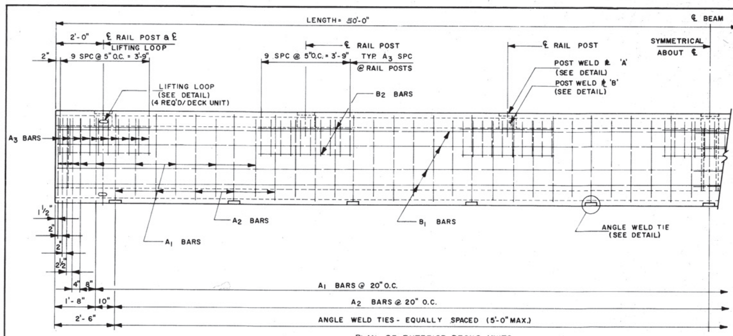
HOLD-DOWN DETAIL

REV:	DESCRIPTION:	DATE:	BY:
1	FIXED TYPE OF RAIL POST ASSEMBLY SHOWN	01/25/16	JWB
2	ADDED INTERMEDIATE DIAPHRAGM NOTES & DETAILS TO TOP VIEW	03/08/16	JWB

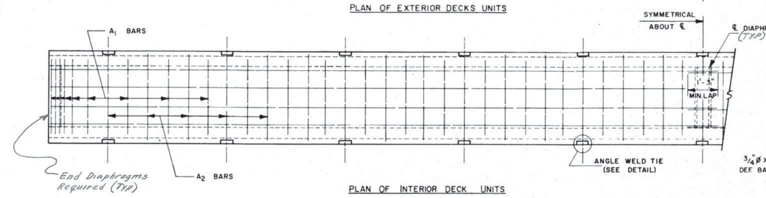
FORTERRA
Rapid City, South Dakota
2046 Samco Road, Suite 2
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: CASTING DETAILS 23" X 50'-0" DTEE (0° SKEW) PARKER, SD
DATE: 01/14/16	
OR#: 4115314BR1	
DR'N BY: JWB	CUSTOMER: HOLLAWAY CONSTRUCTION
CHK'D BY: –	DWG NAME: 4115314BR1-02

E.4 – 30-in. Double-Tee Sections Built before 2005



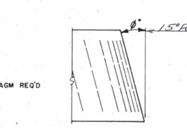
PLAN OF EXTERIOR DECKS UNITS



PLAN OF INTERIOR DECK UNITS

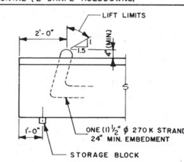
No. Req'd	Section Length	Weight (LBS)	Ø° Skew	Number Of Rail Posts & Rail Post Spacing	C.G.S. End Of Span	C.G.S. Center Of Span	No. Of Strds	f'ci P.S.I.	f'c P.S.I.	Strand Profile	AASHTO Loading
	20'-0"	11000		3 RAIL POSTS @ 2'-0", 2 SPS @ 8'-0", 2'-0"	12.00"	12.00"	6	4500	5000	STGT	HS20
	25'-0"	13800		4 RAIL POSTS @ 2'-0", 3 SPS @ 7'-0", 2'-0"	12.00"	12.00"	6	4500	5000	STGT	HS20
	30'-0"	16600		4 RAIL POSTS @ 2'-0", 3 SPS @ 8'-8", 2'-0"	7.67"	7.67"	6	4500	5000	STGT	HS20
	35'-0"	19300		5 RAIL POSTS @ 2'-0", 4 SPS @ 7'-9", 2'-0"	8.00"	8.00"	8	4500	5000	STGT	HS20
	40'-0"	22100		5 RAIL POSTS @ 2'-0", 4 SPS @ 9'-0", 2'-0"	12.00"	4.00"	8	4500	5000	DRAPED	HS20
	45'-0"	24800		6 RAIL POSTS @ 2'-0", 5 SPS @ 8'-2 3/6", 2'-0"	13.00"	8.77"	12	4500	5000	DRAPED	HS20
8	50'-0"	27600	15° RHF	6 RAIL POSTS @ 2'-0", 5 SPS @ 9'-2 3/8", 2'-0"	12.00"	7.83"	14	4500	5000	DRAPED	HS20
	55'-0"	30400		7 RAIL POSTS @ 2'-0", 6 SPS @ 8'-6", 2'-0"	12.00"	7.01"	16	4500	5500	DRAPED	HS20
	60'-0"	33100		8 RAIL POSTS @ 2'-0", 7 SPS @ 8'-0", 2'-0"	12.00"	7.31"	18	5000	6000	DRAPED*	HS20
	65'-0"	35900		8 RAIL POSTS @ 2'-0", 7 SPS @ 8'-8 5/8", 2'-0"	15.00"	4.50"	20	5000	6000	DRAPED*	HS20

* CENTER .2L OF STRANDS ARE HORIZONTAL (2 DRAPE HOLDDOWNS)

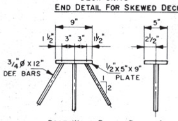


NOTE: A1 & A2 BARS SHALL BE SPACED AS SHOWN FOR INTERIOR OR EXTERIOR DECK UNITS

END DETAIL FOR SKEWED DECK UNITS

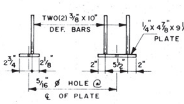


LIFTING LOOP DETAIL



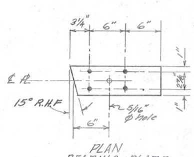
POST WELD PLATE - DETAIL A

(12 Req'd)

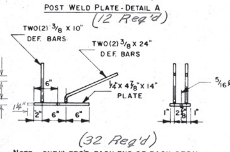


POST WELD PLATE - DETAIL B

(12 Req'd)



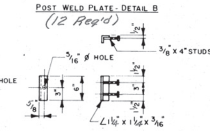
PLAN BEARING PLATE (Top view)



(32 Req'd)

NOTE: ONE (1) REQ'D EACH END OF EACH STEM. SEE REVISED DETAIL WHEN SKEWED PLATES ARE REQ'D.

BEARING PLATE DETAIL

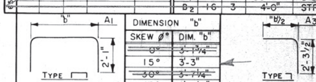


ANGLE WELD TIE DETAIL

(140 Req'd)

Reinforcing Steel Schedule

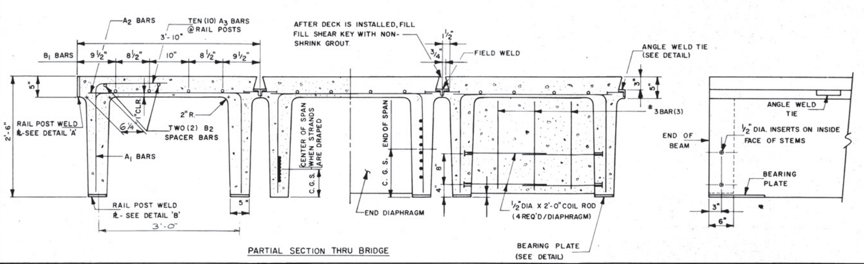
Span	6 Interior Deck Units: Mark	No.*	Size	Length	Type	2 Exterior Deck Units: Mark	No.*	Size	Length	Type
20'-0" SPAN	A1	21	4	"b"+(4'-2")	⊓	A1	21	4	"b"+(4'-2")	⊓
	A2	10	4	"b"+3"	STR.	A2	10	4	"b"+3"	STR.
						A3	30	6	"b"/2+(2'-3 1/2")	¬
	B1	4	4	19'-10"	STR.	B1	4	4	19'-10"	STR.
						B2	6	3	4'-0"	STR.
25'-0" SPAN	A1	24	4	"b"+(4'-2")	⊓	A1	24	4	"b"+(4'-2")	⊓
	A2	13	4	"b"+3"	STR.	A2	13	4	"b"+3"	STR.
						A3	40	6	"b"/2+(2'-3 1/2")	¬
	B1	4	4	24'-10"	STR.	B1	4	4	24'-10"	STR.
						B2	8	3	4'-0"	STR.
30'-0" SPAN	A1	27	4	"b"+(4'-2")	⊓	A1	27	4	"b"+(4'-2")	⊓
	A2	16	4	"b"+3"	STR.	A2	16	4	"b"+3"	STR.
						A3	40	6	"b"/2+(2'-3 1/2")	¬
	B1	4	4	29'-10"	STR.	B1	4	4	29'-10"	STR.
						B2	8	3	4'-0"	STR.
35'-0" SPAN	A1	30	4	"b"+(4'-2")	⊓	A1	30	4	"b"+(4'-2")	⊓
	A2	19	4	"b"+3"	STR.	A2	19	4	"b"+3"	STR.
						A3	50	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	18'-1"	STR.	B1	8	4	18'-1"	STR.
						B2	10	3	4'-0"	STR.
40'-0" SPAN	A1	33	4	"b"+(4'-2")	⊓	A1	33	4	"b"+(4'-2")	⊓
	A2	22	4	"b"+3"	STR.	A2	22	4	"b"+3"	STR.
						A3	50	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	20'-7"	STR.	B1	8	4	20'-7"	STR.
						B2	10	3	4'-0"	STR.
45'-0" SPAN	A1	36	4	"b"+(4'-2")	⊓	A1	36	4	"b"+(4'-2")	⊓
	A2	25	4	"b"+3"	STR.	A2	25	4	"b"+3"	STR.
						A3	60	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	23'-1"	STR.	B1	8	4	23'-1"	STR.
						B2	12	3	4'-0"	STR.
50'-0" SPAN	A1	39	4	"b"+(4'-2")	⊓	A1	39	4	"b"+(4'-2")	⊓
	A2	28	4	"b"+3"	STR.	A2	28	4	"b"+3"	STR.
						A3	60	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	25'-7"	STR.	B1	8	4	25'-7"	STR.
						B2	12	3	4'-0"	STR.
55'-0" SPAN	A1	42	4	"b"+(4'-2")	⊓	A1	42	4	"b"+(4'-2")	⊓
	A2	31	4	"b"+3"	STR.	A2	31	4	"b"+3"	STR.
						A3	70	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	28'-1"		B1	8	4	28'-1"	STR.
						B2	14	3	4'-0"	STR.
60'-0" SPAN	A1	45	4	"b"+(4'-2")	⊓	A1	45	4	"b"+(4'-2")	⊓
	A2	34	4	"b"+3"	STR.	A2	34	4	"b"+3"	STR.
						A3	80	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	30'-7"	STR.	B1	8	4	30'-7"	STR.
						B2	16	3	4'-0"	STR.
65'-0" SPAN	A1	48	4	"b"+(4'-2")	⊓	A1	48	4	"b"+(4'-2")	⊓
	A2	37	4	"b"+3"	STR.	A2	37	4	"b"+3"	STR.
						A3	80	6	"b"/2+(2'-3 1/2")	¬
	B1	8	4	33'-1"	STR.	B1	8	4	33'-1"	STR.
						B2	16	3	4'-0"	STR.



Skew Ø°	Dim. "b"
~~0°~~	~~3'-1 3/4"~~
15°	3'-3"
~~30°~~	~~3'-7 1/4"~~
~~45°~~	~~4'-5 1/2"~~

DIMENSIONS ARE OUT TO OUT OF BARS

* QUANTITIES ARE FOR ONE (1) DECK UNIT



PARTIAL SECTION THRU BRIDGE

GENERAL NOTES:

1. DESIGN SPECIFICATIONS: A.A.S.H.T.O. STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1973, WITH INTERIM SPECIFICATIONS FOR 1974 & 1975.
2. LIVE LOAD: A.A.S.H.T.O. HS 20-44.
3. CONCRETE: USING TYPE II CEMENT, $f'c = 5000$, $f'c_i = 4500$.
4. REINFORCING STEEL: A.S.T.M. SPEC. A-615 GRADE 60.
5. PRESTRESSING STEEL: STRANDS SHALL CONSIST OF SEVEN (7) WIRE UNCOATED STRESS RELIEVED, A.S.T.M. DESIGNATION A-416, 1/2" DIAMETER, 270K GRADE, MIN. ULTIMATE STRENGTH OF 41,300 POUNDS PER STRAND. AN INITIAL TENSILE FORCE OF 28,910 POUNDS SHALL BE APPLIED TO EACH STRAND.

SHOP DRAWING
FOR
PRESTRESSED BRIDGE DECK UNITS
FOR
BRIDGE OVER RAPID CREEK
PROJECT NO. SRS 0152 (1) PENNINGTON COUNTY
Contractor: Moore Construction Co.
Project Plans: Banner Associates, Inc.

PRESTRESSED CONCRETE BRIDGE DECK UNITS
AS MANUFACTURED BY
SOUTH DAKOTA CONCRETE PRODUCTS COMPANY

MFGD @ Mitchell Plant			R/S 40712 6
DESIGNED BY	DRAWN BY	CHECKED BY	APPROVED
	[illegible]		BRIDGE ENGINEER

Final: 12/27/77
12/13/77
~~REV: 10/4/76~~

3N

E.5 – 30-in. Double-Tee Sections Built after 2005

Brian Anderson

Rapid City, SD 57701
605-737-5208 (TEL)
(FAX)
Brian.Anderson@forterrabp.com

FORTERRA

/t

Attn:

0.0' DTEE Coxes-Mirror Lake Bridge Repl. t

Mark Junkert
t

R1t

1	Set of	Shop Drawing Submittalst	sheets	

t Landy tenbot rian Anderson t

____ PRODUCTION CANNOT BE SCHEDULED OR BEGIN UNTIL APPROVALS ARE RECEIVED.

X For production as noted X For jobsite use X For your files

____ Per your request ____ For your information ____ Other

Approved for Production and Shipping. Please note the following:

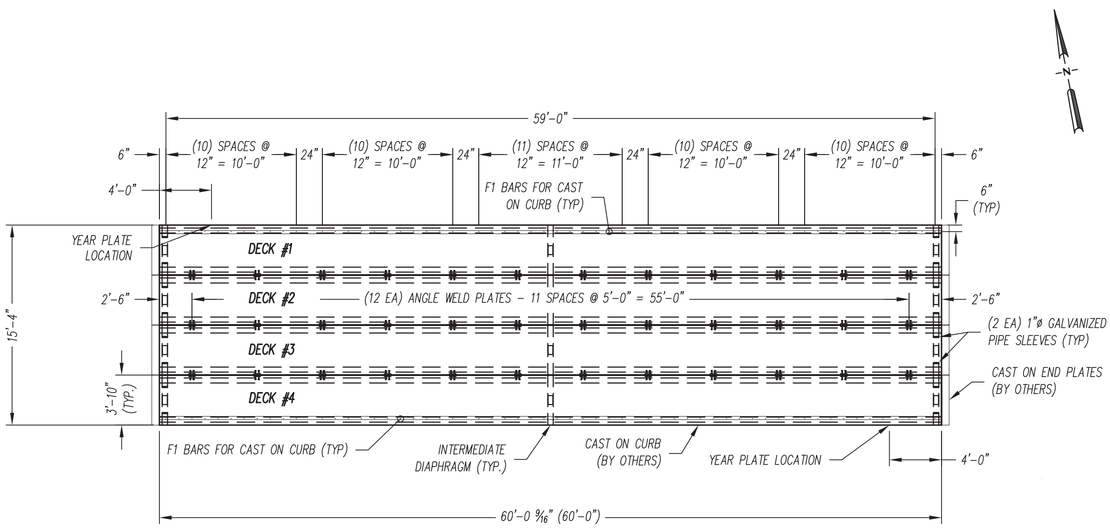
1.

2. Forward copies of approved shop drawings to the engineer.

3. Provide copies to field personnel responsible for product installation.

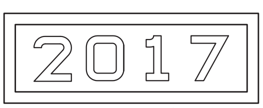
4. Carefully review installation guide.

Brian Anderson

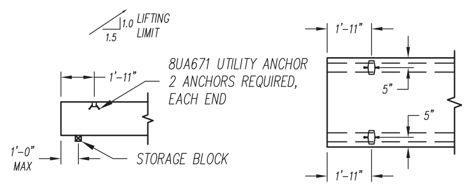


30" X 3'-10" X 60'-0" DTEE (0° SKEW)

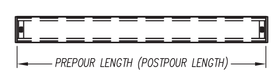
34,900 LBS EACH



LOCATE YEAR PLATE ON FACE OF EXTERIOR GIRDERS AS SHOWN. CENTER THE PLATE VERTICALLY ON THE EDGE OF THE SLAB.



LIFTING & STORAGE DETAIL



DIMENSION KEY

MISCELLANEOUS ITEMS	
16	4" X 15" X ½" LOOSE BEARING PADS
36	¼" X 1" X 5" WELD PLATES
8	1" X 29" DOWEL PIN
1	5.5 X 50 BOND BREAKER
8	⅛" X 6" X 10" EXP. JOINT FILLER

PROJECT INFORMATION	
PLACE OF FABRICATION	MITCHELL
PROJECT #	MIRR17WA
STRUCTURE #	41-019-032
COUNTY	LAWRENCE COUNTY
FORTERRA PROJECT #	4417119BR1
ENGINEER	AASON ENGINEERING
CONTRACTOR	JV BAILEY
STATE INSPECTION REQ'D	NO

SECTION INFORMATION	
BEAM TYPE	30" DTEE - 0° SKEW
CONCRETE	TYPE I-II LA
f'ci (DETENTION)	5,000 psi
f'c (28 DAY)	6,000 psi
PS STRAND	1/2" DIA. X 270K LOW RELAXATION STRAND PER ASTM A-416
INITIAL TENSION	30,980 lbs
RE-BAR	ASTM A-615 GRADE 60
RE-MESH	ASTM A-497

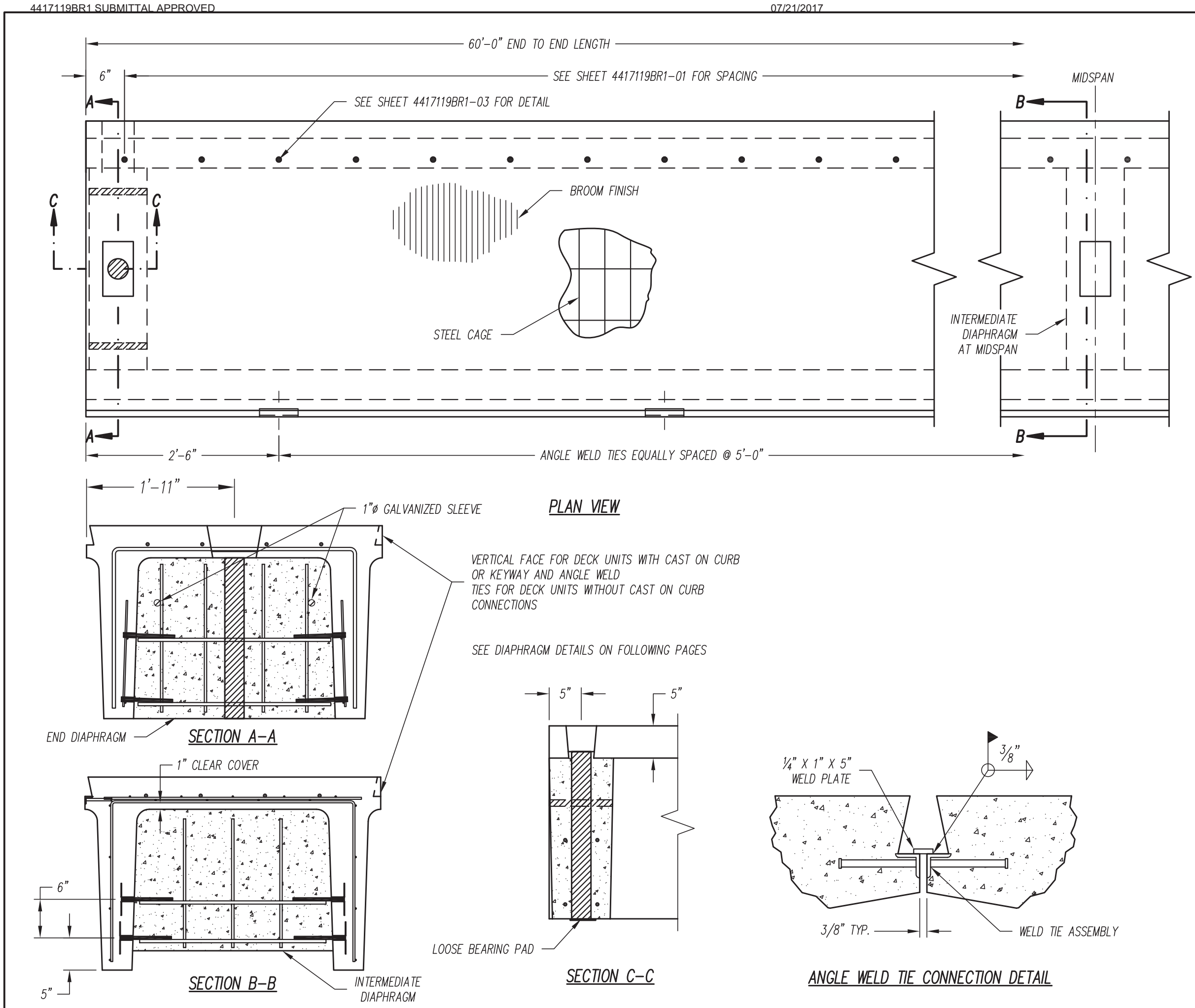
1. STENCIL EACH DECK WITH THE INFORMATION LISTED BELOW. PLACE STENCIL ON EXTERIOR FACE OF INTERIOR BEAMS - INSIDE FACE OF EXTERIOR BEAMS. CENTER STENCIL VERTICALLY ON LEG 4'-0" FROM END OF BEAM.

DATE OF MANUFACTURE
FORTERRA
MITCHELL, SD
MIRR17WA / 41-019-032
DECK # 1 2 3 4
LAWRENCE COUNTY, SD

2. THE TOP SURFACE SHALL BE ROUGHED BY BROOMING IN THE TRANSVERSE DIRECTION. THE AREA UNDER EACH RAILPOST SHALL BE FINISHED TO A SMOOTH AND UNIFORM SURFACE TO ASSURE PROPER FIT.
3. CONTRACTOR TO GROUT DEPRESSION AROUND LIFT ANCHORS IN FIELD (TYP.) - GROUT NOT PROVIDED.

FORTERRA
Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57701
(605) 718-4111

SCALE: NONE	PROJECT: 30" X 60'-0" DTEE (0° SKEW) PRO. MIRR17WA - STR. 41-019-032 LAWRENCE COUNTY, SD
DATE: 7/18/17	
OR#: 4417119BR1	
DR'N BY: LKB	CUSTOMER: JV BAILEY
CHK'D BY: -	DWG NAME: 4417119BR1-01

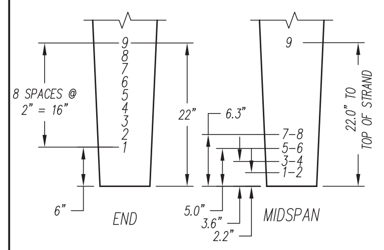


STRESSING INFORMATION

STRAND STRESSING

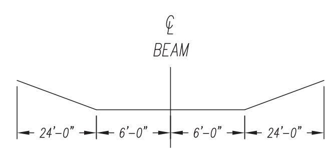
GAUGE PRESSURE = 3165

ELONGATION = 10 3/4"



CONCRETE STRENGTH = 5,000 AT DETENTION
6,000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)

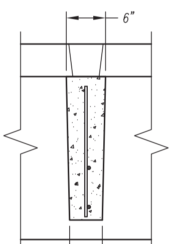


HOLD-DOWN DETAIL

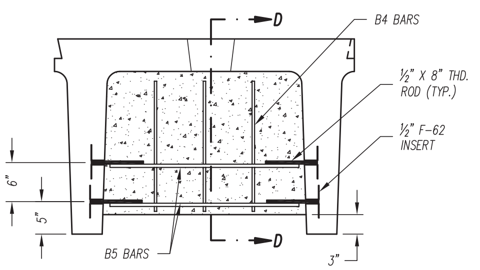
FORTERRA
Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57701
(605) 718-4111

SCALE: NONE	PROJECT: CASTING DETAILS PRO. MIRR17WA - STR. 41-019-032 LAWRENCE COUNTY, SD
DATE: 7/18/17	
OR#: 4417119BR1	
DR'N BY: LKB	CUSTOMER: JV BAILEY
CHK'D BY: –	DWG NAME: 4417119BR1-02

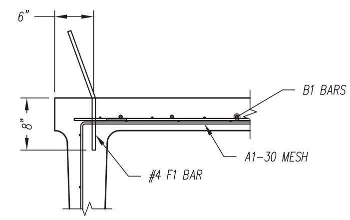
INTERMEDIATE DIAPHRAGMS ARE REQUIRED
ON DECK UNITS THAT ARE 45'-0" AND LONGER



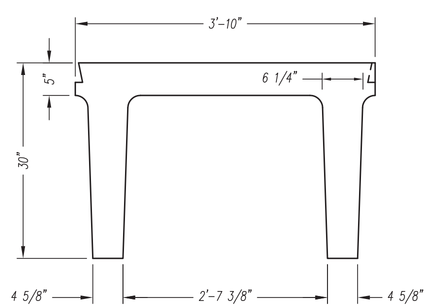
SECTION D-D



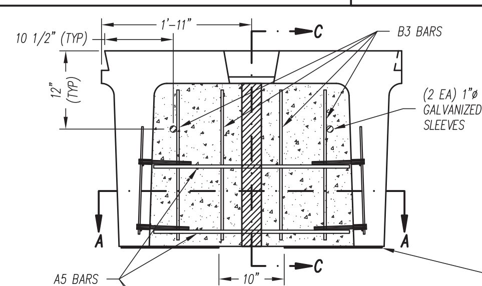
INTERMEDIATE DIAPHRAGM



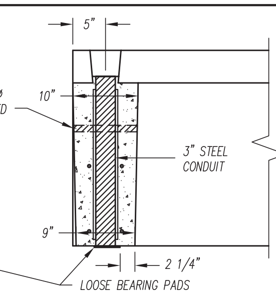
F1 BAR



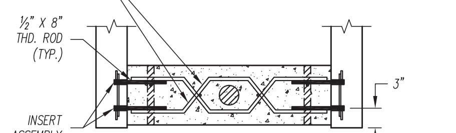
TYPICAL SECTION VIEW
30" DEEP PRESTRESSED CONCRETE CHANNEL



SECTION THRU DIAPHRAGM



SECTION C-C

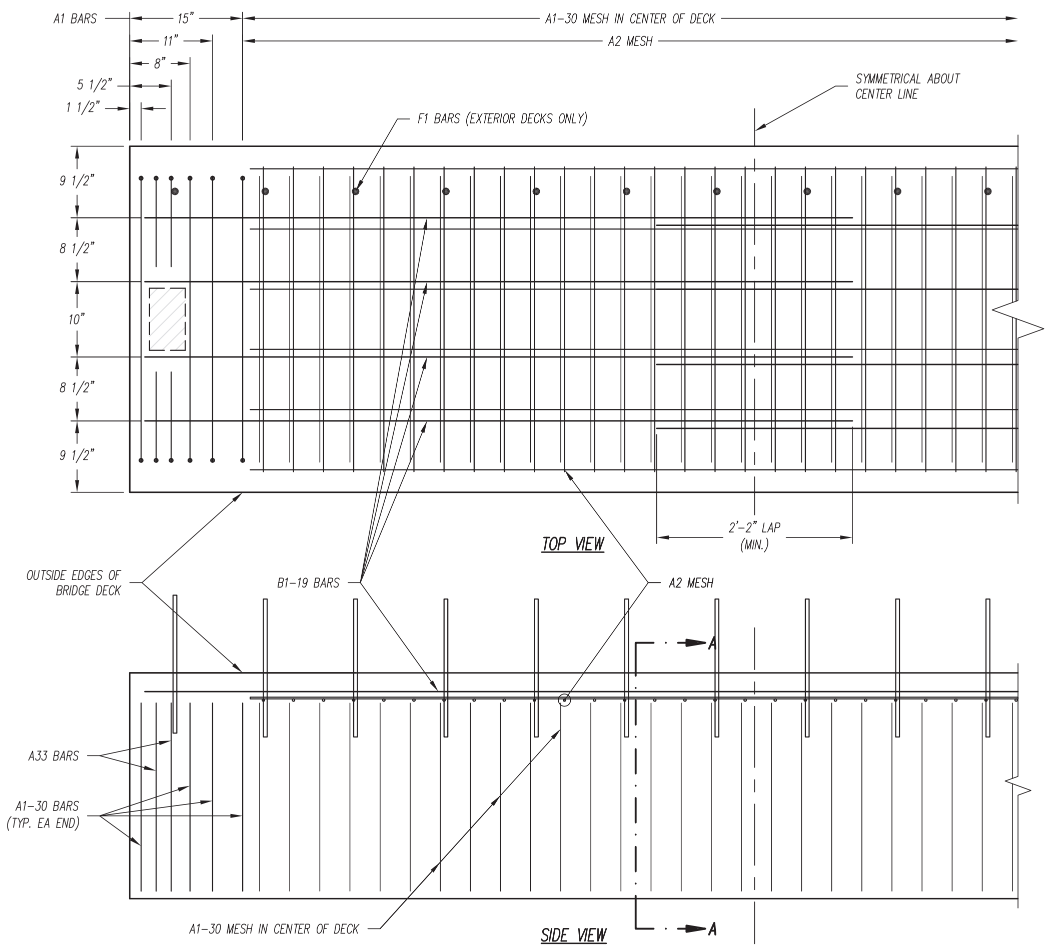


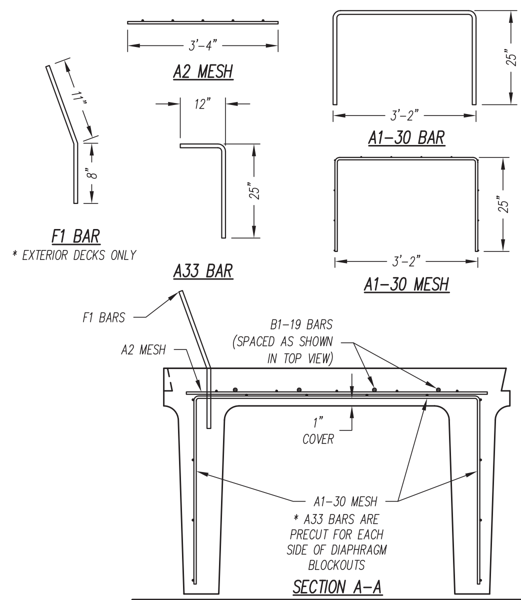
SECTION A-A

END DIAPHRAGM

FORTERRA	Rapid City, South Dakota 4310 Pendleton Drive Rapid City, SD 57701 (605) 718-4111
SCALE: NONE	PROJECT: RAIL POST & DIAPHRAGM DETAILS PRO. MIRR17WA - STR. 41-019-032 LAWRENCE COUNTY, SD
DATE: 7/18/17	
OR#: 4417119BR1	
DR'N BY: LKB	CUSTOMER: JV BAILEY
CHK'D BY: -	DWG NAME: 4417119BR1-03

MARK	# PER DECK	SIZE	LENGTH	SHAPE	TOTAL
A1-30 BAR	8	4	7'-4"		32
A1-30 MESH	8	4X8 MESH D8.0XD4.0	7'-4"		32
A2 MESH	8	4X8 MESH D8.0XD4.0	3'-4"	STR.	32
B1-20	12	4	20'-0"	STR.	48
B1-10	1	4	10'-0"	STR.	4
* A33	8	4	3'-1"		32
* F1 BAR	56	4	1'-7"		112

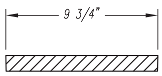




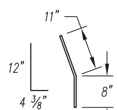
FORTERRA

Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57701
(605) 718-4111

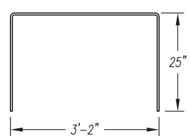
SCALE:	NONE	PROJECT:	STEEL CAGE LAYOUT PRO. MIRR17WA - STR. 41-019-032 LAWRENCE COUNTY, SD
DATE:	7/18/17		
OR#:	4417119BR1		
DR'N BY:	LKB	CUSTOMER:	JV BAILEY
CHK'D BY:	–	DWG NAME:	4417119BR1-04



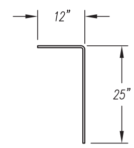
1"ø GALVANIZED SLEEVE



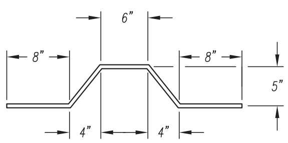
F1 BAR



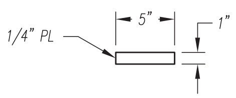
A1-30 BAR



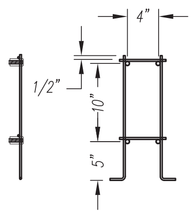
A33 BAR



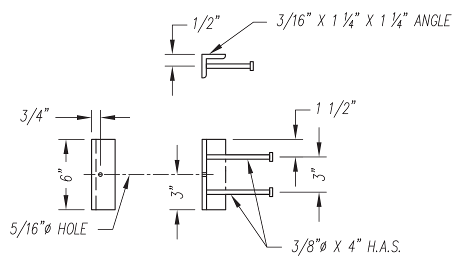
A5



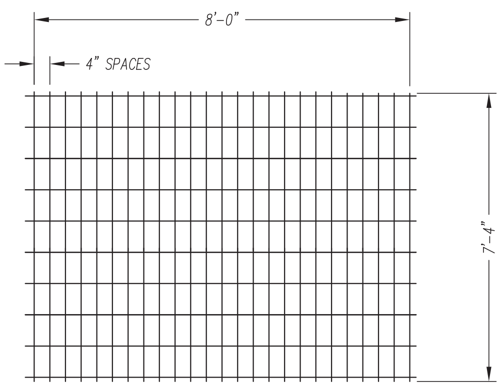
WELD PLATE



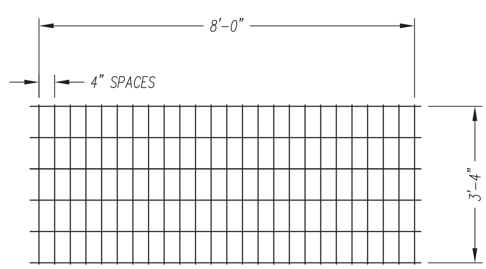
INSERT ASSEMBLY



ANGLE WELD TIE



A1-30 MESH



A2 MESH

NOTE:
1) REINFORCING STEEL DIMENSIONS ARE OUT TO OUT
2) REINFORCING STEEL AS PER ASTM 1-615 GRADE 60
3) STRUCTURAL STEEL AS PER ASTM A-36

MARK	# / DECK INT/EXT	SHAPE	SIZE	DIMENSION	TOTAL
60'-0" DOUBLE TEE					
A1-30 BAR	8		4	7'-4"	32
A1-30 MESH	8		8.0/4.0	7'-4" X 8'-0"	32
A2-MESH	8	STR.	8.0/4.0	3'-4"	32
A33	8		4	3'-1"	32
ANGLE WELD TIE	24 / 12				72
B1-20	12	STR.	4	20'-0"	48
B1-10	1	STR.	4	10'-0"	4
A-4400 HOLD DOWN	4				16
DIAPHRAGM					
INSERT ASSEMBLY	4				16
INSERT	4		½" F-62		16
THD. ROD	20		½" X 8"		80
A5	8		4	2'-10"	32
B3	8	STR.	3	1'-11"	32
B4	3	STR.	3	1'-8"	12
B5	2	STR.	4	2'-5"	8
CONDUIT	2		3" DIA.	26"	8
SLEEVE	4		1" DIA.	9 ¾"	16
CAST ON CURB					
F1	0 / 56	\	5	1'-7"	112
LOOSE ITEMS					
WELD PLATE	12		¼" PL	1" X 5"	36
BRG. PAD	4	70 DURO	½" X 4"	15"	16
DOWEL PINS	2	STR.	1"	29"	8
EXP. FILLER	2		⅜"	6" X 10"	8
POLY SEAL BOND BREAKER				50'	1 ROLL

* GALVANIZED PER ASTM A153
* BOLT PER ASTM A449

FORTERRA
Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57701
(605) 718-4111

SCALE: NONE	PROJECT: MATERIALS LIST PRO. MIRR17WA – STR. 41-019-032 LAWRENCE COUNTY, SD
DATE: 7/18/17	
OR#: 4417119BR1	
DR'N BY: LKB	CUSTOMER: JV BAILEY
CHK'D BY: –	DWG NAME: 4417119BR1-05

FORTERRA™

Concrete Mix Design Data for Mitchell

Mix Designation	Targets (psi)		Date
6000 PS	1 Day:	3000	1/26/2016
	28 Day:	6000	

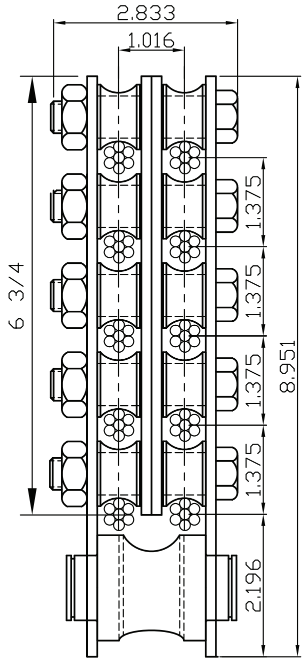
	Material	Cubic Yard Quantity		Specific Gravity	Cubic Yard Volume
Cement	Cement, Type I-II	700	lbs	3.150	3.56 ft³
Cementitious Materials					
Aggregates	Coarse Aggregate, 3/4" CA	1805	lbs	2.653	10.90 ft³
	Fine Aggregate, Washed Sand	1203	lbs	2.604	7.40 ft³
Chemical Admixtures	Air Entrainer	0.7	oz/CWT	1.050	4.9 oz
	HRWR SN	16.4	oz/CWT	1.200	114.8 oz
Water	Water	212	lbs	1.000	3.40 ft³
Air	Air Content, %	6.0%	+ / -	1.5% / 1.0%	1.62 ft³
				Total Volume	27.00 ft³
Batch Properties	Pozzolans, %	0%		Yd³ Weight	3929 lbs/yd³
	Total Cementitious	700	lbs	Unit Weight	145.53 lbs/ft³
	Water Cement Ratio*	0.31			
	Slump	6	in +/-	2	in

*Water Cement Ratio includes water from liquid admixtures

Material	Type/Classification	Supplier
Cement	Type I-II	GCC Dakota, Rapid City, SD
Coarse Aggregate	3/4" CA	Spencer Quarries Inc., Spencer, SD
Fine Aggregate	Washed Sand	Bitterman Sand Pit, Delmont. SD
Air Entrainer	Daravair M	WR Grace
HRWR SN	Daracem 19	WR Grace

Contact:

Name	Title	Phone	Email
John Kallemeyn	Materials Engineer	763-241-8274	jkallemeyn@cretex.com



NOTES:
1. MAX 1.5 KIPS (SWL) PER STRAND
2. USE 0.5" STRAND
3. 1 (3-1/2) COIL ROD

CAUTION: TOTAL STRAND SWL MUST NOT EXCEED UNIT SWL.

S.W.L.(kip)	QTY	PART #	PART NAME	DWG #
48	1	150947	SWIVEL	A-2652
12	2		SIDEFRAME .156" THICK	A-2678
	2		MIDDLE SIDEFRAME	A-2678
	2	151946	WALDES TRUARC SNAP RINGS	A-2806
	10	579886	SEAMLESS BUSHING	A-2646
	10	150646	ROLLER	A-2647
	5		GRADE 8, 1/2 X 2-1/2 BOLTS	
	5	150808	GRADE 8, 1/2 NUTS	

REVISION NO. | REVISION DATE | REVISION

TOLERANCES: UNLESS NOTED OTHERWISE

DECIMALS
.000 = ±.005
.00 = ±.010
.0 = ±.020

FRACTIONS
0 to 1 ± 1/64
over 1 to 12 ±1/32
over 12 ± 1/16

ANGLES
± 1/2°

APPROVED BY:/DATE: L. FERNANDEZ

DRAWN BY: \6000\6840-SH458 SPECIAL

COMPUTER NAME: 6-10-08

DATE

SCALE: 7/16

MeadowBurke
innovating concrete construction

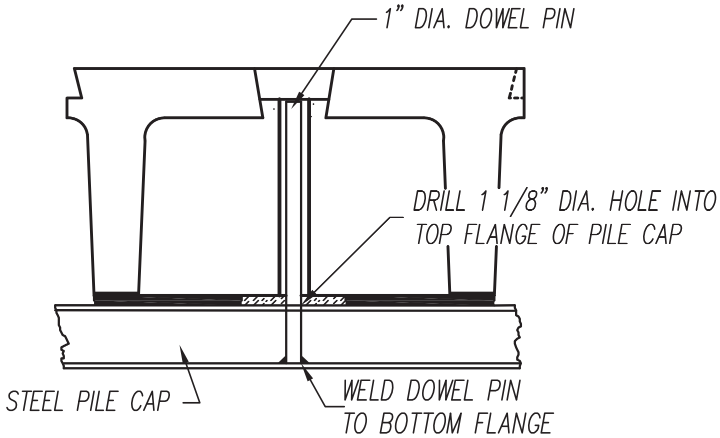
SPECIAL STRAND HOLD DOWN

TITLE

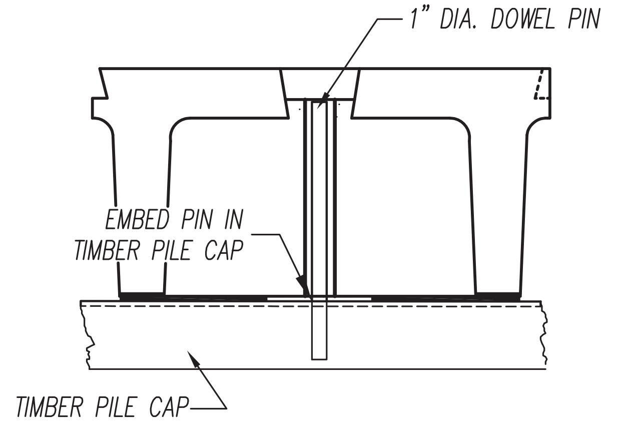
ITEM NO.: ----

PRODUCT SERIES: HOLD DOWNS

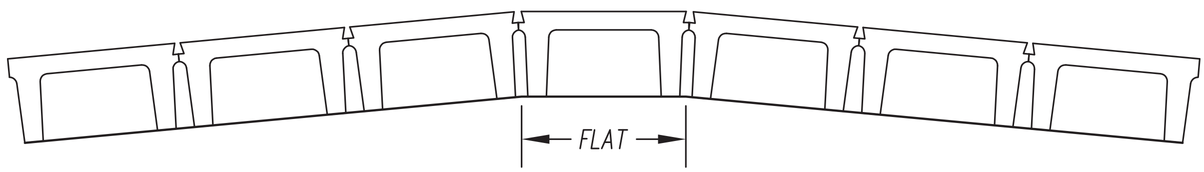
DRAWING NO.: A-4400



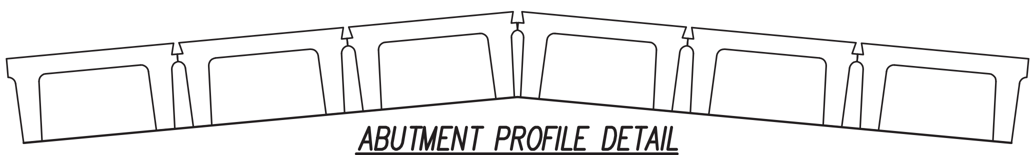
PRESTRESS DECK UNIT TO STEEL
ABUTMENT CONNECTION



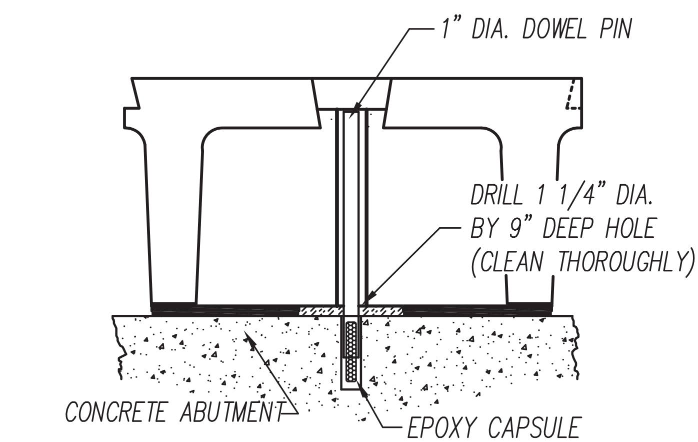
PRESTRESS DECK UNIT TO TIMBER
ABUTMENT CONNECTION



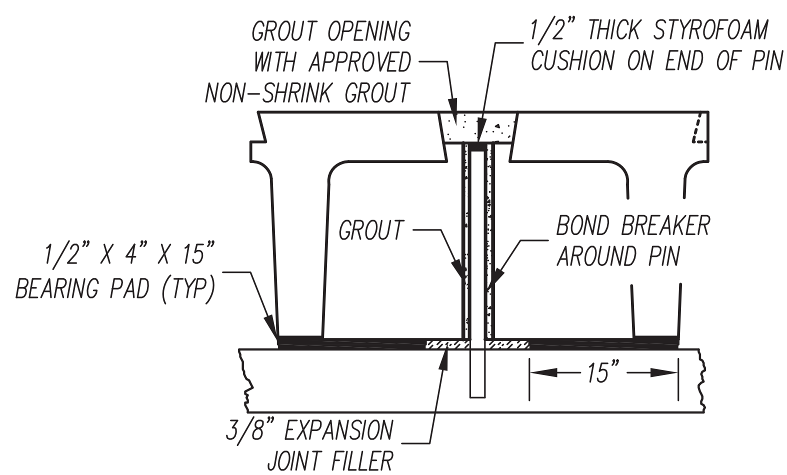
ABUTMENT PROFILE DETAIL
ODD#-UNITS WIDE - CROWNED SLOPE



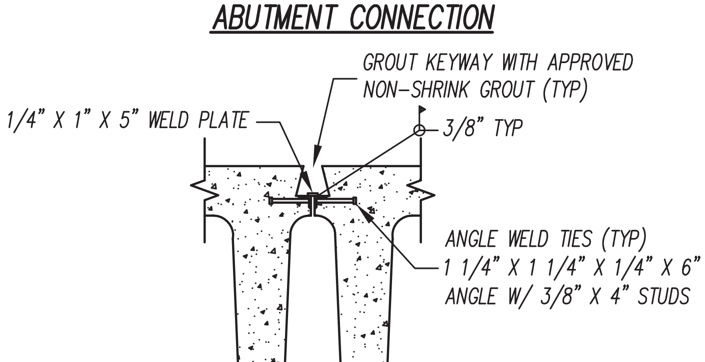
ABUTMENT PROFILE DETAIL
EVEN#-UNITS WIDE - CROWNED SLOPE



PRESTRESS DECK UNIT TO CONCRETE
ABUTMENT CONNECTION



BEARING AND GROUTING DETAILS

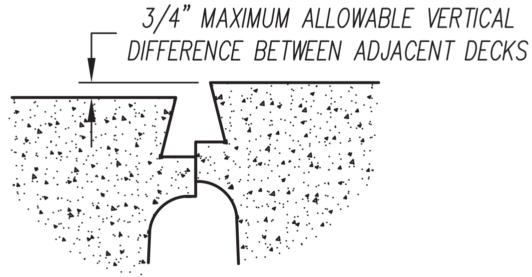


DTEE CONNECTION DETAIL

FORTERRA

Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: DOUBLE TEE DECK UNIT ABUTMENT CONNECTION DETAILS
DATE: 6/3/04	
DR'N BY: RTF	
REV: LKB	DWG NAME: DTEE CONNECTION (2020-10)

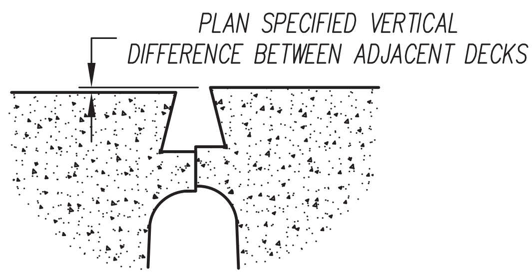


SEE NOTE A

<u>FABRICATION SPECIFICATION</u>

Producer Responsibility

A. *Maximum Allowable camber difference as specified in PCI Quality Control Manual Section 6.4.4 is limited to 1/4" per 10 ft. (3/4" max). This is a production tolerance only. This tolerance may not apply to beams that have a span–to–depth ratio approaching or exceeding 30.*



SEE NOTE B

<u>INSTALLATION SPECIFICATION</u>

Contractor Responsibility

B. *Installation tolerances are specified in the project plans. If the differential is more than allowed by plan notes, this tolerance can be achieved by the contractor by several methods including but not limited to the following: lifting the end of a deck and stitch welding at points along the deck to meet specifications, loading a deck unit to bring it into tolerance prior to welding, or in some situations shimming the end of the deck unit.*

Application of any method to install deck units to meet this specification is completely the Contractor's responsibility and needs to be approved by the manufacturer and the Project Engineer prior to performing the work.

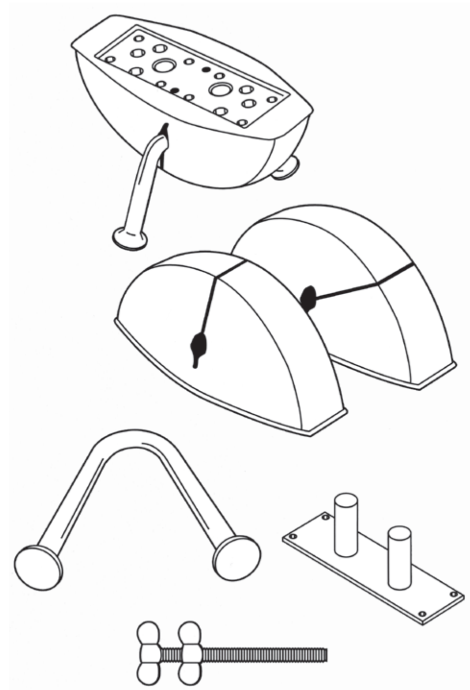
NOTE: These methods apply for both Double Tee Deck Units and Wide Flange Bulb Tee Deck Units.

FORTERRA

Rapid City, South Dakota
4310 Pendleton Drive
Rapid City, SD 57702
(605) 718-4111

SCALE: NONE	PROJECT: CONCRETE DECK UNIT INSTALLATION
DATE: 6/3/04	
DR'N BY: RTF	
REV: LKB	DWG NAME: DECK INSTALLTION (2010–35)

Utility Anchor System

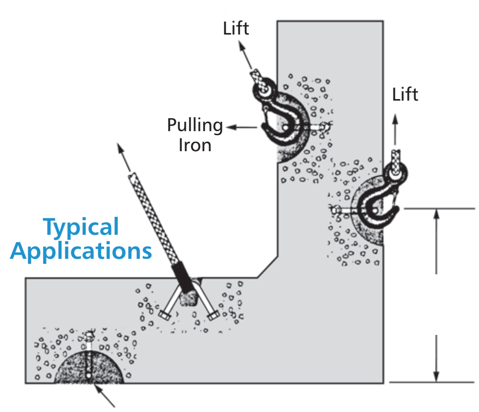


Key Advantages

Added Benefit

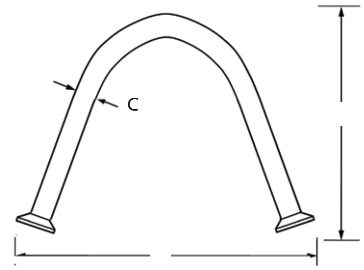
Utilize the Utility Anchor System to:

Anchor Placement

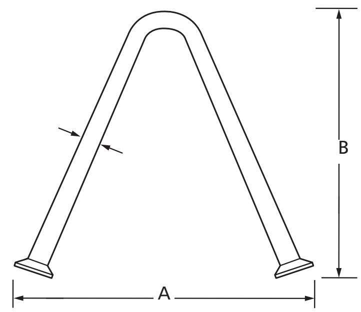


P75 and P75H Utility Anchor®

P75 and P75H Utility Anchor						
	4UA444	121877	5-1/4"	3-1/8"	0.444"	Swift Lift
	5UA444	123442	6"	3-3/4"	0.444"	Swift Lift
	5UA671	123441	6-7/16"	3-3/4"	0.671"	Swift Lift
	6UA671	121889	7-3/8"	4-3/4"	0.671"	Swift Lift
	8UA671	121891	9-3/4"	6-3/4"	0.671"	Swift Lift



P75 Utility Anchor



P75-H Utility Anchor

Anchor	Type	Product Code No.	Minimum Panel Thickness	Safe Working Load Tension 90	Safe Working Load Shear 90	Safe Working Load Tension/ Shear 45	Minimum Edge Distance
	5US444	123442	5"	3,860	7,710	2,730	10"
	5UA671	123441	5"	4,560	8,430	3,220	10"
	8UA671	121801	7 5/8"	10,830	18,850	7,660	16"

Note:

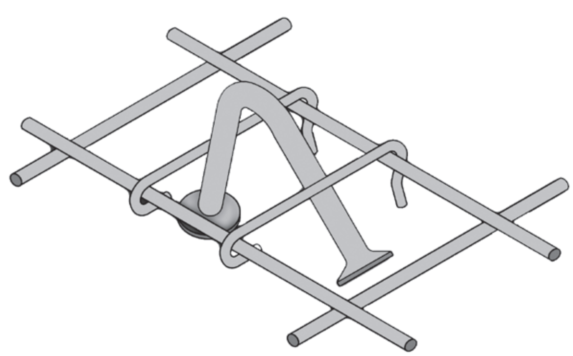
To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P75 Utility Anchors, 5UA444.

P75C Utility Anchor® with Clip

To Order:
Specify: (1) quantity, (2) name, (3) product code (4) anchor size, (5) wire spacing (6) wall thickness.

Example:
200, P75C, #121443, 5UA444anchor, 9" wire spacing, 5" wall.



Product Code	Utility Anchor	Wire Clip Lengths	Wall Thickness
121890	5UA671	9"	5"
121893	6UA671	9"	6"

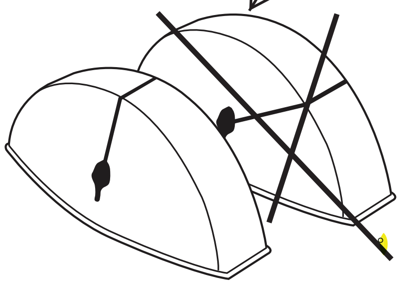
P76 Utility Anchor® Setting Plugs

NOT USED

P76 Utility Anchor Setting Plug					
Type	Product Code No.	Length	Width	Depth	Color
~~45P444~~	~~123176~~	~~8.00"~~	~~3.25"~~	~~3"~~	~~Blue~~
90P671	127786	9.00"	4.58"	3.35"	Orange
90P875	124685	15.00"	6.13"	5"	Blue

NOT USED

45° NOT USED



P76 Utility Anchor Setting Plugs

To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P76 Utility Anchor Setting Plugs, 90P444.

BLUE PLUG USED FOR UA444
ORANGE PLUG USED FOR UA671
LARGE BLUE PLUG USED FOR UA875

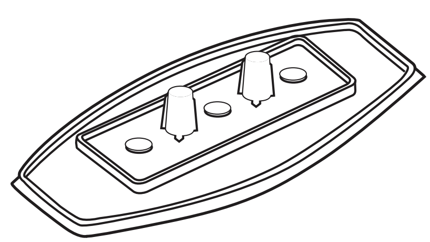
P76D Disposable Setting Plugs

P76D Disposable Utility Anchor Setting Plugs 0.671

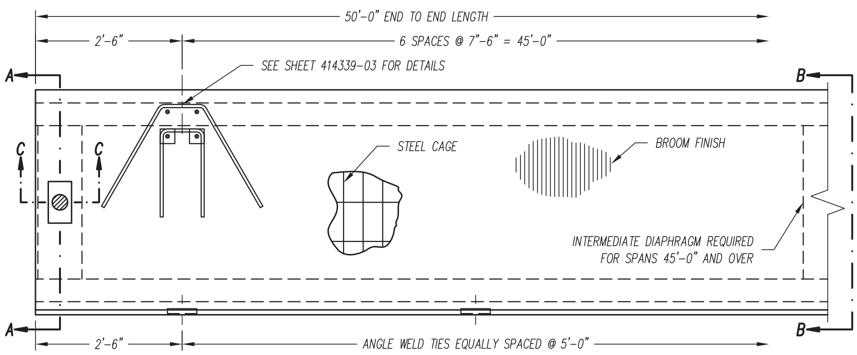
To Order:
Specify: (1) quantity, (2) name, (3) product code.

Example:
200, P76D, #126214.

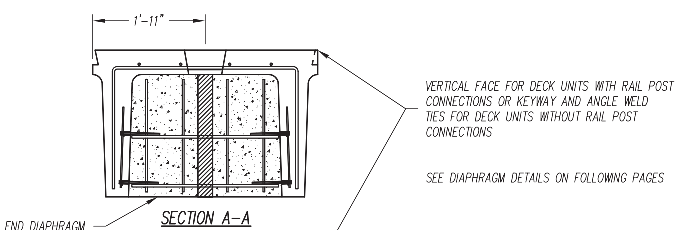
P76C Utility Anchor Cover/Patch



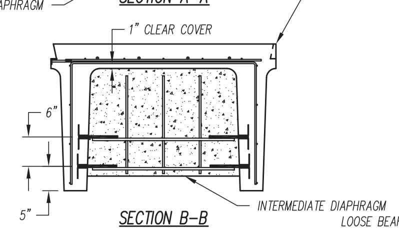
P76C Utility Anchor Cover/Patch



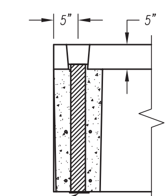
TOP VIEW



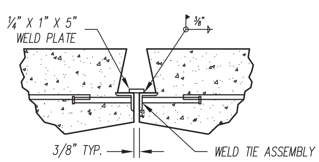
SECTION A-A



SECTION B-B



SECTION C-C

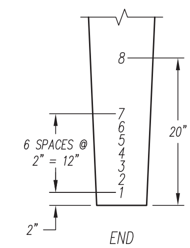


ANGLE WELD TIE CONNECTION DETAIL

STRESSING INFORMATION

STRAIGHT STRAND

GAUGE PRESSURE = 3165 ELONGATION = 10 13/16"



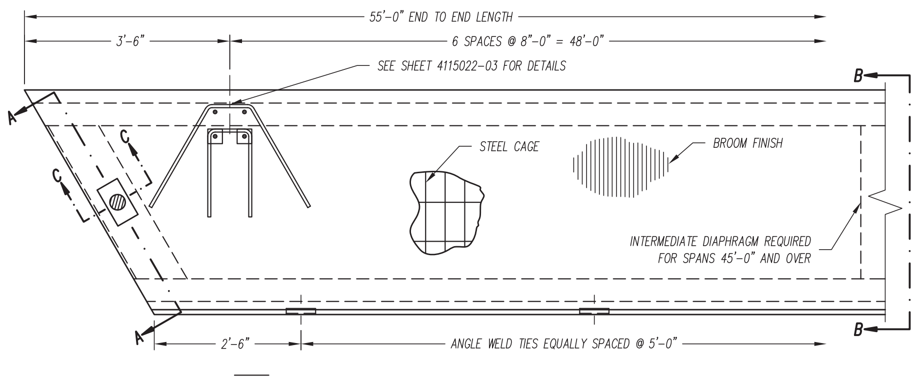
CONCRETE STRENGTH = 5000 AT DETENTION
6000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHIO M203, SUPPLEMENT 1 (LOW LAX STRAND)

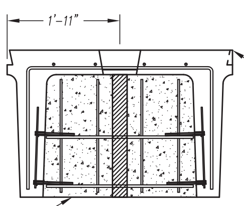
CHANGED STRAND DESIGN

Cretex Concrete Products — OFFICES IN: BISMARCK HELENA RAPID CITY

SCALE NONE	TITLE CASTING DETAILS
DATE 2/4/15	
DR'N BY JWB	
RS# 414260	CUSTOMER HOLLAWAY CONSTRUCTION CO.
REV DATE 7/14/15 JWB	DWG NAME 414260-02



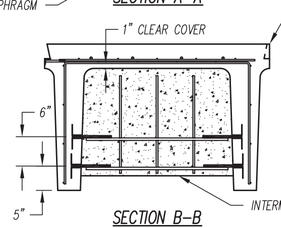
TOP VIEW



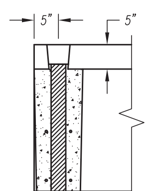
SECTION A-A

VERTICAL FACE FOR DECK UNITS WITH RAIL POST CONNECTIONS OR KEYWAY AND ANGLE WELD TIES FOR DECK UNITS WITHOUT RAIL POST CONNECTIONS

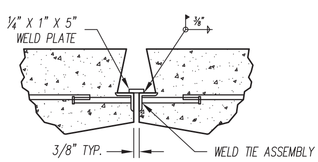
SEE DIAPHRAGM DETAILS ON FOLLOWING PAGES



SECTION B-B



SECTION C-C

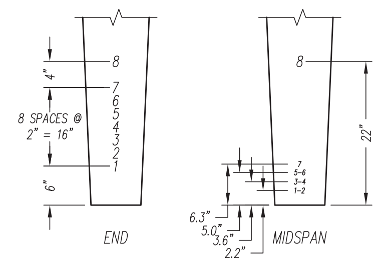


ANGLE WELD TIE CONNECTION DETAIL

STRESSING INFORMATION

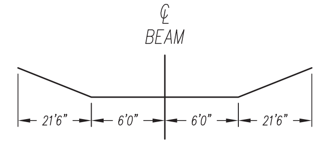
STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



CONCRETE STRENGTH = 5000 AT DETENTION
6000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHIO M203, SUPPLEMENT 1 (LOW LAX STRAND)

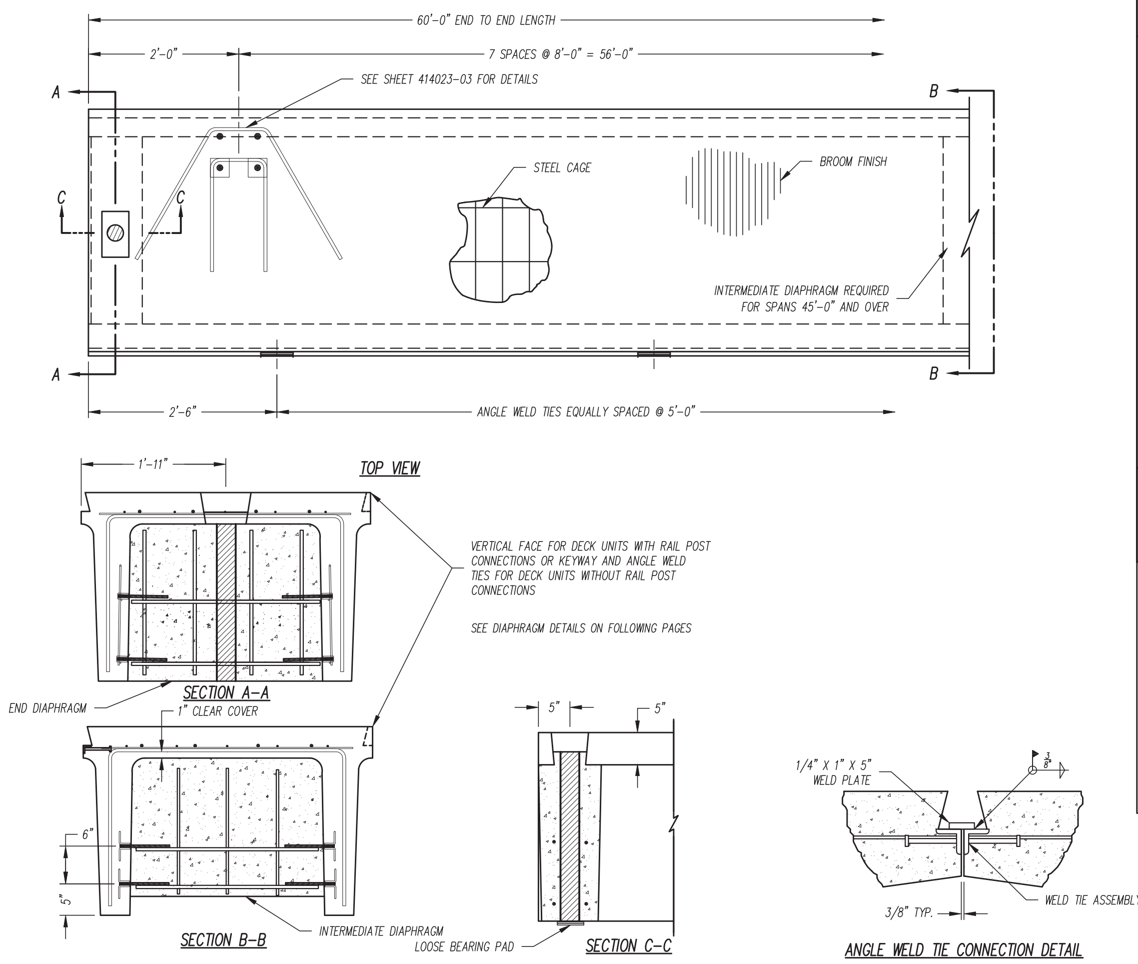


HOLD-DOWN DETAIL

Cretex Concrete Products

OFFICES IN: BISMARCK HELENA RAPID CITY

SCALE NONE	TITLE		
DATE 02/12/15	CASTING DETAILS		
DR'N BY JWB			
RS# 414261		CUSTOMER	HOLLAWAY CONSTRUCTION
REV DATE –		DWG NAME	414261-02



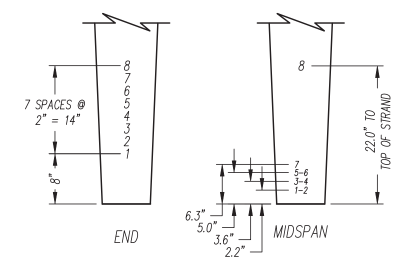
STRESSING INFORMATION

DRAPED STRAND

GAUGE PRESSURE = 2979 ELONGATION = 10"

STRAIGHT STRAND

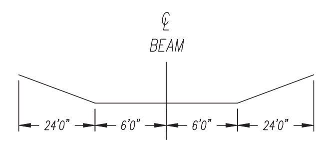
GAUGE PRESSURE = 3164 ELONGATION = 10 3/4"



STRAND PATTERN

CONCRETE STRENGTH = 5000 AT DETENTION
6000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHTO M203, SUPPLEMENT 1 (LOW LAX STRAND)

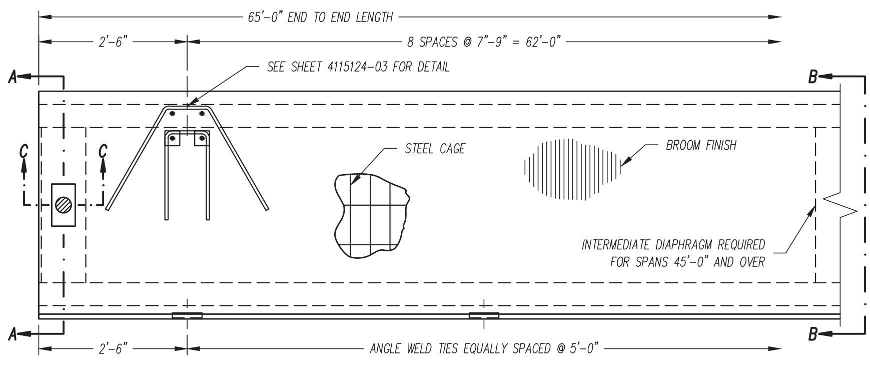


HOLD-DOWN DETAIL

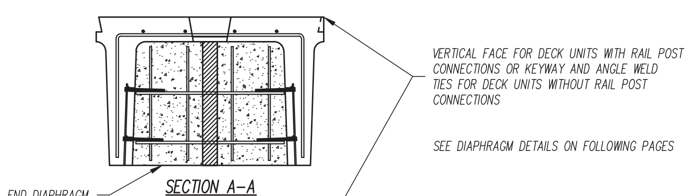
Cretex Concrete Products

OFFICES IN: BISMARCK HELENA RAPID CITY

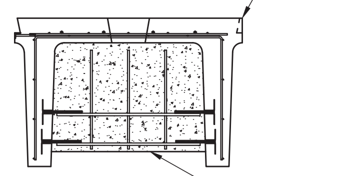
SCALE none	TITLE CASTING DETAILS		
DATE 1/17/14			
DR'N BY REM			
RS# 414023	CUSTOMER BROWN COUNTY		
REV DATE none	DWG NAME 414023-02		



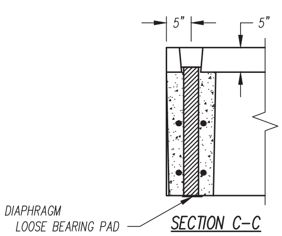
TOP VIEW



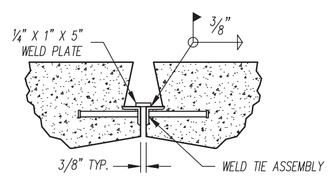
SECTION A-A



SECTION B-B



SECTION C-C

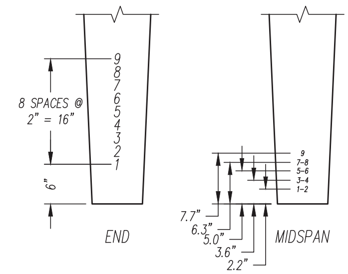


ANGLE WELD TIE CONNECTION DETAIL

STRESSING INFORMATION

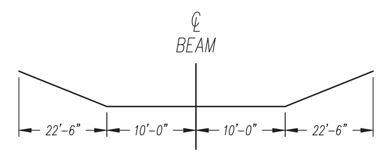
STRAND STRESSING

GAUGE PRESSURE = 3165 ELONGATION = 10 ¾"



CONCRETE STRENGTH = 5,500 AT DETENTION
6,000 AT 28 DAYS

PRESTRESSING STRAND CONFORMS TO AASHIO M203, SUPPLEMENT 1 (LOW LAX STRAND)



HOLD-DOWN DETAIL

Cretex Concrete Products

OFFICES IN: BISMARCK HELENA RAPID CITY

SCALE NONE	TITLE CASTING DETAILS		
DATE 05/22/15			
DR'N BY JWB			
RS# 4115124		CUSTOMER	HOLLOWAY CONSTRUCTION
REV DATE –		DWG NAME	4115124-02