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Abstract

Post-tensioned box girder bridges are very common form of bridge construction. The post-tensioning anchorage zone is the location where very large prestressing forces are applied to the box girder, and then spread into the box section. The spreading of the large compressive forces creates transverse tension forces that must be considered in design. The diaphragm, web, deck and soffit adjacent to prestressing anchors are affected by force spreading and included in the post-tensioning general anchorage zone. These parts need to have adequate reinforcement and proper concrete placement. As part of this study, a database of Caltrans bridges was developed, which included 29 anchorage zones of box girder bridges. Based on this database, it was concluded that the diaphragm reinforcement selection varies substantially between bridges, and seems to be based more on “rules of thumb”. Lack of an accurate methodology of design and detailing for anchorage zone has led to highly congested anchorage zones with construction issues and cracking problems.

In order to study the performance of anchorage zones, four box girders end zones were instrumented in the field in California. These four bridges cover a wide variety of anchorage zone configuration including different diaphragm width, number of girders, box girder height and openings in the end diaphragm. Strain gauges were used in order to capture strains in reinforcing bars and within the concrete elements. End diaphragm cracking was observed during post-tensioning for the investigated bridges. Through the captured strains, the flow of forces was estimated as well as the different parameters affecting force spreading in the general anchorage zone.
Experimental work included two phases. The first phase included two single half-scale I-section girders with rectangular solid end diaphragms. The main parameter investigated in these specimens was the diaphragm width. The second phase contained two double girder half-scale box section with different openings in the end diaphragms as well as one solid diaphragm. Loading was applied with post-tensioning tendons to represent different design levels as well as to reach the ultimate load of the anchorage zone. Increasing of diaphragm width reduces the effect of bursting forces developed in different directions of the general anchorage zones. Effect of diaphragm openings were studied. Anchorage zone failure occurred in the double girders with openings in the end diaphragm.

Finite element models were developed for the experimental specimens using the DIANA finite element package. Results of the developed models showed good correlation with the experimental results of tested specimens. These models were used to extend the investigated parameters affecting performance of general anchorage zone including: geometry of box girder end zone, edge eccentricity, number of anchors, tendon inclination and skew angle.

Results obtained from field monitoring of bridges, experimental work and finite element modeling were combined using the strut-and-tie methodology to develop a set of design equations for bursting forces of anchorage zone. A simplified design table is developed based on the proposed equations. It provides percentage of bursting forces to ultimate jacking force based on effective diaphragm width to box girder height ratio as well as girder spacing to box girder height ratio. The proposed equations and simplified design table provide adequate procedures to design general anchorage zone of box girder bridges.
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Notation

\( A_{\text{ducts}} \) = total area of prestressing ducts (in.\(^2\))

\( A_g \) = gross area of box section (in.\(^2\))

\( A_{g\text{-web}} \) = gross area of the web section (in.\(^2\))

\( A_s \) = amount of reinforcement (in.\(^2\))

\( A_{s\text{-web}} \) = total area of longitudinal reinforcement included in the web (in.\(^2\))

\( a \) = the vertical dimension of the anchorage device (in.)

\( a_d \) = the vertical dimension of the anchorage device affecting the deck (in.)

\( a_s \) = the vertical dimension of the anchorage device affecting the soffit (in.)

\( a_w \) = the vertical dimension of the anchorage device affecting the web (in.)

\( a' \) = the distance between region centroid of anchorage device affecting the deck to region centroid of anchorage device affecting the soffit (in.)

\( b \) = the lateral dimension of the anchorage device (in.)

\( b_d \) = diaphragm width (in.)

\( b_{de} \) = effective diaphragm width (in.)

\( b_w \) = web girder width (in.)

\( d_{\text{burst}} \) = distance from anchorage device to the centroid of the bursting force, \( T_{\text{burst}} \) (in.)

\( E_s \) = reinforcing steel modulus of elasticity (ksi)

\( E_{\text{PC}} \) = Equivalent prestressing contribution ratio

\( E_{\text{PC} \text{Exterior}} \) = Equivalent prestressing contribution ratio for exterior girders

\( E_{\text{PC} \text{Interior}} \) = Equivalent prestressing contribution ratio for interior girders

\( e \) = base of Napierian logarithms

\( e_{\text{edge}} \) = Tendon profile edge eccentricity (in.)

\( F_{\text{deck}} \) = axial force acting on the deck due to prestressing at distance \((h/4+S/2)/2\) from interior diaphragm face (kip)
\[ F_{\text{soffit}} = \text{axial force acting on the soffit due to prestressing at distance (h/4+S/2)/2 from interior diaphragm face (kip)} \]

\[ F_{\text{web}} = \text{axial force acting on the web due to prestressing at distance (h/4+S/2)/2 from interior diaphragm face (kip)} \]

\[ F_y = \text{reinforcing steel yield force (kip)} \]

\[ f_{\text{cu}} = \text{concrete cube compressive strength (ksi)} \]

\[ f_{pj} = \text{stress in the prestressing steel at jacking (ksi)} \]

\[ f_{sp} = \text{concrete splitting tensile strength (ksi)} \]

\[ f_y = \text{reinforcing steel yield strength (ksi)} \]

\[ f_{y \text{ act.}} = \text{actual yield strength of reinforcement based on reinforcement testing (ksi)} \]

\[ f'_{c} = \text{concrete compressive strength (ksi)} \]

\[ f'_{ci} = \text{initial concrete compressive strength at time of prestressing (ksi)} \]

\[ f'_{c \text{ act.}} = \text{actual concrete compressive strength at test day (ksi)} \]

\[ H_o = \text{Diaphragm opening height (in.)} \]

\[ h = \text{box girder height (in.)} \]

\[ h' = \text{the distance between the deck centroid and the soffit centroid (in.)} \]

\[ K = \text{wobble friction coefficient (per ft of tendon)} \]

\[ n = \text{number of box section girders} \]

\[ P_{\text{du}} = \text{design factored jacking force per girder (kip)} \]

\[ P_J = \text{prestressing jacking force (kip)} \]

\[ P_{J \text{ Exterior}} = \text{prestressing jacking force for exterior girders (kip)} \]

\[ P_{J \text{ Interior}} = \text{prestressing jacking force for interior girders (kip)} \]

\[ P_{Ju} = \text{ultimate prestressing jacking force (kip)} \]

\[ P_{n\text{-web}} = \text{nominal axial resistance of the web girder (kip)} \]

\[ r^2 = \text{coefficient of determination} \]
S = spacing between girders (in.)

$T_{\text{burst}}$ = tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis (kip)

$T_{\text{deck}}$ = tensile bursting force in the anchorage zone acting in the deck in transverse direction at S/2 from the diaphragm inner face (kip)

$T_{\text{diaph. transverse}}$ = tensile bursting force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis on the diaphragm interior face in transverse direction (kip)

$T_{\text{diaph. vertical}}$ = tensile bursting force in the anchorage zone acting ahead of the anchorage device on the diaphragm inner face in vertical direction (kip)

$T_{\text{dv-acute}}$ = vertical tensile bursting force in the anchorage zone acting ahead of the anchorage device on the diaphragm inner face in the acute side in case of skew diaphragm (kip)

$T_{\text{dv-obtuse}}$ = vertical tensile bursting force in the anchorage zone acting ahead of the anchorage device on the diaphragm inner face in the obtuse side in case of skew diaphragm (kip)

$T_{\text{soffit}}$ = tensile bursting force in the anchorage zone acting in the soffit in transverse direction at S/2 from the diaphragm inner face (kip)

$T_{\text{web}}$ = tensile bursting force in the anchorage zone acting in the web in vertical direction at h/4 from the diaphragm inner face (kip)

$t_{\text{bo}}$ = prestressing block-out depth measured parallel to tendon axis (in.)

$t_d$ = deck thickness (in.)

$t_s$ = soffit thickness (in.)

$W_o$ = diaphragm opening width (in)

$x$ = length of a prestressing tendon from the jacking end to any point under consideration (ft)

$Y_{\text{c.g.}}$ = distance from section base to centroid (in.)
\[ y = \text{transverse bursting forces ordinate at inner diaphragm face (in.)} \]

\[ \alpha = \text{sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad.)} \]

\[ \alpha_{\text{inc.}} = \text{tendon inclination angle (degree)} \]

\[ \alpha_{\text{skew}} = \text{skew angle of diaphragm (degree)} \]

\[ \beta_{\text{end}} = \text{amplification factor due to stressing both bridge ends} \]

\[ \Delta f_{pF} = \text{loss due to friction (ksi)} \]

\[ \mu = \text{coefficient of friction} \]
Chapter (1) – Introduction

1.1 General

Construction issues and cracking problems have occurred in anchorage zones of box girder bridges. These local problems affect the global performance of the box girder. Current design codes do not provide a clear method for design of anchorage zones at the end diaphragm. Available design equations can be used only for rectangular sections. In the case of a box girder, the cross section changes from a rectangular section through the diaphragm to an irregular shape at the webs of the box girder.

The anchorage zone for any prestressed member consists of two regions, local zone and general zone. The general anchorage zone consists of the diaphragm, web, deck and soffit adjacent to prestressing anchors; these regions need to have adequate reinforcement and proper concrete placement. Sufficient reinforcement must be provided to handle the spreading of forces in the general zone of the end anchorage. Adequate dimensioning and detailing of box girder end is critical for proper performance of post-tensioned bridges. Current typical reinforcement details have led to highly congested anchorage zones with construction issues and cracking problems.

1.2 Problem Statement

Recently, California Department of Transportation (Caltrans) implemented anchorage zone design procedures in Memo to Designers 11-25 Anchorage Zone Design [1]. These procedures included minimum widths for the diaphragms and web girders as well as minimum vertical reinforcement in the webs. These recommendations led to an increase in the diaphragm width and web flares. No detailed design procedures were
presented. None of the available design codes or methodologies considers the cross section changes from a wide rectangular section through the diaphragm to an I-shape section at the webs of the box girder.

Case studies of bridges constructed in California have shown significant cracking due to improper dimensioning and detailing of the anchorage zones. For example, a box girder bridge with 11.5’ depth had significant cracking in the deck and the girders due to post-tensioning. This problem was evaluated afterwards by Richmond [2] using detailed finite element analysis. The recommendation of that analysis was to increase the widths of end diaphragm and web flares. Also, several bridges were subjected to crack propagation out of prestressing block-outs.

Anchorage zones need to have proper reinforcement and proper concrete placement. Realistic models are needed that provide sufficient safety margins and reduce congestion. Placement of concrete is critical and the reduction of congestion will improve the chances of having high quality concrete in the anchorage zone.

1.3 Research Scope, Objectives and Methodology

The main objectives of this study were to understand the behavior of anchorage zones located in end diaphragms and in-span hinges, and to develop a set of design equations for anchorage zone of box girder bridges. The methodology to achieve the main objectives of the research can be summarized as follows:

- Develop bridge database to determine dimensions and detailing of an average prototype specimen.
• Perform field investigation to measure the actual flow of strains in the general anchorage zone.

• Execute experimental work to assess main parameters affecting performance of anchorage zones.

• Develop a verified finite element model to extend the investigated parameters affecting performance of general anchorage zone.

• Combine the results to develop a set of design equations for the anchorage zone bursting forces.

1.4 Dissertation Outline

The dissertation outline is divided into the following chapters:

Chapter (1) is an introduction to this research discussing the problem statement, and scope, methodology and objectives of the research program.

Chapter (2) summarizes previous and recent research work done to study and simulate behavior of anchorage zones. The available design methods of anchorage zones are presented as well as a comparison between different design codes and guidelines is conducted.

Chapter (3) presents a database of Caltrans bridges, which included 29 anchorage zones of box girder bridges. The data was analyzed to determine dimensions and detailing of an average prototype specimen. Obtaining the scaled dimensions and the average reinforcement ratios for experimental specimens was done through this database. The developed bridge database was used as well to choose a set of convenient exemplary bridges for field investigation.
Chapter (4) presents configurations of the monitored bridges. It includes the developed protocol for field monitoring including preliminary finite element analysis as well as instrumentation methodology. Field monitoring results are provided including comparisons between investigated bridges. A description of the end diaphragm cracking due to post-tensioning along with potential reasons for the developed cracks are illustrated.

Chapter (5) presents different parameters affecting the general anchorage zone performance based on field monitoring results. Effect of stressing sequence is investigated. Three different methods are presented for distributing jacking force between different girders. The effect of stressing one end on the bursting forces developed at the other end is discussed.

Chapter (6) presents experimental program, which included two phases. Phase I had two single I-section girders with different end diaphragm configurations at each end. While Phase II had two double girder specimens to investigate the effect of openings in the end diaphragm. This chapter illustrates experimental specimens development comprising: study of specimen shape, effect of specimen length and specimen scaling. Test matrix, instrumentation, test setup and loading protocol are presented.

Chapter (7) presents experimental results including: crack patterns, strain results and deformations. Crack widths were measured at different loading stages to compare between specimens. Strain distributions along instrumented reinforcement are presented. The adequacy of local zone performance was calibrated. Failure modes and reasons are illustrated for the damaged double girder specimens.
Chapter (8) presents finite element model for the general anchorage zone including: modeling criteria, model verification, development of prototype and parametric study to widen the investigated factors affecting anchorage zone. Parametric study was conducted to investigate the parameters that were not included through the field investigation or the experimental work.

Chapter (9) presents a strut-and-tie methodology based on the field, experimental and finite element results to develop a set of design equations for bursting forces of anchorage zone. These forces include transverse forces in the deck, soffit and diaphragm inner face and vertical forces in the web and diaphragm inner face. Safety check is proposed based on the observed failure mode for experimental specimens, where web axial capacity shall be able to maintain the whole prestressing force during post-tensioning.

Chapter (10) presents the summary and the conclusions of the research, in addition to recommendations for future studies and research topics relevant to the subject.
Chapter (2) – Literature Review

2.1 Introduction

Post-tensioning anchorage zones are a critical part in prestressed box girder bridges. Construction issues and cracking problems occurred in this zone affect the global performance of box girder bridges. Current design codes do not provide a clear method for design of anchorage zones at the end diaphragm. Available design equations can be used only for rectangular sections. In the case of a box girder, the cross section changes from a rectangular section through the diaphragm to an irregular shape at the webs of the box girder.

This chapter summarizes previous and recent research work done to study and simulate behavior of anchorage zones. Different mathematical and finite element models were proposed to analyze the bursting forces and the distribution of transverse stresses in the anchorage zone. Also, the available design methods of anchorage zones are presented as well as a comparison between different design codes and guidelines is conducted. The presented literature enhanced the research objectives and procedures.

2.2 Previous and Current Research Work

The anchorage zone is defined as the volume of concrete through which the concentrated prestressing force at the anchorage device spreads transversely to a more linear stress distribution across the entire cross-section at some distance from the anchorage device. It consists of two regions: general anchorage zone and local anchorage zone. The region subjected to tensile stresses due to spreading of the tendon force into the structure is the general zone, however, the region of high compressive stresses immediately ahead
of the anchorage device is the local zone [3]. The principle tensile stresses at the general zone and the principle compressive stresses surrounding the local zone are shown in Figure 2-1.

Analytical methods for plastic analysis of anchorage zone was conducted by Ibell and Burgoyne [4]. They were able to accurately predict behavior and strength of concrete prisms, based upon a single failure criterion for concrete. Fenwick and Lee [5] investigated the anchorage zone tensile stresses and discussed the way in which they arise using experimental work and finite analyses; their results showed that once a bursting crack forms the resultant loss of transverse stiffness in the member leads to a decrease in the magnitude of bursting force and a change in its location. But none of them investigated an anchorage zone with variable cross-section as the end diaphragm of box girder bridges.

Rowe [6] developed an empirical formula for calculating bursting forces. His work was conducted based on theoretical analyses and experimental work relevant to the problem of the stress distribution in an end anchorage zone. The present state of practice for the design of bearing zones is mixed between solutions based on empirical equations,
linear-elastic stress analyses and strut-and-tie models. If, however, the design requires large redistribution of stresses, the end zone may perform poorly at service and/or overload. Foster and Rogowsky [7] illustrated a plane stress finite-element investigation of concentrated loads on reinforced concrete panels. The numerical investigation suggested that the bursting force distribution is substantially different from that calculated using elastic design methods currently used in some codes of practice. The distribution of bursting stresses were found to be flatter and occur over a longer disturbed region than is predicted by the linear solution.

Anchorage zone transverse stresses for prestressed members of uniform cross-section were determined using elastic finite element procedure for three-dimensional solids by Yettram and Robbins [8]. Stress distributions were obtained for multiple and eccentric anchorages on rectangular and I-section beams. Both bursting and spalling stresses were considered.

In the early 1990s, extensive study of design requirements for anchorage zone reinforcement for post-tensioned concrete girders was carried out at the University of Texas at Austin. Dozens of experimental tests of anchorage zones were investigated by Breen et al. [9] in 1994. This was the last substantial experimental research on post-tensioned concrete anchorage zones; it was done as part of the National Cooperative Highway Research Program (NCHRP) 10-29. Based on the results of these tests and the finite element analyses [10] [11], the bursting force equation for a typical rectangular anchorage zone was developed, which has been adopted by AASHTO LRFD Bridge Design Specifications [3] since 1994. Through the same research project, three half-scale
specimen modeling a diaphragm for the anchorage zone of external tendons of a box girder bridge were tested experimentally. The prestressing forces were not applied directly to the web, as the tendons were external. The main performance of the anchorage zone was bending of the diaphragm, which differs in case of using tendon implemented in the web girders. Finite element analysis and strut-and-tie models were developed to investigate the performance of these specimens. Both finite element analysis and strut-and-tie model prediction indicated that failure should be controlled by the capacity of the diaphragm bending and the web bursting reinforcement (vertical tension reinforcement). These predictions were very conservative and inconsistent for several reasons. For one, the contribution of uncracked concrete in the massive diaphragm specimen is very significant. In addition, reinforcement distortions noticed in the specimens after completion of the tests indicated substantial dowel action between diaphragm and flanges. Both contributions are difficult to assess and are not normally relied on in design.

Anchorage zones in prestressed concrete I-beams are designed to accommodate anchorage hardware and to provide adequate space for the reinforcement needed to distribute the highly concentrated post-tensioning force. Based on analytical and full-scale experimental studies, optimized anchorage zone details have been developed by Ma et al. [12]. This proposed standardized anchorage zone is suitable for use with a post-tensioning of I-beams without end diaphragm as in case of box girder bridges.

An updated mathematical model has been proposed to analyze the bursting forces and the distribution of transverse stresses in the anchorage zone by Zhou et al. [13]. They used iso-static lines or stress trajectories to obtain transverse stresses in end anchorages as
shown in Figure 2-2, but their methodology was applied only on rectangular sections. The bursting forces derived by this updated analytical model was the same as the formula given by the current AASHTO LRFD Bridge Design Specifications [3].

![Diagram of compressive forces](image)

- a) Iso-static lines of compression

- (b) Distribution of transverse stresses along the tendon axis

Figure 2-2: Dispersion of compression in an anchorage zone of rectangular section [13].

One of the innovative techniques of enhancing the performance of anchorage zones was implementing new materials like steel fiber reinforced concrete. This was investigated by Robinson et al. [14] to enhance the performance of the anchorage zone and to minimize the amount of mild steel reinforcement required by the code. The basic steel fiber reinforced concrete properties were obtained and then used in a thorough finite element analysis on 3D models of concrete blocks representing the anchorage zone. Results showed that the addition of steel fibers improved the loading capacity of the anchorage blocks.
However, the wide variety of available literature including linear as well as non-linear analysis of the anchorages zones, none of these researches considered variations of cross-section at the end anchorage. Most of these literature results cannot be wisely applied in case of anchorage zone for box girders. In the case of box girder, the cross section changes from a wide rectangular section through the diaphragm to an I-Shape section at the webs of the box girder.

2.3 Available Design Methods

Current design methods lead to highly congested anchorage zones. Several design methods are available for design and detailing of anchorage zones. These design methods include approximate design equations, elastic analysis method, modeling using elastic finite element analysis and strut-and-tie technique. None of these methods has a direct procedures that can be applied for the end anchorage of box girders.

2.3.1 Approximate design equations

Different design codes and guidelines can be used for design of anchorage zones. British Standard BS8110 [15] as well as CEP-FIP 1990 [16] are applicable for the design of end anchorages with rectangular cross sections only. AASHTO LRFD Bridge Design Specifications [3] and ACI 318-14 Building Code Requirements for Structural Concrete [17] possess a set of approximate equations for design of anchorage zone with rectangular cross-section, although, when the end anchorage is as complex as in the box girder, they refer to strut-and-tie method or finite element analysis. Both methods cannot be simply applied, as they need many iterations and verification.
Recently, the California Department of Transportation (Caltrans) implemented an anchorage zone design procedure in Memo to Designers 11-25 Anchorage Zone Design [1]. These procedures included minimum thicknesses for the end diaphragm and the web girder as well as minimum vertical reinforcement in girder webs. On the other hand, no detailed design procedures were presented.

A comparison was conducted between the following design codes:

- AASHTO LRFD Bridge Design Specifications [3].
- Caltrans Anchorage Zone Design - Memo to Designers 11-25 [1].
- ACI-318, Building Code Requirements for Structural Concrete [17].
- CEB-FIP Model Code 1990, Structural Concrete - Volume 1 [16].
- EURO Code 2: Design of concrete structures [18].

The results of this comparison are summarized in Table 2-1. It includes the input data required to apply the design codes equations and the output of these equations from bursting forces, spalling forces or bearing stress. Finally, limits introduced by Caltrans in Memo to Designers 11-25 [1] are compared to the remaining design codes.
Table 2-1 Comparison between design codes.

<table>
<thead>
<tr>
<th>Design Codes</th>
<th>BS 8110</th>
<th>AASHTO</th>
<th>CALTRANS MEMO 11-25</th>
<th>ACI-318</th>
<th>CEB-FIP Model Code</th>
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<td>√</td>
<td>×</td>
<td>×</td>
<td>×</td>
</tr>
</tbody>
</table>

Where:
- \( T_{burst} \) = Tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis.
- \( d_{burst} \) = Distance from anchorage device to the centroid of the bursting force, \( T_{burst} \).
- \( h \) = Section height or box girder height.
- \( P_{tu} \) = Ultimate prestressing jacking force.
- \( f_{cu} \) = Concrete cube compressive strength.
- \( A_{s min} \) for web = Minimum web reinforcement at anchorage zone.
- \( b_{d min} \) = Minimum diaphragm width.
- \( b_{w min} \) = Minimum web girder or web flare width.
Based on this comparison, none of the available design codes is applicable for anchorage zone of box girder bridges except Caltrans Memo to Designer 11-25 and EURO Code 2. Caltrans memo to designer has no procedures, it just provides minimum thicknesses for the end diaphragm and the web girder as well as minimum vertical reinforcement in the girder web. EURO Code 2 provides only dispersion angle for prestressing force spreading (33.7°) without any detailed procedures of design. It does not contain any limits for spalling or bearing stresses. Shortage in the available design equations for anchorage zone of box girders is illustrated based on this comparison and in the following subsections.

2.3.2 Elastic analysis method

The elastic analysis method presented by Nawy [19] and Namaan [20] for design of end anchorage is based on the design of anchorage zone method developed by Gergely and Sozen [21] in 1967. Free body model forces for this design methodology are illustrated in Figure 2-3. This method is applicable for rectangular as well as irregular sections, but it cannot be applied in the case of end anchorages of box girder bridges due to cross-section variation between the diaphragm and the web.
2.3.3 Elastic finite element analysis

AASHTO LRFD Bridge Design Specifications [3] and ACI 318-14 Building Code Requirements for Structural Concrete [17] possess a set of approximate equations for design of anchorage zone with rectangular cross-section. However, when the end anchorage is as complex as in the box girder, they refer to elastic finite element analysis. Although finite element modeling is a powerful tool, it needs verification before it can be used in design.

A linearly elastic finite element analysis results are more accurate determination of the state of stresses in the anchorage zone. However, the process of computation is time-consuming and costly. Also, the results can be limited because of the difficulty of developing adequate models that can correctly model the cracking in the concrete. A non-linear finite element analysis used to predict the post cracking response could resolve this discrepancy, but it cannot be applied on daily basis [19].
2.3.4 Strut-and-tie models

In 1980s, strut-and-tie models have been emerging as an efficient tool for modeling and detailing disturbed regions (D-regions) in structural concrete members. Some typical strut-and-tie models have been suggested by Marti [22], Schlaich et al. [23] and Breen et al. [9] to obtain the bursting forces behind the anchorage devices. Strut-and-tie models are referenced by AASHTO LRFD Bridge Design Specifications [3] and ACI 318-14 Building Code Requirements for Structural Concrete [17]. Strut–and-tie models for anchorage zones with different configurations are presented in Figure 2-4.

Strut-and-tie method is a convenient way for design of such complex zones, but it needs a lot of iterations in order to obtain a reasonable solution. There are clear strut-and-tie models for problem like: corbels, beams, frame joints, anchorage zones of rectangular cross sections, footings and pile caps [23]; however, the models are not so clear for box girder anchorage zones.

Based on the literature review conducted, there is a shortage in the available design equations for anchorage zones of box girder bridges. Realistic models are needed to provide safe and adequate design procedures. Applicability and simplicity of the design technique must be satisfied, so it can be applied by designer on daily basis.
Figure 2-4: Strut-and-tie models for anchorage zones with different configurations [21].
Chapter (3) – Database for Sample of Constructed Bridges by Caltrans

3.1 Introduction

Bursting forces in the anchorage zones of post-tensioned box girder bridges extend through different bridge elements including: diaphragm, deck, web and soffit. The dimensions of these elements are not related to the design of general anchorage zone. In different design codes, AASHTO LRFD Bridge Design Specifications [3], ACI 318-14 [17] and Caltrans Highway Design Manual [24], the diaphragm dimensions or detailing are not related to prestressing anchorages. In order to obtain an average prototype specimen for the general anchorage zone, a database of bridges provided by Caltrans was developed. This database included 29 anchorage zones of box girder bridges. The data was analyzed to determine dimensions and detailing of the prototype specimen. Choice of representative specimens and bridges for experimental work and field investigation was a challenge, especially when there is no consistent way to design the general anchorage zone. Obtaining the average reinforcement ratios for experimental work was investigated through the developed bridge database. The average prototype specimen was used to develop a scaled representative experimental specimen. The developed bridge database was used as well to choose a set of convenient exemplary bridges for field investigation.

3.2 Database Bridges

The developed database included bridges that were designed and constructed between 2000 and 2016. This database was used to identify average dimensions for box sections and typical reinforcement configurations. Also, it provided data for the existing reinforcement ratios in different locations in the end anchorage zone of post-tensioned box
The database was developed using drawings provided by Caltrans for 29 box girder bridges listed in Table 3-1. The table presents the bridge name as well as code number assigned by Caltrans.

Table 3-1: List of bridges included in the developed database.

<table>
<thead>
<tr>
<th>No.</th>
<th>Bridge Name</th>
<th>Bridge No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mariposa Road OC</td>
<td>29-0325</td>
</tr>
<tr>
<td>2</td>
<td>Main Street OC</td>
<td>29-0327</td>
</tr>
<tr>
<td>3</td>
<td>Dr. MLK Jr. Blvd. OC</td>
<td>29-0329</td>
</tr>
<tr>
<td>4</td>
<td>Quail Meadows OH</td>
<td>10-0171</td>
</tr>
<tr>
<td>5</td>
<td>Quail Meadows UC</td>
<td>10-0173</td>
</tr>
<tr>
<td>6</td>
<td>Upp Creek Bridge</td>
<td>10-0174</td>
</tr>
<tr>
<td>7</td>
<td>FV-Frame 6 Start</td>
<td>10-0165-F6B</td>
</tr>
<tr>
<td>8</td>
<td>FV-Frame 6 End</td>
<td>10-0165-F6E</td>
</tr>
<tr>
<td>9</td>
<td>FV-Frame 7 Start</td>
<td>10-0165-F7B</td>
</tr>
<tr>
<td>10</td>
<td>FV-Frame 7 End</td>
<td>10-0165-F6E</td>
</tr>
<tr>
<td>11</td>
<td>FV-Frame 8 Start</td>
<td>10-0165-F8B</td>
</tr>
<tr>
<td>12</td>
<td>FV-Frame 8 End</td>
<td>10-0165-F8E</td>
</tr>
<tr>
<td>13</td>
<td>Route 101/20 Separation</td>
<td>10-0128RL</td>
</tr>
<tr>
<td>14</td>
<td>S101-W20 Connector</td>
<td>10-0129F</td>
</tr>
<tr>
<td>15</td>
<td>Haehl Creek (Left Bridge)</td>
<td>10-0129-L</td>
</tr>
<tr>
<td>16</td>
<td>Haehl Creek (Right Bridge)</td>
<td>10-0129-R</td>
</tr>
<tr>
<td>17</td>
<td>Haehl Creek</td>
<td>10-0159</td>
</tr>
<tr>
<td>18</td>
<td>East Hill Road UC</td>
<td>10-0157</td>
</tr>
<tr>
<td>19</td>
<td>Smith Creek</td>
<td>37-0606</td>
</tr>
<tr>
<td>20</td>
<td>McGonigle Creek (Left)</td>
<td>57-1082-L</td>
</tr>
<tr>
<td>21</td>
<td>McGonigle Creek (Right)</td>
<td>57-1082-R</td>
</tr>
<tr>
<td>22</td>
<td>Camino Ruiz Under Cr. (Left)</td>
<td>57-1083-L</td>
</tr>
<tr>
<td>23</td>
<td>Camino Ruiz Under Cr. (Right)</td>
<td>57-1083-R</td>
</tr>
<tr>
<td>24</td>
<td>Duenda Road OC</td>
<td>57-1102</td>
</tr>
<tr>
<td>No.</td>
<td>Bridge Name</td>
<td>Bridge No.</td>
</tr>
<tr>
<td>-----</td>
<td>----------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>25</td>
<td>Green Valley Creek (Left)</td>
<td>57-1133-R</td>
</tr>
<tr>
<td>26</td>
<td>Green Valley Creek (Right)</td>
<td>57-1133-L</td>
</tr>
<tr>
<td>27</td>
<td>Lake Hodges (Left)</td>
<td>57-1134-L</td>
</tr>
<tr>
<td>28</td>
<td>Lake Hodges (Right 1)</td>
<td>57-1134-R1</td>
</tr>
<tr>
<td>29</td>
<td>Lake Hodges (Right 2)</td>
<td>57-1134-R2</td>
</tr>
</tbody>
</table>

3.3 Comparison between Data

This bridge database was developed in order to obtain an average prototype specimen for the anchorage zone of post-tensioned bridges. The dimensions and reinforcement ratios were compared for the area of the bridge adjacent to the prestressing anchors. These comparisons included dimensions of different elements, ratio of diaphragm width to box girder height as well as the ratio of spacing between girders to the height, level of prestressing jacking force and finally diaphragm reinforcement ratios.

3.3.1 Different elements dimensions

The geometry of different elements are analyzed in order to obtain the average, maximum and minimum dimension for each element. Investigated parameters include the diaphragm width, box girder height, girder spacing, internal and external web thicknesses as well as deck and soffit thicknesses. Variation of diaphragm width for different bridges is shown in Figure 3-1. Maximum, minimum and average thicknesses are 4.75’, 2.5’ and 3.5’, respectively. These values are not significantly representative, as the ratios between different dimensions for elements are considered more adequate. Box girder height for the considered bridges varies between 3.75’ to 9.75’ as shown in Figure 3-2, while the average box girder height is 6.75’. Figure 3-3 illustrates the variation in spacing between girders.
The maximum and minimum spacing between girders are 11.75’ and 7.25’, respectively and average girder spacing is 10.5’.

Figure 3-1: Diaphragm width for database bridges.

Figure 3-2: Box girder height for database bridges.
Variation of thickness for interior and exterior web girders at end diaphragm are shown in Figures 3-4 and 3-5, respectively. Nearly all internal webs are 12” with rarely web flares done. However, external web thickness is 18” or 20” for most of the considered bridges.
Variation of thickness for deck and soffit are shown in Figures 3-6 and 3-7, respectively. Maximum, minimum and average thicknesses for the deck are 9.25”, 7.5” and 8.5”, respectively. On the other hand, these values for the soffit are 8”, 6” and 7.5”, respectively. It is obvious that the deck is always thicker than the soffit due to the concrete cover restriction provided by AASHTO LRFD Bridge Design Specifications [3].
Tendon inclination angle at the diaphragm for different bridges is presented in Figure 3-8. The inclination angle varies between 3.34° to 5.83° with an average value of 4.34°. However, the AASHTO LRFD Bridge Design Specifications [3] sets limits in the simplified equations for the inclination angle to range between -5° to 20°, the practical values are way less than these limits due to the small height of post-tensioned box girder relative to its span length.
3.3.2 Ratio of diaphragm width to box girder height

Ratio of diaphragm width to box girder height is investigated in order to obtain an average representative value of diaphragm in research work. This ratio is presented in Figure 3-9, where 24 of the investigated anchorage zones represent end diaphragms and the remaining five represent diaphragms at in-span hinges. The minimum values occur at in-span hinges, as the designer always tries to reduce the bridge weight at the hinge by using thinner diaphragms. Using thinner diaphragms for in-span hinges decreases the box girder section flexural and shear demands. The ratio of diaphragm width to girder height ranges between 0.3 and 0.9 with an average value of 0.5. These values are one of the main concerns for selection of bridges for field monitoring and the dimensions of experimental specimens.

![Figure 3-9: Diaphragm width to box girder height for database bridges.](image)

3.3.3 Ratio of girder spacing to box girder height

Ratio of spacing between girders to box girder height is calculated for bridges in the database as shown in Figure 3-10. This ratio represents the aspect ratio of the box girder cells (width to height). Maximum, minimum and average value for the ratio of girder
spacing to box girder height are 2.1, 1.0 and 1.55, respectively. The average value of 1.55 was used for the prototype specimen of experimental work.

![Figure 3-10: Girder spacing to box girder height for database bridges.]

### 3.3.4 Prestressing jacking force

Prestressing jacking force cannot be compared for different bridges, as each bridge has its own geometry, material properties and conditions. Therefore, the ratio between jacking force ($P_J$) and the axial compressive strength of the bridge section is considered for comparison. The axial compressive strength is calculated by multiplication of the target concrete compressive strength ($f'_c$) of the bridge by the gross area ($A_g$) of the box section at mid span. This ratio is representative for the design level of prestressing force for each bridge, as design of prestressed elements depends mainly on checking service stresses. This comparison for different bridges is plotted in Figure 3-11. This figure presents three design levels of prestressing jacking force starting from minimum design level of 0.1 to maximum design level of 0.3. The average design level of 0.2 is the considered value for the loading protocol of the experimental work.
3.3.5 Diaphragm reinforcement ratios

The diaphragm is a congested element with several types of reinforcement. The reinforcement include top and bottom transverse bars, distributed transverse horizontal bars in the inner and outer faces of the diaphragm as well as vertical confining stirrups. Figure 3-12 presents top and bottom reinforcement, horizontal inner and outer diaphragm faces reinforcement and vertical diaphragm stirrups for database bridges. There is wide variation between reinforcement ratios in different locations. For example the transverse horizontal reinforcement in the outer side of the diaphragm ranges between 0.04% and 0.16%, so the ratio between maximum and minimum is 4 times. A large range also occurred in the ratio between maximum and minimum for vertical confining, 2.90. This variation highlights an inconsistency in the detailing of anchorage zones. The relationship between diaphragm reinforcement ratios and prestressing force per girder for database bridges is plotted in Figure 3-13. A random relationship is obtained from this figure. The relationship between diaphragm reinforcement ratios and total prestressing force ratio for database
bridges is plotted in Figure 3-14. There is no correlation developed out of this relationship as well.

Figure 3-12: Diaphragm reinforcement ratios for database bridges.

Figure 3-13: Relationship between diaphragm reinforcement ratios and prestressing force per girder for database bridges.
No meaningful correlation is obtained from the previous three figures. Therefore, it is concluded that diaphragm reinforcement selection is likely based on a “rule of thumb” by each designer based on a) typical bar diameters used in the end diaphragm, b) maximum spacing between bars and c) previous experience.

3.4 Summary of Bridge Database Results

Using the developed bridge database, the average, minimum and maximum for different parameters defining anchorage zones as well as the box girder are summarized in Table 3-2. These parameters include concrete material properties, prestressing jacking force per girder, number of girders, number of spans, maximum span length, tendon inclination angle and maximum edge eccentricity. This table also contains the dimension of the box girder including box girder height, girder spacing, width of diaphragm, deck thickness, soffit thickness, interior and exterior girder width, and overhang dimensions. Based on these data, the cross-sectional area of the box girder as well as the height to the centroid are calculated and presented in the same table. The reinforcement ratios at the diaphragm and vertical ties of the webs are calculated in order to develop the experimental
specimens based on the average values concluded from this database. Different reinforcement ratios are included in this table. Finally, several ratios of these parameters are developed in order to understand factors governing design of post-tensioned box girders. These ratios include maximum edge eccentricity to box girder height, box girder height to maximum span length, prestressing force ratio, diaphragm width to box girder height and girder spacing to box girder height.

Table 3-2: Parameters defining anchorage zones obtained from developed database.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Average</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength ([f'_c]) (ksi)</td>
<td>4.48</td>
<td>4.00</td>
<td>5.08</td>
</tr>
<tr>
<td>Initial concrete compressive strength at time of prestressing ([f'_c]_i) (ksi)</td>
<td>3.71</td>
<td>3.50</td>
<td>4.06</td>
</tr>
<tr>
<td>Prestressing jacking force ([P_J]) per girder (kips)</td>
<td>2171</td>
<td>1260</td>
<td>3016</td>
</tr>
<tr>
<td>No. of girders ([n])</td>
<td>4</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>No. of box cells ([n-1])</td>
<td>4</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>No. of spans</td>
<td>2</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>Maximum bridge span length (ft)</td>
<td>152.9</td>
<td>102.8</td>
<td>242.7</td>
</tr>
<tr>
<td>Deck total width (ft)</td>
<td>53.5</td>
<td>27.3</td>
<td>105.0</td>
</tr>
<tr>
<td>Tendon inclination angle ([\alpha_{inc.}]) (degree)</td>
<td>4.34</td>
<td>3.34</td>
<td>5.83</td>
</tr>
<tr>
<td>Maximum edge eccentricity ([e_{edge}]) (in)</td>
<td>6.61</td>
<td>3.04</td>
<td>12.82</td>
</tr>
<tr>
<td>Box girder height ([h]) (ft)</td>
<td>6.75</td>
<td>3.75</td>
<td>9.75</td>
</tr>
<tr>
<td>Girder spacing ([S]) (ft)</td>
<td>10.5</td>
<td>7.25</td>
<td>11.75</td>
</tr>
<tr>
<td>Diaphragm width ([b_d]) (ft)</td>
<td>3.5</td>
<td>2.5</td>
<td>4.75</td>
</tr>
<tr>
<td>Deck thickness ([t_d]) (in)</td>
<td>8.5</td>
<td>7.5</td>
<td>9.25</td>
</tr>
<tr>
<td>Soffit thickness ([t_s]) (in)</td>
<td>7.5</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>Internal girder width (in)</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>External girder width (in)</td>
<td>12.75</td>
<td>12</td>
<td>16.25</td>
</tr>
<tr>
<td>Internal girder width at diaphragm (in)</td>
<td>12</td>
<td>12</td>
<td>18</td>
</tr>
<tr>
<td>External girder width at diaphragm (in)</td>
<td>18</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>Parameter</td>
<td>Average</td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>-----------------------------------------------------</td>
<td>---------</td>
<td>---------</td>
<td>---------</td>
</tr>
<tr>
<td>Overhang clear length (ft)</td>
<td>4.5</td>
<td>2.7</td>
<td>5.15</td>
</tr>
<tr>
<td>Overhang outer thickness (in)</td>
<td>8</td>
<td>7.25</td>
<td>9.25</td>
</tr>
<tr>
<td>Overhang inner thickness (in)</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Area of box section $[A_g]$ (ft$^2$)</td>
<td>92.64</td>
<td>43.83</td>
<td>179.67</td>
</tr>
<tr>
<td>Height to centroid $[Y_{c.g.}]$ (in)</td>
<td>46.68</td>
<td>24.46</td>
<td>66.32</td>
</tr>
<tr>
<td>Minimum stirrups ratio for interior girders</td>
<td>0.0058</td>
<td>0.0028</td>
<td>0.0092</td>
</tr>
<tr>
<td>Minimum stirrups ratio for exterior girders</td>
<td>0.0040</td>
<td>0.0017</td>
<td>0.0085</td>
</tr>
<tr>
<td>Diaphragm top reinforcement ratio</td>
<td>0.0009</td>
<td>0.0004</td>
<td>0.0015</td>
</tr>
<tr>
<td>Diaphragm bottom reinforcement ratio</td>
<td>0.0009</td>
<td>0.0003</td>
<td>0.0015</td>
</tr>
<tr>
<td>Diaphragm interior face horizontal transverse</td>
<td>0.0007</td>
<td>0.0004</td>
<td>0.0013</td>
</tr>
<tr>
<td>reinforcement ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diaphragm exterior face horizontal transverse</td>
<td>0.0007</td>
<td>0.0004</td>
<td>0.0016</td>
</tr>
<tr>
<td>reinforcement ratio</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diaphragm vertical stirrups reinforcement ratio</td>
<td>0.0015</td>
<td>0.0009</td>
<td>0.0023</td>
</tr>
<tr>
<td>Maximum edge eccentricity to box girder height</td>
<td>0.0832</td>
<td>0.0403</td>
<td>0.1687</td>
</tr>
<tr>
<td>$[e_{edge} / h]$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Box girder height to maximum span length</td>
<td>0.0409</td>
<td>0.0230</td>
<td>0.0462</td>
</tr>
<tr>
<td>Prestressing force ratio $[P_t / f_c A_g]$</td>
<td>0.19</td>
<td>0.13</td>
<td>0.28</td>
</tr>
<tr>
<td>Diaphragm width to box girder height $[b_d / h]$</td>
<td>0.50</td>
<td>0.30</td>
<td>0.90</td>
</tr>
<tr>
<td>Girder spacing to box girder height $[S / h]$</td>
<td>1.55</td>
<td>1.0</td>
<td>2.1</td>
</tr>
</tbody>
</table>

### 3.5 Average Prototype Specimen

Developing a representative prototype specimen is essential as a base for parametric investigation. The main defining parameters for this prototype will be constants, while changing other variables during parametric study. Based on the results presented in Table 3-2, a prototype specimen is obtained. Typical dimensions for interior and exterior girders representing average values of the prototype specimen are presented in Figure 3-15. The
configurations of this prototype specimen are summarized in Table 3-3. This specimen is scaled in order to develop all experimental specimens. Also, it is a main aspect affecting the choice of bridges for field monitoring.

![Diagram of interior and exterior girders representing average values extracted from bridge database.]

Table 3-3: Parameters defining prototype specimen.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength ([f'_{c}]) (ksi)</td>
<td>4.5</td>
</tr>
<tr>
<td>Concrete compressive strength at time of initial prestressing ([f'_{ci}]) (ksi)</td>
<td>3.75</td>
</tr>
<tr>
<td>No. of girders ([n])</td>
<td>4</td>
</tr>
<tr>
<td>Tendon inclination angle ([\alpha_{inc.}]) (degree)</td>
<td>4.34</td>
</tr>
<tr>
<td>Edge eccentricity ([e_{edge}]) (in)</td>
<td>6.61</td>
</tr>
<tr>
<td>Box girder height ([h]) (ft)</td>
<td>6.75</td>
</tr>
<tr>
<td>Girder spacing ([S]) (ft)</td>
<td>10.5</td>
</tr>
<tr>
<td>Diaphragm width ([b_d]) (ft)</td>
<td>3.5</td>
</tr>
<tr>
<td>Deck thickness ([t_d]) (in)</td>
<td>8.5</td>
</tr>
<tr>
<td>Soffit thickness ([t_s]) (in)</td>
<td>7.5</td>
</tr>
<tr>
<td>Internal girder width (in)</td>
<td>12</td>
</tr>
<tr>
<td>External girder width (in)</td>
<td>18</td>
</tr>
<tr>
<td>Overhang clear length (ft)</td>
<td>4.5</td>
</tr>
<tr>
<td>Overhang outer thickness (in)</td>
<td>8</td>
</tr>
<tr>
<td>Overhang inner thickness (in)</td>
<td>12</td>
</tr>
</tbody>
</table>
3.6  Bridge Selection for Field Monitoring

Field monitoring during post-tensioning was done in order to capture the spreading of forces within the general anchorage zone. Several parameters were considered during choice of bridges for field monitoring. The project included four post-tensioned bridges constructed in California. These bridges were Willits Bypass Floodway Viaduct Frame 6 and Frame 8, South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing and Stockton Main Street Overcrossing. The reasons for choosing each one of these bridges are illustrated in the following sections.

3.6.1  Willits Bypass Floodway Viaduct Frame 6 (10-0165-F6B)

Three main reasons were considered for selection of this bridge. First, this bridge has in-span hinge diaphragms, which have a different configuration than an end diaphragm. This diaphragm has a low value of diaphragm width to box girder height of 0.3. Second, it possesses access openings between all of the box girders. This parameter is a main concern affecting the performance of anchorage zone, as it significantly reduces the continuity of the reinforced diaphragm. Finally, post-tensioning of box girder was done from both ends due to the bridge length of 259’. A photo for in-span hinge diaphragm of Willits Bypass Floodway Viaduct Frame 6 is shown in Figure 3-16. Diaphragm reinforcement details and configurations based on Caltrans drawings is presented in Figure 3-17.
Figure 3-16: In-span hinge diaphragm for Willits Bypass Frame 6 (10-0165-F6B).

Figure 3-17: Diaphragm reinforcement details for Willits Bypass Frame 6 (10-0165-F6B) based on Caltrans drawings (SI Units).

3.6.2 Willits Bypass Floodway Viaduct Frame 8 (10-0165-F8E)

The reason that this bridge was considered for field monitoring is its similarity to the developed prototype specimen. All of its dimensions are nearly the same as the developed prototype specimens. Diaphragm reinforcement details and configurations based on Caltrans drawings is presented in Figure 3-18. It has a solid end diaphragm with no
openings. Post-tensioning of this box girder was done from both ends due to the bridge length of 146’. A photo for end diaphragm of Willits Bypass Floodway Viaduct Frame 8 is shown in Figure 3-19.

Figure 3-18: End diaphragm reinforcement details for Willits Bypass Frame 8 (10-0165-F8E) based on Caltrans drawings (SI Units).

Figure 3-19: End diaphragm for Willits Bypass Frame 8 (10-0165-F8E).
3.6.3 South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing (29-0329)

The main reason considered for selection of this bridge is that it has relatively high prestressing jacking force. The ratio between jacking force and the axial compressive strength of the bridge section $[P_j/F_{cA_g}]$ is 0.28 for this bridge as shown previously in Figure 3-11. This high value represents the upper design level for post-tensioning, where the effect of this high prestressing force on the general anchorage zone needs to be investigated. A general view photo for Stockton Martin Luther King Overcrossing is shown in Figure 3-20. Diaphragm reinforcement details and configurations based on Caltrans drawings is presented in Figure 3-21.

Figure 3-20: General view for Stockton Martin Luther King Overcrossing (29-0329) after construction.
Figure 3-21: End diaphragm reinforcement details for Stockton Martin Luther King Overcrossing (29-0329) based on Caltrans drawings (US Units).

3.6.4 South Stockton Widening Main Street Overcrossing - Replace (29-0327)

Three main reasons were considered for selection of this bridge. First, this bridge has relatively large utility opening in the end diaphragm. The vertical dimension of the opening represents nearly 50% of the box girder height. Second, the diaphragm width is relatively large in this bridge. The ratio of diaphragm width to box girder height is 0.9 as presented previously in Figure 3-9. Finally, this bridge has 12 girders, which represents the maximum number of girders according to the developed bridge database. Using this large number of girders, the effect of stressing a certain girder on the adjacent ones can be monitored. A photo for diaphragm of Stockton Main Street Overcrossing is shown in Figure 3-22. Diaphragm reinforcement details and configurations based on Caltrans drawings is presented in Figure 3-23.
Figure 3-22: End diaphragm for Stockton Main Street Overcrossing (29-0327).

Figure 3-23: End diaphragm reinforcement details for Stockton Main Street Overcrossing (29-0327) based on Caltrans drawings (US Units).
3.7 Findings

From the database of box girder bridges designed by Caltrans, the following results were determined:

- The diaphragm reinforcement selection varies substantially and seems to be based more on “rules of thumb” depending on a) typical bar diameters used in the end diaphragm, b) maximum spacing between bars and c) previous experience.

- A prototype specimen was developed that will then be scaled to develop all experimental specimens.

- Key parameters affect the choice of bridges for field monitoring included: a) the level of prestressing force, b) opening in the diaphragm, c) ratio between diaphragm width to box girder height, d) number of prestressing ends, e) number of girders and f) diaphragm type.
Chapter (4) – Field Monitoring

4.1 Introduction

Field investigation was performed for the end zone adjacent to prestressing anchors of four post-tensioned box girder bridges in order to study the performance of general anchorage zones. Different types of strain gauges were used in order to capture strains on reinforcing bars, and within the concrete elements. End diaphragms, in-span hinges diaphragms, web girders, decks and soffits were instrumented in the field using different types of strain gauges. The strain values were recorded through the entire stressing of the prestressing tendons. Even in case of both ends stressing, the data was collected during stressing both ends. The field investigation enabled the measurement of the actual flow of strains in the structure, leading to determination of force trajectories. Several previous research projects included field measurement for bridges in order to study various parameters such as: aging effect, deflection and rebars debonding and soil structure interaction (Guo and Chen 2016; Beltran and Nassif 2014; Kim and Laman 2012) [25] [26] [27]; however, none of these research projects studied the performance of general anchorage zones in box girders.

This chapter presents configurations of the monitored bridges. It includes the developed protocol for field monitoring including preliminary finite element analysis as well as instrumentation methodology to minimize installation time. Then instrumentation preparation, plans and installation techniques are illustrated. The instrumentation plans are shown to indicate the exact location for each strain gauge. Field monitoring results are provided including comparisons between investigated bridges. Strain distribution along
instrumented reinforcement is presented for the diaphragm inner face transverse and vertical directions, the web vertical direction as well as the deck and soffit transverse directions. The chapter includes a description of the end diaphragm cracking due to post-tensioning along with potential reason for the developed cracks. Finally, concluded remarks are highlighted at the end of this chapter.

4.2 Investigated Bridges

The project includes four post-tensioned bridges constructed in California. These bridges are Willits Bypass Floodway Viaduct Frame 6 and Frame 8, South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing and Stockton Main Street Overcrossing. Several parameters affect the choice of bridges for field monitoring including: the level of prestressing force, opening in the diaphragm, ratio between diaphragm width to box girder height, number of prestressing ends, number of girders, and diaphragm type. The parameters affecting the choice of those four bridge were clarified in Chapter (3). The detailed configurations of the instrumented bridges are presented in Table 4-1; it includes bridge type, number of girders, skew angle of investigated end diaphragm, total jacking force, target as well as initial concrete compressive strength, prestressing force ratio, box girder height, girder spacing, width of diaphragm, deck thickness, soffit thickness, interior and exterior web thicknesses at diaphragm, maximum span length, ratio of diaphragm width to girder height, and number of prestressing ends. At the end of the table, notes are provided in order to clarify the existence of openings within the diaphragm.

Bridge I - WB Frame 6 and Bridge II - WB Frame 8 are post-tensioned from both ends; however, Bridge III - Stockton MLK OC and Bridge IV - Stockton Main St. OC are
post-tensioned from only one end. One of the main differences between the instrumented bridges is the configuration of the openings in the end diaphragm. Bridge I has access openings between all box girder webs. However, Bridge II has solid end diaphragm. Both Bridges III and IV have utility openings in the end diaphragm, but Bridge IV has a relatively larger one. These openings in the end diaphragm may affect the performance of the end anchorage during prestressing.

Table 4-1 Instrumented bridges’ configurations

<table>
<thead>
<tr>
<th>Bridge Title</th>
<th>Bridge I WB Frame 6</th>
<th>Bridge II WB Frame 8</th>
<th>Bridge III Stockton MLK OC</th>
<th>Bridge IV Stockton Main St. OC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge type</td>
<td>Straight</td>
<td>Curved</td>
<td>Curved</td>
<td>Skew</td>
</tr>
<tr>
<td>No. of girders [n]</td>
<td>4</td>
<td>4</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>Skew angle (degree)</td>
<td>0</td>
<td>0</td>
<td>41</td>
<td>21.5</td>
</tr>
<tr>
<td>Total jacking force (kips)</td>
<td>11060</td>
<td>7239</td>
<td>16750</td>
<td>15120</td>
</tr>
<tr>
<td>Concrete compressive strength [$f'_c$] (ksi)</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Concrete compressive strength at time of initial prestressing [$f'_{c1}$] (ksi)</td>
<td>4</td>
<td>4</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td>$P_J/f'_c A_g$</td>
<td>0.190</td>
<td>0.132</td>
<td>0.281</td>
<td>0.211</td>
</tr>
<tr>
<td>Box girder height [h]</td>
<td>8’ – 8”</td>
<td>7’ – 4”</td>
<td>5’ – 6”</td>
<td>3’ – 9”</td>
</tr>
<tr>
<td>Girder spacing [S]</td>
<td>10’ – 8”</td>
<td>10’ – 8”</td>
<td>11’ – 0”</td>
<td>7’ – 3”</td>
</tr>
<tr>
<td>Width of end diaphragm [b_d]</td>
<td>2’ – 8”</td>
<td>3’ – 0”</td>
<td>4’ – 0”</td>
<td>3’ – 4”</td>
</tr>
<tr>
<td>Deck thickness [t_d] (in)</td>
<td>8 3/4</td>
<td>8 3/4</td>
<td>8 5/8</td>
<td>7 3/8</td>
</tr>
</tbody>
</table>
### Bridge Title

<table>
<thead>
<tr>
<th>Bridge Title</th>
<th>Bridge I WB Frame 6</th>
<th>Bridge II WB Frame 8</th>
<th>Bridge III Stockton MLK OC</th>
<th>Bridge IV Stockton Main St. OC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soffit thickness ([t_s]) (in)</td>
<td>7 5/8</td>
<td>7 5/8</td>
<td>7 1/2</td>
<td>6</td>
</tr>
<tr>
<td>Web thickness Interior/Exterior ([b_w]) (in)</td>
<td>12/20</td>
<td>12/20</td>
<td>12/18</td>
<td>12/18</td>
</tr>
<tr>
<td>Max. span length</td>
<td>220’– 0”</td>
<td>166’– 8”</td>
<td>137’– 6”</td>
<td>100’– 0”</td>
</tr>
<tr>
<td>Diaphragm width / Girder height</td>
<td>0.308</td>
<td>0.409</td>
<td>0.727</td>
<td>0.889</td>
</tr>
<tr>
<td>No. of prestressing ends</td>
<td>Two ends</td>
<td>Two ends</td>
<td>One end</td>
<td>One end</td>
</tr>
<tr>
<td>Notes</td>
<td>Access openings for all bridge girders</td>
<td>Solid end diaphragm</td>
<td>Utility opening in end diaphragm</td>
<td>Relatively large utility opening in end diaphragm</td>
</tr>
</tbody>
</table>

One of the major defining parameters for post-tensioning process is the stressing sequence. The stressing order for anchors is conducted to reduce the twisting of bridge about the longitudinal axis. Also, stressing of anchors is always executed from top to bottom. The reason for that sequence is to facilitate stressing the top strands by lowering the stressing jack from a crane as shown in Figure 4-1. Then, the strands of the stressed anchor are bundled in two groups and separated using tie wires as presented in Figure 4-2. Making two groups facilitates lowering the prestressing jack to the bottom anchors as shown in the same figure.
Figure 4-1: Lowering prestressing jack from a crane to reach target strands.

Figure 4-2: Strands of top anchors are bundled after stressing then stressing the lower anchor.
The stressing sequence affects propagation of strains at each of the investigated anchors. Also, the amount of force applied on each anchor differs based on the amount of strands in each individual tendon. The stressing sequence, number of strands in each tendon as well as the total jacking force per tendon for Bridge I are illustrated in Figures 4-3 and 4-4 for both stressing ends. The top two rows of tendons have 27 strands; however, the bottom row has only 18 strands. On the other side, Bridge II has only two rows of tendons and number of strands in each tendon ranges between 20 to 21 as shown in Figures 4-5 and 4-6 for both stressing ends.

Figure 4-3: Stressing sequence for Bridge I – First End.

Figure 4-4: Stressing sequence for Bridge I – Second End.

Figure 4-5: Stressing sequence for Bridge II – First End.
Figure 4-6: Stressing sequence for Bridge II – Second End.

Bridge III and Bridge IV have larger number of girders. Bridge III has six girders while Bridge IV has twelve girders. The stressing sequence of both bridges are shown in Figures 4-7 and 4-8, respectively.

Figure 4-7: Stressing sequence for Bridge III.

Figure 4-8: Stressing sequence for Bridge IV.

4.3 Field Monitoring Protocol

Construction issues and cracking problems have occurred in anchorage zones of box girder bridges. These local problems affected the global performance of box girder bridges. Monitoring performance of the general anchorage zones was one of the main tasks in this research project. In order to prepare convenient instrumentation plans, preliminary finite element models were developed. These models were used to determine the critical
locations for instrumentation of the anchorage zone of box girders. The instrumentation procedures were different depending on each individual case of construction and dimensions of the box girder end zone.

A primary goal in the selection of field instrumentation was to obtain the information needed while minimizing any delays in the time schedule of the bridge construction. Several precautions were made to reduce the chances that the instrumentation process would interfere with construction.

4.3.1 Preliminary finite element analysis

Preliminary analysis was needed to determine the critical zones of stress concentration due to prestressing. These zones were instrumented in the field in order to measure the actual increase in strains during post-tensioning. Preliminary finite element models were developed for typical end anchorages of a box girder based on the prototype specimen developed from the bridge database.

Finite element modeling of end anchorage is complex due to the geometry of this zone in addition to congested reinforcement. Three-dimensional model for a repetitive girder was developed using DIANA R 9.6 [28] as shown in Figure 4-9. The mesh of the end diaphragm, web girders, soffit, deck and loading plates of the model are shown in Figure 4-9 (a) and the reinforcement details are shown in Figure 4-9 (b). The box section has the typical dimensions of the prototype average specimen developed from the bridge database. The section is 81” height (h), 126” spacing between girders (S), 12” web thickness (bw), 8.5” deck thickness (td), 7.5” soffit thickness (ts), and 40” end diaphragm width (bd). Two eccentric prestressing straight ducts are modeled as voids. The applied
load per girder is 2100 kips, which represents 15% of the concrete section axial capacity. Loads are applied on circular loading plates.

Locations of stress concentration and critical zones are determined using the developed model as shown in the longitudinal section in Figure 4-10 [29]. The transparent portions of the figure represent the compression zones as the tensile vertical bursting stresses are only plotted. It is illustrated from the stress concentration of vertical tensile stresses that the critical zones are the inner face of the end diaphragm and approximately 50% h of the web. These zones are affected by bursting tension forces. This is the reason that instrumentations are placed in these critical zones only.

Figure 4-10: Vertical tensile stresses (ksi) of longitudinal section of the box girder.
The vertical and transverse tensile stresses on the inner face of end diaphragm are plotted in Figure 4-11 (a) and (b), respectively. The distribution of the vertical stresses illustrates that end diaphragm vertical reinforcement shall be designed for post-tensioned bursting forces. The pattern of the transverse tensile stresses extends only to 30% S on both sides of the girder. Therefore, horizontal transverse reinforcement on the end diaphragm’s inner face shall also be designed for post-tensioned bursting forces. As a result of this preliminary modeling, critical zones are determined. These zones are defined by high bursting tensile stresses.

(a) Vertical tensile stresses

(b) Transverse tensile stresses

Figure 4-11: Inner face of end diaphragm tensile stresses (ksi) contours.

4.3.2 Instrumentation methodology

The instrumentation was placed in the critical zones concluded from preliminary modeling. Different techniques of instrumentation were investigated. Two different types of strain gauges were investigated in order to capture the performance of box girder anchorage zones: concrete gauges to measure the strains inside the concrete elements and steel gauges to measure the developed strains in reinforcing bars. Attaching strain gauges
to the reinforcing bars in the field was nearly impossible, as this process require accessibility to reinforcing bars and a longer time frame for installation. The construction time schedule would not allow for such delays. So, an alternative methodology was used. Reinforcing bars were prepared with strain gauges in the lab. These bars were called “Sister Bars” see Figure 4-12. However, since these bars would increase the reinforcement ratio, #3 bars were used to minimize the impact. The increase in reinforcement ratios in all the instrumented bridges did not exceed 10%, which could be considered as minimal due to the random values of existing reinforcement as illustrated previously in Chapter (3).

Figure 4-12: Sample of instrumented Sister Bars

4.4 Field Instrumentation

4.4.1 Instrumentations preparation

Several types of strain gauges were investigated in order to choose the appropriate type for both concrete and steel. The main parameters in choosing strain gauges were accuracy, applicability, level of recorded data, interfering with construction, safety during construction and sensitivity to concrete placement. For reinforcing bars, foil strain gauges with a maximum strain of 5% was chosen. As the strains in the field would not exceed the yield strain of the reinforcing bars, strain up to 5% was acceptable. Also, several procedures have been considered to maintain safety of the strain gauge before the prestressing process. The reinforcing bars were grinded to prepare the bar surface. Strain gauges were attached
using a matching type of adhesive. Thick layer of wax was added above the strain gauge to seal it from water during concrete placement. Flexible tape was used to provide soft layer above strain gauge. A thick epoxy layer was added to prevent damage of gauges during concrete casting. Summary for strain gauging steps is shown in Figure 4-13. Due to the severe environment for the reinforcing instrumentation, mastic water sealant tape was used as an additional coating surface for the bars placed in the bridge deck. This black mastic tape is shown in Figure 4-13 – Step 5.

The concrete gauges were dumbbell shaped. The gauge length was 50 mm and backing thickness was 4 mm diameter. These concrete strain gauges were attached in the field using thin aluminum mechanical wires as shown in Figure 4-14. Several procedures were considered during attaching these type of gauges in order not to apply any initial strain on them before measuring stage. Mechanical wires were always attached with an angle to the gauge axis, so during tightening these mechanical wires would apply no significant tension force to the gauge. All strain gauge wires were inserted in heat shrink tubes for protection as presented in Figure 4-15.
Step 1: Grinding bar surface and attaching strain gauges with matching type of adhesive

Step 2: Adding thick layer of wax above the strain gauge to seal it from water

Step 3: Adding flexible tape to provide soft layer above the gauge

Step 4: Adding thick epoxy layer to protect the gauges during construction

Step 5: Adding mastic water sealant tape

Figure 4-13: Steps of strain gauging for reinforcement bars.
Figure 4-14: Dumbbell shaped concrete gauge attached using aluminium mechanical wires.

Figure 4-15: Strain gauge wires placed in heat shrink tubes.

4.4.2 Instrumentation plans

Instrumentation plans were prepared to study the performance of the anchorage zone in the four investigated box girder bridges. Detailed drawings including the locations and labels of strain gauges are presented in Appendix (A). Each one of the investigated bridges had its own instrumentation plans based on the presence of diaphragm openings and the geometry of anchorage zones. In all of the investigated bridges at least two girders were monitored during post-tensioning, one of them was interior girder and the other was exterior.
4.4.2.1 Bridge I - Willits Bypass Floodway Viaduct Frame 6 (10-0165-F6B)

Horizontal as well as vertical sister bars were implemented in the inner face of the diaphragm in order to capture bursting force spreading. Labeling of transverse horizontal bars starts with (H); however, vertical bars starts with (V). The second character in the horizontal bars represents the three levels of the strain gauges: Top (T), Middle (M) and Bottom (B). For vertical bars, the second character in the label represents imaginary gridlines from A to F, while the last character represents level of the strain gauges: Top (T), Middle (M) and Bottom (B). The implemented sister bars in the inner face of Bridge I diaphragm are shown in Figure 4-16.

![Figure 4-16: Instrumentation plan for inner face of Bridge I diaphragm.](image)

The web girders were instrumented in two directions, vertical and transverse. Sister bars instrumented with strain gauges were implemented in the vertical direction of the web. The transverse direction of the web was instrumented using concrete strain gauges. The reason for installing the transverse concrete gauges was to study the need of small
transverse hoops within the web in the general anchorage zone. The web gauges were labeled WI and WX for interior and exterior girders, respectively. The instrumentation plans for both webs are illustrated in Figures 4-17 and 4-18.

Figure 4-17: Instrumentation plan for interior web of Bridge I.

Figure 4-18: Instrumentation plan for exterior web of Bridge I.
Two lines of strain gauges were installed using sister bars in the deck between top and bottom layers of reinforcement. The first line adjacent to the anchors, was to study the reason for transverse cracking ahead of the anchors. The second line of strain gauges ahead of the diaphragm, was added to study the transverse splitting force spreading in the deck. All the gauges installed in the deck were labeled by (D) as shown in Figure 4-19.

![Figure 4-19](image)

**Figure 4-19:** Instrumentation plan for deck of Bridge I.

4.4.2.2 Bridge II - Willits Bypass Floodway Viaduct Frame 8 (10-0165-F8E)

The same labeling technique was used for instrumentation plans of Bridge II. The implemented sister bars in the inner face of Bridge II solid diaphragm are shown in Figure 4-20. Three horizontal rows of sister bars in addition to six vertical bars were installed at the interior Girder B as well as the exterior Girder A. Interior and exterior web instrumentation plans are shown in Figures 4-21 and 4-22, respectively. Also, one row of strain gauges was installed in the deck in order to study the transverse splitting force spreading. All the gauges installed in the deck are illustrated in Figure 4-23.
Figure 4-20: Instrumentation plan for inner face of Bridge II diaphragm.

Figure 4-21: Instrumentation plan for interior web of Bridge II.
4.4.2.3 Bridge III - South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing (29-0329)

The implemented sister bars in the inner face of Bridge III diaphragm are shown in Figure 4-24. The same labeling technique was used for all instrumented bridges. Three horizontal rows of sister bars in addition to six vertical bars were installed at the interior Girder B as well as the exterior Girder A. Interior and exterior web instrumentation plans are shown in Figures 4-25 and 4-26, respectively. Two locations were instrumented in the
deck in order to study the difference between developed strains in the interior and exterior girders as shown in Figure 4-27.

Figure 4-24: Instrumentation plan for inner face of Bridge III diaphragm.

Figure 4-25: Instrumentation plan for interior web of Bridge III.

Figure 4-26: Instrumentation plan for exterior web of Bridge III.
4.4.2.4 Bridge IV - South Stockton Widening Main Street Overcrossing - Replace (29-0327)

In this bridge four girders were instrumented with strain gauges. Three interior girder and one exterior girder were investigated in order to study the effect of stressing a certain girder on adjacent girders. The implemented sister bars in the inner face of Bridge IV diaphragm are shown in Figure 4-28. Two horizontal rows of sister bars in addition to seven vertical bars were implemented. Instrumentation plans for interior Girders B, C and D are shown in Figures 4-29, 4-30 and 4-31, respectively. The exterior Girder A was also instrumented as shown in Figure 4-32.
Figure 4-28: Instrumentation plan for inner face of Bridge IV diaphragm.

Figure 4-29: Instrumentation plan for interior Girder (B) of Bridge IV.

Figure 4-30: Instrumentation plan for interior Girder (C) of Bridge IV.
Figure 4-31: Instrumentation plan for interior Girder (D) of Bridge IV.

Figure 4-32: Instrumentation plan for exterior Girder (A) of Bridge IV.

Girders A, B and C were instrumented just ahead of the prestressing block-out to study the reason for transverse cracking ahead of the anchors. The deck and soffit at Girder C were instrumented in order to compare between the deck and soffit transverse bursting forces. Instrumentations implemented in the soffit were labeled with S. The instrumentation plan for both deck and soffit of Bridge IV is shown in Figure 4-33.
4.4.3 Instrumentations installation

Installation of the sister bars and the concrete strain gauges was a critical stage. Several safety cautions were considered to make sure the instrumentation was not interfering with construction so that they would survive the construction process. Construction workers usually step on reinforcing bars during construction. They use the horizontal secondary bars as a ladder to climb up and down. Hiding the wires under the reinforcing bars was an important task during placing the strain gauge wires as shown in Figure 4-34. Concrete strain gauges are very fragile and no coating can be applied to them. These gauges were fixed in place using thin mechanical wires. To prevent damage of such fragile gauges, they were hidden under prestressing ducts as shown in Figure 4-35.
Figure 4-34: Installed instrumentation in the field including sister bars and concrete gauges.

Figure 4-35: Concrete gauges hidden under prestressing ducts.

The final position of sister bars after being installed in the inner face of Bridge I diaphragm is presented in Figure 4-36. Six vertical bars are highlighted using dotted red boundaries and three horizontal rows are highlighted using dotted white boundaries. The installed sister bars in the interior as well as exterior webs are presented in Figure 4-37. These vertical bars are clarified using dotted red boundaries.
Figure 4-36: Sister bars installed in the inner face of Bridge I diaphragm.

(a) Exterior Girder  
(b) Interior Girder

Figure 4-37: Sister bars installed in the web girders of Bridge I.

After concrete placement, all strain gauges were checked using a digital ohmmeter. The measured resistance showed that more than 90% of the wires were still connected to the strain gauges. Finally, additional protection was made to the strain gauge wires coming out of the specimen by bundling the wires and putting them into water sealed bags.
4.5 Field Data Recording and Correction

The data was recorded for the implemented strain gauges and the prestressing jack pressure gauge using a Data Acquisition System. Data Acquisition System consists of Data Acquisition Box (DAC) and recording laptop as shown in Figure 4-38. In order to connect the strain gauge wires to the DAC, intermediate converters (strain gauge boxes) were used as shown in Figure 4-39. The data was transferred from the strain gauge wires into the measuring cables through these strain gauge boxes.

Figure 4-38: Data Acquisition System used for field measurements.
In order to record the prestressing jack pressure, a digital pressure gauge was attached to the pressure line as shown in Figure 4-40. This pressure gauge was attached to a measuring cable that transfer the data to the DAC. The prestressing contractor was determining prestressing levels based on a dial pressure gauge as shown in Figure 4-40. The data recorded from the implemented pressure gauge was always calibrated with the dial pressure gauge used by the prestressing contractor, in order to check the validity of the measurements during stressing process. Also, the data was recorded in each stressing stage for each anchor. The measured pressure values were always transferred into prestressing forces based on the constant cross-sectional area of the prestressing jack.
Final form of strain gauge results was developed after applying several corrections to the raw data including data resampling, correction for temperature effects and zero error reading correction. These corrections are illustrated as follows:

### 4.5.1 Data resampling

The minimum sample rate used for data recording was 24 Hz. As this type of prestressing loading is considered as static loading, there was no need for this high sample rate of recording. Therefore, data was resampled for 3 Hz by taking the average of each eight successive records. This resampling facilitates handling and correcting the captured data. The data resampling process is illustrated in Figure 4-41. However, this resampling
may reduce the maximum values, the reduction did not exceed 5 microstrains. This small value is considered as minimal.

![Image](image)

**Figure 4-41**: Example for raw data resampling from 24Hz to 3Hz.

### 4.5.2 Temperature effects

Stressing each individual anchor was considered as an individual loading stage. Few discrepancies were observed between the end of the loading stage and the beginning of the following one. The main reason for this difference was the temperature variation especially when prestressing stage starts in the early morning and continues till after noon. Temperature effects were included by correcting the end of each loading stage to match the beginning of the subsequent stage. This error occurred in few stressing steps due to waiting a long time between steps. Moving the prestressing jack was the reason for these delays. During some loading stages at in-span hinge diaphragm, the clear space ahead of the prestressing anchors was extremely congested with reinforcement. That makes handling the prestressing jack from one anchor to another consume a lot of time. The temperature effect correction is clarified in Figure 4-42.
4.5.3 Zero error reading

A few strain gauge measurements started with an initial relatively high values; however, the initial measurement before starting prestressing should be zero. Reasons were observed for this initial error including temperature effect or initial strain that occur to the strain gauge wires during connecting them to the strain gauge boxes. This error was also corrected for all of the measured strain gauges as shown in Figure 4-43.

Figure 4-42: Correcting temperature effect between subsequent loading stages for a sample strain gauge.

Figure 4-43: Zero error reading correction for a sample strain gauge.
4.5.4 Trend approximation

The loading methodology of prestressing forces was considered as a slow static test, due to the slow rate of applying prestressing forces. So minor strain variations that happened within different loading stages were neglected by capturing the final results at the end of each loading stage. The transfer of force from concrete to reinforcing bars is really hard to estimate as it is significantly affected by the concrete material properties, which may cause these minor variations. Approximation trend lines were developed for each strain gauge as shown in Figure 4-44. These linear trend lines represent the beginning and the end of each stressing stage. This methodology facilitates processing and interpretation for the measured data.

![Figure 4-44: Trend line approximation for a sample strain gauge.](image)

4.6 Field Results

Field monitoring for the investigated bridges developed significant amount of data. This data includes strains developed on implemented sister bars, strains within concrete element and pressure values for the prestressing jack.
4.6.1 Bridge I - Willits Bypass Floodway Viaduct Frame 6

Stressing of strands is an elongation process that is applied using a hydraulic jack. The prestressing jack has a certain stroke capacity. As bridge length is relatively high, several strokes shall be applied using the hydraulic jack based on the target elongation required to satisfy the required prestressing force and the maximum stroke the jack can perform. The strands are clamped using the jack till they reach the maximum stroke, then the prestressing wedges clamp the strands while the jack releases all its force. During second stressing stage, the jack starts to apply more elongation on the strands causing higher prestressing forces. These stages of prestressing are presented in Figure 4-45 for different anchors of Bridge I. The blue solid line in this chart represents the multiple stroke performed by the hydraulic jack on each individual anchor. The red dotted line clarifies the envelope of prestressing forces applied at each anchor. The accumulative of the envelope peaks prestressing forces is used for plotting the resulting strains developed on sister bars or within concrete elements, which represent the total prestressing force applied on the whole box section.

![Figure 4-45: Rate of actual and envelope prestressing forces applied using hydraulic jack for multiple anchors of Bridge I.](image)
Measured strains of sister bars at both interior and exterior webs of Bridge I are shown in Figures 4-46 and 4-47, respectively. The sister bars at the interior web girder include transverse bars in the diaphragm inner face and the deck as well as vertical bars in the inner face of the diaphragm and the web. The ones at the exterior web girder include transverse and vertical bars in the diaphragm inner face as well as vertical bars in the web. The transverse concrete strains developed within both interior and exterior web girders are shown in Figure 4-48.

Figure 4-46: Measured strains of sister bars at the interior web of Bridge I.
It can be noticed that there is a plateau at the end of the recorded data for each individual strain gauge. This increase in the strain values at the same prestressing force presents the effect of stressing the other end of the bridge. Although, stressing the other end should not affect the investigated end theoretically, increase in the strains recorded was observed in all of the recorded data. This occurred in Bridge I and Bridge II, which are stressed from both ends.

Figure 4-47: Measured strains of sister bars at the exterior web of Bridge I.
4.6.2 Bridge II - Willits Bypass Floodway Viaduct Frame 8

Measured strains of sister bars at both interior and exterior webs of Bridge II are shown in Figures 4-49 and 4-50, respectively. The sister bars at the interior web girder include transverse bars in the diaphragm inner face and the deck as well as vertical bars in the inner face of the diaphragm and the web. The ones at the exterior web girder include transverse and vertical bars in the diaphragm inner face. The transverse concrete strains developed within both interior and exterior web girders are shown in Figure 4-51.
Figure 4-49: Measured strains of sister bars at the interior web of Bridge II.

Figure 4-50: Measured strains of sister bars at the exterior web of Bridge II.

Figure 4-51: Measured transverse concrete strains at the webs of Bridge II.
4.6.3 Bridge III - South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing

Measured strains of sister bars at both interior and exterior webs of Bridge III are shown in Figures 4-52 and 4-53, respectively. The sister bars at the interior web girder include transverse bars in the diaphragm inner face and the deck as well as vertical bars in the inner face of the diaphragm and the web. The developed strains in the transverse direction of the diaphragm are divided in two groups with two opposite signs (compression and tension). Reasons of these opposite signs will be clarified later in this chapter during comparing the results. The sister bars at the exterior web girder include transverse bars in the diaphragm inner face and the deck as well as vertical bars in the inner face of the diaphragm and the web. The transverse concrete strains developed within both interior and exterior web girders are shown in Figure 4-54.
Figure 4-52: Measured strains of sister bars at the interior web of Bridge III.
Figure 4-53: Measured strains of sister bars at the exterior web of Bridge III.

Figure 4-54: Measured transverse concrete strains at the webs of Bridge III.
4.6.4 Bridge IV - South Stockton Widening Main Street Overcrossing – Replace

Measured strains of sister bars at interior web Girder B, C and D of Bridge IV are shown in Figures 4-55, 4-56 and 4-57, respectively. The sister bars at the interior web Girder B include transverse bars in the diaphragm inner face and vertical bars in the web. The ones at the interior web Girder C include transverse bars in the diaphragm inner face, the deck and the soffit as well as vertical bars in the inner face of the diaphragm and the web. Sister bars implemented at web Girder D include transverse and vertical bars in the diaphragm inner face as well as vertical bars in the web. Two concrete gauges were installed above and below the utility opening at Girder D. They were implemented in the transverse direction on the inner face of the diaphragm. The measured data out of both concrete gauges are plotted in Figure 4-58.

![Graph](image1)

(a) Diaphragm transverse direction  
(b) Web vertical direction

Figure 4-55: Measured strains of sister bars at Girder B of Bridge IV.
Figure 4-56: Measured strains of sister bars at Girder C of Bridge IV.
Figure 4-57: Measured strains of sister bars at Girder D of Bridge IV.

Figure 4-58: Measured transverse concrete strains at the inner face of the diaphragm around utility opening of Bridge IV.
Measured strains of sister bars at exterior web Girder A of Bridge IV are shown in Figure 4-59. The sister bars at the exterior web Girder A include transverse and vertical bars in the diaphragm inner face as well as vertical bars in the web. The transverse concrete strains developed within both interior and exterior web girders are shown in Figure 4-60. In order to compare the difference in transverse bursting force in the deck ahead of the prestressing block-out and inside the deck ahead of the diaphragm, measured transverse sister bars strains were implemented at the deck ahead of the prestressing block-out for girders A, B and C. The recorded strains out of these sister bars are shown in Figure 4-61.

Figure 4-59: Measured strains of sister bars at exterior Girder A of Bridge IV.
4.6.5 Instrumentations adequacy

Using all the recorded data from different instrumentations including reinforcement as well as concrete strain gauges and pressure gauges, the adequacy of the instrumentation methodology is determined. The number of measured instrumentations compared to the number of failed ones is illustrated using the bar chart plotted in Figure 4-62. The ratio of measured instrumentation to the total implemented ones is 88% for Bridge I, 85% for Bridges II and III, and 90% for Bridge IV. Several issues were considered in choosing the type of strain gauges, suitable coating materials, cover for the gauge wires and installation
technique [29]. While not fully successful, the installation technique and placement provided sufficient data for understanding the flow of forces within the anchorage zone.

![Comparison between measured and failed instrumentations](image)

Figure 4-62: Comparison between measured and failed instrumentations for the investigated bridges.

### 4.7 Comparison between Results

In this section of the chapter, effect of concrete contribution in resisting tensile strain is discussed. The strain distributions along different elements are developed including transverse and vertical directions of the diaphragm inner face, web vertical direction, transverse direction in the deck and soffit as well as transverse strain distribution ahead of the prestressing block-outs. Also, the observed transverse strains of the web using concrete gauges are compared. Finally, comparisons outcomes are presented.

#### 4.7.1 Concrete contribution to developed strains

Concrete compressive strength is much higher than its tensile strength. Concrete tensile strength is very sensitive to the concrete mixture, cement type, water to cement ratio, aggregate size and type, concrete curing, existence of admixtures and concrete age [16] [30]. Also, maximum tensile strain of concrete is sensitive for the same parameters. Tensile strength of concrete affects the transfer of forces from concrete to reinforcing bars.
The higher the strain level reached, the lower the effect of concrete contribution on the recorded reinforcement strain.

Two concrete gauges were installed adjacent to two sister bars placed above and below utility opening at Girder D of Bridge IV. The exact location for the installed sister bars and concrete gauges is illustrated previously in Figure 4-28. The results of these two gauges explain concrete contribution to the developed strain. Comparison between developed strains of sister bars and concrete gauges in the transverse direction of the inner diaphragm face is shown in Figure 4-63. The strains developed in concrete gauges have the same trend as those developed in the adjacent sister bars. As a rough assumption, maximum tensile strain of concrete was set as 150 microstrain. Tensile strength of concrete varies between 8 and 15 % of the compressive strength [31]. Therefore, the assumed value of 150 microstrain is based on tensile strength of 12% of the compressive strength. When the concrete strains values are lower than the assumed maximum tensile strain of concrete, concrete will have significant contribution in resisting splitting tensile strain.

![Figure 4-63: Developed strains of sister bars and concrete gauges at interior Girder D of Bridge IV in the transverse direction of the inner diaphragm face around utility opening.](image-url)
For more clarification, transverse strain distribution above and below the utility opening at Girder D of Bridge IV are plotted in Figure 4-64. Two strain levels are compared. The solid line represents the developed strain after stressing the top anchor of Girder D and the dotted line represents the strain level after stressing the second anchor. Both stress levels - above and below the utility opening - show that strains developed by concrete gauges are nearly the same as those on reinforcing bars. That illustrates the role of concrete in resisting tensile strains. It also shows that the concrete contribution in resisting the transverse splitting is higher than the reinforcing bars at these low levels of strains. At higher strain level, concrete will lose its tensile capacity and the effect of tension stiffening will be minimized. At a cracked section of reinforced concrete member, all tension is carried by reinforcement. However, the concrete continues to carry tensile stresses between the cracks because of bond action, which effectively stiffens the member response and reduces deformations. This phenomenon, commonly known as tension stiffening, is effectively responsible for above-zero average tensile stresses existing in cracked concrete [32]. Tension stiffening of concrete affects the transfer of forces from concrete to reinforcing bars. The higher the strain level reached, the lower the effect of tension stiffening on the total recorded strain value. The concrete contribution in resisting the splitting strains is higher, when strain levels are relatively low.
4.7.2 Diaphragm transverse strain distribution

The transverse strain distribution for the sister bars at interior girders of Bridge I and II are shown in Figures 4-65 and 4-66, respectively. The strain values always increase towards the web centerline and decrease distant from it. Only in the middle level of Bridge II, the strain increase near the web face, the most probable reason for that is a crack was initiated at the web face. This crack amplifies the captured strains.

The level of strains in case of Bridge I is nearly two times the strains in Bridge II. The reasons for this difference are the prestressing level, ratio of diaphragm width to box girder height, presence of access opening in diaphragm and tension stiffening of concrete. Prestressing force ratio is 0.190 for Bridge I and 0.132 for Bridge II. Diaphragm width to box girder height is 0.308 and 0.409 for Bridge I and II, respectively. End diaphragm of Bridge II is solid; however, in-span hinge diaphragm of Bridge I possesses access opening between all of its webs. Bridge I has higher prestressing force and thinner diaphragm with access opening, which cause significant increase in the transverse strain values in the inner face of the diaphragm than Bridge II. Tension stiffening of concrete affects the transfer of 

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Figure 4-64: Transverse strain distribution above and below utility opening at interior Girder D of Bridge IV on the inner face of the diaphragm.
forces from concrete to reinforcing bars. The higher the strain level reached, the lower the effect of tension stiffening on the recorded strain values.

Based on the results shown in Figures 4-65 and 4-66, it is noticed that stressing the other bridge end anchors causes slight increase in strain values. This increase is approximately 6.8% in Bridge I and 8.6% for Bridge II.

![Figure 4-65: Transverse strain distribution of the diaphragm inner face at interior web of Bridge I.](image)

![Figure 4-66: Transverse strain distribution of the diaphragm inner face at interior web of Bridge II.](image)

The transverse strain distribution for the sister bars at interior girders of Bridge III and IV are shown in Figures 4-67 and 4-68, respectively. Both bridges have skew end
diaphragm, which changes the strain distribution. The obtuse side of the angle between the diaphragm and the web always possesses higher strain values than the acute side. The obtuse side has higher potential to cracking rather than the acute one. Both acute and obtuse sides of the diaphragm were illustrated previously in Figure 4-33.

![Figure 4-67: Transverse strain distribution of the diaphragm inner face at interior web of Bridge III.](image1)

![Figure 4-68: Transverse strain distribution of the diaphragm inner face at interior web of Bridge IV.](image2)

This phenomena of change in the strain distribution is clarified through creating two simplified 2D strut-and-tie models. Figures 4-69 and 4-70 illustrate a comparison between simplified 2D strut-and-tie models for the end diaphragm in case of straight bridge and
skew one, respectively. These developed models represent a top view for the diaphragm, where the web, deck and soffit axial stiffness are defined using springs. The resulting axial forces of these models are presented in the form of rectangular blocks (blue for tension and red for compression). The developed strut-and-tie models also confirm tension on the obtuse side of the inner diaphragm face in case of skew bridge. On the other hand, tension will occur on both sides of the diaphragm face in case of straight bridge.

Figure 4-69: Simplified 2D Strut-and-tie proposed model for end diaphragm in case of straight bridge.

Figure 4-70: Simplified 2D strut-and-tie proposed model for end diaphragm in case of skew bridge.
The acute side of Bridge III is subjected to compressive strain; however, the acute side of Bridge IV is subjected to tensile strains. This difference in performance occurred due to difference in skew angle. The skew angle for Bridge III is 41° and the skew angle for Bridge IV is 21.5°. The difference in strains between obtuse and acute sides increases in case of large skew angle. Also, the strain levels in Bridge III is higher than that in Bridge IV due to the prestressing level, ratio of diaphragm width to box girder height and tension stiffening of concrete. More illustration for Bridge III transverse strain distribution as well as higher strain levels reached are presented in the cracking case study at the end of this chapter.

The transverse strain distribution for the sister bars at exterior girders of Bridge I and II are shown in Figures 4-71 and 4-72, respectively. The strain values always increase towards the exterior web and decrease distant from it. The maximum strains always occur at the web face, where cracks can occur.

The level of strains in case of Bridge I is nearly four times the strains in Bridge II. The reasons for this difference are the prestressing level, ratio of diaphragm width to box girder height, presence of access opening in diaphragm and tension stiffening of concrete as discussed previously. Due to the low value of strains in Bridge II, the concrete contribution in resisting tensile strains, significantly affects the developed strains on sister bars.
Figure 4-71: Transverse strain distribution of the diaphragm inner face at exterior web of Bridge I.

Figure 4-72: Transverse strain distribution of the diaphragm inner face at exterior web of Bridge II.

The transverse strain distribution for the sister bars at exterior girders of Bridge III and IV are shown in Figures 4-73 and 4-74, respectively. The strain distribution is the same as for Bridges I and II. However, the strain levels in Bridge III should be higher than strain values of Bridge IV due to the prestressing level and ratio of diaphragm width to box girder height, the level of strains in case of Bridge III is nearly the same as strain values of Bridge IV. This happens because of the low value of strains in both bridges, the concrete tension stiffening, significantly affects the developed strains. The concrete contribution in resisting the splitting strains is higher when strain levels are low.
Finally, in order to compare the transverse strains of the diaphragm inner face at interior girders to those developed at exterior girders, a bar chart is drawn for the maximum strains developed at different levels (Top, Middle and Bottom) as shown in Figure 4-75. The strains developed at interior girders are always higher than those at exterior ones except at the top level of Bridge II. The reason for this exception is the low level of strain values at top level of Bridge II. Most of the strain values are less than the maximum tensile strain of concrete assumed earlier (150 microstrain). In this case the contribution of concrete is very high in resisting splitting forces than reinforcing bars. That's why this exception case
is not representative for the difference between interior and exterior girders strains. The average ratio of maximum transverse strain of interior to exterior girders for Bridges I, II, II and IV are 1.40, 1.68, 1.88 and 1.21, respectively. So the splitting forces developed at interior girders are higher than those at exterior girder by at least 20%.

Figure 4-75: Comparison between maximum transverse strains of the diaphragm inner face at interior as well as exterior webs of Bridges I to IV.

4.7.3 Diaphragm vertical strain distribution

Horizontal as well as vertical strain distribution are essential to understand prestressing force spreading within the diaphragm. The vertical strain distribution at inner face of the diaphragm at both interior as well as exterior girders are shown in Figures 4-76 and 4-77 for Bridge I and II, respectively. The exact locations of vertical bars labeled A-F for Bridge I and A-E for Bridge II are illustrated previously in Figures 4-16 and 4-20, respectively. The strain distribution is always maximum within the middle third of the diaphragm. The middle third represents the location of prestressing anchors. Maximum strain value of sister bar A in Bridge I is 2600 microstrain, which exceeds the yield strain for the used Grade 60 sister bars. The reason for this yielding is the presence of access opening between all webs in the diaphragm and the small value of diaphragm width. The
diaphragm inner face vertical strain values at interior girders are always higher than those at exterior ones. Also, strain values for Bridge I are higher than Bridge II values due to the same reasons explained in the previous section 4.7.1.

Based on the diaphragm vertical strain results for Bridges I and II at interior as well as exterior girders, it is noticed that stressing the other bridge end anchors causes slight increase in strain values. This increase for Bridge I is approximately 6.9% for interior girder and 6.8% for exterior one. However, for Bridge II, the increase is approximately 8.2% for interior girder and 7.7% for exterior one.

![Graph](image)

**Figure 4-76:** Vertical strain distribution at inner face of Bridge I diaphragm.
The vertical strain distribution at inner face of the diaphragm at both interior as well as exterior girders are shown in Figures 4-78 and 4-79 for Bridge III and IV, respectively. The exact locations of vertical bars labeled B-F for Bridge III and A-G for Bridge IV are illustrated previously in Figures 4-24 and 4-28, respectively. The strain distribution is always maximum within the middle third of the diaphragm as in Bridges III and IV. The diaphragm inner face vertical strain values at interior girders are slightly higher than those at exterior ones. The reason for small difference between interior and exterior girders is the low level of strain values, which is slightly higher than or equal to the tensile strain of concrete. In this case the contribution of concrete is high in resisting splitting forces than reinforcing bars. Tensile concrete resistance as well as tension stiffening play an effective role in reducing the reinforcement strain values.
Figure 4-78: Vertical strain distribution at inner face of Bridge III diaphragm.

Figure 4-79: Vertical strain distribution at inner face of Bridge IV diaphragm.
In order to compare the vertical strains of the diaphragm inner face at interior girders to those developed at exterior girders, a bar chart is drawn for the maximum strains developed at both girders as shown in Figure 4-80. The strains developed at interior girders are always higher than those at exterior ones. The average ratio of maximum vertical strain of interior to exterior girders for Bridges I, II, II and IV are 1.88, 1.59, 1.29 and 1.33, respectively. So the splitting forces developed at interior girders are higher than those at exterior girder by at least 20%.

Figure 4-80: Comparison between maximum vertical strains of the diaphragm inner face at interior as well as exterior webs of Bridges I to IV.

4.7.4 Web vertical strain distribution

Vertical strain distribution in the web has significant importance, as the only reinforcement value provided by Caltrans Memo to Designer (11-25) - Anchorage Zone Design [1] is the web reinforcement ahead of the anchorage zone. The vertical strain distributions at interior and exterior web girders of Bridge I are shown in Figure 4-81. The strain distribution is always maximum within the middle third of the height. The middle third represents the location of prestressing tendons. Maximum strain values of sister bar at 7” from the diaphragm face in Bridge I was 2000 microstrain, which is nearly the yield
strain for the used Grade 60 sister bars. The reason for this yielding was the presence of access opening between all webs in the diaphragm and the small value of diaphragm width. The vertical strain values of interior girder are significantly higher than those at exterior girder. The reason for this significant difference is the web width as the interior web width is only 12” but the exterior one is 20”. This difference caused by web flares increases the concrete contribution in case of exterior girder rather than interior one.

Based on the web vertical strain results for Bridges I at interior as well as exterior girders, it is noticed that stressing the other bridge end anchors causes slight increases in strain values. This increase is approximately 6.6% for interior girder and 6.8% for exterior one.

![Figure 4-81: Vertical strain distribution at web girders of Bridge I.](image)
The vertical strain distributions at web girders of Bridges II, III and IV are shown in Figures 4-82, 4-83 and 4-84, respectively. The strain values in all of the webs for the three bridges are relatively low compared to the concrete tensile strain, which magnify the role of concrete contribution in resisting vertical tensile bursting forces. In this case the contribution of concrete is higher in resisting splitting forces than reinforcing bars. Tensile concrete resistance as well as tension stiffening have an effective role in reducing the reinforcement strain values. In this case there is no significant difference in strains between exterior girder and interior girders. Also, the strain distribution along the webs is nearly constant. Also, strain values for Bridge I are higher than Bridges II, III and IV values due to ratio of diaphragm width to box girder height, presence of access opening in diaphragm and tension stiffening of concrete.

Figure 4-82: Vertical strain distribution at interior web girder of Bridge II.
The farther the measured vertical web strain from the diaphragm face, the lower its value is. This is clarified in the strain distribution of Bridge I in Figure 4-81. Strain values at 7” from the diaphragm face are higher than those at 23” or 43” for interior girder. Strains values at 9” from diaphragm face are higher than those at 25” or 45” for exterior girder. Also, the strain values are significantly reduced at 43” for interior girder or 45” for exterior girder. This reduction clarifies that the effective zone of the web along the bridge length is 50% of the box girder height (8’-8”) measured from the diaphragm face. This conclusions cannot be extracted from any of the remaining figures for vertical strain distribution of the web due to low strain levels for Bridges II, III and IV.
Figure 4-84: Vertical strain distribution at web girders of Bridge IV.
4.7.5 Web transverse strains

Transverse strains developed in the web girders caused by prestressing forces were captured using concrete gauges. These gauges were implemented in both interior and exterior girders for all of the investigated bridges. Two gauges were installed at each web girder one of them near to the diaphragm inner face and the other was distant from it. The exact locations of these gauge are discussed in section 4.4.2. A comparison between the maximum strains recorded using these concrete gauges is shown in Figure 4-85. The maximum tensile strain of concrete is sensitive to multiple parameters. As a rough assumption maximum tensile strain of concrete was set as 150 microstrain. From the bar chart, it can be noticed that none of strain values exceed the assumed value of concrete tensile strains.

Also, the distant gauge in exterior girder is subjected to compression strain. The reason for this compressive strains in the exterior girders is the in-plane curvature of exterior tendons. The exterior tendons are always curved with a concave centroid inside the box section. This horizontal in-plan curvature is created as the minimum clearance required for prestressing head plate is always larger than the edge distance from exterior girder center line to the bridge edge. This tendon curvature develops in-plan component of prestressing force causing transverse compression forces in the exterior webs. Finally, as resulting transverse concrete strains in web girders are less than tensile strain of concrete, transverse concrete strains in web girders are negligible.
4.7.6 Deck and soffit transverse strain distribution

Two transverse sister bars were implemented in the deck of Bridge I at the interior girder. The first one was installed 5” from the block-out and the other one was installed 2” from the inner diaphragm face. The strain distributions for both bars are shown in Figure 4-86. A plateau of maximum strains occurs at the web. The strain values within the deck beyond the diaphragm inner face are significantly higher than those ahead of the prestressing block-out.
Transverse sister bar was implemented in the deck of Bridge II at the interior girder 2” from the inner diaphragm face. The strain distribution along this bar is shown in Figure 4-87. Based on the deck transverse strain results for Bridge I and II, it is noticed that stressing the other bridge end anchors causes slight increase in strain values. This increase is approximately 6.7% in Bridge I and 8.5% for Bridge II.

![Figure 4-87: Transverse strain distribution of the deck at interior web of Bridge II.](image)

Transverse sister bars were implemented in the deck of Bridge III at both interior and exterior girders. The strain distributions along the implemented bars are shown in Figures 4-88 and 4-89 for interior and exterior girders, respectively. Due to the low level of strains captured, no accurate assessment can be done between exterior and interior girders results.

![Figure 4-88: Transverse strain distribution of the deck at interior web of Bridge III.](image)
In Bridge IV, three groups of transverse sister bars were implemented in the deck and soffit. The first two groups were identical bars placed in the deck as well as the exact same location in the soffit, in order to compare between them. The strain distributions along these bars are shown in Figure 4-90. The strain levels of reinforcement installed in the soffit were slightly higher than those in the deck. The third group of sister bars was implemented at different girders ahead of the prestressing block-out. The strain distribution of these bars is presented in Figure 4-91. The strain levels of these bars ahead of the block-outs are significantly lower than those implemented in the deck beyond the diaphragm face.
Based on the results obtained from the strains developed ahead of the prestressing block-outs for Bridge I and Bridge IV, the tensile strain at this location is lower than that in the deck beyond the diaphragm inner face. The lateral spreading of the strain developed ahead of the prestressing block-out is significantly less than the spreading beyond the inner diaphragm face.

4.8 Bridge III - End Diaphragm Cracking due to Post-tensioning

End diaphragm cracking was observed after post-tensioning for Bridge III. These cracks were not noticed during regular inspection because they occurred on the inner face of the diaphragm inside the box section. Skew angle of the end diaphragm is a governing factor affecting cracks distribution. This bridge has a skew end diaphragm with large block-outs. Prestressing block-outs are a part of concrete box section that is removed in order to be capable of stressing. The block-out dimension increases with the increase of skew angle. Increasing dimensions of these block-outs decreases the remaining effective thickness of the end diaphragm. Cracking of bridges especially during construction will affect bridge
serviceability life. Conventional design of the end diaphragm may cause cracking if the post-tensioning effects are not taken into account.

4.8.1 Post-inspection after prestressing

The instrumented diaphragm of Bridge III has two utility openings in the end diaphragm with dimension of 24” horizontal and 22” vertical. The total prestressing force of the bridge is 16,750 kips, which is considered a high value as presented in Chapter (3). This prestressing force represents 28.1% of the compressive strength capacity of the box girder concrete section.

After the bridge was constructed and post-tensioning forces were applied to the prestressing strands, the formwork of the whole bridge was removed. After this stage, regular inspection was done for all of the bridge elements and nothing significant was observed. In this stage as a part of the research project further inspection was done through the accessibility inspection openings in the soffit as shown in Figure 4-92. The webs as well as the inner face of the diaphragms were inspected. Cracks were observed in the inner face of the diaphragm as shown in Figure 4-93 [33]. Red lines in this figure are drawn at the crack locations. The accessibility inspection openings were available for two cells only of the bridge, between Girders B - C and Girders D - E. These cracks were observed on the obtuse side of the skew diaphragm. The width of these cracks ranges between 0.005” to 0.02” as presented in Figure 4-94. This figure shows the crack width relative to the concrete crack width gauge.
Figure 4-92: Access utility inspection openings in the soffit of Bridge III.

(a) Outside view  (b) Inner view

Schematic drawing for crack patterns observed during inspection

Cracks in cell between girders D and E  Cracks in cell between girders B and C

Figure 4-93: Developed cracks monitored during bridge inspection.

Figure 4-94: Cracks compared to concrete crack width gauge.
4.8.2 Reasons for developed cracks

Three main reasons were determined for the developed cracks in the inner face of the diaphragm. First, cracks were developed on the obtuse side of the diaphragm, where all tensile strains developed. Second, the main vertical crack line was developed at the part of the diaphragm with the smallest concrete section at prestressing block-out. Third, the prestressing force level in this bridge, as this box girder was subjected to relatively high value of prestressing force.

4.8.2.1 Tension side of the diaphragm

The transverse strain distribution on inner face of the diaphragm clarifies the presence of two opposite strains on both side of the web as shown previously for the transverse strain distribution in Figure 4-67. The acute side of the skew diaphragm is subjected mainly to compressive strains except the part directly adjacent to the web. The obtuse side of the skew diaphragm is subject to tensile strains only. As the compressive strength of concrete is significantly higher than its tensile strength, cracks only occurred on the obtuse side of the diaphragm.

4.8.2.2 Prestressing block-out size

Prestressing block-outs are a part of concrete section that is removed in order to be capable of stressing. These parts are relatively bigger in case of skew end diaphragms in order to develop enough surface perpendicular to the anchors. In this investigated bridge the skew angle of the end diaphragm was 41.13°. This skew angle was the reason to construct big block-outs as shown in Figure 4-95. The difference between the total
diaphragm width (4’) and the thickness at block-out (1’-10”) is clearly shown in the same figure.

Figure 4-95: Top view for prestressing block-out before casting the deck.

Skew angle of the end diaphragm affects the dimensions of block-outs required for post-tensioning, which leads to reduction in the diaphragm width. Vertical crack was initiated exactly at the smallest section of the diaphragm. The net effective thickness of the diaphragm should be considered in design of the anchorage zone of box girder bridges.

4.8.2.3 Value of prestressing force

The average value for prestressing force ratio obtained from the developed bridge data base was 0.2 and the maximum design level was 0.3. Bridge III has a prestressing force ratio of 0.281 as shown in Table 4-1. This value is considerably high compared to the maximum design level. It is obvious that prestressing force level significantly affects strains developed in the diaphragm, which causes cracking.
4.9 Findings and Observations

Based on the preliminary finite element models for the end anchorage zones, the critical zones of stress concentration due to prestressing were determined. The inner face of the diaphragm and approximately 50% of the girder height along the web are affected by bursting tension forces. The pattern of the transverse tensile stresses in the inner face of the diaphragm extends 30% of the girder spacing on both sides of the web.

Several issues were considered in choosing the type of strain gauges: suitable coating materials, cover for the gauge wires and installing techniques. Based on the field investigation results the following findings are determined:

- Developed transverse strains on the inner face of the diaphragm always increase towards the web centerline and decrease distant from it for both interior as well as exterior girders. The maximum strain values always occur at the web face, where cracks can occur at the smallest section.

- Major parameters affecting difference in strains between investigated bridges are level of prestressing force, ratio of diaphragm width to girder height, presence of opening in the diaphragm and concrete contribution in resistance based on strain level reached.

- Tensile strength of concrete affects the transfer of forces from concrete to reinforcing bars. The higher the strain level reached, the lower the effect of concrete contribution on the recorded strain of the reinforcement.

- Tension stiffening of concrete affects the transfer of forces from concrete to reinforcing bars. The higher the strain level reached, the lower the effect of tension stiffening on the recorded strain values.
• The vertical strain distribution on the inner face of the diaphragm and at the web is always maximum within the middle third of the height.

• Vertical web strains are inversely proportional with the distance from the diaphragm face. These strain values are significantly reduced at 50% of the girder height along the web, which coincide with the results of the preliminary finite element models. This reduction clarifies that the effective zone of the web is 50% of the box girder height along the bridge length.

• In case of stressing both ends of the bridge, stressing the second bridge end anchors causes slight increase in strain values for the first end. This increase is approximately 6.7% in Bridge I and 8.5% for Bridge II.

• In case of skew diaphragm, the obtuse side of the angle between the diaphragm and the web always possesses higher strain values than the acute side. The obtuse side has higher potential to cracking rather than the acute one. The higher the skew angle is, the larger difference in strains between obtuse and acute sides is developed.

• Transverse concrete strains in web girders are less than tensile strain of concrete. So these strains are considered negligible.

• The transverse exterior web forces distant from the diaphragm face are always compression. The reason for this compressive strains in the exterior girders is the in-plane curvature of exterior tendons. This in-plan horizontal curvature of the tendon develops transverse component of prestressing force, which causes transverse compression forces in the exterior webs.
• Developed transverse strains in the deck and soffit have a constant plateau of maximum strains that occurs at the web. The strain level of the reinforcing in the soffit is slightly higher than those in the deck.

• The strain levels of the bars ahead of the block-outs are significantly lower than those implemented in the deck beyond the diaphragm face.

• The diaphragm inner face transverse and vertical as well as the web vertical strain values at interior girders are higher than those at exterior ones. So the splitting forces developed at interior girders are higher than those at exterior girders by at least 20%.

• Openings in the box girder diaphragm affect the performance of end anchorage. These openings have significant effect on the continuity of stresses in end diaphragms as well as the strains developed in reinforcing bars.

Based on the cracking of the diaphragm of Bridge III the following findings are determined:

• Skew of the end diaphragm significantly affects transverse strain distribution within the inner side of the diaphragm causing tensile strains on the obtuse side and compressive strains on the acute side.

• Skew angle of the end diaphragm affects the dimensions of block-outs required for post-tensioning, which leads to reduction in the diaphragm width.

• The net effective thickness of the diaphragm should be considered in design of the anchorage zone of box girder bridges.

• Prestressing force value affects strains developed in the diaphragm, which causes cracking.
Chapter (5) – Analytical Work for Field Monitoring Results

5.1 Introduction

Recorded results from field investigation include strains in both sister bars and concrete elements as well as pressure values of the prestressing jack. The obtained data is presented in this chapter to show the different parameters affecting the general anchorage zone performance. The effect of diaphragm width on prestressing force spreading and the difference between developed bursting forces at both interior and exterior girders is shown.

Stressing sequence is examined in this chapter. It is shown that the stressing of one girder can impact the adjacent ones, and this effect is significantly influenced by the number of bridge girders. Three different methods are presented for distributing jacking force between different girders. Two of the investigated bridges are stressed from both ends. The effect of stressing one end on the bursting forces developed at the other end is discussed.

Finally, CSi Bridge [34] models are presented, which were developed in order to increase the number of data points for performance assessment of anchorage zones. The developed elastic models were verified using the obtained field results, then a parametric study was conducted to extend the field results. Furthermore, the developed models were used to verify the obtained relationships for the effect of stressing a certain girder on adjacent ones or the effect of stressing both ends. At the end of this chapter, concluded remarks are highlighted.
5.2 Effect of Diaphragm Width on Prestressing Force Spreading

The bridge geometry affects the spreading of prestressing forces. The main geometrical parameters observed from field monitoring affecting force spreading were diaphragm width \(b_d\) and box girder height \(h\). To examine the flow of forces, the average maximum strain values for the bars in a given location was converted to force knowing the amount of reinforcement at each investigated location. The main reason for using the average of the maximum strains is to minimize the concrete contribution in resisting tension forces. For the vertical direction of the diaphragm, the total reinforcement from centerline to centerline between girders in the inner face of the diaphragm was the amount considered. The total horizontal reinforcement in the inner face of the diaphragm including top and bottom bars in the diaphragm corners was used for determining the diaphragm transverse forces. The distributed reinforcement along the web for a length of \((h/2)\) was the range of reinforcement used to calculate vertical web bursting forces. For the deck, included range of reinforcement extended a distance equal to the girder spacing \(S\) along the bridge length. However, this method of obtaining force neglects the tensile strength and tension stiffening of concrete; it was considered acceptable to understand the performance and flow of forces within the general anchorage zones.

The obtained forces from the field strains in different locations and directions within the anchorage zone divided by the jacking prestressing force for both interior and exterior girders are plotted against the diaphragm width in Figure 5-1, and then against box girder height in Figure 5-2. Both figures show that the estimated bursting forces in different elements are inversely proportional with the diaphragm width and directly proportional
with the box girder height. Due to different configuration of the investigated bridges, these comparisons are considered not representable, but do show a general trend.

Figure 5-1: Relationship between percentage of estimated bursting forces and diaphragm width for both interior and exterior girders.

Figure 5-2: Relationship between percentage of estimated bursting forces and box girder height for both interior and exterior girders.

In order to provide a more realistic comparison for the investigated bridges, the ratio between diaphragm width and box girder height was considered. This ratio was also presented in the Memo to Designers 11-25 by Caltrans [1], where it has a minimum limit
of 0.3. The ratio between estimated bursting forces and the jacking prestressing force per girder is plotted against the ratio of diaphragm width to box girder height for both interior and exterior girders as shown in Figure 5-3. The figure shows that the estimated bursting forces in different elements are inversely proportional with the diaphragm width to box girder height ratio. This relationship highlights the role of diaphragm width in reducing the effect of prestressing bursting forces.

Estimated bursting forces developed in interior girder are higher than those developed at exterior girders as shown in Figure 5-3. Indicating that interior girders are more critical than exterior girders in resisting bursting forces.

Figure 5-3: Relationship between percentage of estimated bursting forces and ratio of diaphragm width to box girder height.

Another factor the spreading of forces is the bridge skew angle. The effective diaphragm width changes according to the skew angle of the bridge due to prestressing block-outs. The thickness of these prestressing block-outs can reach up to 50% of the diaphragm width. In order to define the effect of these block-outs, the effective diaphragm
width was considered. This effective diaphragm width \( b_{de} \) was defined as the distance measured from the diaphragm outer face at the anchorage centroid to the diaphragm inner face along the web longitudinal axis.

The ratio of estimated bursting forces is plotted against the ratio of effective diaphragm width to box girder height for both interior and exterior girders in Figure 5-4. It shows that the estimated bursting forces in the diaphragm transverse and vertical directions, web vertical direction and deck transverse direction are inversely proportional with the effective diaphragm width to box girder height ratio. The field investigated bridges had effective diaphragm width to box girder height of 0.29 to 0.72, and a range of total diaphragm width to box girder height from 0.31 to 0.89. The thicker the diaphragm, the lower the bursting forces.

![Diagram showing relationship between percentage of estimated bursting forces and ratio of effective diaphragm width to box girder height.](image)

**Figure 5-4:** Relationship between percentage of estimated bursting forces and ratio of effective diaphragm width to box girder height.
5.3 Comparison between Interior and Exterior Girders Bursting Forces

Based on the field investigation results, a detailed comparison was performed between the bursting forces developed at both interior and exterior girders. It was observed from Figures 5-1, 5-2, 5-3 and 5-4 that bursting forces of interior girders are higher than those of exterior girders. A comparison between percentage of estimated bursting forces for interior and exterior girders is presented in Figure 5-5. It includes the results in the diaphragm transverse and vertical directions as well as the web vertical direction for the investigated bridges. Also, the ratios of developed bursting forces between the interior and exterior girders are plotted on a secondary vertical axis. This ratio has wide variation between 1.15 to 2.66 for the different investigated bridges due to the concrete contribution in resisting bursting forces as well as the level of the recorded strains. The interior girders bursting forces are always higher than bursting forces of exterior girders by at least 15% based on the minimum value obtained from the developed chart.

Two reasons were determined for the lower values of bursting forces in exterior girders than interior girders. First, the exterior web girders are thicker than the interior ones for all investigated bridges. Second, the in-plane horizontal curvature of exterior tendons provides a transverse compression force in the diaphragm, deck and soffit as shown in Figure 5-6. The component of the applied prestressing force and the tensile force in the strands develop an inward transverse force that decreases the effect of bursting forces. This force counteract the bursting forces developed in the diaphragm, deck and soffit transverse direction.
Figure 5-5: Comparison between percentage of estimated bursting forces for exterior and interior girders for different investigated bridges.

Figure 5-6: Horizontal plan for exterior girder showing developed inward compression force in the diaphragm decreasing bursting forces.
Based on these results the bursting forces developed at interior girders are considered more critical than those developed at exterior ones. The interior girders bursting forces are always higher than those of exterior girders by at least 15%.

5.4 Effect of Stressing Sequence

Anchorage zone design of box girder bridge is a shared responsibility between the post-tensioning contractor and the bridge designer. The anchorage distribution between girders, ducts sizes, number of strands per duct, local anchorage zone detailing and stressing sequence are determined by post-tensioning contractor. However, the bridge designer is responsible for the general anchorage zone including diaphragm, web, deck and soffit at the end of the box girder adjacent to prestressing anchors. The designer has to depend only on the target design force to design the general zone. According to AASHTO LRFD Bridge Design Specifications [3], the design ultimate force for post-tensioning anchorage zones shall be taken as 1.2 times the maximum jacking force. However, no statements are available to show how to distribute these forces between interior and exterior girders or what is the effect of stressing adjacent girders on each other. Through the data obtained from the field investigation, these effects were investigated and are described in the subsequent subsections.

5.4.1 Stressing sequence rearrangement method

In order to obtain a correlation between prestressing force and developed strains, two procedures were implemented. First, normalization of prestressing forces and developed strains, then rearranging the stressing sequence to get the force causing total strain in the investigated girder. Since the monitored bridges have different prestressing forces, stress
levels and dimensions, the resulting strains are not directly comparable. In order to be capable of comparing the results both strain values and prestressing forces were normalized. Two new parameters were developed, the strain ratio and the Prestressing Force Contribution Ratio (PCR). The strain ratio is defined as the ratio between resulting strain and its maximum value. The PCR is the prestressing force divided by the maximum value for the whole box girder. All strain ratios and PCRs range between 0 and 1.0 as shown in Figure 5-7 for average strain values for the diaphragm and the web at exterior girder of Bridge I.

Figure 5-7: Relationship between average strain ratio and PCR for exterior girder of Bridge I before and after rearranging stressing steps.

After normalization of the results, rearranging of the stressing steps is done. The stressing sequence is modified by starting with steps applied at the investigated girder followed by the adjacent girders and the remaining stressing steps of the box girder at the end. The stressing sequence, number of strands per tendon and tendon forces for Bridge I are shown previously in Figure 4-3. So, in order to rearrange the stressing steps for external
girder D, the initial steps (3) and (7) are applied on the investigated girder itself, followed by steps (2), (6) and (10) of the adjacent Girder C. Then, the remaining steps (1), (5), (9), (4) and (8) are added to the sequence as illustrated in Figure 5-7. This figure clarifies that the effect of stressing steps applied on the girder itself leads to more strain ratio than applying the load on adjacent girders or the remaining girders of the bridge. The slope of the tangent line in this curve represents the rate of strain ratio progression at the anchorage zone to the PCR. Stressing steps (3) and (7) applied on the investigated exterior girder, develop strain ratio of 0.48; the remaining steps led to 0.52 of the strain ratio. By the end of this section a proposed value for the PCR is developed based on the field results of the monitored bridges, which is presented in all of the prepared charts.

To summarize the idea of rearranging the stressing steps, reordered sequences for interior Girder C and exterior Girder D of Bridge I are shown in Table 5-1. This table illustrates the original stressing sequence and the modified sequence for the investigated interior and exterior girders. The stressing steps numbering for the investigated girder is presented in a bold font, the stressing steps for adjacent girders are underlined and the remaining steps are in regular form. This presentation technique is implemented in all of the rearrangement stressing sequence tables.

<table>
<thead>
<tr>
<th>Original stressing sequence</th>
<th>Rearranged sequence for interior Girder C</th>
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<tr>
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<td>2 – C</td>
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<tr>
<td>Original stressing sequence</td>
<td>Rearranged sequence for interior Girder C</td>
<td>Rearranged sequence for exterior Girder D</td>
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<tr>
<td>10</td>
<td>8 – A</td>
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</tr>
</tbody>
</table>

5.4.2 Bridge I field results processing

The rearranged stressing sequences for interior Girder C and exterior Girder D of Bridge I are presented in Table 5-1. For interior Girder C, loading is rearranged starting from the investigated Girder C, followed by the adjacent Girders D and B, then the remaining Girder A. However, for exterior Girder D, loading is rearranged starting from the investigated Girder D, followed by the adjacent Girders C, then the remaining Girders B and A.

After rearranging stressing sequence, relationship between strain ratio propagation with PCR for Bridge I is shown in Figure 5-8. It includes average strain ratio for the diaphragm inner face transverse and vertical directions and web vertical direction for both interior Girder C and exterior Girder D. Deck transverse direction average strain ratio is presented for interior Girder C, as well. This figure clarifies that the effect of stressing steps applied on the girder itself leads to more strain ratio than applying the load on adjacent girders or the remaining girders of the bridge. Stressing steps applied on the interior investigated Girder C itself develop strain ratio of 0.58; the remaining steps led to 0.42
strain ratio. Stressing steps applied on the exterior investigated Girder D itself develop strain ratio of 0.45; the remaining steps led to the 0.55 strain ratio.

These plots elaborate that applying prestressing force on a certain girder has major effect on the developed strains; however, strain propagation due to loading remaining girders cannot be neglected. In order to make use of these relationships, the average tangent for loading the investigated girder itself is plotted as a relationship between strain ratio and PCR.

The implemented tangent represents the force required to be applied at a certain girder to attain the total strain developed from loading the whole box girder. This concept is used to obtain an adequate PCR for both exterior and interior girders of box section in
service state. The reason that developed total strain at a certain anchorage zone does not occur during stressing the same girder is the group action effect of girders as well as service state of loading the bridge. During stressing a certain girder in the bridge, the axial force within the girder increases but it does not match the applied prestressing force. As portion of this applied force is distributed to the remaining girders. That’s why the group action affects the developed strains in the anchorage zone. The service state performance is totally different than ultimate state. During the service state, the concrete contribution cannot be neglected due to concrete tensile strength and tension stiffening. However, in ultimate state, concrete tensile contribution will be set to zero and no transfer of loads will happen between girders.

5.4.3 Bridge II field results processing

The same procedure was performed for Bridge II using the average of the recorded data. After normalization of the results, rearranging stressing steps was done. The stressing sequence was modified by starting with steps applied at the investigated girder followed by the adjacent girders and the remaining ones. Stressing sequence, number of strands per tendon and tendon forces for Bridge II are shown previously in Figure 4-5. The rearranged stressing sequences for interior Girder B and exterior Girder A of Bridge II are presented in Table 5-2. For interior Girder B, loading was rearranged starting from the investigated Girder B, followed by the adjacent Girders A and C, then the remaining Girder D. However, for exterior Girder A, loading was rearranged starting from the investigated Girder A, followed by the adjacent Girder B, then the remaining Girders C and D.
Table 5-2 Rearranged stressing sequence for interior Girder B and exterior Girder A of Bridge II.

<table>
<thead>
<tr>
<th>Original stressing sequence</th>
<th>Rearranged sequence for interior Girder B</th>
<th>Rearranged sequence for exterior Girder A</th>
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<tbody>
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</table>

The relationship between strain ratio propagation with PCR for Bridge II is shown in Figure 5-9. It includes average strain ratio for the diaphragm inner face transverse and vertical directions and web vertical direction for both interior Girder B and exterior Girder A. Deck transverse direction average strain ratio is presented for interior Girder B. The slope of the tangent line in this curve represents the rate of developing strain at the anchorage zone to the PCR. PCR required to develop final strain on the interior investigated Girder B is 0.51 and on the exterior investigated Girder A is 0.48.
5.4.4 Bridge III field results processing

The same procedure was performed for Bridge III using the average of the recorded data. Stressing sequence, number of strands per tendon and tendon forces for Bridge III are shown previously in Figure 4-7. The rearranged stressing sequences for interior Girder B and exterior Girder A of Bridge III are presented in Table 5-3. For interior Girder B, loading is rearranged starting from the investigated Girder B, followed by the adjacent Girders A and C, then the remaining Girders D, E and F. However, for exterior Girder A, loading is rearranged starting from the investigated Girder A, followed by the adjacent Girders B, then the remaining Girders C, D, E and F.
Table 5-3 Rearranged stressing sequence for interior Girder B and exterior Girder A of Bridge III.

<table>
<thead>
<tr>
<th>Original stressing sequence</th>
<th>Rearranged sequence for interior Girder B</th>
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<td>1</td>
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After rearranging stressing sequence, relationship between strain ratio propagation with PCR for Bridge III is shown in Figure 5-10. It includes average strain ratio for the diaphragm inner face transverse direction for both compression as well as tension values, diaphragm vertical directions, web vertical direction and deck transverse direction for the interior Girder B. While for exterior Girder A, it contains the average strain ratio for the diaphragm inner face transverse and vertical directions, web vertical direction and deck.
transverse direction. PCR ratio required to develop final strain in the interior investigated Girder B is 0.34 and in the exterior investigated Girder A is 0.31.

Figure 5-10: Strain ratio propagation with PCR after stressing steps rearrangement for Bridge III.

5.4.5 Bridge IV field results processing

The same normalization of the results as well as rearranging of stressing steps procedures were performed for Bridge IV using the average of the recorded data. Stressing sequence, number of strands per tendon and tendon forces for Bridge IV are shown previously in Figure 4-8. The rearranged stressing sequences for interior Girder B, C and D as well as exterior Girder A of Bridge IV are presented in Table 5-4. For exterior Girder A, loading was rearranged starting from the investigated Girder A, followed by the adjacent Girder B, then the remaining Girders C to L. For interior Girder B, loading was rearranged starting from the investigated Girder B, followed by the adjacent Girders A and C, then the
remaining Girders D to L. On the other side, rearranging for interior Girder C begins with the investigated Girder C, followed by the adjacent Girders B and D, then the remaining Girders A and E to L. While for interior Girder D, loading was rearranged starting from the investigated Girder D, followed by the adjacent Girders C and E, then the remaining Girders B, F, A and G to L.

Table 5-4 Rearranged stressing sequence for interior Girders B, C and D as well as exterior Girder A of Bridge IV.

<table>
<thead>
<tr>
<th>Original stressing sequence</th>
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</table>
After rearranging stressing sequence, relationship between strain ratio propagation with PCR for Bridge IV is shown in Figure 5-11. It includes average strain ratio for the diaphragm inner face transverse as well as vertical directions and web vertical direction for the exterior Girder A and exterior Girder D. Also, it contains the average strain ratio for the diaphragm inner face transverse direction and web vertical direction for the interior Girder B as well. Then, average strain ratio for the diaphragm inner face transverse and vertical directions, web vertical direction and deck as well as soffit transverse directions for the interior Girder C are presented. These figures clarify that the effect of stressing steps applied on the girder itself leads to more strain ratio than applying the load on adjacent girders or the remaining girders of the bridge. The slope of the tangent line in this curve represents the rate of developing strain at the anchorage zone to the PCR.
Figure 5-11: Strain ratio propagation with PCR after stressing steps rearrangement for Bridge IV.
5.4.6 Prestressing Force Contribution Ratio (PCR) for both interior and exterior girders

Based on the developed relationships between strain ratio propagation and PCR for different girders of the investigated bridges, an idealized relationship is plotted in Figure 5-12 in order to provide sufficient illustration for the performance. This relationship is divided into three zones. First zone includes the PCR through loading the investigated girder itself and the corresponding strain ratio. The developed curve in this zone has steep slope representing high contribution of stressing the investigated girder on the strain ratio. The slope of this zone is defined by the 1st tangent line. Second zone defines the relationship between PCR and strain ratio during stressing of the adjacent girders. The slope of this zone is medium representing the moderate effect of stressing the adjacent girders on the developed strain ratio. This moderate effect is presented by the 2nd tangent line. The third zone includes the effect of stressing the remaining girders on the strain ratio. Last part of the idealized curve has a mild slope represented by the 3rd tangent line.

![Idealized strain ratio propagation with PCR](image-url)

Figure 5-12: Idealized strain ratio propagation with PCR.
Idealized relationship plot elaborates that applying prestressing force on a certain girder has the major effect on the developed strains; however, strain propagation due to loading remaining girders is at a lower rate. The 1st tangent line is used to calculate a representative PCR causing the full strain ratio as shown in Figure 5-12. This tangent represents PCR required to be applied at a certain girder to attain the total strain developed from loading the whole box girder. This value is defined as Equivalent Prestressing Contribution Ratio (EPCR). Using the plotted tangents, an adequate values for EPCR for both interior and exterior girders of box section are presented in equations 5-1 and 5-2, respectively. The developed equation for calculating EPCR is based on the number of girders (n).

\[ EPCR_{\text{Interior}} = 0.03 + \frac{1.85}{n} \quad \text{Eq. 5-1} \]

\[ EPCR_{\text{Exterior}} = 0.02 + \frac{1.75}{n} \quad \text{Eq. 5-2} \]

The proposed EPCR values are plotted for comparison with the field investigation results in Figures 5-7 to 5-11 using a vertical red line. In all of these previous relationships the difference between the 1st tangent and the proposed value for EPCR did not exceed 15%. These proposed equations for EPCR is compared to the field investigation results as shown in Figure 5-13. This bar chart presents the EPCR obtained from the average 1st tangent for the field results and the proposed EPCR value for both interior and exterior girders of the investigated bridges. As the proposed equations are based on number of girders, the number of girders for each of the investigated bridges is provided at the bottom of the chart. EPCR obtained from field results includes strains developed in the diaphragm transverse and vertical directions, web vertical direction, deck transverse direction and the
average of all data for each of the investigated bridges. At each location, the average strain from several strain gauges was used. Results of the proposed equations provide a good match with the average results, which gives an indication for equation adequacy.

A normal distribution for variation between field investigation results and proposed EPCR is plotted in Figure 5-14. This figure presents the frequency of the field investigation results in a form of a bar chart. It clarifies that the results provide an acceptable bell shaped normal distribution. The coefficient of determination ($r^2$) obtained between the collected data points and the proposed equation is 0.97, which indicates that the proposed equations for EPCR are in a good fit with the obtained field results.
5.5 Methods for Jacking Force Distribution between Girders

Three methods for distribution of jacking force between girders are presented in this section. Design of post-tensioned box girder bridges depends on checking service stresses to satisfy code requirements under different loading condition. A uniform stresses distribution applied on a rigid box section is considered. This assumption is valid actually in most cases, but what will happen in case of having large number of girders with unequal forces applied on each girder? This assumption will not be valid. Based on observation of bridges shop drawings provide by Caltrans, variation between girders jacking forces did not exceed 10%. Therefore, distribution of jacking force equally between girders with maximum variation of 10% is the first distribution method.

The second method for distribution of the forces is to apply jacking force on each girder based on the tributary area of the box section it supports. The interior girder tributary area is the one enclosed by the centerline between girders, while for exterior girder, it is...
the area bounded by the centerline between girders and the edge of the box section including overhang. This method will minimize the routes of force trajectories needed to satisfy uniform distribution within the box section. That will also decreases the strains developed within the anchorage zone including diaphragm, web, deck and soffit.

As EPCR represents PCR required to be applied at a certain girder to attain the total strain developed from loading the whole box girder. EPCR can be used to determine the amount of jacking force applied on each individual girder. EPCR is calculated for interior as well as exterior girders, then it is normalized to get the applied force on each girder as presented in equations 5-3 and 5-4. Based on this technique, the bursting force developed in the anchorage zone will be minimized and the uniform stress distribution within the box section will be satisfied.

\[
P_{J_{\text{Interior}}} = P_{J} \frac{EPCR_{\text{Interior}}}{2EPCR_{\text{Exterior}}+[(n-2)EPCR_{\text{Interior}}]} \quad \text{Eq. 5-3}
\]

\[
P_{J_{\text{Exterior}}} = P_{J} \frac{EPCR_{\text{Exterior}}}{2EPCR_{\text{Exterior}}+[(n-2)EPCR_{\text{Interior}}]} \quad \text{Eq. 5-4}
\]

The three methods are compared using two example of prototype bridges with four and twelve girders. The dimension of the bridge cross-section is illustrated previously in the prototype specimens developed in Chapter (3) – Figure 3-15. The results of these examples are illustrated in Figures 5-15 and 5-16. These figures present the percentage of jacking force per girder for the three methods. Also, the tributary area for each girder is illustrated using a secondary vertical axis in the same chart. In both figures, the percentage of jacking force distributed on exterior girder is lower than that distributed on interior girder in case of using tributary area method or EPCR.
In order to investigate the applicability of using tributary area or EPCR methods for jacking force distribution, the percentage of variation between the investigated methods is plotted in Figure 5-17 for four and twelve girders. This figure illustrates that both tributary area and EPCR methods satisfy Caltrans typical method that the variation between girders jacking forces shall not exceed 10%.

As long as both tributary area and EPCR methods for distributing jacking force possess theoretical or experimental concepts, these methods are more realistic to be applied...
in case of distributing jacking force rather than equal distribution of forces between girders. This will minimize the routes of force trajectories needed to satisfy uniform distribution within the box section and will also decrease the strains developed within the anchorage zone including diaphragm, web, deck and soffit.

![Percentage of variation between different methods for jacking force distribution in case of using four or twelve girders.](image)

**Figure 5-17**: Percentage of variation between different methods for jacking force distribution in case of using four or twelve girders.

### 5.6 Effect of Stressing Both Bridge Ends

Two of the investigated bridges are post-tensioned from both ends. After stressing the first end and recording the developed strain data at this end, the propagation of strains due to stressing the other end was recorded. The only types of prestressing losses contributing through stressing the second end are friction losses, anchorage seating loss and elastic shortening. Anchorage seating is neglected because the maximum jacking force is obtained just before seating. Anchorage seating loss are typically a smaller value compared to friction losses and elastic shortening due to bridges having long tendons. As the box girder section was already stressed from the first end, the effect of elastic shortening can be neglected while considering the strain propagation due to stressing at the end that was initially stressed.
Friction losses are calculated based on the methodology presented in AASHTO LRFD Bridge Design Specifications [3] (5.9.5.2.2b-1) as shown in equation 5-5. The defining parameters for this equation are prestressing stress at jacking, wobble friction coefficient and prestressing duct coefficient of friction. The summary of friction losses calculations for both bridges is shown in Table 5-5. This table clarifies that Bridge I has longer span length of 258.8’ than Bridge II with a span length of 145.4’, which is the reason that Bridge I has a ratio of friction losses equal to 40.6% while Bridge II only has 26.9%.

\[
\Delta f_{pF} = f_{pj} \left( 1 - e^{- (Kx + \mu \alpha)} \right)
\]

Eq. 5-5 [3]

Where:

\( \Delta f_{pF} \) = loss due to friction (ksi)

\( f_{pj} \) = stress in the prestressing steel at jacking (ksi)

\( x \) = length of a prestressing tendon from the jacking end to any point under consideration (ft)

\( K \) = wobble friction coefficient (per ft of tendon)

\( \mu \) = coefficient of friction

\( \alpha \) = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad.)

\( e \) = base of Napierian logarithms
Table 5-5 Defining parameters for friction losses for both bridges I and II.

<table>
<thead>
<tr>
<th></th>
<th>x (ft)</th>
<th>K (per ft)</th>
<th>μ</th>
<th>α</th>
<th>Δf_pF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bridge I</strong></td>
<td>258.8</td>
<td>0.000656</td>
<td>0.2</td>
<td>1.754</td>
<td>0.406</td>
</tr>
<tr>
<td><strong>Bridge II</strong></td>
<td>145.4</td>
<td>0.000656</td>
<td>0.15</td>
<td>1.450</td>
<td>0.269</td>
</tr>
</tbody>
</table>

The increase of strains due to stressing the other end are compared to friction losses along the whole bridge length. In order to compare the percentage of strain increase in different elements of the anchorage zone with the friction losses for both bridges, a bar chart is plotted in Figure 5-18. It includes the strain variation ratio for the diaphragm transverse and vertical directions, web vertical direction and deck transverse direction for interior girders of both bridges. It also includes the strain variation ratio for the diaphragm transverse and vertical directions and web vertical direction for exterior girders. In order to understand the effect of friction losses on the strain variation, the ratio of friction losses is plotted on a secondary vertical axis. As only two of the investigated bridges were stressed from both ends, only two data points are captured illustrating this performance. Having only two data points is not enough to develop a representative equation, a finite element model is developed to capture the same performance. The following section introduces the developed CSi Bridge 2016 [34] model and its verification.
Figure 5-18: Percentage of strain propagation due to stressing the other end compared to ratio of friction losses for both Bridges I and II.

5.7 Finite Element Model Development

Elastic finite element model was developed using CSi Bridge 2016 [34] in order to provide more data points assessing the bridge global performance. The developed model is verified using the field monitoring results for strain ratio and PCR relationship as well as the strain propagation due to stressing both ends. More data points are developed using the verified model, which are used to validate the developed equations of EPCR for both interior as well as exterior girders. Furthermore, the verified model is used to develop an equation for the effect of stressing both bridge ends.

5.7.1 Modeling verification

Defining parameters for the investigated bridges are illustrated previously in Chapter (4). These defining parameters including geometry, prestressing forces, concrete properties, and number of prestressing ends are used to develop the CSi Bridge models. Elastic models can be representative at this stage of loading due to low level of concrete cracking that occurs for the whole box girder during the post-tensioning stage. Even observed cracks at the end diaphragm just after construction were localized, which did not
affect the global performance. Extruded view for the developed CSi Bridge model is shown in Figure 5-19 for Bridge II. Area object model was used to define the box girder section; however, frame elements were used to define the columns and bent caps. Area object model was used for the box girder in order to capture the effect of stressing each individual girder on the adjacent ones.

Figure 5-19: Extruded view of CSi Bridge model developed for Bridge II.

The model was verified through the relationship between the developed strain ratios and PCR for the monitored bridges. Prestressing force for each individual girder was defined in separate load cases. The prestressing force applied at each girder as a ratio of the total applied prestressing force on the whole box section was considered as PCR. However, the developed axial force in each load case at the investigated girder interior or exterior represented the strain. This value of axial force at each girder was divided by the final axial force developed on the investigated girder to demonstrate the strain ratio.

The verification results for interior and exterior girders for Bridge I and II are shown in Figures 5-20 and 5-21, respectively. The relationships between strain ratio and PCR show good correlation between model and field investigation results including diaphragm transverse and vertical directions, web vertical direction and deck transverse direction. However, the results slightly deviate between the model and the field results within the
zone of adjacent girders and the remaining ones, the 1\textsuperscript{st} tangent zone for the investigated girder itself provides good matching between model output and field results. This 1\textsuperscript{st} tangent is the parameter used to obtain the EPCR for each girder. The main reason for this slight deviation between the model and the field results is the elastic properties considered for model elements. The actual strain values measured in the field depend on concrete contribution in resisting strains, concrete tensile strength, tension stiffening of concrete and level of measured strains.

Figure 5-20: CSi Bridge model verification for field results of Bridge I.

Figure 5-21: CSi Bridge model verification for field results of Bridge II.
The verification results for interior and exterior girders for Bridge III and IV are shown in Figures 5-22 and 5-23, respectively. For Bridge IV, interior girders B, C and D are included in addition to exterior Girder A. The relationships between strain ratio and PCR show good correlation between model and field investigation results including diaphragm transverse and vertical directions, web vertical direction and deck transverse direction. The same good correlation is observed in the investigated girder zone and slight deviation is observed in the adjacent and remaining girders zones.

Figure 5-22: CSi Bridge model verification for field results of Bridge III.
Figure 5-23: CSi Bridge model verification for field results of Bridge IV.

In order to verify the effect of stressing both ends, prestressing cables were defined in the model using a new technique. Due to unavailability in CSi bridge software for a method to capture the performance after stressing each of the bridge ends individually, a new modeling technique was developed. Two identical cables were implemented as frame elements in the model inside the web girders. These cables possess the same area, properties and profile. The applied prestressing forces in the cables were defined in two separate loading cases, which facilitates obtaining the results for each case individually. Each one of these cables was stressed from one end only. The prestressing forces applied
in these cables were calculated based on the elongation required to compensate the losses. The elongation was calculated assuming that the force within the tendon equals the average prestressing force expected on both ends. This is the same technique used by Caltrans to calculate required elongation during construction. The first identical cable stressed from the first bridge end will have higher prestressing force, representing the elongation that will occur in the first end. While, the second identical cable will have smaller prestressing force representing the elongation that will occur at the second stressing stage.

After applying these prestressing forces, the axial force increases due to stressing the second end was determined using the individual load cases defined for each cable. The same methodology was used for both Bridges I and II, which were stressed from both ends. The increase in axial force representing the increase in strain was compared to the field results as shown in Figure 5-24. This figure elaborates the adequacy of the developed modeling strategy to incorporate the effect of stressing both end causing increase in the strain levels applied at the anchorage zone.

Figure 5-24: Comparison between field investigation and CSi bridge model results with respect to strain increase due to stressing both ends for both Bridges I and II.
5.7.2 Model sensitivity

Model sensitivity is essential in order to validate the model for investigating a certain parameter. In order to validate the developed equation for EPCR, other parameters including deck thickness, web thickness, girder spacing, tendon profile, diaphragm width and span length girders were investigated. The prototype model is built based on the prototype bridge section developed previously in Chapter (3). The model included four girders and single span with total length of 150’. The investigated parameters are summarized Table 5-6, where the model labels are presented as well as the modification done in each model rather than the prototype. The reasons for each modification is also clarified in this table.

Table 5-6 Developed models for sensitivity study.

<table>
<thead>
<tr>
<th>Model Label</th>
<th>Modified Parameter</th>
<th>Reason</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prototype</td>
<td>Model developed based on the prototype section developed previously</td>
<td>Base model for comparison with other investigated parameters</td>
</tr>
<tr>
<td>1</td>
<td>Deck thickness is two times prototype thickness</td>
<td>Changing deck thickness modifies the section centroid, also it represents any difference between deck and soffit thicknesses</td>
</tr>
<tr>
<td>2</td>
<td>Web thickness is 1.5 times prototype thickness</td>
<td>Changing web thickness modifies the area and inertia between of the bridge section</td>
</tr>
<tr>
<td>3</td>
<td>Girder spacing equals box girder height</td>
<td>Changing girder spacing from 1.55 h to be 1.0 h affects the aspect ratio of the box cell</td>
</tr>
<tr>
<td>4</td>
<td>Girder spacing equals two times box girder height</td>
<td>Changing girder spacing from 1.55 h to be 2.0 h affects the aspect ratio of the box cell. This model clarifies the difference between wide and narrow box girder cell.</td>
</tr>
<tr>
<td>Model Label</td>
<td>Modified Parameter</td>
<td>Reason</td>
</tr>
<tr>
<td>-------------</td>
<td>------------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>5</td>
<td>Straight tendons are used instead of curved tendon profile</td>
<td>Changing the cable profile is one of the investigated parameters</td>
</tr>
<tr>
<td>6</td>
<td>Diaphragm width equals box girder height</td>
<td>Changing diaphragm width from 0.5 h to 1.0 h is a main concern in the sensitivity study.</td>
</tr>
<tr>
<td>7</td>
<td>Bridge span length is 100’</td>
<td>Changing span length for 150’ to 100’ is a significant parameter for investigation.</td>
</tr>
<tr>
<td>3 Girders</td>
<td>Three web girders are used in this model</td>
<td>Using three web girders instead of four is the main parameter for investigation, which affects EPCR</td>
</tr>
<tr>
<td>5 Girders-A</td>
<td>Five web girders are used in this model, the first interior girder is considered</td>
<td>Using five web girders instead of four is the main parameter for investigation, which affects EPCR</td>
</tr>
<tr>
<td>5 Girders-B</td>
<td>Five web girders are used in this model, the middle interior girder is captured</td>
<td>Capturing the model results through the middle interior girder instead of the first interior one shall be considered to obtain the difference</td>
</tr>
</tbody>
</table>

The relationship between strain ratio and the PCR for all of the sensitivity study models are plotted in Figures 5-25 and 5-26 for both interior and exterior girders, respectively. In both figures, the parameters included in models 1 to 7 do not have significant effect on the EPCR compared to the prototype model. However, in few case there is slight deviation between the developed model and the prototype, this deviation does not exist in the 1st tangent zone used to obtain the EPCR. Based on these figures, the investigated parameters including deck thickness, web thickness, girder spacing, tendon profile, diaphragm width and span length do not have significant effect on EPCR.
Figure 5-25: CSi bridge models sensitivity study for interior girders.

In case of using three girders or five girders instead of four girders in the prototype model, the results deviate from the other models. This is observed in both interior as well as exterior girders. The tangent in the first zone clearly shows the variation due to the number of girders. As this tangent represents the EPCR for each individual girder, the more the number of girders is, the lower the value of EPCR.

One more parameter was included in the sensitivity study was examining different interior girders. In case of using five girders, results from two interior girders are
monitored. The first and the middle interior girders in model 5 Girders – A and B are presented in Figure 5-25. The comparison between both interior girders clarifies that the different interior girders do not affect the EPCR for the bridge.

All of these parameters included in the sensitivity study improve the confidence in the developed model to assess the effect of stressing bridge girders on each other or the effect of stressing both bridge ends. Also, it enhances the confidence level in the data points developed to validate the EPCR equations or to build an equation for the effect of stressing both bridge ends.

5.7.3 Model results

The parametric study developed using this elastic CSi Bridge model includes two parameters: changing number of girders in order to validate the developed equation for EPCR and changing friction losses in order to propose an equation for the effect of stressing both bridge ends. More models were developed to increase number of girders up to eighteen girder. Also, the friction loss was investigated starting from 10% to 70% of the jacking force.

5.7.3.1 Validation of EPCR equation

One of the main reasons for developing this elastic CSi model is to validate and verify the proposed equation for EPCR. This EPCR presents the PCR value applied at a certain girder to attain the total strain developed from loading the whole box girder. Equations 5-1 and 5-2 were developed to define the EPCR for both interior and exterior girders, respectively. Several CSi Bridge models were developed using different number of girders. Number of girders ranged from two to eighteen. The results of the created CSi Bridge
models are compared to the proposed EPCR equation as well as the field results as shown in Figure 5-27. This figure presents the verification for both interior as well as exterior girders.

Figure 5-27: CSi Bridge models results compared to proposed equation for EPCR as well as field results for both interior and exterior girders.

Based on the results of this figure, the proposed equations for the EPCR provide a good correlation with both results obtained from the field monitoring as well as CSi bridge models. The coefficient of determination ($r^2$) obtained between the developed models and the proposed equation is 0.99 for both interior as well as exterior girders, which elaborates that the proposed equations are a good fit with the results.

5.7.3.2 Effect of stressing both bridge ends

Based on the CSi bridge models developed to capture the performance of stressing both ends of the box girder, several data point were obtained. The variable in each of the developed models is the friction losses by modifying the wobble friction coefficient $(K)$ and the duct coefficient of friction $(\mu)$. As strain propagation due to stressing the other end is inversely proportional with friction losses, the same correlation was obtained using CSi
Bridge models as shown in Figure 5-28. This figure presents the relation between amplification factor due to stressing both ends ($\beta_{\text{end}}$) and friction losses. The amplification factor due to stressing both ends is the magnification at the initial stressing end due to stressing the opposite end. The friction losses were calculated based on the AASHTO equation (5.9.5.2.2b-1) presented in equation 5-3.

\[
\beta_{\text{end}} = 1.117 - \left[ 0.117 \left( 1 - e^{-(Kx + \mu \alpha)} \right) \right] \quad \text{Eq. 5-6}
\]

The developed equation is presented in Figure 5-28. Based on the results of this figure, the proposed equation for $\beta_{\text{end}}$ provides a good correlation with both results obtained from the field monitoring as well as CSi bridge models. The coefficient of determination ($r^2$) obtained between the developed models and the proposed equation is 0.99, which elaborates that the proposed amplification factor is in a good fit with the results.
5.8 Concluded Remarks

Based on the analytical work performed on the field investigation results the following remarks can be concluded:

- Bursting forces in different elements are inversely proportional with the effective diaphragm width to box girder height ratio. This relationship highlights the role of diaphragm width in impacting the magnitude of the bursting forces caused by spreading.

- The interior girders bursting forces are always higher than those of exterior girders by at least 15%. This highlights that interior girders are more critical than exterior girders in resisting bursting forces.

- The low values of the bursting forces in exterior girders is due to the thicker exterior web girder than interior ones for all investigated bridges. Also, the in-plane horizontal curvature of exterior tendons implement transverse compression force in the diaphragm, deck and soffit. The compressive component of the applied prestressing force develops an inward force that decreases the effect of bursting spreading forces.

- At the service stage, the total strain at a giving anchorage zone is caused by the stressing of the anchorages in that girder as well as the anchorages in other girders.

- The relationship between strain ratio propagation and PCR for different girders of the monitored bridges is divided into three zones. First zone has a steep slope representing high contribution of stressing the investigated girder on the strain ratio. Second and third zones have moderate and mild slopes representing the effect of adjacent and remaining girders on the strain ratio.
• Equivalent Prestressing Contribution Ratio (EPCR) represents PCR required to be applied at a certain girder to attain the total strain developed from loading the whole box girder. EPCR decreases as the number of bridge girders increases.

• Three methods for distribution of jacking force between girders in the post-tensioned bridges are presented. Both the tributary area method and EPCR method are more realistic ways distribute the jacking force rather than equal distribution of forces between girders. Using the tributary area method or the EPCR method minimizes the routes of force trajectories needed to satisfy uniform distribution within the box section and therefore decreases the strains developed within the anchorage zone including diaphragm, web, deck and soffit.

• In case of stressing both bridge ends, bursting strain values increase at the first end due to stressing the second end. This increase in the strains is defined as amplification factor due to stressing both ends ($\beta_{\text{end}}$), which is inversely proportional with friction losses.

• Elastic CSi Bridge models can be representative during post-tensioning stage due to low level of concrete cracking that occurs for the whole box girder. The model results show good correlation with the field investigation results.

• The developed equations for both EPCR and $\beta_{\text{end}}$ provide a good correlation with both results obtained from the field monitoring as well as CSi bridge models.
Chapter (6) – Experimental Work

6.1 Introduction

The experimental program was carried out to investigate the impact of anchorage zone reinforcing on the performance of post-tensioned box girder bridges anchorage zones. The program incorporated two phases in which specimens were designed and cast to study spreading of forces within the general anchorage zone as well as checking the adequacy of the local zone. Phase I included two single I-section girders with different end diaphragm configurations at each end. While Phase II had two double girder specimens to study the difference between single and double girders as well as to investigate the effect of openings in the end diaphragm.

This chapter presents experimental specimens development comprising study of specimen shape, effect of specimen length and specimen scaling. Preliminary SAP [35] models are developed to compare the proposed scaled specimen with a full-scale prototype bridge in order to make sure that the scaled specimens are representative. In addition, design of specimens to satisfy the design code limits and obtain the desired mode of failure is illustrated. Also, test matrix and instrumentation plans for both experimental phases are presented.

Specimen preparation included several steps: attaching strain gauges, building formworks and reinforcement cages, adding prestressing anchors, and finally concrete casting. Test setup, instrumentation and loading protocol are presented in this chapter. All specimens were cast and tested at the Large Scale Structures Laboratory of University of Nevada, Reno.
6.2 Experimental Specimen Development

The experimental specimen were developed to represent the average dimensions and configuration of the bridges being constructed. The dimensions of the prototype specimen were extracted from the developed bridge database in Chapter (3). Several parameters were investigated to assure that scaling and testing of the experimental specimens were representative to the prototype. Preliminary elastic SAP 2000 models were developed to validate that experimental specimens satisfy testing objectives. Different elements at the anchorage zone including diaphragm, web, deck and soffit are defined as elastic shell elements in order to capture the developed stress distribution. Stress distribution at different elements was compared to check the adequacy of the developed specimen to represent the prototype case. Elastic models were considered acceptable in case of early loading stages of the bridge, due to low level of strains during post-tensioning and cracking that would only be local or minor.

The parameters studied were used to determine if the proposed test setup, specimen dimensions, and loading protocol would adequately represent an actual bridge. A comparison was conducted between the developed specimen and a full prototype bridge girder in order to assure that developed stress distributions were matching.

6.2.1 Study of test methodology and specimen shape

Two testing methodology alternatives were investigated. The first alternative was to load the specimen through tendons in an end block reaction frame. Due to the high prestressing forces applied, the dimension of this reaction frame become very large. Therefore, casting the end block reaction frame as a part of the beam was a better idea. The
second alternative was using specimens with diaphragms on both end, providing more experimental results by testing both end at the same time. Also, this specimen configuration is a closer representation of the actual bridge case.

Comparisons were conducted between the two alternatives using elastic SAP models as shown in Figures 6-1 and 6-2. The specimen length in both cases was four times the section height (h), representing two times of specimen height ahead of the anchors on each side. The length of the first alternative specimen containing reaction block was 324” including 81” for the reaction frame. The second alternative specimen length was 324” as well, representing two times of specimen height ahead of the anchors on each side. The tendon profile and the supporting system change in both alternatives. In case of the reaction block, the specimen was tightened to the lab floor using prestressing bars which change its performance as a rigid fixed block. This made the prestressing system indeterminate causing external reaction on the lab floor due to post-tensioning. The second alternative was supported using a hinged-roller determinate system, eliminating any external reactions due to prestressing; this was considered a better system. The prestressing force levels in both models was 0.1 $f'_{c}A_g$, which is the lowest level of prestressing force extracted from the bridge database. Due to the elasticity of the developed SAP models, the prestressing force level will not affect the output results.
Figure 6-1: Full-scale specimen with reaction frame developed SAP models investigating effect of specimens shape.

Figure 6-2: Double sided full-scale specimen developed SAP models investigating effect of specimens shape.

The stress distributions observed in the web were compared for both alternatives including the longitudinal compressive stresses as well as the vertical tensile stresses as shown in Figures 6-3 and 6-4, respectively. Both specimens have uniform longitudinal compressive stresses based on the tendon profile after a distance equal to the section height (h) measured from the outer face of the diaphragm. The distribution of the vertical tensile stresses represents the expected bursting force in the web. Also, these results clarify the good agreement between the developed models, which illustrate the adequacy of using either of the alternatives.
One of the main aspects considered using the developed SAP models was the stress distribution at the inner face of the diaphragm. As the diaphragm was subjected to high bursting forces; however, no code or design guideline include it in the design procedures of anchorage zones for box girder bridges. The transverse as well as the vertical tensile stresses developed on the inner diaphragm face are compared for both alternatives as shown.
in Figure 6-5. Both developed models show identical results, which clarifies that either methodology for testing experimental specimens would be sufficient.

![Comparison between tensile stresses at inner face of the diaphragm for alternatives investigating specimen shape.](image)

While both specimens’ types were shown to be adequate, it was decided to use the double sided specimens because two anchorage zones could be tested at the same time and the system was statically determinate.

### 6.2.2 Effect of specimen length and tendon profile

In order to reach an optimum specimen length representing the actual bridge case with no interfering between the results of both ends on each other, different specimens with variable length were investigated. Five SAP models were developed for full-scale double sided specimens with different length including 2.0 h, 2.5 h, 3.0 h, 3.5 h and 4.0 h. Three curved tendons were used in each specimen. Inclined tendon profiles with average tendon inclination angle of 4.34° obtained from the developed bridge database were implemented.
In order to compare the interference between the ends, longitudinal compressive stresses as well as vertical tensile stresses in the web for the developed models were examined, as shown in Figures 6-6 and 6-7, respectively.

Longitudinal compressive stresses of the web clarify that there is no interference between specimen ends starting with a specimen length of 2.5 h, where uniform longitudinal compressive stresses based on the tendon profile are achieved at mid span. However, the vertical tensile stresses distribution in the web indicates interference continue until the specimen length reaches 4.0 h. Although, the interference between both ends in case of vertical tensile stresses in the web occurs at very low stress values. Therefore, length of 4.0 h was chosen to eliminate any interference between ends.
Figure 6-6: Comparison between longitudinal compressive stresses at the web for full-scale specimens with different length in case of multiple curved tendons.
Figure 6-7: Comparison between vertical tensile stresses at the web for full-scale specimens with different length in case of multiple curved tendons.
Five more SAP models were developed that investigated only one straight tendon implemented in the specimen centroid. This specimen would have the maximum spreading of forces. The longitudinal compressive stresses as well as vertical tensile stresses in the web for the developed models are shown in Figures 6-8 and 6-9, respectively.

Longitudinal compressive stresses of the web clarify that there was no interference between specimen ends starting from a specimen length of 2.5 h, where uniform longitudinal compressive stresses were obtained at mid span. However, the vertical tensile stresses distribution in the web indicates interference between both ends occurred until the specimen length reached 4.0 h. These are same observations resulting from models developed in case of using three curved tendons. This highlights that the tendon profile and number of tendons do not affect geometry or configurations of the designed experimental specimens.

Comparing the developed vertical tensile stresses in the web in case of using three curved tendons or one straight tendon clarify the significant difference in the developed bursting forces. This difference is obvious through the stresses values and distribution. That’s why, the number of tendon as well as inclination angle are significant parameters affecting performance of anchorage zones.
Figure 6-8: Comparison between longitudinal compressive at the web for full-scale specimens with different length in case of single straight tendon.
Figure 6-9: Comparison between vertical tensile stresses at the web for full-scale specimens with different length in case of single straight tendon.
6.2.3 Specimen scaling

Scaling and similitude of experimental specimens are main aspects that must be satisfied in order to produce a representative specimen. Target prestressing level was a main concern during specimen scaling. Reaching high level of prestressing force was a main aim to achieve high level of strain within concrete and reinforcing bars. It was important to apply a load that was greater than design load to determine the factor of safety of the designs. Loading for the double sided specimens was done using implemented strands in the duct. In order to reach higher strain levels more strands are required to be added which consequently need bigger duct. It was a concern that implementing bigger duct would affect specimen scaling regarding the web thickness by either changing web thickness or accepting smaller concrete cover over the prestressing ducts.

The target prestressing force level was determined based on the developed bridge database as well as ultimate failure load for specimens obtained from previous literature. Based on the developed bridge database, the maximum prestressing force level was 0.3 $f'_cA_g$. Based on the experimental work done by Sanders in 1990 [11], failure loads for different experimental specimens are plotted as shown in Figure 6-10. All of the specimens with label B or TPT were rectangular specimens; however, only F1 specimen was T-section. Even this T-section specimen did not have an end diaphragm where the prestressing forces were directly applied to the web. The average failure load obtained from this comparison was 0.375 $f'_cA_g$. Due to the different nature of the problem in case of end diaphragm, the target value for the developed experimental specimen was set to the maximum value of 0.475 $f'_cA_g$. As the presence of end diaphragm will contribute significantly in resisting the developed bursting forces during post-tensioning.
Different scale factors were investigated in order to choose the best one causing maximum prestressing force without either changing web thickness or accepting smaller concrete parts surrounding the prestressing ducts. Different scaling factors were chosen from 0.5 to 1.0, where scale factor 1.0 represents the prototype specimens based on the average dimensions from the bridge database. Different proposed scaled specimens with the maximum ultimate force that can be applied for each one are summarized in Table 6-1. This table contains section properties, which are web width ($b_w$), section height ($h$), girder spacing ($S$), deck thickness ($t_d$), soffit thickness ($t_s$) and the total cross-sectional area ($A_g$). Prestressing data including: prestressing duct outer diameter (OD), number of ducts, number of strands per duct based on 0.6” strands, and maximum jacking force can be applied based on these data are presented in this table as well. The ratio between duct outer diameters to the web thickness (OD/$b_w$) was the governing factor in scaling procedures in order to maintain an acceptable amount of concrete surrounding the prestressing ducts. Finally, concrete compressive strength, assumed prestressing force at working level (0.2 $f'_c A_g$) and target ultimate value of prestressing force (0.5 $f'_c A_g$) are presented. At the end
of the table, the ratio between maximum prestressing forces obtained from the number of strands to the maximum target ultimate force (0.5 $f'_c A_g$) is shown. This value presents how much does this scale factor satisfying the target ultimate prestressing force.

Table 6-1: Alternative scaling factor for prototype experimental specimens.

<table>
<thead>
<tr>
<th>Scale</th>
<th>1.0</th>
<th>0.9</th>
<th>0.8</th>
<th>0.7</th>
<th>0.6</th>
<th>0.5</th>
<th>0.5*</th>
<th>0.5**</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_w$ (in)</td>
<td>12</td>
<td>10.8</td>
<td>9.6</td>
<td>8.4</td>
<td>7.2</td>
<td>6</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>$h$ (in)</td>
<td>81</td>
<td>72.9</td>
<td>64.8</td>
<td>56.7</td>
<td>48.6</td>
<td>40.5</td>
<td>40.5</td>
<td>40.5</td>
</tr>
<tr>
<td>$S$ (in)</td>
<td>126</td>
<td>113.4</td>
<td>100.8</td>
<td>88.2</td>
<td>75.6</td>
<td>63</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td>$t_d$ (in)</td>
<td>8.50</td>
<td>7.65</td>
<td>6.80</td>
<td>5.95</td>
<td>5.10</td>
<td>4.25</td>
<td>4.31</td>
<td>4.31</td>
</tr>
<tr>
<td>$t_s$ (in)</td>
<td>7.50</td>
<td>6.75</td>
<td>6.00</td>
<td>5.25</td>
<td>4.50</td>
<td>3.75</td>
<td>3.69</td>
<td>3.69</td>
</tr>
<tr>
<td>$A_g$ (in²)</td>
<td>2796</td>
<td>2265</td>
<td>1789</td>
<td>1370</td>
<td>1007</td>
<td>699</td>
<td>699</td>
<td>732</td>
</tr>
</tbody>
</table>

| OD (in) | 4.78 | 4.19 | 4.19 | 4.19 | 3.18 | 2.83 | 3.18 | 3.18 |
| OD / $b_w$ | 0.40 | 0.39 | 3.82 | 0.50 | 0.44 | 0.47 | 0.53 | 0.45 |
| No. of duct | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| Strands per duct | 27 | 19 | 15 | 15 | 12 | 9 | 12 | 12 |
| Max. force (kips) | 4035 | 2839 | 2241 | 2241 | 1793 | 1345 | 1793 | 1793 |
| $f'_c$ (ksi) | 5 | 5 | 5 | 5 | 5 | 5 | 5 | 5 |
| $P_{working}$ (kips) | 2796 | 2265 | 1789 | 1370 | 1007 | 699 | 699 | 732 |
| $P_{ultimate requ.}$ (kips) | 6990 | 5662 | 4474 | 3425 | 2516 | 1748 | 1748 | 1829 |
| Max. Force / $P_{ultimate requ.}$ | 0.58 | 0.50 | 0.50 | 0.65 | 0.71 | 0.77 | 1.03 | 0.98 |

(*) 0.5 scaled specimen with bigger (3.18”) duct
(**) 0.5 scaled specimen with bigger (3.18”) duct and thicker web (7.0”)

Scale factor 0.5 with 0.77 ratio between maximum prestressing force and the target ultimate value was considered the optimum compared to other scale factors; however, the value of 0.77 does not satisfy the requirement to assure high level of strain to develop in
concrete and reinforcement. In order to be capable of applying more prestressing force on the same section with scale factor of 0.5, larger duct was implemented with outer diameter of 3-3/16” (3.18”) in the scaled specimen 0.5*. This larger duct decreases the amount of concrete surrounding the prestressing duct, so the ratio OD/bw increased from 0.4 for the prototype to 0.53 in the scaled model 0.5*. In order to maintain an acceptable ratio of OD/bw, the web thickness was increased by 1” in the scale model 0.5**. In this specimen, web thickness increased from 6” to 7” and the ratio of OD/bw satisfies the requirement by 0.45, which was nearly the same as the prototype. Also, maximum force applied on this section was 0.98 from the target required ultimate force. This scale factor of 0.5** satisfies all the requirement; however, it deviated from the scaling laws by using larger web thickness.

In order to investigate this increase in web thickness, two scaled SAP models were developed with scale factor of 0.5 and different web thicknesses of 6” and 7”. To compare the developed models, longitudinal compressive stresses as well as vertical tensile stresses in the web are shown in Figures 6-11 and 6-12, respectively. No significant difference was observed between the longitudinal compressive stresses. However, the vertical stresses in the web were slightly higher in the case of using 6” web rather than the 7” one, where vertical tensile stresses are inversely proportional with web thickness. As the vertical tensile stresses ranges between 0.04 ksi to 0.1 ksi, this slight difference between results is considered insignificant.
Figure 6-11: Comparison between longitudinal compressive stresses at the web for half-scale specimens with different web thicknesses.

Figure 6-12: Comparison between vertical tensile stresses at the web for half-scale specimens with different web thicknesses.

The transverse as well as the vertical tensile stresses developed on the inner diaphragm face are compared for both models as shown in Figure 6-13. Both developed models showed identical results, which clarify that slightly changing in web thickness does not affect developed stresses in the inner face of the diaphragm in either transverse or vertical directions.
As changing the web thickness deviated from the scaling laws, checking all elements affected by spreading bursting forces was a must. Therefore, longitudinal compressive stresses as well as transverse tensile stresses in the deck for both models with web thickness 6” or 7” are compared in Figures 6-14 and 6-15, respectively. Both figures developed identical results in case of using 6” web or 7” web, which shows that a slight change in web thickness does not affect developed stresses in the deck in either longitudinal or transverse directions. This indicates that changing web thickness to 7” instead of 6” does not affect specimen scaling or change the stress distribution. Therefore, for the purposes of the specimens, a 7” web thickness was selected.
Figure 6-14: Comparison between longitudinal compressive stresses at the deck for half-scale specimens with different web thicknesses.

(a) 6" web thickness

(b) 7" web thickness

Figure 6-15: Comparison between transverse tensile stresses at the deck for half-scale specimens with different web thicknesses.

(a) 6" web thickness

(b) 7" web thickness
6.2.4 Final comparison between full box section and scaled specimen

The anchorage zone experimental specimens passed through several stages to reach the shape of half-scale double sided specimens. Starting from multiple girders box section to a single girder out of the whole bridge. After that specimen length changed from the length of a whole bridge to a short specimen, which possesses a length of four times the specimen height. Then after these stages, the prototype specimen was scaled to a half-scale specimen.

Four elastic SAP models were developed to check different stages of specimen scaling. First, full-scale bridge box with five girders, simple span, 150’ span length and three prestressing tendons with 4.34° inclination angle was studied. The geometry and configuration of this full-scale bridge box matched the developed prototype specimen in Chapter (3). The second model was a single girder extracted out of the whole box section. The third model represented a double sided specimen that matches exactly the single girder with shorter specimen length equal to four times the section height. Finally, the fourth developed model represents half-scale double sided specimen obtained from the prototype. The scaling process was performed by scaling the geometry with ½ scale factor and the force by ¼ scale factor in order to maintain constant stresses developed within different elements. Extruded views for the developed models are shown in Figure 6-16.
Longitudinal compressive stresses in the web of the developed four models are shown in Figure 6-17. The same flow of stresses was observed in different models. The significant length that is affected by non-uniform stress distribution is equal to the specimen height (h) along the web measured from the diaphragm. Vertical tensile stresses in the webs of the developed scaling models are shown Figure 6-18. Slight difference was observed in the distribution of stresses between different models. This difference was considered as insignificant due the small range of plotted stresses between 0 to 0.1 ksi. These comparisons of the longitudinal compressive stresses as well as the vertical tensile stresses of the web indicate the adequacy of scaling procedures applied.
As the diaphragm is considered as the first element resisting bursting forces of prestressing, stresses of the inner diaphragm face are compared. Transverse and vertical tensile stresses developed on the inner face of the diaphragm for different models are plotted in Figures 6-19 and 6-20, respectively. Both figures clarify that the developed stress distribution is identical for different cases. This highlights that the developed half-scale specimen is representative for the multiple girder bridge.
Deck longitudinal compressive stresses as well as transverse tensile stress for developed models are shown in Figures 6-21 and 6-22, respectively. In case of longitudinal compressive stresses, the results were exactly the same, which indicate that the developed scaled specimen was representative. However, in case of transverse tensile stresses, different performance was noticed. The reason for this different performance was the
continuity of multiple girder. This difference in stresses was neglected, as the performance of the single girders matched the performance of exterior girder in case of multiple bridge girder. The exterior girder performance is illustrated as in the top of Figure 6-22 (a). Therefore, the developed single girder specimen represents an exterior girder. Based on all the comparisons done, the developed half-scale specimen was adequate and representative for the actual full prototype bridge.

Figure 6-21: Comparison between longitudinal compressive stresses at the deck for scaling models.
6.2.5 Specimen self-weight compensation

Presence of support reaction affects the performance of anchorage zones [36]. In order to represent adequately the reactions developed at the diaphragm during post-tensioning, scaling for this reaction was performed. During post-tensioning of a box girder bridge, the superstructure is supported by formwork as shown during construction of Bridge I - Willits Bypass Floodway Viaduct Frame 6 in Figure 6-23. This formwork was used to hold fresh concrete during casting. This formwork remains until prestressing stage
was completed. During post-tensioning, the box section started to camber separating from the formwork. This separation developed reactions at the end diaphragm due to the structure weight. As the self-weight of the bridge was significant, the reactions must considered for testing experimental specimens.

![Formwork supporting superstructure for Bridge I - Willits Bypass Floodway Viaduct Frame 6.](image)

The self-weight of a typical prototype girder was calculated based on a simple span length of 150’ and an average diaphragm of 3’ - 5” width. The reaction for this system was 231.51kips; this reaction represents 8.29% of the average prestressing force (0.2f’cAg) of the prototype specimen. In order to maintain the same percentage of the self-weight reaction at the diaphragm for the half-scale specimen, additional load must be added to the specimens. The self-weight of the specimen was 15.68kips, causing a reaction of 7.84 kips each end, which represents 1.07% of the average prestressing force (0.2f’cAg). In order to increase the reaction in the specimens, two concentrated loads were applied vertically on the test specimens. The value for these loads compensated for the difference of the self-weight to the average prestressing force ratio from 1.07% to 8.29%. This difference was
52.8 kips which needs to be added at each end. The total load applied on the single girder experimental specimens was 105.6 kips. This load was applied in the experimental test by using four high strength rods connected to the lab floor and stressed using center-holes hydraulic jack. The applied load on each rod is 26.40 kips in case of single girder specimens. For double girder specimens, this force was doubled to be 52.8 kips per rod.

### 6.2.6 Design of experimental specimens

Design of developed experimental specimens was performed to assure that non-desirable failure modes did not occur during testing. This design included checking service stresses acting at mid-span of the specimen during testing. Also, ultimate stages were checked for both shear and flexure. Checking shear was done to make sure that no shear failure or excessive cracking due to shear would happen. Checking flexural performance was done to assure that the ultimate flexural capacity of the section exceeds the maximum moments applied on the specimens during testing.

The specimens were designed according to Caltrans Memo to Designer (11-25) - Anchorage Zone Design [1] in order to make sure it satisfied Caltrans requirements for anchorage zones. Detailed calculations for specimen design are illustrated in Appendix (B). Also, final detailed construction drawings and instrumentation plans for experimental specimens are presented in Appendix (C).

### 6.3 Test Matrix

Experimental half-scale specimens included two single I-section girder in addition to two double girder specimens. The two single I-section girders had different end diaphragm configurations for each end. The first specimen had diaphragm widths of 16.25”
and 20.25”, which represented 40% and 50% of the specimen height (h). Those ends were labeled S04 and S05L, respectively as shown in the schematic drawing in Figure 6-24. The second specimen had diaphragm widths of 24.25” and 20.25”, which represented 60% (S06) and 50% (S05H) of the specimen height as shown in Figure 6-25. All double girder specimens had diaphragm widths of 16.25”, which represented 40% of the specimen height (h).

A bridge database was developed for 29 different diaphragm configuration that were provided by Caltrans. The cross-sectional dimensions of the developed specimens were based on a half-scale factor from the prototype specimen determined from the bridge database. The cross-section concrete dimensions and side view for single girder specimens are shown in Figure 6-26.

![Figure 6-24: Elevation schematic drawing for single girder specimen S45.](image1)

![Figure 6-25: Elevation schematic drawing for single girder specimen S56.](image2)
All of the single girder specimens were reinforced with the average reinforcement ratio concluded from the developed bridge database; however, end diaphragm labeled S05H was reinforced with higher reinforcement ratio to study the effect of this parameter. The reinforcement ratio for this end diaphragm was 1.5 times the average value. Also in this end diaphragm, a grillage of #3 at 6” was implemented at the external face of the diaphragm representing the typical grillage of #4 at 4” as presented in CALTRANS – Cast-in-place prestressed girder details B8-5 [37]. This grillage was added in this specimen to study its effect on the general anchorage zone performance. The test matrix for all experimental specimens is summarized in Table 6-2.

Table 6-2: Test matrix for experimental test specimens.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1- S45</td>
<td>S04</td>
<td>16 ¾</td>
<td>N/A</td>
<td>TYP. S05H</td>
<td>20 ¼</td>
<td>N/A</td>
</tr>
<tr>
<td>2- S56</td>
<td>S05L</td>
<td>20 ¼</td>
<td>N/A</td>
<td>TYP. S06</td>
<td>24 ¼</td>
<td>N/A</td>
</tr>
<tr>
<td>3- DS6</td>
<td>DST</td>
<td>16 ¾</td>
<td>Solid Diaph.</td>
<td>TYP. D6T</td>
<td>16 ¾</td>
<td>0.6h × 0.6S</td>
</tr>
<tr>
<td>4- D55</td>
<td>D5T</td>
<td>16 ¾</td>
<td>0.5h × 0.5S</td>
<td>TYP. D5H</td>
<td>16 ¾</td>
<td>0.5h × 0.5S</td>
</tr>
</tbody>
</table>
This table includes specimen number, label for each end, end diaphragm width ($b_d$), opening dimensions - if any - and reinforcement variation. The opening were not available in the single girder specimens, as these openings are always added between the two girders in the end diaphragm. The reinforcement variation represented the diaphragm reinforcement for single girder specimens. For the double girder specimens, the reinforcement included the additional reinforcement around openings.

Labeling characters for the diaphragm ends were based on dividing the specimens into two groups. First character (S) or (D) stood for single or double girder specimens, respectively. The second character in the single girders group 04, 05 and 06 represented the diaphragm width of 40%, 50% and 60% of the section height ($h$). The last character in the single girders group (L) and (H), represented the reinforcement ratio in the diaphragm, where specimen end S05L had a typical average reinforcement ratio obtained from the developed database and specimen end S05H had 1.5 times the reinforcement ratio in S05L. As specimen ends S04 and S06 had the typical value of reinforcement, no character representing reinforcement ratio was added to them.

In the double girder specimens, the second character (S), (5) and (6) stood for the opening dimensions of the diaphragm based on box section height ($h = 40.5''$) and girder spacing ($S = 63''$), where (S) stood for solid diaphragm, (5) stood for opening dimensions $0.5h \times 0.5S$ and (6) stood for opening dimensions $0.6h \times 0.6S$. The final character in the double girder specimens represented the reinforcement around the opening based for a CALTRANS – Utility opening box girder B7-10 [38]. In case of using Caltrans recommendation, specimens were labeled with (T). However, in specimen end D5H, last
character (H) represented the usage of double reinforcement to study the effect of adding such high reinforcement around the opening to minimize crack width.

The two double girder specimens possessed different end diaphragm configurations investigating effect of openings on prestressing force spreading within the anchorage zones. Both double girder specimens DS6 and D55 possessed the same diaphragm width of 16.25” as shown in Figure 6-27. Cross-section for double girder specimens at mid-span is presented in Figure 6-28; it illustrates concrete dimension for all element web, deck, soffit as well as girder spacing.

![Figure 6-27: Elevation schematic drawing for double girder specimens DS6 and D55.](image)

![Figure 6-28: Cross-section for double girder specimens at mid-span.](image)

Configurations of openings in the end diaphragm are illustrated in the side view drawings. The utility openings regulations were based on CALTRANS – Utility opening
box girder B7-10 [38]; minimum opening width was 1/3 of the box girder height or 18”, which was greater and the minimum height was 1/3 of the box girder height or 24”, which was greater. These regulations by Caltrans provided minimum dimension only; however, the maximum dimensions were developed based on the design engineer experience and judgment. For specimen DS6, side view for ends DST and D6T are illustrated in Figures 6-29 and 6-30, respectively. Both figures illustrate the block-out limits, prestressing anchors location as well as the opening dimensions for specimen end D6T. This opening represented a maximum case, where opening width was 60% of girder spacing (38”) and the height was 60% of the section height (24 ½”). Opening configuration of both ends for specimen D55 is illustrated in Figure 6-31. The opening dimension represented the maximum dimension ever constructed by Caltrans, where opening width was 50% of girder spacing (31 ½”) and the height was 50% of the section height (20 ½”).

![Figure 6-29: Side view for double girder specimen end DST.](image-url)
Figure 6-30: Side view for double girder specimen end D6T.

Figure 6-31: Side view for double girder specimen D55.

The double girder specimens included additional large top access opening to facilitate inspection during testing. The plan view for the developed double girder specimens is shown in Figure 6-32. Access opening with dimensions 36” × 36” was developed in the deck as shown in this figure. Also, lifting eyes were implemented in order to facilitate specimens handling during testing preparations.
6.4 Instrumentation Plans

Instrumentation plans were prepared to study the performance of the anchorage zone in the experimental specimens. Detailed drawings including the locations and labels of strain gauges are presented in Appendix (C). Each one of the specimens has its own instrumentation plans based on the presence of diaphragm openings, the geometry as well as reinforcement of the anchorage zones. Reinforcing bars strain gauges were implemented in the inner and outer faces of the diaphragm, transverse bars in the deck and soffit as well as vertical stirrups in the web. Foil strain gauges were attached to the prestressing strands in order to capture the actual forces in the tendon by correlating the strain values to the pressure measured from the prestressing jack. Concrete gauges were implemented in the web as well as the inner and outer faces of the diaphragm.
6.4.1 Experimental specimens Phase I – single girders

Instrumentation devices were implemented in the experimental specimens including steel and concrete gauges. The strain gauge labels were started with the diaphragm end side label (S04, S05L, S05H and S06) then the gauge label. Reinforcing bars as well as concrete gauges were implemented in the inner and outer faces of the diaphragm as shown for specimen end S04 in Figures 6-33 and 6-34 respectively. Labeling of transverse horizontal gauges started with (H); however, vertical bars started with (V). The inner face of the diaphragm were labeled (VI) and (HI), where (I) stood for inner side. The outer face of the diaphragm were labeled (VE) and (E) stood for the exterior side. After the designators just described, a series of number were used to label each gauge in its exact location as shown in the figures. Most of the gauges implemented in the diaphragm were on the inner face, as this was the critical side determined from the preliminary finite element analysis.

Figure 6-33: Instrumentation plan for the inner diaphragm face of specimen end S04.
Steel gauges were attached to the prestressing strands and the spiral in the local zone as well as web stirrups as shown in Figure 6-35 for specimen end S04. The gauges attached to the prestressing strands were labeled (TN) standing for tendon followed by (T or M or B) standing for tendon location top (T), middle (M) and bottom (B). The gauges attached to spiral reinforcement were labeled (A) standing for anchors, followed also by (T or M or B) standing for location.

Strain gauges attached to web stirrups were labeled (W). These gauges extended to a distance equal to specimen height to assure that the critical zone of the web will match the distance obtained from preliminary analysis. Critical distance obtained from preliminary analysis was half the specimen height (h/2) along the web.
Strain gauges were attached to the transverse bars in the deck and soffit as shown for specimen end S04 in Figures 6-36 and 6-37, respectively. The strain gauges were labeled (D) and (S) standing for deck and soffit. Most of the gauges in the deck were implemented in the bottom layer, which was considered to be critical based on the preliminary analysis. Only few gauges were installed in the top layer for comparison. On the other side, most of the gauges implemented in the soffit were in the top layer, which was the critical one.

Steel strain gauges were attached to the transverse bars at the top and the bottom of the diaphragm. These gauges were labeled (T) and (B) standing for top and bottom. The importance of these gauges was to compare the difference in strains at the deck inside and outside the diaphragm as well as at the soffit inside and outside the diaphragm. A couple of gauges labeled (L) were implemented in longitudinal direction at mid-span of the specimens in the deck and soffit. These gauges were used to check the flexural deformation that occurs in the specimens during testing.

Figure 6-35: Instrumentation plan for the web of specimen end S04.
Figure 6-36: Instrumentation plan for the deck of specimen end S04.

Figure 6-37: Instrumentation plan for the soffit of specimen end S04.
In order to record the global performance of the test specimens, string pots as well as linear variable displacement transducers (LVDTs) were attached to it as shown in Figure 6-38 for specimen end S04. LVDTs were attached at the specimen bottom (S04-DGC, S04-DGE, S45-DGC and S45-DGE) to capture specimen deflected shape. Two LVDTs were recording at each location, one at the specimen center and the other at the edge. Also, two LVDTs (S04-DGV1 and S04-DGV2) were attached to the specimen outer face to measure the relative deformation between specimen’s edges to assure that no rotation or twisting along specimen axis will occur during testing. Two string pots (S45-DGT and S45-DGB) were used to measure the top and bottom axial deformation for the specimen.

Three threaded rods were inserted in the web at mid-span of the specimens. These rods were located at three different levels (top, middle and bottom). Nine string pots (S04-
DG01 to 09) were connected to these rods in order to form a grid, measuring the deformation of the diaphragm. This grid of string pots is shown in Figure 6-39. In order to calibrate this technique of measurements using long string pots, two LVDTs (S04-DG07R and S04-DG09R) were connected to the top and bottom flanges using steel rods to capture the same readings as string pots (S04-DG07 and S04-DG09).

Figure 6-39: Grid of string pots measuring diaphragm deformations for specimen end S04.

### 6.4.2 Experimental specimens Phase II – double girders

The strain gauge labels were started from the specimen diaphragm end label (DST, D6T, D5T and D5H) then the gauge label. Reinforcing bars and concrete gauges were implemented in the inner and outer faces of the diaphragm as shown for specimen end D6T in Figures 6-40 and 6-41, respectively. Labeling was done using the same technique used for single girder specimens. Larger number of gauges were used in the double girder specimens due to the larger size of the specimen as well as to investigate the force trajectories around the diaphragm openings. Most of the gauges were implemented
between the two girders surrounding the end diaphragm opening in order to study the stress concentrations at this zone.

In all of the double girder experimental specimens, reinforcement grillage was implemented at one girder in the outer face of the diaphragm as shown in Figure 6-41. This grillage was #3 at 6” represented the typical grillage of #4 at 4” as illustrated in CALTRANS – Cast-in-place prestressed girder details B8-5 [37]. The grillage was implemented on one side only to compare the effect of its existence. Strain gauges attached to the grillage bars were labeled (G) standing for grillage. On the other girder, where there were no-grillage reinforcement implemented, concrete gauges were installed and labeled (N) standing for no-grillage.

![Figure 6-40: Instrumentation plan for the inner diaphragm face of specimen end D6T.](image)
Steel gauges were attached to the prestressing strands, the spiral in the local zone as well as web stirrups as shown in Figure 6-42 for specimen end D6T. The gauges attached to the prestressing strands were labeled (TN) standing for tendon followed by (G) or (N) standing for the grillage side or the no-grillage side. After that it was followed by (T or M or B) standing for tendon location top (T), middle (M) and bottom (B). Strain gauges attached to web stirrups were labeled (W). These gauges extended to a distance equal to half of the specimen height (h/2).
Strain gauges were attached to the transverse bars in the deck and soffit as shown for specimen end D6T in Figures 6-43 and 6-44, respectively. The strain gauges were labeled (D) and (S) standing for deck and soffit as in the single girder specimens. Also, steel strain gauges were attached to the transverse bars at the top and the bottom of the diaphragm.

Figure 6-43: Instrumentation plan for the deck of specimen end D6T.
A couple of gauges labeled (L) were implemented in longitudinal direction at mid-span of the specimens in the deck and soffit. These gauge were used to check the flexural deformation that occurs to the specimens during testing.
In order to record the global performance of the test specimens string pots as well as LVDTs were attached as shown in Figure 6-45 for specimen end D6T. LVDTs were attached at the specimen bottom (DS6-DGA and DS6-DGB) to capture specimen deflected shape. Two LVDTs were placed under each of the investigated girders A and B, where Girder A represents the grillage side and Girder B represents the side with no-grillage. Also, two transverse string pots (DS6-SP and D6T-SP) were attached to the specimen soffit to measure the relative deformation between specimen’s edges. These string pots were used to record transverse twisting of the specimen due to stressing each individual girder. Four string pots (DS6-DGDA, DS6-DGDB, DS6-DGSA and DS6-DGSB) were used to measure the top and bottom axial deformation for the specimen.

Figure 6-45: Elevation for double girder specimen D6T illustrating attached string pots and LVDTs during testing.
Threaded rods were inserted in the web at mid-span of the specimens at mid-height. Four string pots (D6T-DGA1, D6T-DGA2, D6T-DGB1 and D6T-DGB2) were connected to these rods in order measure the deformation of the diaphragm. String pots (D6T-DGA1 and D6T-DGA2) were placed at grillage side (Girder A), however, string pots (D6T-DGB1 and D6T-DGB2) were placed at no-grillage side (Girder B). The string pots (D6T-DGA1 and D6T-DGB1) were placed 4” from specimen’s edge, however, string pots (D6T-DGA2 and D6T-DGB2) were placed at 4” for the web outer face.

The same labeling technique was used for the remaining specimens including single as well as double girder specimens. This facilitates instrumentation as well as processing of the output results. Detailed instrumentation plans are shown in Appendix (C).

6.5 Specimen Preparation

Specimen preparation included formwork preparation, attaching strain gauges to reinforcement, placement of reinforcement, attaching prestressing anchors to the wooden formworks and installing prestressing ducts with the target cable profile. Before concrete casting, concrete gauges were placed. Finally, concrete casting and preparing concrete test cylinders and prisms were performed.

6.5.1 Attaching strain gauges to reinforcement bars and prestressing strands

Certain reinforcing bars were selected based on the instrumentation plans prepared. These reinforcing bars were separated and instrumented with strain gauges. Foil strain gauges with a maximum strain of 10% were used for reinforcing bars. As the strains in the experimental tests were expected to be higher than that obtained in the field, strain up to 10% was acceptable to capture 5 times the yield strain. The same procedures applied for
field sister bars were used to attach the strain gauges to reinforcing bars as explained previously in section 4.4.1. Mastic water sealant tape was used as an additional coating surface for all the strain gauges to assure its safety during concrete casting as shown in Figure 6-46. After that the instrumented reinforcing bars were placed in the formwork. Strain gauge wires were inserted in heat shrink tubes for protection as presented in Figure 6-46.

![Figure 6-46: Reinforcing bar strain gauge covered with mastic tape.](image1)

After casting concrete specimens, prestressing strands were placed inside the ducts. Foil strain gauges were attached to the strands in order to capture the strain developed in them as shown in Figure 6-47. These strains were used to estimate the exact force inside each of the prestressing strand.

![Figure 6-47: Foil strain gauge attached to prestressing strand.](image2)
6.5.2 Building formwork and steel cages

Building formworks was complex due to the sloped face of the prestressing block-out as shown in Figure 6-48. This slope was needed in order to satisfy the required tendon inclination. The figure also illustrates the connection between prestressing anchorage components including: multiplane anchor, trumpet and prestressing duct. After finishing the formwork, the reinforcing bars were installed. The strain gauge wires were attached to the reinforcing bars and bundled in order to eliminate any loose wires inside the specimen. These loose wires are more probable to get damaged during concrete casting than other wires bundled with reinforcing bars. These wires were placed into water sealed bags outside the specimen for protection during concrete casting as shown in Figure 6-48 (b).

Figure 6-48: Formwork and reinforcing bars at prestressing anchorages.

After installing all of the reinforcement and constructing the formwork, the formwork was strengthened using transverse stud as shown in Figure 6-49 for specimen
S45. These transverse studs prevent any deformations in the formwork surface to maintain a vertical good shape for the specimen surface.

Figure 6-49: Final formwork for specimen S45 before concrete casting.

6.5.3 Prestressing anchors and ducts compiling

Installing prestressing anchors and attaching the corrugated metal duct inside the web of the specimens was a critical stage. It was important to satisfy the specified cable profile for each of the implemented ducts. Each web in the experimental specimens contained three prestressing ducts with a profile as shown in Figure 6-50. The anchorage system used was multiplane anchorage with 12-0.6” strands. The prestressing anchorage system consisted of four components as shown in Figure 6-51: multiplane anchor, wedge plate, trumpet and spiral confining the local zone. The spiral for this anchorage system was six loops of #5 spiral with 1 7/8” pitch.
Figure 6-50: Cables profile for the implemented ducts.

(a) Trumpet  (b) Multiplane anchor with spiral reinforcement  (c) Wedge plate

Figure 6-51: Prestressing anchorage system components.

First stage for installing the prestressing anchors was to attach the multiplane anchor to the formwork as shown in Figure 6-52. Then the trumpet was attach to the multiplane anchor to connect the corrugated metal ducts. The cable profile was satisfied by tying the ducts to the adjacent web stirrups using tie wires and check the profile using measuring tape at different points. The final shape for the implemented prestressing system satisfying the required profile was as shown in Figure 6-53. After concrete casting, the prestressing strands were implemented inside the ducts, then the wedge plates are added. After that, the wedges are added to each individual strand before stressing.
6.5.4 Concrete gauges installation

The concrete gauges are dumbbell shaped with a gauge length of 50 mm and 4 mm diameter. These concrete strain gauges were attached in the field using thin aluminum mechanical wires as shown in Figure 6-54. Several procedures were considered for attaching this type of gauge with no initial strain. The mechanical wires were always attached at an angle to the gauge axis, so during tightening these mechanical wires would
create no significant tension force to the gauge. All strain gauge wires were inserted in heat shrink tubes for protection as presented in Figure 6-54.

![Figure 6-54: Concrete gauges installed in experimental specimens.](image)

6.5.5 Concrete casting

For the single girder specimens, specimens were cast in one pour for the whole specimen. Both sides of this specimen were accessible for concrete pouring and finishing as shown in Figure 6-49 for specimen S45. For the double girder specimens, the specimens were cast using two concrete pours. First one was for the soffit, web and the diaphragm as shown in Figure 6-55. The reason for these two pours is that the inner side of the box will not be accessible to be finished if only one concrete pour was used.

During concrete casting two techniques were used for concrete compaction to assure concrete flowing between congested reinforcement bars as well as to prevent any void or pockets inside the concrete. First method for compaction was using the tradition concrete compactor as shown in Figure 6-55. The second way was by impacting the surface of
formwork using jack hammer as shown in Figure 6-56. Both methods provide good compaction for the developed concrete.

Figure 6-55: Casting first concrete pour for double girder specimens.

Figure 6-56: Compacting concrete using jack hammer by impacting the formwork surface.

After casting the first concrete pour for double girder specimens, the surface of the soffit was finished in order to make it smooth and facilitate monitoring cracks during testing as shown in Figure 6-57. Then, formwork inside the box was removed and the top
slab formwork was constructed. The second concrete pour for double girder specimens was for the deck. The surface of the deck was finished as shown in Figure 6-58. The construction stages for the double girder specimens including two pours matches the process of bridge construction in the field. The typical methodology for bridge construction is to perform two concrete pours, the first includes soffit, web, bent caps and diaphragm and the second includes only the deck. Performing the same construction procedures as constructed in the field made the double girder specimens more representative of the actual construction case.

Figure 6-57: Finishing the surface of the soffit for double girder specimens.

Figure 6-58: Second batch for the deck of the double girder specimens and finishing deck surface.
In order to check workability of concrete during construction, slump test was performed as shown in Figure 6-59. The slump test was repeated three times for each concrete batch and the average results was considered according to ASTM C143/C143M − 15a [39]. The slump results for the first single girder specimens was 5 ¾”. However, for the double girder specimens the average results for slump was 4 ½” for the first pour (soffit, web and diaphragm) and 5 ¼” for the second one (deck).

(a) Preparation of slump cone  
(b) Concrete slump test

Figure 6-59: Measurement of slump value for fresh concrete.

In order to check the properties of concrete used in the experimental specimens, several concrete cylinders were cast including 6” × 12” cylinders for compressive strength test and splitting tensile strength test. Also 4” × 8” concrete cylinders were used for static modulus of elasticity test. Prisms with dimensions 6” × 6” × 20” were prepared for flexural strength of concrete in order to determine the modulus of rupture. All the cylinders and prisms were prepared according to ASTM C470/C470M − 15 [40] and ASTM C78/C78M − 15b [41]. Sample of prepared cylinders and prisms are shown in Figure 6-60.
After concrete placement, all strain gauges were checked using a digital ohmmeter. The measured resistance showed that more than 95% of the wires are still connected to the strain gauges installed inside the end anchorage.

### 6.6 Test Setup and Instrumentation

After cleaning the specimen to remove any dust during construction, the specimen was white-washed to aid observations of the crack. Strands were placed into prestressing ducts and strain gauges were attached at both ends of each tendon. Instrumentation stages include connecting all strain gauge wires to strain gauge boxes then to the DAC, attaching all displacement gauges (String pots and LVDTs) to the specimen, connecting them to the DAC and connecting pressure gauges to the DAC. General layout for Phase I and II experimental specimens are shown in Figures 6-61 and 6-62, respectively.
Several aspects were considered during the design of experimental test setup in order to represent the prototype bridge. The specimen was subjected to its self-weight during testing; however, this weight was not sufficient to model the prototype case. Load was
applied to the experimental specimen to model the true dead weight of the prototype girder using two transverse beams, center-hole rams and vertical threaded rod connected to the lab floor as in test setup general layout shown in Figure 6-61 and Figure 6-62. For double girder specimens, 1.5” holes were provided during construction in the deck and soffit to pass the threaded rods as shown in Figure 6-62. During stressing the tendon in the ducts, the specimen will camber up due to the parabolic profile for the tendons. This camber will add more force in the threaded rods, an accumulator was attached to vertical rams to maintain constant force.

The specimens were supported on two transverse beams representing the supporting abutment. Rubber pads were used to allow rotation as well as translation of the specimens representing bridge bearings. Schematic drawing for experimental test setup is illustrated in Figure 6-63.

![Figure 6-63: Schematic drawing for experimental test setup.](image)

More than 150 strain gauges were used in each specimen. The wires from these gauges were connected to strain gauge boards as shown in Figure 6-64 and to the data
acquisition. Displacement gauges (String Pots and LVDTs) were used to monitor the shortening of specimen during stressing as well as the deformation of the end diaphragm. In order to capture the post-tensioning jack pressure during stressing, a pressure gauge and pressure transducer were attached to the pressure line of the jack. Six strain gauges were attached to the strands to measure the actual strain in the tendons during stressing. The strains developed in the strands were correlated to the measured pressure of the stressing jack.

![Image](image.png)

Figure 6-64: Instrumentation connected to data acquisition system.

### 6.7 Loading Protocol

Stressing sequence during experimental testing affects the output results especially in the case of testing the double girder specimen. The stressing level differs between bridges according to bridge configuration including span length, bridge width, skew angle, flexural behavior and shear behavior. In order to obtain the typical levels of stressing, the
developed database for Caltrans bridges was considered. The average prestressing force ratio \( \left( \frac{P_j}{f'_c A_g} \right) \) of 0.20 was considered as the design level for the anchorage zones. Three levels of stressing force were considered in the loading sequence: 0.1, 0.2 and 0.3 \( f'_c A_g \). Tendon load was increased until the maximum capacity of the tendons was reached. The effect of stressing only two tendons was investigated by stressing the bottom and middle tendons. According to preliminary finite element analysis, stressing the middle and bottom tendons was more critical than stressing the top and middle ones. Therefore, stressing force level of 0.2 \( f'_c A_g \) was reached by stressing only the middle and bottom tendons for all of the tested specimens. As a rule of thumb from the field monitoring work, stressing for tendons always take place from upper tendons to the lower ones. That’s why, this sequence was applied in the test protocol.

### 6.7.1 Phase I – single girder specimens

The bending moment due to the beam self-weight, in addition to the moment due to external vertical load, represented 58\% of the ultimate flexural capacity of the specimen as a non-prestressed section. This value was considered relatively high and could cause cracking at mid-section of the specimen unless the stressing of tendons was performed. Therefore, the loading protocol considered applying the vertical load in stages with stressing the tendons. The applied stressing sequence for the first experimental single girder specimen S45 is illustrated in Table 6-3. The table includes for each loading step, the total applied vertical load on the specimen, tendon stressing side and location (top, middle or bottom), design level, additional prestressing force that was applied in each step, the stress level in each tendon after each loading and the target extension of strands.
Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. The first two loading stages represented prestressing force value of 0.1 $f'_cA_g$ using the middle and bottom tendons only, which was the critical case. Stages three and four represented a prestressing force value of 0.2 $f'_cA_g$ using the middle and bottom tendons only. Furthermore, a prestressing force value of 0.3 $f'_cA_g$ was applied using the top tendon through stages five and six. After that stressing side changed from S04 to S05L to minimize deterioration of strands from one end due to several stressing steps as well as stressing from the end that has thicker diaphragm. Through stages seven to nine, stressing continues using the typical stressing sequence from top to bottom to reach 0.4 $f'_cA_g$. Finally, the last three steps were applied to reach 0.5 $f'_cA_g$ using the typical stressing sequence from top to bottom as well as attaining maximum stress for tendons which is 0.85 $f_{pu}$.

Table 6-3: Applied loading protocol for single girder specimen S45.

<table>
<thead>
<tr>
<th>#</th>
<th>Total Vertical Load (kips)</th>
<th>Tendon</th>
<th>Spec. End</th>
<th>Design Level * (%)</th>
<th>Add. Load (kips)</th>
<th>Stress Level of Tendons (ksi)</th>
<th>Stress /$f_{pu}$</th>
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(*) Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1350 kips for single girder specimens.

A couple of issues were observed from the first experimental test. First, no cracking or excessive deflection occurred due to applying the vertical load, that’s why staged application of vertical loads was cancelled in the following tests. The strands were very loose within the duct before stressing which caused excessive extension.

For the second single girder specimen S65, three initial loading steps were added to accommodate adequate seating of wedges into anchorage plate before stressing. These steps are equivalent to the 20% of the design value (0.2f'cAg), which represent the same initial steps applied in the field during stressing. These initial stage labels start with (B) standing for beginning stages. Also, the vertical load was applied in one step to facilitate testing procedures for specimen S65. The applied loading protocol for the second experimental specimen S65 is provided in Table 6-4.
Table 6-4: Applied loading protocol for single girder specimen S65.

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<th>Add. Load (kips)</th>
<th>Stress Level of Tendons (ksi)</th>
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</table>

(* Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1350 kips for single girder specimens.)
6.7.2 Phase II – double girder specimens

In case of double girder specimens Phase II, the same loading procedures were applied. Six anchors on each end of the specimens were stressed. Numbering of anchors was performed for both ends as shown in Figures 6-65 and 6-66 for both ends of double girder specimen DS6 to facilitate stressing procedures.

Figure 6-65: Anchors labels for specimen end DST.

Figure 6-66: Anchors labels for specimen end D6T.

Loading protocol of the double girder specimen DS6 is summarized in Table 6-5. Loading started with initial stressing of tendons (B1-B6) to accommodate adequate seating of wedges into anchorage plate before starting the full stressing protocol. These initial stressing stages provide accurate results for tendons extension. After that vertical load
compensating actual prototype self-weight was applied in step (L1). Total vertical load applied through threaded rods was 211.2 kips.

Loading stages (1-4) created the target prestressing force value of 0.1 $f'_c A_g$ using the middle and bottom tendons only. Then stages (5-8) created a prestressing force value of 0.2 $f'_c A_g$ using the middle and bottom tendons only. Prestressing force value of 0.3 $f'_c A_g$ was applied using the top tendon through stages (9-12). Due to the large number of stressing stages, testing was performed on two days. The first day included loading stages up to step 12. At the beginning of the second day, applied vertical load on the specimen was checked in step (L2). The pressure accumulator was working efficiently, no changes occurred in the vertical loads applied. Through stages (13-18), stressing continued using the typical stressing sequence from top to bottom to reach 0.4 $f'_c A_g$. Finally, the last six steps (19-24) were planned to reach 0.5 $f'_c A_g$; however, failure occurred at D6T Girder B (no-grillage side) during step 21. Therefore, the last three steps (22-24) were cancelled and unloading for the vertical load applied on the specimen was done through step (L3).

Table 6-5: Applied loading protocol for double girder specimen DS6.

<table>
<thead>
<tr>
<th>#</th>
<th>Spec. End</th>
<th>Tendon Location</th>
<th>#</th>
<th>Design Level * (%)</th>
<th>Add. Load (kips)</th>
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L1 Applying vertical load using four high strength threaded rods, Total vertical load = 211.2 kips

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L2 Check applied vertical load using four high strength threaded rods, Total vertical load = 211.2 kips

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L3 Unloading:
Remove applied vertical load on the four high strength threaded rods.

(*) Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1075 kips for single girder specimens.

The same loading protocol was applied for double girder specimen D55. Numbering of anchors is shown in Figures 6-67 and 6-68 for both ends of double girder specimen D55 to facilitate stressing procedures. Loading protocol of double girder specimen D55 is summarized in Table 6-6. However, failure occurred at D5H Girder B (no-grillage side) after step 23. Therefore, the last step 24 was cancelled and unloading for the vertical load applied on the specimen was done through step (L3). During unloading stage sudden failure
occurred at D5T Girder A (grillage side), indicating crack propagation after stressing due to short term creep effect.

Figure 6-67: Anchors labels for specimen end D5T.

Figure 6-68: Anchors labels for specimen end D5H.

Table 6-6: Applied loading protocol for double girder specimen D55.

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<th>Stress Level of Tendons (ksi)</th>
<th>Stress /f_{pu}</th>
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<td>70</td>
<td>27</td>
<td>0.1</td>
<td>0.16</td>
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<td>Bottom</td>
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L1 Applying vertical load using four high strength threaded rods,
Total vertical load = 211.2 kips

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L2 Check applied vertical load using four high strength threaded rods,
Total vertical load = 211.2 kips

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**L3**

Remove applied vertical load on the four high strength threaded rods.

(*) Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1075 kips for single girder specimens.

### 6.8 Experimental Output Recording and Correction

This section introduces the methodology of data collection using data acquisition system. The corrections applied for the recorded strains is presented. After that correlating the strains captured from the strands to the jacking forces collected using pressure gauge attached to the prestressing jack was performed.
6.8.1 Data recording methodology

The data was recorded for the implemented strain gauges, displacement gauges and the pressure gauges using Data Acquisition System. Data Acquisition System consists of Data Acquisition Box (DAC) and recording laptop. In order to connect the strain gauge wires to the DAC, intermediate converter named as strain gauge board was used as shown previously in Figure 6-64. The data is transferred from the strain gauge wires into the measuring cables through these strain gauge boards.

In order to record the prestressing jack pressure, a digital pressure gauge was attached to the pressure line as shown in Figure 4-40. This pressure gauge was attached to the measuring cable that transfer the data to the DAC. The data recorded from the implemented pressure gauge was calibrated with a dial pressure gauge, in order to check the validity of the measurements during stressing process. Data was recorded in each stressing stage for each anchor. The forces obtained from the strands are more representative, as they eliminated the losses and directly obtained the actual forces applied on the anchorage zone.
Final form of strain gauge results was developed after applying several corrections to the raw data including data resampling, zero error reading correction and trend line approximation. These corrections are exactly the same ones performed for field investigation results, as described in Chapter 4. The minimum sample rate used for data recording was 24 Hz. As this type of prestressing loading is considered as static loading, there was no need for this high sample rate of recording. Therefore, data was resampled for 3 Hz by taking the average of each eight successive records. This resampling facilitates handling and correcting the captured data. As the test was done inside the lab no effect for temperature was considered for results’ correction.
The initial measurement before starting experimental testing were set to zero. The loading methodology of prestressing forces was considered as a slow static test, due to the slow rate of applying prestressing forces. So, minor strain variations that happened within different loading stages were neglected. Approximation trend lines were developed for each strain gauge as shown previously in Figure 4-44. These linear trend lines represent the beginning and the end of each stressing stage. This methodology facilitates processing and interpretation for the measured data.

6.8.3 Estimation of actual prestressing forces using strand strains

Obtaining prestressing force representing the actual force applied on the anchorage zone was essential. The measured pressure values using the pressure gauge were transferred into prestressing forces based on the constant cross-sectional area of the prestressing jack. These measured forces represent the external force applied on the specimen during stressing; however, during anchorage seating, after stressing or on the other side of the specimen, the forces are not same. The reason for different forces at various locations as well as force reduction with time was the prestressing losses.

In order to obtain the actual force applied at each side of the specimen, the pressure gauge force was correlated to the strains developed in the strands. The regular correlation between stress and strains in the strands were based on the constant elastic modulus of the strands. The direction of the attached gauge changes with the elongation of strands since the stands are made of up seven twisted wires. The developed strains in the strands were correlated each loading step with the prestressing force applied using the jack. The correlation for specimen S45 at loading step 11 is shown in Figure 6-70. This figure
illustrates that the strain value was corrected by dividing the strains with a correction factor of 11.3 to obtain the adequate force values applied on the middle anchor of specimen end S05L.

![Figure 6-70: Correlation between pressure gauge force and develop strand’s strain for specimen S45 at loading step 11.](image)

These forces obtained directly from the strands are more representative, as it eliminate the losses and directly used to obtain actual forces applied on the anchorage zone. The actual force on each end of the specimen was calculated by summation of correlated strain at each end. The developed correlated force for specimen S45 is plotted in Figure 6-71. This figure presents the average force in each tendon (top, middle and bottom) as well as the force on each specimen end.
Figure 6-71: Correlated prestressing force at both ends of specimen S45.

6.9 Material Testing

The concrete, reinforcing bars, prestressing strand and anchorage properties are presented in this section. All the material testing was done according to American Society for Testing and Material (ASTM) specifications.

6.9.1 Concrete properties

Three concrete batches were used for casting experimental specimens. The first one was used for the single girder specimens. The second batch was used for the soffit, web and diaphragms of the double girder specimens. Last concrete batch was used for the deck of the double girder specimens. Casting the double girder specimens in two batches matches exactly the actual construction procedure in the field, where they make two concrete pours for the box girders.

Concrete compressive strength for the three batches was done according to ASTM C39/C39M – 16 [42]. Also, splitting tensile strength test for different concrete batches was performed using ASTM C496/C496M – 11 [43]. All these tests were repeated three times for each concrete batch. The average properties for each batch are summarized in Table
It includes compressive strength, splitting tensile strength as well as the modulus of elasticity for the three concrete batches. Modulus of elasticity for concrete cylinders was obtained on test day according to ASTM C469/C469M – 14 [44].

Table 6-7: Concrete properties for experimental specimens on test day.

<table>
<thead>
<tr>
<th>Batch</th>
<th>Compressive Strength (ksi)</th>
<th>Splitting Tensile Strength (ksi)</th>
<th>Modulus of Elasticity (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch I for single girder specimens</td>
<td>6.49</td>
<td>0.59</td>
<td>2981.25</td>
</tr>
<tr>
<td>Batch II for double girder specimens (soffit, web and diaphragm)</td>
<td>5.87</td>
<td>0.56</td>
<td>3958.35</td>
</tr>
<tr>
<td>Batch III for double girder specimens (deck)</td>
<td>5.79</td>
<td>0.39</td>
<td>3424.86</td>
</tr>
</tbody>
</table>

Detailed testing results for concrete as well as reinforcing bars are shown in Appendix (D). Stress-strain relationship for one of the tested specimens while obtaining the modulus of elasticity is shown in Figure 6-72. This figure illustrates that the ultimate strain of concrete was nearly 0.005 and the strain at maximum compressive strength was 0.003.

![Figure 6-72: Stress-strain relationship for concrete cylinder of single girder specimens’ concrete batch on test day.](image-url)
6.9.2 Reinforcing bar properties

Two groups of reinforcing bars were used in the experimental specimens. The first one was used in single girder specimens, S45 and S56. The second group was used for the double girder specimens, DS6 and D55. Tensile strength test was done for reinforcing bar samples according to ASTM A370 – 17 [45]. The develop stress-strain curves for #3 and #4 rebars used in single girder specimens S45 and S56 are shown in Figure 6-73. The stress-strain relationships for the second rebar group from specimens DS6 and D55 are shown in Figure 6-74. Based on these stress-strain relationships all the reinforcing bars satisfy the requirements for Grade 60, which is the desired grade.

Figure 6-73: Stress-strain relationship for reinforcing bars used in specimens S45 and S56.
Figure 6-74: Stress-strain relationship for reinforcing bars used in specimens DS6 and D55.

6.9.3 Prestressing strands and anchorages properties

The supplier for strands, ducts, wedges, anchors, head plates and trumpets was DYWIDAG System International. Based on their brochure for DYWIDAG Post-Tensioning Systems [46], the 0.6” diameter strand had a cross-sectional area of 0.217 in$^2$. The yield strength measured at 1% extension under load for these strands was 243 ksi and the ultimate strength was 270 ksi. Elastic modulus for these strands was 28,000 ksi.

The anchor was a multiplane anchorage (MA) with anchorage size of 12-0.6”, which fits 12 strands with a diameter of 0.6”. The block-out diameter for this anchor was 10” and the anchor diameter was 8- 5/8”. Wedge plate diameter was 6- 5/16”, and its thickness was 1-11/16”. The trumpet length was 12-7/8”. The required spirals for local zone was #5 Grade 60 rebars with 1-7/8” pitch.
Chapter (7) – Experimental Results and Discussion

7.1 Introduction

Four experimental specimens were cast and tested to assess the performance of the anchorage zones for box girder bridges. Crack patterns developed in each specimen are presented to capture force spreading as well as to determine critical elements affected by post-tensioning. Crack widths were measured at different loading stages to compare between specimens.

Each specimen contained more than 150 strain gauges. The longitudinal and transverse strains developed in the web girders were investigated. Strain distribution along instrumented reinforcement is presented for the diaphragm inner face transverse and vertical directions, the web vertical direction as well as the deck and soffit transverse directions. Also, strain distribution in the transverse direction ahead of the prestressing block-out was investigated in addition to the strain distribution around diaphragm openings. Effect of grillage implementation in the outer side at the anchors were compared to the case where no-grillage was implemented. The adequacy of local zone performance was calibrated using the developed strain of the anchorage spiral reinforcement. Low level of strains were observed in the anchorage spirals. Based on the string pots results, axial deformation for all of the experimental specimens were determined. Failure occurred in the double girder specimens. Failure modes and reasons are illustrated at the end of this chapter. Finally, findings and conclusions obtained from the experimental investigation are presented.
7.2 Developed Crack Patterns

Crack pattern observations are initial indication of the performance. This section illustrates the developed crack pattern for each specimen. Cracks were observed on the diaphragm inner face, diaphragm side, web, deck and soffit. However, no cracked were observed on the outer diaphragm face. Comparisons between developed cracks were conducted to investigate main parameters affecting crack width.

7.2.1 Single girder specimen S45

The crack patterns after testing of specimen S45 are illustrated for the diaphragm inner face and side in Figures 7-1 and 7-2, respectively. For the diaphragm cracks, they were distributed more on specimen end S04 than S05L. Also, maximum crack widths developed on specimen ends S04 and S05L were 0.045” and 0.04”, respectively. One crack was developed in the middle of the diaphragm side for both specimen ends S04 and S05L, which represents bending of the diaphragm edge. The diaphragm was supported at the top and bottom by the deck and soffit, however, the middle height of the diaphragm was free to deflect.
The crack patterns of the web are illustrated in Figure 7-3. Cracks occurred in the web were always adjacent to the prestressing ducts as shown in the same figure, where the ducts are indicated with blue dotted lines. The low web resistance adjacent to the prestressing ducts and the transverse tension caused the cracks to form.
The number of web cracks was higher on specimen end S04 than S05L, which demonstrates the impact of diaphragm width on the strains developed in the web. The thicker the diaphragm the lower the strain in the web. Maximum crack widths developed on specimen ends S04 and S05L were 0.016” and 0.013”, respectively.

![Figure 7-3: Crack pattern for the web of specimen S45.](image)

Crack patterns developed in the deck and soffit are illustrated in Figures 7-4 and 7-5, respectively. There was no significant difference in the crack pattern in the deck between specimen ends S04 and S05L. However, there were slightly more cracks in the soffit at specimens end S04 than S05L, which may indicate a limited effect of diaphragm width on the distribution of forces within the deck and the soffit of the box girder section.
7.2.2 Single girder specimen S56

The crack patterns after testing of specimen S56 are illustrated for the diaphragm inner face and side in Figures 7-6 and 7-7, respectively. More crack developed in specimen end S05H than S06. Also, maximum crack width developed on both specimen ends S05H and S06 was 0.035”. There are greater distribution of cracks with smaller width on
specimen end S05H showing the effect of having higher reinforcement implemented in this end. The cracks developed in the diaphragm side indicate the flexural performance of the diaphragm edge, where one horizontal crack was developed at mid height and inclined cracks were developed at the top and bottom of the diaphragm side for both specimen ends S05H and S06.

(a) End S06
(b) End S05H
Figure 7-6: Crack pattern for the diaphragm inner face of specimen S56.
Figure 7-7: Crack pattern for the diaphragm side of specimen S56.

The crack patterns of the web are illustrated in Figure 7-8, where developed cracks were adjacent to the prestressing ducts. The number of web cracks developed in specimen ends S05H and S06 were the same, however they were less than those developed in specimen end S04. Maximum crack widths developed on specimen ends S05H and S06 were 0.013” and 0.010”, respectively. This shows the effect of diaphragm width on the strains developed in the web. The thicker the diaphragm the lower the strain in the web.

Figure 7-8: Crack pattern for the web of specimen S56.
Crack patterns developed in the deck and soffit are illustrated in Figures 7-9 and 7-10, respectively. Distribution of cracks was in the shape of a cone, starting from the prestressing anchorage then spreading in the deck and the soffit of the specimens. There was no significant difference in the crack pattern of the deck and soffit between specimen ends S05H and S06. This indicated a limited effect of diaphragm width on the distribution of forces within the deck and the soffit of the box girder section.

(a) End S06
(b) End S05H
Figure 7-9: Crack pattern for the deck of specimen S56.

(a) End S06
(b) End S05H
Figure 7-10: Crack pattern for the soffit of specimen S56.
7.2.3 Double girder specimen DS6

Double girder specimen DS6 has two ends DST and D6T. End DST is a solid diaphragm and end D6T has a large opening $0.6S \times 0.6h$. Crack pattern for each end of the double girder specimen DS6 is presented.

7.2.3.1 Specimen DS6 – End DST

The crack patterns after loading stage 18 of specimen end DST are illustrated for the diaphragm inner face between the girders and outside them in Figures 7-11 and 7-12, respectively. Failure occurred in this specimen at specimen end D6T- Girder B during stressing stage 21 at 136.7% of the actual design level as presented previously in Table 6-5. Maximum crack width developed on the inner diaphragm face between girders was 0.045” and 0.03” outside the girders. Vertical cracks occurred adjacent to the girders, however horizontal cracks extended through the whole diaphragm. This illustrates that maximum transverse bursting forces in the diaphragm occur at the web.

Diaphragm side crack patterns after loading stage 18 of specimen end DST are shown in Figure 7-13. Cracks developed on the diaphragm side indicating the flexural performance of the diaphragm edge, where horizontal cracks developed at different levels along specimen height. Maximum crack widths of 0.03” on the diaphragm side were the same as the cracks on the diaphragm inner face outside the girders.
Figure 7-11: Crack pattern for the diaphragm inner face of specimen end DST between girder.

Figure 7-12: Crack pattern for the diaphragm inner face of specimen end DST outside the girders.

(a) Girder B   (b) Girder A
The crack patterns of the web inner and outer faces are illustrated in Figures 7-14 and 7-15, respectively. The developed cracks were adjacent to the prestressing ducts, where the ducts are indicated with blue dotted lines. The cracks that developed on the inner and outer faces were identical, which shows the uniform distribution of bursting stresses on both sides of the web. Maximum crack width developed on webs was 0.02".

(a) Girder B
(b) Girder A
Figure 7-13: Crack pattern for the diaphragm sides of specimen end DST.

(a) Girder B
(b) Girder A
Figure 7-14: Crack pattern for the webs’ inner faces of specimen end DST.
Crack patterns developed in the deck top and bottom faces are illustrated in Figures 7-16 and 7-17, respectively. Distribution of cracks was like a cone, starting from the prestressing anchorage spreading in the deck top face, however, nearly no cracks were observed at the bottom face of the deck. This may have occurred because the web intersecting the bottom face of the deck resists or minimizes the strain distribution along the deck bottom face.
Figure 7-16: Crack pattern for the deck top face of specimen end DST.

(a) Between girders
(b) Outside Girder A

(a) Outside Girder B
(b) Outside Girder A

Figure 7-17: Crack pattern for the deck bottom face of specimen end DST.
7.2.3.2 Specimen DS6 – End D6T

The crack patterns after loading stage 18 of specimen end D6T are shown for the diaphragm inner face between the girders and outside the girders in Figures 7-18 and 7-19, respectively. Failure occurred in this specimen at end D6T-Girder B during stressing stage 21. Between girders, vertical cracks occurred adjacent to the girders and horizontal cracks were spreading around the opening on both sides. Few cracks occurred at the top and bottom of the opening. Outside the girders, vertical cracks occurred adjacent to the girders, however horizontal cracks extended through the whole diaphragm. This illustrated that maximum transverse bursting forces in the diaphragm occur at the web.

Maximum crack width developed on the inner diaphragm face between girders was 0.075” and 0.06” outside the girders. Crack widths clarify the effect of diaphragm opening in increasing the crack width in the inner diaphragm face compared to specimen end DST, which has a solid diaphragm.

Crack patterns on the sides of the opening after loading stage 18 of specimen end D6T are shown in Figure 7-20. The cracks developed on the side were inclined starting from specimen centroid spreading to the top and bottom developing the shape of a punching cone. This spreading indicates the punching failure of the anchors inside the diaphragm, which was demonstrated by the failure modes of the specimens illustrated at the end of this chapter.
Figure 7-18: Crack pattern for the diaphragm inner face of specimen end D6T between girder.

Figure 7-19: Crack pattern for the diaphragm inner face of specimen end D6T outside the girders.
Figure 7-20: Crack pattern for the opening sides of specimen end D6T.

Diaphragm side crack patterns of specimen end D6T are shown in Figure 7-21. The cracks developed on the side face of the diaphragm indicate the flexural performance of the diaphragm edge, where one horizontal crack is developed at mid-height of the specimen. At the top of Girder A diaphragm side, long horizontal crack was also observed. This crack occurred due to the construction joint implemented between the diaphragm and the deck, as two concrete batches were used to cast the double girder specimens. The first batch was used for the soffit, web and diaphragm, however, the second concrete batch was used for the deck.
The crack patterns of the web inner and outer faces are shown in Figures 7-22 and 7-23, respectively. The developed cracks were adjacent to the prestressing ducts, where the ducts are indicated with blue dotted lines. The cracks developed on the inner and outer faces were identical, which demonstrated the uniform distribution of bursting stresses on both sides of the web. Maximum crack width developed on webs was 0.025”. The crack patterns were similar to the ones for specimen end DST, which indicate slight effect of the openings on the bursting forces developed in the web.
Crack patterns developed in the deck top face are illustrated in Figure 7-24. Distribution of cracks was like a cone, starting from the prestressing anchorage spreading in the deck top face. Maximum crack width observed in the deck was 0.02”.

Figure 7-24: Crack pattern for the deck top face of specimen end DST.
7.2.4 Double girder specimen D55

Double girder specimen D55 had two ends D5T and D5H. Both ends had openings with the same dimension of $0.5S \times 0.5h$, however they had different reinforcement around the openings. Reinforcement around the opening in specimen end D5T is based on CALTRANS – Utility opening box girder B7-10 [38]. The specified reinforcement was doubled in specimen end D5H.

7.2.4.1 Specimen D55 – End D5T

The crack patterns after loading stage 18 of specimen end D5T are illustrated for the diaphragm inner face between the girders and outside them in Figures 7-25 and 7-26, respectively. Failure occurred in this specimen at end D5H - Girder B after stressing stage 23, then occurred at end D5T - Girder A during removing the applied vertical loads. Between girders, vertical cracks occurred adjacent to the girders and horizontal cracks were spreading around the opening on both sides. No cracks occurred at the top and bottom of the opening. Outside the girders, vertical cracks occurred adjacent to the girders, however horizontal cracks extended through the whole diaphragm. This illustrates that maximum transverse bursting forces in the diaphragm occurred at the web.

Maximum crack width developed on the inner diaphragm face between girders was 0.06” and 0.05” outside the girders. Crack widths show the effect of the diaphragm opening in increasing the crack width in the inner diaphragm face compared to specimen end DST, which had a solid diaphragm. Crack widths illustrate that the larger the opening, the wider the cracks for the same amount of reinforcement compared to specimen end D6T.
Crack patterns on the sides of the diaphragm opening after loading stage 18 of specimen end D5T are shown in Figure 7-27. The cracks developed on the side were inclined starting from specimen centroid spreading to the top and bottom. This spreading indicated the punching of the anchors inside the diaphragm, which was also demonstrated by the failure modes of the specimens illustrated at the end of this chapter.

Figure 7-25: Crack pattern for the diaphragm inner face of specimen end D5T between girder.
Diaphragm side crack patterns of specimen end D5T are shown in Figure 7-28. The cracks developed on the side indicate the flexural performance of the diaphragm edge, where horizontal cracks were developed along the height of the specimen. At the top of
Girder A diaphragm side, long horizontal crack was also observed. This crack occurred due to the construction joint implemented between the diaphragm and the deck.

![Girder B](image1.png) ![Girder A](image2.png)

Figure 7-28: Crack pattern for the diaphragm sides of specimen end D5T.

The crack patterns of the web inner and outer faces are illustrated in Figures 7-29 and 7-30, respectively. The developed cracks were adjacent to the prestressing ducts, where the ducts are indicated with blue dotted lines. The cracks developed on the inner and outer faces were identical. Maximum crack width developed in the webs was 0.025”. The crack patterns were similar on both ends of specimen DS6, which indicate the slight effect of openings on the bursting forces developed in the web.
Crack patterns developed in the deck top face are illustrated in Figure 7-31. Distribution of cracks was like a cone, as observed in the other specimens. Maximum crack width observed in the deck was 0.02”. The crack patterns were similar to the developed ones on both ends of specimen DS6, which indicate the slight effect of openings on the bursting forces developed in the deck.
7.2.4.2 Specimen D55 – End D5H

The crack patterns after loading stage 18 of specimen end D5H are illustrated for the diaphragm inner face between the girders and outside them in Figures 7-32 and 7-33, respectively. Between girders, vertical cracks occurred adjacent to the girders and horizontal cracks were spreading around the opening on both sides. No cracks occurred at the top and bottom of the opening. Outside the girders, vertical cracks occurred adjacent to the girders, however horizontal cracks extended through the whole diaphragm. This showed that maximum transverse bursting forces in the diaphragm occurred at the web.

Maximum crack width developed on the inner diaphragm face between girders was 0.05” and 0.04” outside the girders. By comparing the developed crack width to the end that has identical opening dimensions specimen end D5T, the doubled opening reinforcement effect was illustrated. The specimen end D5H that possesses higher opening reinforcement had smaller crack width. Therefore, implementation of more reinforcement around the diaphragm opening decreased the developed crack widths. Crack widths clarify the effect of diaphragm opening in increasing the crack width in the inner diaphragm face.
compared to specimen end DST, which had a solid diaphragm. It also clarifies that the larger the opening, the wider cracks it has compared to specimen end D6T due to stress concentration that occurs at opening corners.

Crack patterns of the opening sides after loading stage 18 of specimen end D5H are shown in Figure 7-34. The cracks developed on the side were inclined starting from specimen centroid spreading to the top and bottom forming a punching cone. This spreading indicates the punching of the anchors inside the diaphragm, which is demonstrated by the failure modes of the specimens illustrated at the end of this chapter.

Figure 7-32: Crack pattern for the diaphragm inner face of specimen end D5H between girder.
Diaphragm side crack patterns of specimen end D5H are shown in Figure 7-35. The cracks developed on the side face of the diaphragm show the flexural performance of the diaphragm edge, where horizontal cracks are longer at the mid-height of the section.
The crack patterns of the web inner and outer faces are illustrated in Figures 7-36 and 7-37, respectively. The developed cracks are adjacent to the prestressing ducts, where the ducts are indicated with blue dotted lines. The cracks developed on the inner and outer faces were identical. Maximum crack width developed on webs was 0.02”. The crack patterns were similar for all double girder specimen ends, which indicate the slight effect of openings on the bursting forces developed in the web.
Crack patterns developed in the deck top face are illustrated in Figure 7-38. Distribution of cracks was like a cone, starting from the prestressing anchorage spreading in the deck top face. Maximum crack width observed in the deck was 0.02". The crack patterns were similar for all double girder specimen ends, which indicate the slight effect of openings on the bursting forces developed in the deck.

7.2.5 Crack pattern discussion

Crack widths were measured at different loading stages during testing using a crack comparator as shown in Figure 7-39. The crack widths developed for the single girder
specimens are summarized in Table 3-2. It includes cracks developed in the web, the diaphragm inner face and the top face of the deck at different loading stages. The cracks started in all specimens after loading step #4 at 51.7% of the actual design level in the diaphragm inner face and were in all major elements after loading step #9 at 108.9% of the actual design level. This likely means that most of the constructed bridges reach the cracking limit in the diaphragm inner face just due to stressing.

Figure 7-39: Crack comparator used to measure developed cracks’ widths.
Table 7-1: Crack widths after loading stages for single girder specimens.

<table>
<thead>
<tr>
<th>Spec.</th>
<th>End</th>
<th>Loading step #</th>
<th>Design Level* (%)</th>
<th>Diaphragm</th>
<th>Web</th>
<th>Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td>S45</td>
<td>S04</td>
<td>4</td>
<td>51.9</td>
<td>0.009</td>
<td>0.005</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>77.8</td>
<td>0.025</td>
<td>0.007</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>108.9</td>
<td>0.035</td>
<td>0.010</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>133.3</td>
<td>0.045</td>
<td>0.016</td>
<td>0.013</td>
</tr>
<tr>
<td>S45</td>
<td>S05L</td>
<td>4</td>
<td>51.9</td>
<td>0.007</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td></td>
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</tr>
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<td></td>
<td></td>
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<td>108.9</td>
<td>0.030</td>
<td>0.010</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>133.3</td>
<td>0.040</td>
<td>0.013</td>
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</tr>
<tr>
<td>S45</td>
<td>S05H</td>
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<td>51.9</td>
<td>0.005</td>
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<td>77.8</td>
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<td>108.9</td>
<td>0.030</td>
<td>0.010</td>
<td>0.010</td>
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<td></td>
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<td>133.3</td>
<td>0.035</td>
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<td>9</td>
<td>108.9</td>
<td>0.030</td>
<td>0.009</td>
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<td>133.3</td>
<td>0.035</td>
<td>0.010</td>
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</tr>
</tbody>
</table>

All measured crack widths are in inches

(*) Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1350 kips for single girder specimens.

A comparison was conducted between developed cracks in single girder specimens as shown in Figure 7-40. The main difference between single girder specimens was changing the diaphragm width. The cracks developed in the inner diaphragm face significantly change in different specimens. This illustrates that the crack width in the
diaphragm inner face is inversely proportional with the diaphragm width. The same effect was observed in the crack widths developed in the web, but with a slight difference in crack size. There is no significant difference between crack widths developed in the deck for diaphragm width, which clarifies that no significant effect of the diaphragm width on the cracks developed in the deck.

Figure 7-40: Comparison between maximum crack widths for single girder specimens.

The crack widths developed in double girder specimens are summarized in Table 7-2. It includes cracks developed in the diaphragm inner face between webs and outside them, the web and the top face of the deck at different loading stages. Cracks were seen in all specimens after loading step #4 in the diaphragm inner face and seen in all elements after loading step #12. This likely means that most of the constructed bridges reach the cracking limit in the diaphragm inner face just due to stressing.
Table 7-2: Crack widths measured after loading stages for double girder specimens.

<table>
<thead>
<tr>
<th>Spec.</th>
<th>End</th>
<th>Loading step #</th>
<th>Design Level* (%)</th>
<th>Diaphragm outside webs</th>
<th>Diaphragm between webs</th>
<th>Web</th>
<th>Deck</th>
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</thead>
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<td>0.0042</td>
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</tr>
<tr>
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<td></td>
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<td>0.0188</td>
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<td>D6T</td>
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<td>0.0042</td>
<td>0.0042</td>
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</tr>
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<td></td>
<td></td>
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<td>71.6</td>
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<td>0.0188</td>
<td>0.0104</td>
<td>0.0042</td>
</tr>
<tr>
<td></td>
<td></td>
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<td>97.7</td>
<td>0.0208</td>
<td>0.0292</td>
<td>0.0125</td>
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<tr>
<td></td>
<td></td>
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All measured crack widths are in inches

(*) Design level was calculated according to actual compressive strength on test day using Caltrans Memo to designer 11-25, as shown in Appendix B. Design level = 1075 kips for single girder specimens.

A comparison was conducted between developed cracks in double girder specimens as shown in Figure 7-41. The main difference between double girder specimens was the existence of diaphragm openings. The crack widths developed between the webs are larger
than those developed outside the webs. The effect of crack continuity between webs may be the reason of having bigger cracks in addition to the opening existence between the webs for specimens ends D5T, D5H and D6T. The cracks developed in the inner diaphragm face significantly change in different specimens. This showed that the crack widths in the diaphragm inner face were inversely proportional with the opening dimensions. The larger the opening, the higher the stress concentration that occur at the corners. This enlarged the crack width and caused the difference between tested specimens. The same effect was observed in the crack widths developed in the web, but with slight effect. Maximum crack widths at the top face of the deck was identical, which clarifies that no significant effect of the diaphragm openings on the cracks developed in the deck.

![Figure 7-41: Comparison between maximum crack widths for double girder specimens.](image)

### 7.3 Experimental Results

Experimental testing developed significant amount of data. This data includes strains developed on the instrumented reinforcement, concrete strains, displacement of the specimens during testing, and pressure values for the prestressing jack. Results of more than 150 strain gauges implemented in each specimen were collected. This data was used to study the performance of the general anchorage zone of box girder bridges.
Comparisons were conducted between strain gauges results, in order to assess the performance of the anchorage zone at different locations. Results of strains in the anchorage zone spiral were compared to study its adequacy. Strains on the reinforcement on the outer diaphragm face surrounding anchorages strains were plotted to investigate the performance for the condition of using the typical reinforcement grillage of #4 at 4” stated by Caltrans [37]. This will help to determine if this reinforcement is necessary. Spalling strains developed at the outer diaphragm face were compared and recommendations to resist it were developed. Web transverse as well as longitudinal concrete strains are plotted and discussed. Axial deformations of the specimens are presented, which were measured using string pots attached to the specimens during testing.

### 7.3.1 Major bursting strains

This section presents the major bursting reinforcement strains for both single and double girder specimens. These bursting strains included: vertical and transverse directions of the diaphragm inner face, deck and soffit transverse direction and web vertical direction.

#### 7.3.1.1 Single girder specimens

Measured strains of reinforcement for specimen S45 ends S04 and S05L are shown in Figures 7-42 and 7-43, respectively. These results include strains of the diaphragm inner face vertical and transverse directions, diaphragm top and bottom transverse direction, deck and soffit transverse direction and web vertical direction. The same bursting strains are plotted for specimen ends S05H and S06 as shown in Figures 7-44 and 7-45, respectively. These results were used to develop the strain distribution along different element at various loading stages. These strain distributions are presented in the following section 7.4.
Figure 7-42: Developed reinforcement strains of single girder specimen end S04.
Figure 7-43: Developed reinforcement strains of single girder specimen end S05L.
Figure 7-44: Developed reinforcing strains of single girder specimen end S05H.
Figure 7-45: Developed reinforcement strains of single girder specimen end S06.
7.3.1.2 Double girder specimens

Measured strains of reinforcement for specimen ends DST and D6T are shown in Figures 7-46 and 7-47, respectively. These results include strains of the diaphragm inner face vertical and transverse directions, diaphragm top and bottom transverse direction, deck and soffit transverse direction and web vertical direction.

The same bursting strains are plotted for specimen D55 ends D5T and D5H as shown in Figures 7-48 and 7-49, respectively. These results were used to develop the strain distribution along different element at various loading stages. These strain distributions are presented in the following section 7.4.
Figure 7-46: Developed reinforcement strains of single girder specimen end DST.
Figure 7-47: Developed reinforcement strains of single girder specimen end D6T.
Figure 7-48: Developed reinforcement strains of single girder specimen end D5T.
Figure 7-49: Developed reinforcement strains of single girder specimen end D5H.
7.3.2 Local anchorage zone reinforcement strains

Strain gauges were implemented on the spirals of the local anchorage zone as shown in Figure 7-50 for single girder specimens S45 and S56. Three strain gauges were attached at each end with a total of 12 gauges installed in both specimens. The developed strains in these gauges are shown in Figure 7-51. The ultimate capacity of the used multiplane anchorage with 12-0.6” strands was 703.2 kips. Typically, the applied service load on the anchors is 75% of its ultimate capacity, which is 527.4 kips. The applied load during testing single girder specimens exceeded these service limits by approximately 20% (640 kips). Low strain levels in the spirals were observed. Even with the high force levels applied on each of the anchorage devices, the strains developed in the spiral did not exceed 50% of the yield strain. This illustrates the adequacy of the local anchorage zone reinforcement.

Figure 7-50: Strain gauge attached to multiplane anchor implemented in single girder specimens.
7.3.3 **Outer diaphragm face surrounding anchorages strains (Grillage strains)**

A reinforcement grillage of #4 at 4” is typically implemented at the outer face of the diaphragm at prestressing anchors as presented in CALTRANS – Cast-in-place prestressed girder details B8-5 [37]. This grillage extends 6” beyond the prestressing block-out limits inside the diaphragm. In the scale models the grillage was #3 at 6” in single girder specimen end D5H. In all of the double girder experimental specimens, reinforcement grillage was implemented at one girder in the outer face of the diaphragm (Girder A). The grillage was implemented at one girder only to compare the two cases if it exists or no. Strain gauges attached to the grillage bars were labeled (G) standing for grillage. In Girder B, there was no grillage reinforcement implemented, concrete gauges were installed and labeled (N) standing for no-grillage.

Strains developed in the grillage implemented at single girder specimen end D5H are shown in Figure 7-52. It clarifies that all developed strains around the anchors are
compressive strains with low value (less than 400 microstrain). Also, no cracks were observed around prestressing anchors for all single girder specimens.

Figure 7-52: Measured strains at outer diaphragm face surrounding anchorages for specimen end S05H.

Strains developed at outer diaphragm face surrounding anchorages of specimen DS6 for both ends DST and D6T are shown in Figures 7-53 and 7-54, respectively. Reinforcement strains for the grillage implemented at Girder A are presented in addition to the concrete strains for the side with no grillage at Girder B. Strains developed at both girders were low compressive strains, which can be handled by concrete compressive strength with no need for grillage reinforcement. As shown in Figure 7-55, no cracks were developed at any of the outer faces of the diaphragms at Girder A with reinforcement grillage or at Girder B with no grillage for specimens DS6 before failure.
Figure 7-53: Measured strains at outer diaphragm face surrounding anchorages for specimen end DST.

(a) Girder A – Grillage Side
(b) Girder B – No Grillage Side

Figure 7-54: Measured strains at outer diaphragm face surrounding anchorages for specimen end D6T.

(a) Girder A – Grillage Side
(b) Girder B – No Grillage Side
Figure 7-55: Outer faces of the diaphragm at prestressing block out for specimen DS6 before failure.

Strains developed at outer diaphragm face surrounding anchorages of specimen D55 for both ends D5T and D5H are shown in Figures 7-56 and 7-57, respectively. Reinforcement strains for the grillage implemented at Girder A are presented in addition to the concrete strains for the side with no grillage at Girder B. At specimen end D5T, five concrete gauges were implemented at the grillage side, which were labeled (GC). These gauges were installed near the locations of the reinforcement gauges attached to the grillage. No significant difference was observed between strains of concrete gauges implemented at Girders A and B, which clarifies that no significant effect for the grillage reinforcement to resist compressive forces developed at this zone. Strains developed at both girders were low compressive strains, which can be handled by concrete compressive strength with no need for grillage reinforcement.
(a) Girder A – Grillage Side – Reinforcement Gauges

(b) Girder A – Grillage Side – Concrete Gauges

(c) Girder B – No Grillage

Figure 7-56: Measured strains at outer diaphragm face surrounding anchorages for specimen end D5T.
Based on the strain results developed at the outer diaphragm face around the prestressing anchorages, the grillage reinforcement has no significant role in resisting compressive forces developed at this zone. However, in case of wide prestressing block-outs grillage reinforcement will be needed to resist concrete shrinkage.

### 7.3.4 Outer diaphragm face strains (Spalling strains)

Reinforcement strain gauges were implemented in the outer face of the diaphragm for single girder specimens to study the spreading of spalling stresses at specimen edge. Also, a vertical concrete gauge was used to investigate force spreading, which was labeled VE02. The developed strains at outer diaphragm face for single girder specimens S45 and S56 are shown in Figures 7-58 and 7-59, respectively. All of the developed vertical strains were compressive strains with low values, which can be resisted by concrete compressive strength. The reason for these vertical compressive strains is the support reaction of the specimens, which causes vertical axial forces in the diaphragm. The transverse strains were
tensile strains with low values, which can be resisted as well by the concrete tensile strength. The only transverse gauge that was subjected to compressive strains was HE03, which was installed at the web centerline very close to the prestressing anchors.

Figure 7-58: Strains at outer diaphragm face for specimen S45.

Figure 7-59: Strains at outer diaphragm face for specimen S56.
Reinforcement strain gauges were implemented in the outer face of the diaphragm for double girder specimens to study the spreading of spalling stresses at the outer face. The developed strains at the outer diaphragm face for double girder specimens DS6 and D55 are shown in Figures 7-60 and 7-61, respectively. All of the developed vertical strains were compressive strains with low values, which can be resisted by concrete compressive strength. The transverse strains were tensile strains with low values for specimen end DST, which has solid diaphragm. These low strain values can be resisted by the tensile strength of concrete. However, specimen ends D6T, D5T and D5H possessed relatively high transverse tensile strains, due to stress concentration around diaphragm openings as well as the reduction that occurred in the concrete section due to existence of the diaphragm openings. The transverse tensile strains developed in specimen end D5H are lower than those developed at specimen end D5T, due to the doubled reinforcement around opening for specimen end D5H.

Figure 7-60: Strains at outer diaphragm face for specimen DS6.
(a) End D5T

(b) End D5H

Figure 7-61: Strains at outer diaphragm face for specimen D55.

Based on these results, the spalling tensile stresses along the loaded face are totally resisted by tensile strength capacity of concrete in case of solid diaphragm. The diaphragm wide section at specimen edge provides sufficient resistance for spalling tensile forces. In case of openings, stress concentration occurs around opening especially at the top and bottom, applying AASHTO LRFD Bridge Design Specifications - Section 5.10.9.3.2 [3] states that the spalling forces shall not be taken to be less than two percent of the total factored tendon force, is adequate.

### 7.3.5 Web transverse strains

Transverse strains developed in the web girders caused by prestressing forces were captured using concrete gauges. These gauges were implemented in the webs of single girder specimens. The developed strains in these concrete gauges are shown in Figure 4-60. The maximum tensile strain of concrete is sensitive to multiple parameters. As a rough assumption, maximum tensile strain of concrete was set as 150 microstrain. From the developed strains, none of strain values exceeded the assumed value of concrete tensile
strains. Since the transverse concrete strains in web girders were less than tensile strain of concrete, they were considered negligible.

Figure 7-62: Transverse concrete strains at the webs of single girder specimens.

7.3.6 Web longitudinal strains

Longitudinal strains developed in web Girder A of specimen end D5T caused by prestressing forces were captured using concrete gauges as shown in Figure 7-63. These gauges were implemented to study the distribution of longitudinal strains in concrete along the web. This specimen end failed after loading step # 23. The developed strains in these concrete gauges up to loading step # 23 before failure are shown in Figure 7-64. All of these gauges were subjected to pure compression. The failure reason can be assessed based on the developed strains. As the theoretical ultimate concrete compressive strain is 0.003, it was assumed that when strains were above 0.003, compressive failure occurred in the web. This failure was followed by the diaphragm failure. Based on these results, as the maximum compressive longitudinal strain developed in the web exceeded 2750 microstrain, web compression failure occurred.
Figure 7-63: Concrete gauges placed longitudinally in web Girder A of specimen end D5T.

Figure 7-64: Longitudinal concrete strains at web Girder A of specimen end D5T.

7.3.7 Specimens axial deformations

Axial deformations of the specimens were recorded using string pots attached to the specimen sides at the deck and the soffit. These devices were attached at one side of the specimen in single girder specimens, however in case of double girder specimens it was attached on both sides. The axial deformation for single girder specimens are shown in Figure 7-65. The top axial strain was larger than on the bottom because the prestressing force cambers up the specimen during stressing.
The axial deformation for double girder specimens are shown in Figure 7-66. The deformations occurred in double girder specimen fluctuated from ascending to descending with each loading step. The reason for this rapid change in the trend is the in-plane twisting of the specimens during stressing. In double girder specimens during stressing Girder A, axial deformation of Girder A increases and axial deformations at Girder B decreases due to specimen twisting. The axial strains developed in double girder specimens are lower than those developed at single girder specimens due to the different elastic modulus of concrete used in both specimens. Modulus of elasticity for concrete of single girder specimens was 2981.25 ksi, however for double girder specimens, it was 3958.35 ksi.
Figure 7-66: Axial strain for double girder specimens.

There were other string pots and LVDTs implemented during experimental testing to measure the deformations of the diaphragm inner face, these devices were attached to threaded rods that were implemented perpendicular to web surface. Due to flexibility of these rods, vibrations that occur during specimens stressing and tiny deformation of the thick diaphragms, significant variations in the output deformations therefore, the results of these devices were cancelled.

Vertical LVDTs used to measure the specimen deflections were compared during testing. Measured specimen deformations included deflecting down due to applied vertical load or cambering up due to prestressing. The longitudinal strain gauges attached to the longitudinal reinforcement at specimen mid-span in the deck and soffit were used to assure that no significant yielding occurred in these bars. This clarifies the adequate flexural design of the specimen to resist bending moments.
7.4 Strain Distribution along Different Elements

In this section of the chapter, the strain distributions along different elements are shown including transverse and vertical directions of the diaphragm inner face, web vertical direction, transverse direction in the deck and soffit as well as transverse strain distribution ahead of the prestressing block-outs. Transverse and vertical strains distributions around diaphragm opening are plotted. Based on these strain distributions, comparisons are conducted and conclusions are highlighted.

7.4.1 Diaphragm transverse strain distribution

The transverse strain distribution for the instrumented reinforcement of the diaphragm inner face for specimen S45 on both ends S04 and S05L are plotted in Figures 7-67 and 7-68, respectively. The strain values always increased towards the web centerline forming a plateau around the web and decreased as the gauges were further from it. This clarifies the importance of extending transverse reinforcement of the diaphragm and prevent any lap splices at web girders. There were no significant difference in strain values between specimen ends. Most of the strains exceed the yield value of the reinforcements. However, to obtain an estimate for the forces in these bars based on elastic modulus of reinforcement, strain level must be below yield. The strain levels were plotted at loading step # 7 (88.1% of actual design level) in addition to the maximum value obtained at loading step # 12 (133.3% of actual design level) in order to correlate the strain and obtain force estimates.

The transverse strain distribution for the instrumented reinforcement of the diaphragm inner face for specimen S56 on both ends S05H and S06 are plotted in Figures
7-69 and 7-70, respectively. There is no significant difference in strain values between specimen ends. Most of the strains exceed the yield value of reinforcement, that’s why the strain levels were plotted at loading step # 7 in addition to the maximum value obtained at loading step # 12. The strains developed at the top and bottom corners are significantly lower than the strains developed along the diaphragm inner face. This shows that the deck and soffit restrain the deformation of the diaphragm at the top and bottom which decreases the developed strains at the corners.

Figure 7-67: Transverse strain distribution of the diaphragm inner face for specimen end S04.

Figure 7-68: Transverse strain distribution of the diaphragm inner face for specimen end S05L.
In order to compare the transverse strains of the diaphragm inner face, the developed strains at loading step # 7 were transformed into bursting forces to study the effect of changing diaphragm width. Percentage of estimated transverse bursting forces in the diaphragm inner face for single girder specimens are presented in Figure 7-71. It illustrates that the transverse bursting forces in the diaphragm inner face were inversely proportional with diaphragm width to box girder height ratio.
Figure 7-71: Percentage of estimated transverse bursting forces in the diaphragm inner face for single girder specimens.

The maximum transverse strain distribution for the instrumented reinforcement of the diaphragm inner face for specimen end DST is plotted in Figure 7-72 for loading step # 21. Most of the strains exceed the yield value of reinforcement. However, to obtain an estimate for the forces in these bars based on elastic modulus of reinforcement, strain level must be below yield. The strain levels are plotted at loading step # 10 as shown in Figure 7-73. The developed strains in this figure did not exceed the actual yield value obtained from rebar testing maximum strains. The strain values always increased towards the web centerline and decreased distant from it. That clarifies the importance of extending transverse reinforcement of the diaphragm and prevent any lap splices at web girders. In this solid diaphragm, measured strains at specimen centerline, which is the center between the investigated girders, were either compression or low tension values. This shows that the double girder specimens perform nearly the same as single girder ones. It is preferable to maintain lap splices for transverse bar at the centerline between girders, as this zone was subjected to the lowest strain values.
Figure 7-72: Transverse strain distribution of the diaphragm inner face for specimen end DST at loading step # 21.

Figure 7-73: Transverse strain distribution of the diaphragm inner face for specimen end DST at loading step # 10.

Strain distribution for specimens with diaphragm opening are plotted for specimens with solid diaphragms to see the impact of openings. The transverse strain distribution for the instrumented reinforcement of the diaphragm inner face for specimen ends D6T, D5T and D5H are plotted in Figures 7-74, 7-75 and 7-76, respectively. Average strain levels for specimen end D6T were lower than the developed values for both end of specimen D55. This indicates that the larger the opening, the lower the strain values developed. The strains developed at the top and bottom corners are lower than the strains developed along the diaphragm inner face, which was observed in single girder specimens as well.
Figure 7-74: Transverse strain distribution of the diaphragm inner face for specimen end D6T.

Figure 7-75: Transverse strain distribution of the diaphragm inner face for specimen end D5T.

Figure 7-76: Transverse strain distribution of the diaphragm inner face for specimen end D5H.
In order to compare the transverse strains of the diaphragm inner face, the developed strains are transformed into bursting forces to study the effect of diaphragm openings. Percentage of estimated transverse bursting forces in the diaphragm inner face for double girder specimens are presented in Figure 7-77. It illustrates that the transverse bursting forces in the diaphragm inner face were inversely proportional with diaphragm opening dimensions. The solid end diaphragm of specimen end DST possessed the higher percentage of bursting forces, however, the diaphragm with largest opening of specimen end D6T has the lowest value.

![Figure 7-77: Percentage of estimated transverse bursting forces in the diaphragm inner face for double girder specimens.](image)

It was observed from the transverse strain distribution at the inner diaphragm face that the strains developed at the top and bottom corners are lower than the strains developed along the diaphragm inner face. This observation is highlighted using the maximum strains in the transverse horizontal bars of the diaphragm inner face as shown in Figure 7-78. The strains developed along the diaphragm were always higher than those developed in the corners. This showed that the deck and soffit restrain the deformation of the diaphragm at the top and bottom which decreased the developed strains at the corners.
7.4.2 Diaphragm vertical strain distribution

Horizontal as well as vertical strain distribution are essential to understand prestressing force spreading within the diaphragm. The vertical strain distribution at inner face of the diaphragm for specimen ends S04 and S05L are shown in Figures 7-79 and 7-80, respectively. The strain distribution was always maximum within the middle third of the diaphragm. The middle third represents the location of prestressing anchors and ducts.

Most of the strains exceeded the yield value of reinforcement. However, to obtain an estimate for the forces in these bars based on elastic modulus of reinforcement, strain level must be below yield. The strain levels were plotted at loading step # 7 in addition to the maximum value obtained at loading step # 12 in order to correlate the strain and obtain force estimates. The strain level developed in specimen end S04 were higher than the developed in specimen end S05L.
Figure 7-79: Vertical strain distribution at inner face of specimen end S04.

Figure 7-80: Vertical strain distribution at inner face of specimen end S05L.

The vertical strain distribution at the inner face of the diaphragm for specimen S56 ends S05H and S06 are shown in Figures 7-81 and 7-82, respectively. The strain distribution was always a maximum within the middle third of the diaphragm. The middle third represents the location of prestressing anchors and ducts. The strain levels were plotted at loading step # 7 in addition to the maximum value obtained at loading step # 12. The strain level developed in specimen end S05H was slightly higher than the developed in specimen end S06.
In order to compare the vertical strains of the diaphragm inner face, the developed strains at loading step # 7 were transformed into bursting forces to study the effect of changing diaphragm width. Percentage of estimated vertical bursting forces in the diaphragm inner face for single girder specimens are presented in Figure 7-83. It illustrates that the vertical bursting forces in the diaphragm inner face are inversely proportional with diaphragm width to box girder height ratio, which is the main difference between single girder specimens.
The vertical strain distribution at inner face of the diaphragm for specimens DS6 and D55 are shown in Figures 7-84 and 7-85, respectively. These plots do not include the developed strains adjacent to the opening, as strains developed around the diaphragm openings will be illustrated later in this chapter. The strain distribution is the same as for single girder specimens, where the maximum values were in the middle third of the height. This illustrates that the double girder specimens performs the same as single girder ones. The developed strains were nearly the same for both specimens in case of a solid diaphragm or diaphragms with openings. This illustrates that the opening existence does not affect the developed vertical bursting forces in the diaphragm inner face.

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<td>3.5</td>
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<tr>
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Figure 7-83: Percentage of estimated vertical bursting forces in the diaphragm inner face for single girder specimens.
Figure 7-84: Vertical strain distribution at diaphragms inner faces of specimen D5S.

Figure 7-85: Vertical strain distribution at diaphragms inner faces of specimen D55.
Transverse distribution for the maximum vertical strains obtained in the diaphragm inner face is plotted in Figure 7-86. This figure illustrates that maximum vertical bursting forces do not occur adjacent to the web face or even at the diaphragm edge. Maximum values occurred at 15” from the web centerline, which nearly represents ¼ of the spacing between girders (S). Even for specimen end DST, maximum value between girders occurred at ¼ of the girder spacing then a plateau of maximum strains developed at the middle between girders. Vertical bursting strains increased distant from the web due to web contribution in resisting these strains.

Figure 7-86: Transverse distribution for maximum vertical strain of the diaphragm inner face.

7.4.3 Web vertical strain distribution

Vertical strain distribution in the web has significant importance, as the only reinforcement value provided by Caltrans Memo to Designer (11-25) - Anchorage Zone Design [1] is the web reinforcement ahead of the anchorage zone. The vertical strain distributions at web girders of single girder specimens S45 and S56 are shown in Figures 7-87 and 7-88, respectively. Strains are plotted at different distances from the diaphragm face including 3”, 9”, 15”, 21”, 27”, 33” and 39”. The strain distribution is always
maximum within the middle third of the height. The middle third represents the location of prestressing tendons.

Strains developed in specimen end S04 were slightly higher than those developed at the specimen end S05L. Also, in specimen S56, strains on end S05L were higher than the values developed on end S06. This shows that the vertical bursting strains in the web are inversely proportional with the diaphragm width to girder height ratio.

Figure 7-87: Vertical strain distribution at web girders of specimen S45.
To compare the vertical strains of the web girders, the maximum developed strains were transformed into bursting forces to study the effect of changing diaphragm width. Percentage of estimated vertical bursting forces in the web for single girder specimens are presented in Figure 7-89. It illustrates that the vertical bursting forces in the web were inversely proportional with diaphragm width to box girder height ratio, which is the main difference between single girder specimens.
Figure 7-89: Percentage of estimated vertical bursting forces in the web for single girder specimens.

The vertical strain distributions of web girders for double girder specimen DS6 ends DST and D6T are shown in Figures 7-90 and 7-91, respectively. Strains are plotted at different distances from the diaphragm face including 3”, 9”, 15” and 21” for both Girders A and B at each side. The strain values were maximum within the middle third of the height like the single girder specimens. The vertical strain distributions of web girders for specimen D55 are shown in Figure 7-92. It includes the results for both ends D5T and D5H.

Comparing the developed strains in all double girder specimens, the strain levels were nearly the same. This illustrates that there is no effect of diaphragm openings on the vertical bursting forces developed in the web girders.
Figure 7-90: Vertical strain distribution at web girders of specimen end DST.

Figure 7-91: Vertical strain distribution at web girders of specimen end D6T.
Longitudinal distribution for the maximum vertical strains obtained in the web girders is plotted in Figure 7-93 for single girder specimens. Significant reduction was observed in the vertical strains after 20” from the diaphragm face, which represents ½ of the girder height (h). The resultant location was calculated based on these developed strains, and it was found to be at 11” from the diaphragm face, which represents ¼ h. The same longitudinal distribution is plotted for double girder specimens as shown in Figure 7-94. The centroid for the developed strains for double girder specimens was calculated to be 9”. This was approximately the same value as obtained for single girder, which represents ¼ h. Based on these figures it can be concluded that the affected zone of the web by the vertical bursting forces extends for a distance of ½ h in the longitudinal direction and the centroid of these bursting forces is located at a distance of ¼ h.
7.4.4 Deck and soffit transverse strain distribution

The transverse strain distributions at the deck and the soffit of single girder specimens are plotted in Figures 7-95 and 7-96, respectively. Strains are plotted at different distances from the diaphragm face including 2” and 14” for the deck and 2” and 13” for the soffit. Maximum strains were developed away from the web centerline, which clarify the role of web section in resisting transverse strains in the deck and soffit. The strains developed in the soffit were slightly higher than those developed in the deck. Based on the strains for both the deck and the soffit at 2” from the diaphragm face, it was noticed that
strains developed in the deck bottom reinforcement layer were higher than those developed in the top layer and strains developed in the soffit top reinforcement layer is higher than those developed in the bottom layer.

(a) 2" from diaphragm face  
(b) 14" from diaphragm face

Figure 7-95: Transverse strain distribution of the deck for single girder specimens at different locations.

(a) 2" from diaphragm face  
(b) 13" from diaphragm face

Figure 7-96: Transverse strain distribution of the soffit for single girder specimens at different locations.
Longitudinal distributions for the transverse strains obtained in the deck and the soffit are plotted in Figures 7-97 and 7-98 for single girder specimens. The longitudinal distribution of the deck was nearly constant, however the strain distribution of the soffit slightly decreased away from the diaphragm face. Also, no significant reduction was observed in strain values, which shows that the transverse strains extended longitudinally along the deck and soffit.

![Figure 7-97: Longitudinal strain distribution of the deck for single girder specimens.](image)

![Figure 7-98: Longitudinal strain distribution of the soffit for single girder specimens.](image)

In order to compare the transverse strains of the deck and soffit for single girder specimens, the developed strains at loading were transformed into bursting forces to study
the effect of changing diaphragm width. Percentage of estimated transverse bursting forces in the deck and soffit for single girder specimens are presented in Figure 7-99. It illustrates that the soffit bursting strains were slightly higher than the deck results. It also clarifies that the transverse bursting forces in the deck and soffit were inversely proportional with diaphragm width to box girder height ratio, which was the main difference between single girder specimens.

Figure 7-99: Percentage of estimated transverse bursting forces in the deck and soffit for single girder specimens.

The transverse strain distribution for the instrumented reinforcement of the deck for specimen DS6 is plotted in Figure 7-100. It includes results of the implemented top and bottom reinforcement of specimen end DST for both girders A and B as well as bottom reinforcement at end D6T. The maximum strains did not exceed the actual yield value obtained from reinforcement testing. It can be noticed from the results of specimen end DST that strains developed in the deck top reinforcement layer were lower than those developed in the bottom layer. The strain values always increase towards the web centerline. In these specimens, measured strains at specimen centerline, which is the center
between the investigated girders, were compression or low tension value. This illustrates that the double girder specimens performs nearly the same as single girder ones.

The transverse strain of the deck for specimen D55 is plotted in Figure 7-101. It includes results of the bottom reinforcement of specimen end D5T for both girders A and B as well as end D5H Girder A. The developed strains had exactly the same distribution as specimen end D6T.

![Graphs of transverse strain distribution](image)

Figure 7-100: Transverse strain distribution of the deck for specimen DS6.
Figure 7-101: Transverse strain distribution of the deck for specimen D55.

The transverse strain distribution for the soffit of specimens DS6 and D55 are plotted in Figures 7-102 and 7-103, respectively. It includes results of the bottom and top reinforcement at different distances 3.5” and 14.5”. It can be noticed from the results of end DST that strains developed in the soffit top reinforcement layer were higher than those developed in the bottom layer. The strain values always increase towards the web centerline. In these specimens, measured strains at specimen centerline, which is the center between the investigated girders, were compression or low tension value. This illustrates that the double girder specimens performed nearly the same as single girder ones. The strains developed in the soffit were higher than those developed in the deck, which indicate the transverse bursting force.

The transverse strain results for both the deck and the soffit show the effect of the diaphragm opening. Specimen end DST possessed lower strain values than specimen ends
D5H, D5T or D6T. This illustrates that diaphragm openings increase the strains developed in the deck and soffit.

![Graphs showing strain distribution](image)

Figure 7-102: Transverse strain distribution of the soffit for specimen DS6.
In order to compare the transverse strains of the deck and soffit for double girder specimens, the developed strains at loading were transformed into bursting forces to study the effect of diaphragm openings. Percentage of estimated transverse bursting forces in the deck and soffit for double girder specimens are presented in Figure 7-104. It illustrates that the soffit bursting strains were higher than the deck results. It also clarifies that the transverse bursting forces in the deck and soffit were inversely proportional with opening dimensions. The correlation was observed from the strain distributions of the deck and soffit. Specimen end DST with solid diaphragm has the lowest percentage of force in the deck and the soffit, however specimen end D6T with largest diaphragm opening has the largest value.
Figure 7-104: Percentage of estimated transverse bursting forces in the deck and soffit for double girder specimens.

Based on the strains for both single and double girder specimens for the deck and the soffit, it was noticed that strains developed in the deck bottom reinforcement layer was higher than those developed in the top layer, and strains developed in the soffit top reinforcement layer were higher than those developed in the bottom layer. This is shown through the percentage of strain difference between bottom and top reinforcement layers for specimens S45, S56 and specimen end DST in Figure 7-105. Strains developed in the inner face of the flanges – bottom face of the deck and top face of the soffit – were subjected to higher strains because they were closer to the prestressing anchors implemented in the web.

Figure 7-105: Percentage of strain difference between bottom and top reinforcement layers for specimens S45, S56 and specimen end DST.
7.4.5 Transverse strain distribution ahead of prestressing block-out

Strain gauges were implemented at different girders ahead of the prestressing block-out for both single and double girder specimens. The strain distributions of these reinforcing bars are presented in Figure 7-106 for single girder specimens and Figure 7-107 for double girder ones. The strain levels of these bars ahead of the block-outs were significantly lower than those in the deck reinforcement beyond the diaphragm face.

Figure 7-106: Transverse strain distribution ahead of prestressing block-out for single girder specimens.

Figure 7-107: Transverse strain distribution ahead of prestressing block-out for double girder specimens DST, D5H and D5T.
The tensile strain ahead of the prestressing block-outs was lower than that in the deck beyond the diaphragm inner face. The lateral spreading of the strain developed ahead of the prestressing block-out was significantly less than the spreading beyond the inner diaphragm face.

### 7.4.6 Vertical and transverse strain distribution around diaphragm openings

Stress levels around diaphragm opening was a main objective for the second phase of the experimental work. Providing adequate reinforcement around the opening will enhance the performance by minimizing the developed cracks. The transverse strain distributions for reinforcement around diaphragm openings of double girder specimens are shown in Figure 7-108. It includes the strains developed on the horizontal and inclined reinforcement above and below diaphragm openings. The reinforcement was implemented based on CALTRANS – Utility opening box girder B7-10 [38], except for specimen end D5H which had double of the required reinforcement.

The strains developed at specimen centerline between girders -which is the opening centerline- were compression or very low tension values. Based on these low strain values, it was concluded that a) the zones above and below the openings were not critical with the presence of Caltrans typical detail reinforcement b) the optimum location for diaphragm opening was at the middle of the diaphragm -centerline between girders-. Maximum strain developed in specimen end D6T was higher than that developed in D5T, which shows higher strains for larger openings. The strain developed in specimen end D5H was significantly lower than that developed in specimen end D5T, because of the doubled reinforcement implemented in specimen end D5H. However, Caltrans details were
considered sufficient to resist the transverse stresses developed at the top and the bottom of the opening. In the case of different opening configuration, reinforcement cut by opening vertical sides must be compensated at the top and bottom of the opening to minimize any stress concentration around diaphragm openings.

Figure 7-108: Transverse strain distribution around diaphragm openings of double girder specimens.
Vertical strain distribution for stirrups above diaphragm openings is plotted in Figure 7-109. It includes the strains for a middle stirrup as well as an edge stirrup above the diaphragm opening. Strain levels were very low indicating that the stirrups specified by Caltrans were sufficient to resist the spreading forces. Also, the strains developed in the edge stirrup were always higher than the middle ones, which clarifies that the top part above the opening behaved as a beam supporting the deck slab. Therefore, the behavior of concrete above the diaphragm opening as a beams shall be considered in the design of the top stirrups.

![Figure 7-109: Vertical strain distribution at inner face of diaphragm for stirrups above openings.](image)

Strains in the vertical reinforcement at inner face of diaphragm on both sides of the diaphragm openings are plotted in Figure 7-110. Large strains were observed at opening corners, where the strain values reached 12,000 microstrain. These very high values occurred as Caltrans details for utility openings do not contain any concentrated vertical reinforcement adjacent to the opening sides. Therefore, any reinforcement that would be cut or removed because of the opening top and bottom edges should be placed on both
sides of the opening to minimize any stress concentration at opening corners. If this concept was applied in specimen end D5T, 3 #3 bars would have been used on both opening sides instead of only 1#3 bar.

Figure 7-110: Vertical strain distribution at inner face of diaphragm openings sides.

7.5 Failure of Specimen Ends

The anchorage zone of three girders failed due to prestressing. All failure occurred in the cases where diaphragm opening exists. This highlights the role of diaphragm openings in decreasing the anchorage zone safety factor. The girders that failed due to stressing were end D6T-Girder B, end D5H-Girder B and end D5T-Girder A.

7.5.1 Specimen end D6T-Girder B

Failure occurred in double girder specimen DS6 at the end of loading step # 21. During this step end DST was loaded at Girder B, however failure occurred at end D6T, which has the large end diaphragm opening. The failure occurred due to concrete crushing of the web as shown in Figures 7-111 and 7-112 for the inner and outer faces of the web, respectively. The concrete crushing was due to compression failure of the web, as shown
by buckling of longitudinal bars between vertical stirrups. This happened on both faces of the web.

Figure 7-111: Compression failure of the web inner face at specimen end D6T-Girder B.

Figure 7-112: Compression failure of the web outer face at specimen end D6T-Girder B.
Web failure extended to the top and bottom of the web in the deck and soffit as shown in Figures 7-113 and 7-114, respectively. The concrete cover was bumped up in the deck in a local zone above the web. Spalling of concrete cover of the soffit occurred in addition to concrete crushing was observed at local zone below the web. Obtaining failure of these local zones in the deck and soffit indicates web failure.

Figure 7-113: Concrete cover bumped up for the deck at specimen end D6T-Girder B.

Figure 7-114: Concrete spalling and crushing for the soffit at specimen end D6T-Girder B.
After compression failure of the web, the diaphragm role was to distribute the total prestressing forces to the deck and soffit. As the diaphragm was not thick enough to distribute the high prestressing forces, punching failure occurred in the diaphragm. Punching cones that occurred in the diaphragm between girders are shown in Figure 7-115. The cone vertex started from the centroid of prestressing anchors. The punching perimeter of the diaphragm outside girders is shown in Figure 7-116. The punching perimeter extended from a horizontal line between the diaphragm and the deck lower face, which was the location of construction joint. The end of the punching cone was the diaphragm inner face.

(a) Diaphragm inner face  
(b) Opening Side  
Figure 7-115: Diaphragm punching failure between girders at specimen end D6T-Girder B.
Figure 7-116: Diaphragm punching failure outside girders at specimen end D6T-Girder B.

During punching failure of the diaphragm, the anchorage head plates penetrated 2” as shown in Figure 7-117. This penetration coincided with the punching failure, where the load was applied at the punching cone vertex.
Figure 7-117: Anchorage head plates penetrate concrete block-out at specimen end D6T-Girder B.

7.5.2 Specimen end D5H-Girder B

Failure occurred in double girder specimen D55 after loading step # 23. During this step end D5T was loaded at Girder B, however failure occurred at end D5H. After finishing loading step # 23 and getting ready for loading step # 24, failure occurred. This indicated that short term creep due to high prestressing forces caused significant crack propagation, which initiated failure. The failure occurred due to concrete crushing of the web as shown in Figures 7-118 and 7-119 for the inner and outer faces of the web, respectively. The concrete crushing observed was due to compression failure of the web, which was determined based on buckling of longitudinal bars between vertical stirrups. This happened on both faces of the web.
Web failure extended to the top and bottom of the web in the deck and soffit as shown in Figures 7-120 and 7-121, respectively. The deck edge at the prestressing block-out was crushed in a local zone above the web. Spalling of concrete cover of the soffit occurred in addition to concrete crushing was observed at local zone below the web. Failure of these local zones in the deck and soffit indicated web failure.
After compression failure of the web, the diaphragm role was to distribute the total prestressing forces to the deck and soffit. As the diaphragm was not thick enough to distribute the high prestressing forces, punching failure occurred in the diaphragm. Punching cones that occurred in the diaphragm between girders are shown in Figure 7-122. The cone vertex started from the centroid of prestressing anchors. The punching perimeter of the diaphragm outside girders is shown in Figure 7-123. The punching perimeter extended from a horizontal line between the diaphragm and the deck lower face, which was the location of construction joint. The end of the punching cone was the diaphragm inner face.
(a) Diaphragm inner face
(b) Opening Side

Figure 7-122: Diaphragm punching failure between girders at specimen end D5H-Girder B.

Figure 7-123: Diaphragm punching failure outside girders at specimen end D5H-Girder B.
During punching failure of the diaphragm, the anchorage head plates penetrated the concrete surface by 1.5” as shown in Figure 7-124. This penetration coincided with the punching failure, where the load was applied at the punching cone vertex.

Figure 7-124: Anchorage head plates penetrate concrete block-out at specimen end D5H-Girder B.

7.5.3 Specimen end D5T-Girder A

A second failure occurred in double girder specimen D55 during unloading the vertical load applied on the specimen after failure of end D5H-Girder B. Cracking sounds were heard, then failure occurred at end D5T-Girder A. This failure also indicate the effect of short term creep due to high prestressing forces causing significant crack propagation. The failure occurred due to concrete crushing of the web as shown in Figures 7-125 and 7-126 for the inner and outer faces of the web, respectively. The concrete crushing observed was due to compression failure of the web. The same exact failure occurred at end D6T-Girder B and end D5H-Girder B.
Web failure extended to the top and bottom of the web in the deck and soffit as shown in Figures 7-127 and 7-128, respectively. The concrete cover was bumped up at the deck edge in a local zone above the web. Spalling of concrete cover of the soffit occurred in addition to concrete crushing was observed at local zone below the web. Obtaining failure
of these local zones in the deck and soffit indicated web failure, which also happened for specimen end D6T-Girder B and specimen end D5H-Girder B.

Figure 7-127: Concrete cover bumped up for the deck at specimen end D5T-Girder A.

Figure 7-128: Concrete spalling and crushing for the soffit at specimen end D5T-Girder A.

After compression failure of the web, the diaphragm role was to distribute the total prestressing forces to the deck and soffit. As the diaphragm was not thick enough to distribute the high prestressing forces, punching failure occurred in the diaphragm. Punching cones that occurred in the diaphragm between girders are shown in Figure 7-129. The cone vertex started from the centroid of prestressing anchors. The punching perimeter
of the diaphragm outside girders is shown in Figure 7-130. The vertical line of the punching perimeter was 16” from the web face. This distance equals to the diaphragm width, that confirms failure type to be diaphragm punching. The same punching failure occurred at end D6T-Girder B and end D5H-Girder B.

During punching failure of the diaphragm, the anchorage head plates penetrated 1.5” as shown in Figure 7-131. This penetration coincides with the punching failure.

Figure 7-129: Diaphragm punching failure between girders at specimen end D5T-Girder A.
Figure 7-130: Diaphragm punching failure outside girders at specimen end D5T-Girder A.

Figure 7-131: Anchorage head plates penetrate concrete block-out at specimen end D5T-Girder A.
7.5.4 Failure load compared to web ultimate capacity

The failure loads for specimen ends D6T-Girder B, D5H-Girder B and D5T-Girder A were 1565.8 kips, 1513.7 kips and 1554.7 kips, respectively. Based on the AASHTO LRFD Bridge Design Specifications - equation 5.7.4.4-3 [3], the ultimate capacity of the web was calculated as an axially loaded member restrained at the top by the deck and at the bottom by the soffit. The modified equation after removing all safety factors is as follows:

\[ P_{n-web} = f'_{c \text{ act.}}(A_g\text{-web} - A_{st-web} - A_{ducts}) + f_{y \text{ act.}}A_{st-web} \]  Eq. 7-1

Where

\( P_{n-web} = \) nominal axial resistance of the web girder (kips)

\( f'_{c \text{ act.}} = \) actual concrete compressive strength at test day obtained from Appendix (D) (ksi)

\( A_g\text{-web} = \) gross area of the web section as shown in Figure 7-132 (in.\(^2\))

\( A_{st-web} = \) total area of longitudinal reinforcement included in the web zone (in.\(^2\))

\( A_{ducts} = \) total area of prestressing ducts (in.\(^2\))

\( f_{y \text{ act.}} = \) actual yield strength of reinforcement obtained from Appendix (D) (ksi)
The axial ultimate capacity of the web obtained based on the previous equation is 1605.36 kips. This value is compared to the failure load of different specimens as shown in Table 7-3. It includes specimen end label, failure load, ultimate capacity and the ratio of failure load to ultimate capacity. This ratio is higher than 94% for the three specimens, which shows that anchorage zone failure likely occurred due to web compression failure.

Table 7-3: Comparison between failure loads and ultimate web capacity.

<table>
<thead>
<tr>
<th>Specimen End</th>
<th>Failure Load (kips)</th>
<th>Ultimate Web Capacity (kips)</th>
<th>Failure load / Ultimate web capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>D6T-Girder B</td>
<td>1565.8</td>
<td>1605.36</td>
<td>97.5 %</td>
</tr>
<tr>
<td>D5H-Girder B</td>
<td>1513.7</td>
<td>1605.36</td>
<td>94.3 %</td>
</tr>
<tr>
<td>D5T- Girder A</td>
<td>1554.7</td>
<td>1605.36</td>
<td>96.8%</td>
</tr>
</tbody>
</table>
However, specimen end DST-Girder B reached 1499.5 kips of prestressing force, no failure occurred. The same also happened for single girder specimen end S04, where jacking force was 1591 kips. This indicates the role of the solid diaphragm in distributing axial prestressing forces on the deck and soffit. When there are diaphragm openings, the loads were concentrated in the web thus reducing the safety factor for the anchorage zone.

7.6 Findings and Conclusions

Based on the experimental results as well as the analytical work done for processing, the following observations, findings and conclusions were determined. These findings and conclusions were obtained based on: a) crack pattern, b) developed strains and deformations, and c) specimens failure.

Findings and observations based on the crack pattern are as follows:

- Cracks were observed in experimental specimens at low levels of loading, which indicates that most of the constructed bridges reached the cracking limit in the diaphragm inner face just due to prestressing.
- Crack widths in the diaphragm inner face were inversely proportional with the diaphragm width. The same effect was observed in the crack widths developed in the web, but with slight effect.
- The crack widths in the diaphragm inner face were inversely proportional with the opening dimensions. The larger the opening, the higher the stress concentration at its corners. Maximum crack widths at the top face of the deck were identical, which clarifies that no significant effect of the diaphragm openings on the cracks developed in the deck.
• Cracks occurred in the web were always adjacent to the prestressing ducts, where reduced concrete section exists at the duct location.

• There was no significant difference in the crack pattern in the deck for single and double girder specimens, which indicates limited effect of diaphragm width and diaphragm opening dimensions on the distribution of forces within the deck and the soffit of the box section.

• Existence of higher reinforcement in the diaphragm causes more distributed cracks with smaller width.

• The cracks developed on the diaphragm side indicated the flexural performance of the diaphragm edge, where one horizontal crack was developed at mid height and inclined cracks were developed at the top and bottom of the diaphragm side.

• Crack patterns at the diaphragm opening sides were inclined starting from specimen centroid spreading towards the deck and the soffit of the specimens developing a punching cone.

• Within the diaphragm inner face, vertical cracks occurred adjacent to the girders, however horizontal cracks extended through the whole diaphragm. This illustrated that maximum transverse bursting forces in the diaphragm occurred at the web.

Findings and conclusions based on the experimental results including strains and displacements are as follows:

• The applied load during experimental testing exceeded service limits of multiplane anchors by 20%, yet the strains developed in the anchor spiral did not exceed 50%
of the yield strain. This illustrated the adequacy of the local anchorage zone reinforcement.

- Based on the strain results developed in the outer diaphragm face around the prestressing anchorages, the grillage reinforcement had no significant role in resisting compressive forces developed at this zone. However, in case of wide prestressing block-outs, grillage reinforcement will be needed to resist concrete shrinkage.

- The spalling tensile stresses along the loaded face were totally resisted by tensile strength capacity of concrete in case of solid diaphragm. The diaphragm wide section at specimen edge provided sufficient resistance for spalling tensile forces. In case of diaphragm openings, stress concentration occurred around the opening especially at the top and bottom. AASHTO LRFD Bridge Design Specifications - section 5.10.9.3.2 [3] states that the spalling forces shall not be taken to be less than two percent of the total factored tendon force, which is sufficient and adequate in cases where a diaphragm opening exists.

- Transverse concrete strains in web girders are less than tensile strain of concrete. So these strains are considered negligible.

- Longitudinal concrete gauges in the web were subjected to pure compression that reached 2750 microstrain, based on these results web compression failure can be concluded.

- Developed transverse strains on the inner face of the diaphragm always increase towards the web centerline. That clarifies the importance of extending transverse reinforcement of the diaphragm and prevent any lap splices at web girders.
The strains developed at the top and bottom corners of the diaphragm inner face are significantly lower than the strains developed along the side. This clarifies that the deck and soffit restrained the deformation of the diaphragm at the top and bottom which decreased the developed strains at the corners.

The transverse bursting forces in the diaphragm inner face were inversely proportional with diaphragm width to box girder height ratio and the diaphragm opening dimensions as well.

The vertical bursting forces in the diaphragm inner face and the web were inversely proportional with diaphragm width to box girder height ratio. The diaphragm opening existence did not affect these vertical bursting forces.

The maximum vertical bursting forces occurred at ¼ of the girder spacing away from the web centerline, then a plateau of maximum strains developed at the middle between girders.

The vertical strain distribution on the inner face of the diaphragm and at the web was always maximum within the middle third of the height.

Vertical web strains were inversely proportional with the distance from the diaphragm face. The affected zone of the web by the vertical bursting forces extends for a distance of ½ h in the longitudinal direction and the centroid of these bursting forces is located at a distance of approximately ¼ h.

The transverse bursting forces in the deck and soffit were inversely proportional with diaphragm width to box girder height ratio and directly proportional with the diaphragm opening dimensions.
• The strains developed in the soffit were slightly higher than those developed in the deck. The strain values always increase towards the web centerline.

• Strains developed in the deck bottom reinforcement layer were slightly higher than those developed in the top layer and strains developed in the soffit top reinforcement layer were slightly higher than those developed in the bottom layer.

• The strain levels of the bars ahead of the block-outs were significantly lower than those in the deck beyond the diaphragm face. The lateral spreading of the strain developed ahead of the prestressing block-out was significantly less than the spreading beyond the inner diaphragm face.

• In cases where a diaphragm opening exists, the strains developed at specimen centerline between girders were compression or very low tension values. This indicated that the zones above and below the openings were not critical with the presence of Caltrans typical detail reinforcement. Also, the optimum location for diaphragm opening was at the middle of the diaphragm.

• Caltrans detail was sufficient to resist the transverse stresses developed at the top and the bottom of the opening, but in case of different opening configuration, reinforcement that would be cut or removed because of the opening edges should be placed above and below the opening.

• High vertical strains were obtained at the diaphragm inner face adjacent to openings. Therefore, reinforcement that would be cut or removed because of the opening top and bottom edges should be placed on both sides to minimize any stress concentration at opening corners.
- Vertical stirrups implemented by Caltrans recommendations above the opening had low strains, which showed that the stirrups provided by Caltrans criteria was sufficient to resist the spreading forces. Also, the strains developed in the edge stirrups are always higher than the middle ones, which clarifies that the top part above the opening behaves as a beam supporting the deck slab.

Based on the failure of the three girders at specimen ends D6T-Girder B, D5H-Girder B and D5T-Girder A, it was concluded that failure occurred due to compression failure of the web, which was assessed based on buckling of longitudinal bars between vertical stirrups. Also, web failure was determined based on the results of longitudinal concrete gauges in the web that reached 2750 microstrain. In addition, short-term concrete creep has significant effect on crack propagation due to high prestressing forces, which initiates failure. Compression web failure was followed by punching failure in the diaphragm, which was assessed based on the developed punching cones, the observed punching perimeter and penetration of anchorage head plates into the concrete.
8.1 Introduction

Finite element (FE) modeling is a powerful tool to expand experimental results. DIANA [28] FE package was used to develop a model that was first verified against the experimental work. In addition, experimental specimens available in the literature for general anchorage zone were used to verify the model. These specimens include six rectangular specimens as well as one T-section specimens that were tested by Sanders in 1990 [11]. This chapter presents the finite element model (FEM) for the general anchorage zone including modeling criteria and the model verification, as well as the development of prototype and parametric study to widen the investigated factors affecting anchorage zone.

The modeling criteria presented includes material models for concrete, reinforcement, anchorage devices and the post-tensioning strands. Also, the procedures for meshing of elements, loading, and numerical solving are included. Development of prototype model based on the verification of scaled specimens was essential. Obtaining the actual spreading bursting forces methodology is presented. The prototype outcomes were checked to be representative for the actual behavior. Parametric study was conducted to investigate the parameters that were not included through the field investigation or the experimental work. These parameters included: geometry of box girder end zone, edge eccentricity, number of anchors, tendon inclination, skew angle and grillage reinforcement. Finally, parametric study outcomes are presented.
8.2 Modeling Criteria

The main variables included in the definition of FEM are: material model, meshing, loading techniques, supporting system and iterative solution procedures. The same modeling criteria was used for all of the developed models to assure that model sensitivity did not affect the model outcomes.

8.2.1 Material models

Four different material models were used to define the FEM elements. These materials were assigned for concrete, anchor devices, reinforcing bars and post-tensioning strands. The definition for each material is illustrated as follows.

8.2.1.1 Concrete

Concrete was defined using Total Strain Crack Model. Parabolic stress-strain relationship was defined for concrete in compression. The parabolic curve is based on fracture energy by the definition of the crack bandwidth of the element, for which DIANA assumes a value $h$ related to the square root of the area of the element. This definition is based mainly on the compressive strength ($f'_c$) and the compressive fracture energy ($G_c$) as shown in Figure 8-1. The compressive strength ($f'_c$) was obtained based on the experimental results of test cylinders. Fracture energy was calculated based on ultimate strain of 0.005 obtained from stress-strain relationship of concrete shown previously in Figure 6-72. Constant shear retention was assumed for the total strain crack model assigned for concrete.
Figure 8-1: Parabolic compressive behavior of concrete material [47].

Linear ultimate strain based model was used to define tension behavior of concrete as shown in Figure 8-2. Tension stiffening was defined using the tensile strength ($f_t$) obtained from splitting strength of concrete cylinders and the end plateau strain ($\varepsilon_u$) to assign brittle concrete tension behavior.

8.2.1.2 Reinforcing bars

Ideal plasticity model was defined for reinforcing bars implemented within concrete elements as shown in Figure 8-3. Yield stress was obtained based on the experimental results for reinforcing bars as well as elastic modulus of elasticity. This model represents
the reinforcement performance, where yield plateau occurs and the performance is governed by the concrete performance.

![Ideal plasticity model for reinforcing bars](image)

Figure 8-3: Ideal plasticity model for reinforcing bars [47].

8.2.1.3 Prestressing strands

The same ideal plasticity model defined for reinforcing bars was defined for prestressing strands. The yield strength defined for the strands was the value defined on the data sheet for the strands. The main difference between the material model for reinforcing bars and the prestressing strands was the bond with surrounding concrete. The bond between prestressing strands and surrounding concrete was eliminated by using the NOBOND definition. It indicates that the embedded reinforcement was not bonded to its mother elements. In this case the stiffness of the reinforcement did not contribute to the stiffness of the mother elements, nor do the bar reinforcement strains and stresses change with deformation of the mother element [47].

8.2.1.4 Anchor devices

High stress concentration occurs around anchor devices, this high compressive stresses cause unexpected premature failure in the local zone. As the main concern of the developed finite element models is the general anchorage zone, elastic materials were
defined for anchor devices to eliminate stress concentration and premature failure around the anchor devices. The same modulus of elasticity for concrete was used for this elastic material to prevent any stress concentration between the devices and the concrete.

8.2.2 Elements and meshing

The meshing of elements was performed using the automatic meshing tool provided by MIDAS FX+ [48]. Solid elements were used to define concrete and anchorage devices, however, beam elements were used to define reinforcing bars and post-tensioning strands. The prestressing ducts were defined as void in the concrete with the exact parabolic shape as in the experimental specimens. Solid elements used were “TE12L – tetrahedron”, 3 sides, 4 nodes, and beam elements used were “bar reinforcement”.

Mesh size changed from 1.25” to 8” based on the size of adjacent elements to maintain integrity and continuity of the mesh as shown in Figure 8-4. The mesh size for the anchor devices and the adjacent concrete parts were 1.25”. The mesh size at the top for loading plates and at the bottom for support locations were 2.5”. The remaining mesh size increased for the remaining elements up to 4” in the scaled specimens and 8” in the full prototype specimens. The maximum mess size for the reinforcement and prestressing strands was 4”, however, smaller mesh sizes are developed due to auto meshing performed for beam elements that breaks beam elements at the faces of all solid elements.
Figure 8-4: Mesh size variation at different locations of the developed models.

8.2.3 Loading implementation

Three loads were defined in these models including self-weight of the specimens, vertical loads applied during experimental testing, and prestressing of post-tensioning strands.

8.2.3.1 Specimen self-weight

Self-weight of the specimens was defined using Body Force available in DIANA software. This body force requires the definition of the gravitational acceleration and its direction. Based on the gravitational acceleration and the density defined for different element material of the model, vertical self-weight is applied.

8.2.3.2 Vertical external load

Vertical loads applied on the experimental specimens to compensate the self-weight of the full prototype specimens were defined as surface pressure as shown in Figure 8-5. This pressure was applied on an area of 7”×7” for scaled specimens and 12”×12” for
prototype specimens. The pressure applied on each area was equivalent to the actual force applied during experimental testing.

8.2.3.3 Post-tensioning forces

Stressing force applied to prestressing strands was applied as post-tensioning for reinforcement load available in DIANA as shown in Figure 8-6. This methodology applies forces at the end of prestressing strands, where the material of the strands was defined as unbonded to the concrete parts surrounding it. The forces were directly applied on the specimen ends considering the profile of the strands connected to the concrete at several points along the tendon profile. Tendon losses were neglected as the actual prestressing force was applied.
8.2.4 Supporting system

The defined support system for each of the specimens was a hinged-roller, where the first end was constrained in the vertical direction only and the other end was constrained in the vertical and longitudinal direction as shown in Figure 8-7. Line supports were added at each end to permit rotation of the specimen ends. To maintain global stability of the model out-of-plane support in the transverse X-direction was implemented at the middle joint on each side.

(a) Hinged support  (b) Roller support

Figure 8-7: Supports applied at both end of the models.

8.2.5 Iterative solution procedures

Several loading steps were applied for each stage in the loading sequence. Minimum five loading steps were applied for the self-weight and ten were applied for the vertical load applied on top of the specimens. Then, the prestressing load was applied in increments of 5 kips for the scaled specimens and 20 kips for the prototype ones.

Regular Newton-Raphson method was used for the iterative solution procedures. In the Regular Newton-Raphson iteration the stiffness relation is evaluated every iteration as shown in Figure 8-8. This means that the prediction of tangential stiffness is based on the last known or predicted situation, even if this is not an equilibrium state.
The Regular Newton-Raphson method yields a quadratic convergence characteristic, which means that the method converges to the final solution within only a few iterations. A disadvantage of the method is that the stiffness matrix has to be set up at every iteration and, if a direct solver is used to solve the linear set of equations, the time consuming decomposition of the matrix has to be performed every iteration as well. Moreover, the quadratic convergence is only guaranteed if a correct stiffness matrix is used and if the prediction is already in the neighborhood of the final solution. If the initial prediction is far from the final solution, the method easily fails because of divergence [28].

8.3 Model Verification

FEM verification was performed using several models based on the experimental work done by Sanders 1990 [11]. These experiments include six rectangular specimens as well as one T-section specimens. All experimental specimens were verified, however, only two specimen ends S04 and D5T are presented here in this verification section. The
verification includes strain plots for implemented strain gauges in addition to specimen axial deformations.

### 8.3.1 FEM verification using Sanders 1990 experimental work

Two types of specimens were verified from the experimental work done by Sanders 1990. The first group included six rectangular specimens (labeled B), five of them with different bursting reinforcement configuration and one of them without any bursting reinforcement. The second group contain only T-section specimen labeled F1.

The reinforcement details and configurations for specimen B1 and B2 are shown in Figure 8-9. #4 smooth spiral – 7” diameter with 1¼” pitch was implemented in all the specimens to assure that the failure does not occur in the local anchorage zone. The difference between specimens were in the location and amount of bursting reinforcement.

(a) Details of specimen B1

(b) Details of specimen B2

(c) Cross section of group B specimens

Figure 8-9: Reinforcement detail and configurations of specimens B1 and B2 [11].
The configurations of the FEM developed for specimen B1 is shown in Figure 8-10. It presents the concrete mesh set as well as the implemented reinforcement details in the model. These specimens were tested vertically under pressure without implementing any prestressing strands. The main load was applied as surface pressure on top of the specimen. The supporting system was a group of rollers to allow lateral movement of the specimen base. Only one node at the bottom center of the specimen was restrained in the three orthogonal directions to maintain global stability of the model. Material properties of experimental specimens modeled using DIANA are summarized in Table 8-1. It includes specimen label, concrete splitting tensile strength, concrete compressive strength, bar size used in each specimens and reinforcement yield strength, elastic modulus and yield strain.

(a) Concrete mesh set  (b) Reinforcement details
Figure 8-10: Developed DIANA model for specimen B1.
Table 8-1: Material information for Sanders experimental specimens [11].

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete</th>
<th>Reinforcing Bars</th>
<th>Yield Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{sp}$</td>
<td>$f'_c$</td>
<td>Bar</td>
</tr>
<tr>
<td>B1</td>
<td>464</td>
<td>5380</td>
<td>#4</td>
</tr>
<tr>
<td>B2</td>
<td>464</td>
<td>5380</td>
<td>#3</td>
</tr>
<tr>
<td>B3</td>
<td>464</td>
<td>5380</td>
<td>#2m*</td>
</tr>
<tr>
<td>B4</td>
<td>464</td>
<td>5380</td>
<td>#2m*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B5</td>
<td>420</td>
<td>5320</td>
<td>No General Zone Reinforcement</td>
</tr>
<tr>
<td>B6</td>
<td>420</td>
<td>5320</td>
<td>#2m*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>316</td>
<td>4540</td>
<td>#2s**</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* #2m As = 0.049 in$^2$
** #2s As = 0.0475 in$^2$

Model verification for specimen B1 was satisfied through bursting strain results of stirrups as shown in Figure 8-11. The FEM results showed good correlation with the experimental results. The same good correlation was obtained through spiral strains of specimen B1 as shown Figure 8-12. The comparison between experimental bursting strains developed in specimen B2 and the FEM results are shown in Figure 8-13. The correlation in specimen B1 was better than the results of specimen B2. Although, obtaining the stiff zone followed by a plateau then ascending with lower stiffness in specimen B2 was satisfied. Satisfying such sensitive performance that is based on concrete tensile strength is a very complex task.
The same modeling procedures were used to develop FEMs for specimens B3, B4, B5 and B6. The reinforcement details and configurations of these specimens are shown in...
Figure 8-14. It includes bursting reinforcement details, spacing and locations. The verification of specimens B3, B4 and B6 was satisfied through comparison of experimental bursting strains and FE results as shown in Figures 8-15, 8-16 and 8-17. The comparisons show acceptable performance for specimen B3 and good correlations for specimen B4 and B6.

As specimen B5 did not contain any bursting stirrups, verification was conducted by comparing experimental load-displacement curve to the FE results as shown in Figure 8-18. This comparison provides a good correlation between FE and experimental results.
Figure 8-15: Verification of bursting strain data for specimen B3 [11].

Figure 8-16: Verification of bursting strain data for specimen B4 [11].

Figure 8-17: Verification of bursting strain data for specimen B6 [11].
Figure 8-18: Verification of load-displacement relationship of specimen B5 [11].

Ultimate failure loads for rectangular B specimens were compared to FEM results as shown in Figure 8-19. The developed models provide good correlations compared to experimental results. The stress-strain relationship in addition to the ultimate loads extracted from FEM illustrate the adequacy and capability of it.

Figure 8-19: Ultimate failure load comparison between experimental and FE results for rectangular B specimens.

Specimen F1 was a T-section with reinforcement details and configurations as shown in Figure 8-20. Total specimen length was 90” and height was 34”. Flange thickness was 4.25” and width was 34”, while web thickness was 6”. Two anchorages were implemented
in this specimen at 22” and 30” from specimen base. The bottom anchorage was labeled A, and the top one was labeled B. The flange reinforcement and location of gauges for specimen F1 are illustrated in Figure 8-21. It includes reinforcement details, location and gauges implemented in the flange. The specimen was loaded using two prestressing rams that were implemented at both ends of the specimen. Schematic test setup for F1 specimen is shown in Figure 8-22. It was not possible to have both prestressing rams at the same end of the specimen as anchorages A and B were very close to each other; there was not enough spacing between them.

Figure 8-20: Reinforcement details and configurations of specimen F1 [11].
The configurations of the FEM developed for F1 specimen is shown in Figure 8-23. It presents the concrete mesh set as well as the implemented reinforcement details in the model. This specimen was tested horizontally using prestressing strands. For simplicity of the model, the main load was applied as surface pressure on the web of the specimen. The supporting system was a group of rollers to allow vertical movement of the specimen end.
Only one node at the edge of the specimen was restrained in the three orthogonal directions to maintain global stability of the model.

(a) Concrete mesh set  (b) Reinforcement details

Figure 8-23: Developed DIANA model for F1 specimen.

The verification of specimen F1 was satisfied through comparison of experimental bursting strains and FE results for the web ahead of anchorages A and B as shown in Figures 8-24 and 8-25, respectively. The comparisons show good performance for the web of specimen F1. In order to confirm the model adequacy, comparisons were conducted between experimental bursting strains and FE results for the flange near and distant from anchorage B as shown in Figures 8-26 and 8-27, respectively. These relationships illustrate the adequacy and applicability of the model to assess the bursting performance of the general anchorage zone.
Figure 8-24: Verification of web reinforcement bursting strains ahead of anchorage A, specimen F1 [11].

Figure 8-25: Verification of web reinforcement bursting strains ahead of anchorage B, specimen F1 [11].

Figure 8-26: Verification of flange reinforcement bursting strains near anchorage B, specimen F1 [11].
Verification was conducted by comparing experimental load-displacement curve to the FE results at both anchorages as shown in Figure 8-28. This comparison provides an acceptable correlation between FE and experimental results, however the failure load is lower in the FE results compared to experimental values.

Figure 8-27: Verification of flange reinforcement bursting strains distant from anchorage B, specimen F1 [11].

Figure 8-28: Verification of load-displacement relationship of specimen F1.
8.3.2 FEM verification for experimental specimen end S04

The experimental single girder specimen S45 was model using DIANA including all specimen details and configurations. Solid elements mesh set of the developed model is shown in Figure 8-29. It includes mesh set for the concrete beam and different mesh set for each anchorage devices, which was defined individually. These anchorage devices were used to transfer the post-tensioning forces to the concrete part without causing stress concentration or failure at the local anchorage zone. The reinforcement details of the developed DIANA model for specimen S45 is shown in Figure 8-30. It illustrates the complex details of reinforcement implemented in the FEM. Each individual bar was modeled as beam element even the spirals of the local anchorage zone was included.

Figure 8-29: Solid elements mesh set of DIANA model for single girder specimen S45.
The verification was performed using the total axial strains at the top and bottom of the specimen as shown in Figure 8-31. It shows that the developed DIANA model can accurately assess the performance of experimental specimens. Sample of strain results were also verified by comparing FE results to the experimental results. The diaphragm inner face transverse as well as vertical strains verification are shown in Figures 8-32 and 8-33, respectively. The developed strains showed good correlation with the experimental results.

Figure 8-31: FEM verification of axial strain for specimen S45.
Figure 8-32: FEM verification of the diaphragm transverse strain values for specimen end S04.

Figure 8-33: FEM verification of the diaphragm vertical strain values for specimen end S04.

The web vertical strains verification for specimen end S04 are shown in Figure 8-34. The model results were compared to experimental results for the transverse strains developed in the deck and soffit as shown in Figures 8-35 and 8-36, respectively. All the verifications of strains show good correlation with the experimental results. This clarifies the adequacy and capability of the FEM to assess the performance of anchorage zone for box girder bridges.
Figure 8-34: FEM verification of the web vertical strain values for specimen end S04.

Figure 8-35: FEM verification of the deck transverse strain values for specimen end S04.

Figure 8-36: FEM verification of the soffit transverse strain values for specimen end S04.
8.3.3 FEM verification for experimental specimen end D5T

The experimental double girder specimen D55 was model using DIANA including all specimen details and configurations. Solid elements mesh set of the developed model is shown in Figure 8-37. It includes mesh set for the concrete beam and different mesh set for each anchorage devices, which was defined individually. The reinforcement details of the developed DIANA model for specimen D55 is shown in Figure 8-38. It illustrates the complex details of reinforcement implemented in the FEM. Each individual bar was modeled as beam element, including the spirals of the local anchorage zone and the grillage reinforcement.

Figure 8-37: Solid elements mesh set of DIANA model for double girder specimen D55.
Figure 8-38: Reinforcement details of DIANA model for double girder specimen D55.

The verification was performed using the total axial strains at the top and bottom on both sides of the specimen at girders A and B as shown in Figures 8-39 and 8-40. These figures do not show a good correlation due to the looseness and vibrations of implemented string pots used to measure the axial specimens strain.

Figure 8-39: FEM verification of axial strain for specimen D55 – Girder A.
Sample of experimental strain results were compared with the FE results. The diaphragm inner face transverse as well as vertical strains verification are shown in Figures 8-41 and 8-42, respectively. Figure 8-43 includes the verification of the web vertical strains for the same specimen. The model results were compared to experimental results for the transverse strains developed in the deck and soffit as shown in Figures 8-44 and 8-45, respectively. All these verifications of strains show good correlation with the experimental results. This clarifies the adequacy and capability of the FEM to assess the performance of anchorage zone for box girder bridges.

Figure 8-40: FEM verification of axial strain for specimen D55 – Girder B.

Figure 8-41: FEM verification of the diaphragm transverse strain values for specimen end D5T.
Figure 8-42: FEM verification of the diaphragm vertical strain values for specimen end D5T.

Figure 8-43: FEM verification of the web vertical strain values for specimen end D5T.

Figure 8-44: FEM verification of the deck transverse strain values for specimen end D5T.
Figure 8-45: FEM verification of the soffit transverse strain values for specimen end D5T.

8.4 Prototype Model Development

Developing a prototype model was essential to extend the experimental results through parametric study. The prototype model analysis focused on the ultimate loading. The concrete properties were modified in the model to force the reinforcement to resist the whole bursting strains; this helped to determine the total bursting force.

8.4.1 Prototype verification

A prototype FEM was developed for the ½ scaled single girder experimental specimen S45. This model was built based on the same modeling criteria for the ½ scaled specimen model. The results of this model was compared to the ½ scaled FEM and the experimental results. The axial longitudinal strain at the specimen centroid is plotted for the three cases as shown in Figure 8-45. The average results of the top and bottom was considered as the axial longitudinal strain at centroid for the experimental specimen S45. Due to scaling and similitude laws the applied prestressing force in the prototype specimen was four time the force of the ½ scaled model. Therefore, the comparison was conducted based on the axial prestressing stress of the specimens plotted on the vertical axis, which
is the same in both prototype and scaled models. The prototype model shows good agreement with the \( \frac{1}{2} \) scaled model results and the experimental results, which indicates the adequacy of the developed prototype model.

![Graph](image)

**Figure 8-46**: Comparison between experimental results and DIANA models for axial longitudinal strain at centroid for specimen S45.

In order to assure that the performance of the prototype model matches the \( \frac{1}{2} \) scaled model, sample strain results at different elements are shown in Figure 8-47. Good correlation was developed between strain results, which presents the accuracy of the developed prototype model. This developed prototype model was used to perform an extensive parametric study that includes major parameters affecting general anchorage of box girder bridges.
8.4.2 Achieving ultimate state for bursting forces.

The bursting forces developed in the FEMs were estimated based on the developed strain in the reinforcement. The strains are transferred into forces depending on the reinforcement elastic modulus. This procedure works before reinforcement yielding, however, ultimate state is reached in most cases after yielding of reinforcement. The problem in obtaining bursting forces before reinforcement yielding is the tensile strength of concrete that resist the bursting strains with the reinforcement. If the concrete contribution is included, the force in the reinforcement will underestimate the bursting force. Therefore, in order to reach a representative bursting forces, concrete tensile strength was eliminated from the developed prototype models.

In order to evaluate the effect of concrete tensile strength elimination from the model, comparisons were conducted between bursting forces developed in the transverse as well as the vertical directions as shown in Figures 8-48 and 8-49, respectively. The bursting forces were obtained based on the strains developed in reinforcement just before yielding.
These forces were obtained for different models. One the primary variables was the effective diaphragm width to girder height \((b_{de}/h)\). The results of the modified FEM that did not include the concrete tensile strength are always higher than those from the experiment. This clarifies that the difference between the modified model and the original are the amount of bursting forces that were resisted by concrete tensile strength as well as concrete tension stiffening. The modified model is more representative for the bursting forces at the ultimate stage, which will be used to perform the parametric study.

![Graph](image1)

**Figure 8-48**: Effect of concrete tensile strength elimination on transverse bursting forces for different \(b_{de}/h\) values.

![Graph](image2)

**Figure 8-49**: Effect of concrete tensile strength elimination on vertical bursting forces for different \(b_{de}/h\) values.
8.5 Parametric Study

Parametric study was developed to extend the results either from the field investigation or the experimental work. Parameters that were not investigated in the experiments or in the field investigations were included in this study. The parametric study was performed using the full prototype specimen presented in Chapter (3). The modified prototype FEM eliminating concrete tensile strength was used to assess the actual bursting forces at ultimate stage. The base prototype specimen had straight tendon in order to facilitate the geometrical modeling. The tendon inclination angle was investigated as an individual parameter.

The parametric study included the effect of diaphragm width to box girder height ratio as well as the girder spacing to box girder height ratio. Changing thickness of elements was investigated through varying thicknesses of deck, soffit and web. Effect of edge eccentricity and number of anchors on bursting forces was determined. The tendon inclination angle, the diaphragm skew angle, diaphragm openings, grillage reinforcement and spalling stresses were studied to assess their effect on the developed bursting forces.

8.5.1 Effective diaphragm width to box girder height ratio

Effective diaphragm width to box girder height ratio was investigated through the single girder experimental specimens. This study highlighted the role of the diaphragm width on different bursting forces developed at the general anchorage zone. This parameter was investigated using eight models with different diaphragm width to box girder height ratios ranging from 0.3 to 1.0. The bursting forces developed from these models are illustrated in Figure 8-50. The ratio of the developed bursting force to jacking force is
plotted against the effective diaphragm width to box girder height ratio in order to study its
effect on various bursting forces. The figure includes transverse bursting forces in the deck,
soffit and diaphragm inner face as well as the vertical bursting forces in the web and
diaphragm inner face.

Figure 8-50: Effect of the diaphragm width to box girder height ratio on the anchorage
zone bursting forces.

Based on these results, all developed bursting forces were inversely proportional with
the effective diaphragm width to box girder height ratio. The diaphragm width significantly
affects the diaphragm transverse and vertical bursting forces as well as vertical bursting
forces of the web. Only a slight effect was observed in the deck and soffit bursting forces.

8.5.2 Girder spacing to box girder height ratio

Girder spacing to box girder height ratio rapidly changes during bridge design based
on several parameters. Changing girder spacing affects number of box girder cells for a
certain bridge width. It also changes the aspect ratio of each individual box girder cell.
Girder spacing to box girder height ratio was investigated using twelve models with
different girder spacing to box girder height ratios ranging from 1.0 to 2.1. The bursting
forces developed from these models are illustrated in Figure 8-51. The ratio of the
developed bursting force to jacking force is plotted against the girder spacing to box girder height ratio in order to study its effect on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

![Graph showing the effect of girder spacing to box girder height ratio on anchorage zone bursting forces.](image)

**Figure 8-51: Effect of the girder spacing to box girder height ratio on the anchorage zone bursting forces.**

Based on these results, all developed bursting forces are directly proportional with girder spacing to box girder height ratio except vertical bursting forces in the web. No significant effect was observed on the web vertical bursting forces.

### 8.5.3 Deck and soffit thicknesses

Changing the geometry of bridge section shall significantly affect the developed bursting forces. Increasing the deck and soffit thickness however maintaining the same web width shall change the distribution of bursting forces between elements. Changing the deck and soffit thicknesses was investigated using five models, where the deck thickness was between 7.5” to 9.5” and the soffit thickness was between 6.5” to 8.5”. The bursting forces developed from these models are illustrated in Figure 8-52. The ratio of the developed bursting force to jacking force is plotted against the deck and soffit thicknesses.
to study the effect of changing geometry on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

![Figure 8-52: Effect of deck and soffit thicknesses on the anchorage zone bursting forces.](image)

Based on these results, all developed bursting forces are directly proportional with the increase in the deck and soffit thicknesses, except the vertical bursting force of the web. Vertical bursting force in the web is inversely proportional with the increase of the deck and soffit thicknesses. This indicates that the thicker the element was, the more bursting force it attracted. Bursting forces in the deck and soffit increase, however, bursting force in the web decrease. This also highlights the role of the diaphragm in transferring the prestressing forces to the deck and the soffit in both vertical and horizontal directions.

### 8.5.4 Web width

Changing the web width was investigated using five models, where the web width was between 12” to 18”. The bursting forces developed from these models are illustrated in Figure 8-53. The ratio of the developed bursting force to jacking force is plotted against the web width to study the effect of changing geometry on various bursting forces. The
figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

![Graph showing bursting forces vs. web width]

Figure 8-53: Effect of web width on the anchorage zone bursting forces.

Based on these results, all developed bursting forces are inversely proportional with the increase in web width, except the vertical bursting force of the web. Vertical bursting force in the web was directly proportional with the increase of web width. This conclusion matches the results obtained from the effect of changing the deck and sofit thicknesses.

8.5.5 Number of prestressing anchors

Variation in number of anchors used in post-tensioning of box girder bridges is very limited. Based on the developed bridge database described in Chapter (3) the implemented number of anchors per girder ranges between 1 to 3. This parameter was investigated using three models. The bursting forces developed from these models are illustrated in Figure 8-54. The ratio of the developed bursting force to jacking force is plotted against number of anchors in order to study its effect on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.
Figure 8-54: Number of anchors effect on the anchorage zone bursting forces.

Based on these results, vertical bursting forces in the web and the diaphragm was inversely proportional with number of anchors. On the other side, no significant effect was observed on the transverse bursting forces of the diaphragm, deck or soffit.

8.5.6 Edge eccentricity ratio

Edge eccentricity rarely exists at the anchorage zone of box girder bridges; in most cases at abutments (edge supports), no eccentricity is required a resist any bending moments. At in-span hinges, edge eccentricity exists. The ratio between edge eccentricity to box girder height was investigated using seven models with different eccentricity ratios from 0.0 to 0.3. The bursting forces developed from these models are illustrated in Figure 8-55. The ratio of the developed bursting force to jacking force is plotted against the eccentricity ratio in order to study its effect on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.
Based on these results, bursting forces are not affected by edge eccentricity except the transverse bursting forces of the deck and soffit. The transverse bursting force of the deck was inversely proportional with the eccentricity ratio, however, the transverse bursting force of the soffit was directly proportional with the eccentricity ratio. Taking into consideration that the applied edge eccentricity in the developed DIANA models was downward below section centroid, this clarifies the increase in the soffit bursting force and the decrease in the deck bursting force.

**8.5.7 Tendon inclination angle**

The base prototype model had straight tendon with zero inclination angle, therefore, studying the effect of changing inclination angle on the bursting forces was essential. This parameter was investigated using six models with inclination angle ranging from -5° to 20° to cover the range of tendon inclination angle stated in AASHTO LRFD Bridge Design Specifications [3]. The bursting forces developed from these models are illustrated in Figure 8-56. The ratio of the developed bursting force to jacking force is plotted against inclination angle in order to study its effect on various bursting forces.
transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

Based on these results, the vertical bursting forces of the diaphragm and web as well the transverse bursting force in the soffit were directly proportional with the tendon inclination angle. However, the deck bursting force was inversely proportional with tendon inclination angle. No significant effect was observed on the diaphragm transverse bursting forces. A plateau of constant bursting force values occurred in the diaphragm and the web at inclination angle ranges between -5° to 5°. This indicates that the tendon inclination angle did not affect bursting forces in the diaphragm and web in this range.

8.5.8 Diaphragm skew angle

Effect of diaphragm skew angle was investigated through the field monitoring of bridges, where the obtuse side of the diaphragm was found to be more critical than the acute side as observed previously in section 4.7.2. The skew angle of the diaphragm was investigated using five models with skew angles ranging from 0° to 60°. The developed DIANA model with diaphragm skew angle of 30° is shown in Figure 8-57. The bursting
forces developed from these models are illustrated in Figure 8-58. The ratio of the developed bursting force to jacking force is plotted against the diaphragm skew angle in order to study its effect on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

Figure 8-57: DIANA model for diaphragm skew angle of 30°.

The transverse bursting forces in the diaphragm, deck and soffit were directly proportional with the diaphragm skew angle. However, the vertical bursting forces of the diaphragm and the web were not affect by changing the skew angle. A plateau of constant busting force values occurred in the deck and soffit up to 30° skew angle. Although, the

Figure 8-58: Effect of diaphragm skew angle on the anchorage zone bursting forces.
vertical bursting forces of the diaphragm were not affect by changing the skew angle, the
distribution of vertical bursting forces on both acute and obtuse sides of the diaphragm
were not the same as shown in Figure 8-59. The vertical bursting force in the obtuse side
increased with the diaphragm skew angle, however, the vertical bursting force in the acute
side decreased. Consequently, the ratio between vertical bursting forces in the acute side
to the force in the obtuse side decreases with the diaphragm skew angle even with the
average vertical bursting force not affected by the diaphragm skew angle.

Figure 8-59: Effect of skew angle on distribution of vertical bursting forces on both acute
and obtuse sides of the diaphragm.

8.5.9 Diaphragm opening dimensions

Opening dimensions were investigated through the double girder experimental
specimens. Diaphragm openings in this parametric study were implemented in the middle
between girders. The height of diaphragm opening was ratio of the box girder height (h)
and its width was ratio of girder spacing (S). This parameter was investigated using seven
models with opening dimensions ranging from $0.1S \times 0.1h$ to $0.6S \times 0.6h$. The bursting
forces developed from these models are illustrated in Figure 8-60. The ratio of the
developed bursting force to jacking force is plotted against opening dimensions in order to
study its effect on various bursting forces. The figure includes transverse bursting forces in the deck, soffit and diaphragm inner face as well as the vertical bursting forces in the web and diaphragm inner face.

Figure 8-60: Effect of diaphragm opening dimensions on the anchorage zone bursting forces.

Based on these results, all developed bursting forces are not significantly affected by openings in the end diaphragm. Slight increase could be observed in the bursting forces of the deck and soffit. Slight decrease occurred in the diaphragm transverse bursting forces. However, no significant effect for diaphragm opening on the bursting forces, failure of experimental specimen was affected by diaphragm openings. Stress concentrations were observed in the developed DIANA models adjacent to diaphragm openings as shown in Figure 8-61. The amount of the reinforcement that would be cut or removed because of the opening vertical sides should be placed above and below the opening and any reinforcement that would be cut or removed because of the opening top and bottom edges should be placed on both sides of the opening to minimize any stress concentration at opening corners.
In order to compare the effect of diaphragm opening on the ultimate failure capacity of anchorage zones, comparison was conducted between axial compressive stresses developed in specimen ends DST and D5T as shown in Figure 8-62. The axial compressive stresses at specimen end D5T was higher than those developed at specimen end DST. This coincides with the web compression failure mode occurred in the experimental specimen end D5T. Therefore, in case of diaphragm opening axial capacity of the web girder must be checked to sustain the whole jacking force.
8.5.10 Grillage reinforcement and spalling stresses

In order to compare the effect of grillage reinforcement, developed vertical and traverse stresses on the outer diaphragm face of specimen end DST are plotted as shown in Figure 8-63. No significant difference was observed on both web girders A and B, however, grillage was implemented at Girder A. Also, the zone adjacent to the anchors were subjected to compressive stresses. The grillage reinforcement had no significant role in resisting compressive forces developed at this zone. However, in case of wide prestressing block-outs, grillage reinforcement will be needed to resist concrete shrinkage.

![Image of developed stresses on the outer diaphragm face of specimen DST.](image)

(a) Vertical stresses  
(b) Transverse stresses

Figure 8-63: Developed stresses on the outer diaphragm face of specimen DST.

Tensile spalling stresses started distant from the anchors; therefore grillage reinforcement did not play a role in resisting spalling stresses. Spalling tensile stresses developed was significantly low (less than 0.6 ksi) in either the vertical or transverse direction, however this stage was considered the ultimate stage of the specimen. Based on these results, the spalling tensile stresses along the loaded face was totally resisted by
tensile strength capacity of concrete in the solid diaphragm case. The diaphragm wide section at specimen edge provide sufficient resistance for spalling tensile forces. Minimum reinforcement will be needed to resist concrete shrinkage.

8.6 Concluded Remarks

FEM verifications clarify the adequacy and capability of the developed FEM to assess the performance of anchorage zone for box girder bridges. The proposed prototype model showed good agreement with the ½ scaled model results and the experimental results, which indicates the competence of the developed prototype model. Based on the parametric study performed, the following outcomes were extracted:

- Anchorage zone bursting forces were inversely proportional with the effective diaphragm width to box girder height ratio.
- Anchorage zone bursting forces were directly proportional with girder spacing to box girder height ratio except vertical bursting forces in the web, where no significant effect was observed.
- Anchorage zone bursting forces were directly proportional with the increase in the deck and soffit thicknesses, except the vertical bursting force of the web, where inverse proportionality was observed.
- The vertical bursting force of the web was directly proportional with the increase of web width. However, all other anchorage zone bursting forces were inversely proportional with the increase in web width.
Vertical bursting forces in the web and the diaphragm were inversely proportional with number of anchors. No significant effect was observed on the transverse bursting forces of the diaphragm, deck or soffit.

The transverse bursting force of the deck was inversely proportional with the eccentricity ratio, however, the transverse bursting force of the soffit was directly proportional with the eccentricity ratio. All remaining bursting forces were not affected by anchorage eccentricity.

Anchorage zone bursting forces were directly proportional with the tendon inclination angle except the transverse deck bursting force, where inverse proportionality was observed.

The transverse bursting forces in the diaphragm, deck and soffit were directly proportional with the diaphragm skew angle. However, the total vertical bursting forces of the diaphragm and the web were not affect by changing the skew angle.

The distribution of vertical bursting forces on both acute and obtuse sides of the diaphragm were affected by the diaphragm skew angle. The vertical bursting force in the obtuse side increased with the diaphragm skew angle, however, the vertical bursting force in the acute side decreased.

Anchorage zone bursting forces were not significantly affected by openings in the end diaphragm. Slight increase was observed in the bursting forces of the deck and soffit.

Diaphragm openings affects the ultimate failure capacity of anchorage zones. Therefore, in case of diaphragm opening axial capacity of the web girder must be checked.
• The grillage reinforcement had no significant role in resisting compressive forces developed adjacent to anchors or any spalling stresses.

• Tensile spalling stresses started distant from the anchors with significantly low values. Therefore, the spalling tensile stresses along the loaded face was completely resisted by tensile strength capacity of concrete in the case of solid diaphragm.
Chapter (9) – Results Implementation

9.1 Introduction

Results obtained from field monitoring of bridges, experimental work and finite element modeling provided very good insight into the performance of general anchorage zones. One of the main objectives of this research was to develop a set of design equations for the anchorage zone that were reliable, adequate and safe. All of these results were combined using strut-and-tie methodology to develop a set of design equations for bursting forces of anchorage zone including: transverse forces in the deck and soffit, transverse force in the diaphragm inner face and vertical forces in the web and diaphragm inner face. The proposed equations include the effects of diaphragm skew angle, opening in the diaphragm and tendon inclination, geometry of box girder end zone, edge eccentricity and number of anchors. A simplified design table is developed based on the proposed equations. It provides percentage of bursting forces to ultimate jacking force based on effective diaphragm width to box girder height ratio as well as girder spacing to box girder height ratio.

9.2 Developed Set of Equations

The proposed set of design equations were developed based on strut-and-tie concepts as well as observations obtained from finite element analysis, experimental work and field monitoring. While the proposed design equations are based on the strut-and-tie model concept, they do not require the user to develop a strut-and-tie model.
9.2.1 Design assumptions

Design assumptions were extracted from the results of the field monitoring, experimental work and literature. The main assumptions obtained from the field monitoring were the effective zone of the web, the diaphragm net width and effect of stressing both bridge ends. The assumptions concluded based on the experimental work were the centroid of web vertical bursting force, the distribution of bursting forces within the deck and the soffit, and the failure mode of the general anchorage zone specimens.

The locations of the centroid of the bursting forces in the deck and soffit were determined based on the strut-and-tie models develop by Wollmann [49]. All the assumptions needed to develop the set of equations are summarized in Table 9-1. It includes the design assumptions and its illustration. The reference section for each assumption is provided.

Table 9-1 Assumptions for the proposed set of equations

<table>
<thead>
<tr>
<th>Assumption</th>
<th>Illustration</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective zone of the web is ½ h in longitudinal direction.</td>
<td>The strain values are significantly reduced at ½ h for both interior and exterior girders of field monitoring as well as the webs of experimental work in longitudinal direction.</td>
<td>Section 4.7.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Section 7.5.3</td>
</tr>
<tr>
<td>The centroid of the web vertical bursting forces is located at a distance of ½ h from diaphragm inner face.</td>
<td>The resultant location was calculated based on the longitudinal strain distribution of the web for experimental specimens.</td>
<td>Section 7.5.3</td>
</tr>
<tr>
<td>The centroid of the deck and soffit transverse bursting forces is located</td>
<td>The longitudinal distribution of the deck and soffit is nearly constant, which shows that the transverse strains extended longitudinally along</td>
<td>Section 7.5.4 Ref. [49]</td>
</tr>
<tr>
<td>Assumption</td>
<td>Illustration</td>
<td>Reference</td>
</tr>
<tr>
<td>------------</td>
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</tr>
<tr>
<td>at a distance of $\frac{1}{2} S$ from diaphragm inner face.</td>
<td>the deck and soffit. Centroid location was obtained based on strut-and-tie models developed by Wollmann 2000 [49].</td>
<td></td>
</tr>
<tr>
<td>In case of stressing both bridge ends, bursting strain values increase at the initial stressing end due to stressing the second end. ($\beta_{\text{end}}$)</td>
<td>Strain propagation due to stressing both ends is inversely proportional with friction losses as presented in Eq. 5-6 for $\beta_{\text{end}}$.</td>
<td>Section 5.6 Section 5.7.3.2</td>
</tr>
<tr>
<td>The net effective thickness of the diaphragm should be considered in design of the anchorage zone of box girder bridges. ($b_{de}$)</td>
<td>Skew angle of the end diaphragm affects the dimensions of block-outs required for post-tensioning, which leads to reduction in the diaphragm width.</td>
<td>Section 4.8.2.2 Section 5.2</td>
</tr>
<tr>
<td>Failure mode of general anchorage is web compression failure.</td>
<td>Longitudinal concrete gauges in the web were subjected to high compression values. Buckling of longitudinal bars for specimen ends D6T, D5T and D5H was observed. The same performance was observed in case of diaphragm openings using FEM.</td>
<td>Section 7.4.5 Section 7.6 Section 8.5.9</td>
</tr>
</tbody>
</table>

Where:

- $h$ = Box girder height
- $S$ = Girder spacing
- $\beta_{\text{end}}$ = Amplification factor due to stressing both ends
- $b_{de}$ = Distance measured from the diaphragm outer face at the anchorage centroid to the diaphragm inner face along the web longitudinal axis.

9.2.2 Strut-and-tie methodology

However, all of these codes do not provide a clear illustrative three dimensional strut-and-tie model for the anchorage zone of box girder bridges. The available example presented by Wollmann [49] included only two dimensional strut-and-tie models. These models neglected the presence of end diaphragm which developed overestimated results.

The dimensions used in developing the set of equations for a typical anchorage zone are presented in Figure 9-1. It illustrates an edge section as well as interior cross section. It includes the block-out limits and the prestressing anchors. The boundaries of the prestressing anchors are defined (a) as the vertical ordinate and (b) as the transverse ordinate. Box girder height (h), girder spacing (S), deck thickness (td), soffit thickness (ts) and web width (bw) are illustrated.

Distribution of force between elements was based on stiffness. In the anchorage zone problem, axial stiffness of members was the governing parameter. Based on the design assumption, the bursting forces of the web were located at h/4 from the diaphragm inner face and the transverse bursting forces of the deck and soffit are located at (S/2) from the diaphragm inner face, see Table 9-1. Therefore, stress distribution required to obtain the force applied at each element was calculated at an average distance of \([(h/4 + S/2)/2]\) from
the diaphragm inner face as shown in Figure 9-2. The developed stresses at the extreme fibers top and bottom were defined as $f_1$ and $f_4$. The stress developed just at interface between the web and the deck was defined as $f_2$. The stress developed at the interface between the web and the soffit was defined as $f_3$. The remaining parameters defining the geometry of anchorage zone are presented in Figure 9-2. Edge eccentricity ($e$), diaphragm width ($b_d$), diaphragm effective width ($b_{de}$), thickness of block-out at anchorage measured along web axis ($t_{bo}$) and average inclination angle of tendons ($\alpha_{inc.}$) are illustrated in the figure.

![Diagram](image)

Figure 9-2: Longitudinal section illustrating the location of investigated section for general anchorage zone calculation.

According to the calculated stresses $f_1$, $f_2$, $f_3$ and $f_4$, the acting force at each part of the box girder is calculated as follows:

$$F_{deck} = \frac{f_1 + f_2}{2} \times S \times t_d$$  \hspace{1cm} Eq. 9-1

$$F_{web} = \frac{f_2 + f_3}{2} \times b_w \times (h - t_s - t_d)$$  \hspace{1cm} Eq. 9-2

$$F_{soffit} = \frac{f_3 + f_4}{2} \times S \times t_s$$  \hspace{1cm} Eq. 9-3
These forces are the reactions of the proposed strut-and-tie model: $F_{\text{deck}}$, $F_{\text{web}}$ and $F_{\text{soffit}}$.

9.2.2.1 Transverse bursting forces calculations

The conceptual strut-and-tie model for transverse bursting forces in the deck and the diaphragm is shown in Figure 9-3. Based on the forces applied on the deck ($F_{\text{deck}}$), the reaction forces as well as the applied prestressing forces were set. The location of the applied force at the end anchorage was spaced by $(b/2)$, where $(b)$ is the transverse ordinate of the anchors boundary. The reaction of the deck was placed at $(S/4)$ from the edge, where each reaction force ($F_{\text{deck}}/2$) was implemented at half of the deck. The target bursting forces of the deck and the diaphragm are labeled $T_{\text{deck}}$ and $T_{\text{diaph. transverse}}$, respectively.

![Figure 9-3: Conceptual strut-and-tie model for transverse bursting forces in the deck and the diaphragm.](image)

From the conceptual strut-and-tie model for transverse bursting forces in the deck and the diaphragm, the ordinate ($y_{\text{initial}}$) can be calculated.
From similarity of triangles $\Delta$ i1, i3, i4 and $\Delta$ i1, i5, i6

$$\frac{y_{initial}}{b_{de}} = \frac{\frac{S}{4} - \frac{b}{4}}{\frac{b_{de}}{S} + \frac{S}{2}}$$

$$y_{initial} = \left(\frac{S}{4} - \frac{b}{4}\right) \times \frac{b_{de}}{b_{de} + \frac{S}{2}} \quad \text{Eq. 9-4}$$

The connecting strut trajectory from node i1 to node i5 is not linear due to the transverse bursting forces developed in the diaphragm. The correct trajectory is an arc connecting nodes i1, i2 and i5. This is the same concept conducted by Zhou et al. [13] in developing iso-static lines for bursting forces in post-tensioned anchorage zones. The relation is raised to a higher power grade as follows:

$$y = \left(\frac{S}{4} - \frac{b}{4}\right) \times \left(\frac{b_{de}}{b_{de} + \frac{S}{2}}\right)^{0.65} \quad \text{Eq. 9-5}$$

The grade of the power 0.65 was determined based on best fitting technique for the results obtained from finite element modeling. Then using the developed value of ordinate $y$, the bursting transverse forces in the deck can be calculated as follows:

$$T_{deck} = \frac{\frac{S}{4} - \frac{b}{4} - y}{\frac{S}{2}} \times \frac{F_{deck}}{2}$$

$$T_{deck} = \left(\frac{S}{4} - \frac{b}{4} - y\right) \times \frac{F_{deck}}{S} \quad \text{Eq. 9-6}$$

The conceptual strut-and-tie model for transverse bursting forces in the soffit and the diaphragm is shown in Figure 9-4. Using the same procedures, the bursting transverse forces in the soffit are calculated as follows:
$$T_{\text{soffit}} = \left(\frac{s}{2} - \frac{b}{4} - y\right) \times \frac{F_{\text{soffit}}}{s}$$  \hspace{1cm} \text{Eq. 9-7}

Figure 9-4: Conceptual strut-and-tie model for transverse bursting forces in the soffit and the diaphragm.

From the conceptual strut-and-tie models shown in Figures 9-3 and 9-4, the diaphragm transverse bursting force in the interior diaphragm face ($T_{\text{diaph. transverse}}$) can be obtained. This force is extracted using equilibrium of forces at node i2 as follows:

$$T_{\text{diaph. transverse}} = \left(\frac{F_{\text{deck}} + F_{\text{soffit}}}{2}\right) \times \frac{y}{b_{\text{de}}} - T_{\text{deck}} - T_{\text{soffit}}$$  \hspace{1cm} \text{Eq. 9-8}

9.2.2.2 Vertical bursting forces calculations

The conceptual strut-and-tie model for vertical bursting forces in the web and the diaphragm inner face is illustrated in Figure 9-5. The top force of the deck ($F_{\text{deck}}$) and the bottom force of the soffit ($F_{\text{soffit}}$) are assumed to be applied in the deck and soffit centroids for simplicity. The force in the web is assumed to be divided equally between the upper
half of the web and the lower half with a spacing of \([(h-t_d-t_s)/2]\). The vertical bursting force of the web is assumed to be at distance \(h/4\) from the diaphragm face based on the assumption explained previously.

![Conceptual strut-and-tie model illustration for vertical bursting forces in the web and the diaphragm.](image)

**Figure 9-5**: Conceptual strut-and-tie model illustration for vertical bursting forces in the web and the diaphragm.

In order to distribute the acting forces at the anchorage side between the deck, web and soffit, the vertical dimension of the anchorage device affecting each part of the box section is calculated as follows:

\[
\begin{align*}
    a_d &= a \times \frac{F_{\text{deck}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} \quad \text{Eq. 9-9} \\
    a_w &= a \times \frac{F_{\text{web}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} \quad \text{Eq. 9-10} \\
    a_s &= a \times \frac{F_{\text{soffit}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} \quad \text{Eq. 9-11}
\end{align*}
\]

The anchorage boundary is divided into three zones with three different heights $a_d$, $a_w$, and $a_s$. The concept of obtaining the vertical bursting forces is that the web force ($F_{\text{web}}$) is directly transferred to the web causing bursting force ($T_{\text{web1}}$), however, the deck and soffit forces ($F_{\text{deck}}$ and $F_{\text{soffit}}$) are transferred to the deck and soffit through the diaphragm inner face as well as the web. Based on this assumption the first portion of bursting forces in the web can be calculated as follows:

$$T_{\text{web1}} = \left( \frac{h-t_d-t_s}{4} - \frac{a_w}{4} \right) \times \left( \frac{F_{\text{web}}}{2} \right) \quad \text{Eq. 9-12}$$

The transferred forces of the deck and soffit ($F_{\text{deck}}$ and $F_{\text{soffit}}$) lead to bursting forces in the diaphragm inner face ($T_{\text{diaph. vertical}}$) and the web ($T_{\text{web2}}$). The distribution of the deck and soffit forces between the diaphragm and the web is a function of the diaphragm effective width and the box girder height. This factor was developed based on best fitting techniques with the FE results. The distribution factor obtained is $(b_{de}/10h)^{0.3}$ for the diaphragm inner face ($T_{\text{diaph. vertical}}$) and $[1- (b_{de}/10h)^{0.3}]$ for the web ($T_{\text{web2}}$). It can be noticed that the summation of the distribution factors equals 1.0, which indicates that the deck and soffit forces are totally distributed causing bursting forces in the diaphragm inner face and the web.

As the web bursting forces ($T_{\text{web2}}$) is obtained from the deck and soffit forces ($F_{\text{deck}}$ and $F_{\text{soffit}}$), the centroid of the bursting forces is shifted by $(b_{de} + h/4)$ measured from the anchors face. The centroid location is modified using a correction factor based on the FE results. It is found to be a function of the diaphragm width and the box girder height. This
factor equals \( \sqrt{\frac{2h}{b_{de}}} \). Based on the strut-and-tie model, the distribution factors and correction factor, the bursting forces of the web and diaphragm can be derived as follows:

\[
T_{\text{diaph. vertical}} = \left( \frac{h - \frac{t_d}{2} - \frac{t_s}{2}}{2b_{de}} \right) \left( a_w + \frac{a_d + a_s}{2} \right) \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10h} \right)^{0.3} \quad \text{Eq. 9-13}
\]

\[
T_{\text{web 2}} = \left( \frac{h - \frac{t_d}{2} - \frac{t_s}{2}}{2(b_{de} + \frac{h}{4})} \right) \times \left( \frac{2h}{b_{de}} \right)^{0.3} \times \left[ 1 - \left( \frac{b_{de}}{10h} \right)^{0.3} \right] \quad \text{Eq. 9-14}
\]

These two equations include approximations in order to maintain simplicity of the design methodology. In order to simplify the form of the equation, the distance between the deck anchor zone centroid \((a_d)\) to the soffit anchor zone centroid \((a_s)\) is defined as \((a')\), see Figure 9-5, which is calculated as follows:

\[
a' = a_w + \frac{a_d}{2} + \frac{a_s}{2} \quad \text{Eq. 9-15}
\]

Also the distance between the deck centroid and the soffit centroid is defined as \((h')\), see Figure 9-5, which is calculated as follows:

\[
h' = h - \frac{t_d}{2} - \frac{t_s}{2} \quad \text{Eq. 9-16}
\]

Using equations 9-15 and 9-16 in order to simplify equations 9-13 and 9-14, the developed vertical bursting forces of the diaphragm inner face and the web are updated as follows:

\[
T_{\text{web 2}} = \left( h' - \frac{t_d}{2} - \frac{t_s}{2} \right) \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10h} \right)^{0.3} \quad \text{Eq. 9-17}
\]

\[
T_{\text{diaph. vertical}} = \frac{h' - a'}{2b_{de}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10h} \right)^{0.3} \quad \text{Eq. 9-18}
\]
The final vertical bursting force of the web is the summation of its two portions $T_{\text{web}1}$ and $T_{\text{web}2}$ as follows:

$$T_{\text{web}} = T_{\text{web}1} + T_{\text{web}2} \quad \text{Eq. 9-19}$$

### 9.3 Comparison between Results

In order to check the adequacy of the developed set of equations, the results of the proposed set of equations are compared to the results of field monitoring, experimental work and FE analysis. These comparisons provide validation for the proposed design equations and assure their applicability. The comparisons include diaphragm transverse and vertical bursting forces, web vertical bursting force as well as deck and soffit transverse bursting forces.

Comparison between results for the diaphragm transverse and vertical bursting forces is shown in Figure 9-6 and 9-7, respectively. These figures illustrate the relationship between percentage of diaphragm bursting force to design prestressing force and the effective diaphragm width to box girder height in both transverse and vertical directions. This comparison includes field monitoring results for both interior and exterior girders, experimental results, FE results considering and neglecting concrete tensile strength and the proposed equation. The proposed equation provides a very good correlation with the FE results neglecting concrete tensile strength. This case represents the ultimate case, where the reinforcement is responsible for resisting the total bursting forces. The coefficient of determination between the proposed equation and the FE results for diaphragm transverse is 0.99 and 0.98 for diaphragm vertical. This illustrates the adequacy of the proposed equation.
All the remaining results including experimental results, field monitoring results and the FE results considering concrete tensile strength are less than the proposed equation. The reason for this difference is the contribution of concrete section in resisting tensile bursting forces due to concrete tensile capacity and tension stiffening. All the plotted bursting force results possess the same inversely proportional relationship with the ratio of the effective diaphragm width to box girder height. This indicates that the proposed equation considers the effect of diaphragm width on bursting forces.

Figure 9-6: Comparison between results for diaphragm transverse bursting forces.

Figure 9-7: Comparison between results for diaphragm vertical bursting forces.
The same good correlation is observed in the comparison between results for the web vertical bursting forces as shown in Figure 9-8. It illustrates the relationship between percentage of web vertical bursting force to design prestressing force and the effective diaphragm width to box girder height. The proposed equation provides a very good correlation with the FE results neglecting concrete tensile strength. The coefficient of determination between the proposed equation and the FE results is 0.99, this illustrates the adequacy of the proposed equation.

All the remaining results are less than the proposed equation. The reason for this difference is the contribution of concrete section in resisting tensile bursting forces due to concrete tensile capacity and tension stiffening. All the plotted bursting force results possess the same inversely proportional relationship with the ratio of the effective diaphragm width to box girder height. This indicates that the proposed equation considers the effect of diaphragm width on bursting forces.

Figure 9-8: Comparison between results for web vertical bursting forces.

Comparisons between results for the deck and soffit transverse bursting forces are shown in Figures 9-9 and 9-10, respectively. These figures illustrate the relationship
between percentage of bursting force to design prestressing force and the effective diaphragm width to box girder height for the deck and soffit. In both figures, the proposed equations provide very good correlation with the FE results neglecting concrete tensile strength. The coefficient of determination between the proposed equations for both deck and soffit and the FE results is 0.96, this illustrates the adequacy of the proposed equations.

All the remaining results including experimental results, field monitoring results and the FE results considering concrete tensile strength are less than the proposed equation. The reason for this difference is the contribution of concrete section in resisting tensile bursting forces due to concrete tensile capacity and tension stiffening. However, the difference in values, all the plotted bursting force results possess the same inversely proportional relationship with the ratio of the effective diaphragm width to box girder height. This indicates that the proposed equations consider the effect of diaphragm width on bursting forces.

Figure 9-9: Comparison between results for deck transverse bursting forces.
9.4 Modifications for the Proposed Set of Equations

The proposed set of equations is modified to include more parameters that were not considered in the previous strut-and-tie conceptual models. These parameters include the effects of tendon inclination, diaphragm openings and diaphragm skew angle. The modifications were implemented based on the FE results neglecting concrete tensile strength. These FEMs are considered representative for the ultimate case scenario, where all the forces are handled only by reinforcement, no concrete tensile capacity.

9.4.1 Tendon inclination modification

Tendon inclination was investigated in the FEMs parametric study. Based on this parameter investigated in section 8.5.7, the vertical bursting forces of the diaphragm and web as well the transverse bursting force in the soffit were directly proportional with the tendon inclination angle. However, the deck bursting force was inversely proportional with tendon inclination angle. No significant effect was observed on the diaphragm transverse bursting forces.
Using the developed set of proposed equations, the transverse bursting forces showed good agreement with the finite element results as shown in Figure 9-11. This good correlation was obtained, as changing tendon inclination affects the eccentricity at the investigated section located \([(h/4 + S/2)/2]\) from the diaphragm face. This consequently changes the developed stress values \(f_1, f_2, f_3\) and \(f_4\). When tendon inclines downward, stresses in the soffit increases and the transverse bursting forces increase as well. However, the stresses in the deck decrease and the transverse bursting forces decrease. That’s why, transverse bursting force in the soffit is directly proportional with the tendon inclination angle and transverse bursting force in the deck is inversely proportional with the tendon inclination angle.

![Figure 9-11: Tendon inclination angle verification for transverse bursting forces.](image)

The proposed equations for the vertical bursting forces in the diaphragm and the web did not include any variable for tendon inclination angle. Therefore, the proposed equations develop constant values for the vertical bursting forces with various inclination angles as shown in Figure 9-12.
In order to modify the developed equations to accommodate the tendon inclination angle in the vertical bursting forces, equations 9-18 and 9-19 were modified by adding a correction factor based on tendon inclination angle as follows:

\[
T_{\text{diaph. vertical}} = \frac{h' - a'}{2 b_{de}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10 h} \right)^{0.3} \times [1 + |\sin(\alpha_{\text{inc.}})|] \quad \text{Eq. 9-20}
\]

\[
T_{\text{web}} = (T_{\text{web1}} + T_{\text{web2}}) \times [1 + |\sin(\alpha_{\text{inc.}})|] \quad \text{Eq. 9-21}
\]

This correction factor of \([1 + |\sin(\alpha_{\text{inc.}})|]\) corresponds to the modification in the bursting force equation 5.10.9.6.3-1 of AASHTO LRFD Bridge Design Specifications [3]. The modification factor in AASHTO was 0.5 \[\sum (P_{tu} \sin(\alpha_{\text{inc.}}))\]. The modification implemented in equations 9-20 and 9-21 provides adequate enhancement for the correlation with FE results as shown in Figure 9-12. The coefficient of determination between the modified equations for diaphragm vertical bursting force and the FE results is 0.88 and 0.98 for the vertical web bursting forces.

Figure 9-12: Tendon inclination angle verification for vertical bursting forces.
9.4.2 Diaphragm opening modification

Openings in the box girder diaphragm affect the performance of end anchorage. These openings have significant effect on the continuity of stresses in diaphragms as well as the strains developed in reinforcing bars. FEMs parametric study included the effect of diaphragm openings. Based on this parameter investigated in section 8.5.9, slight increase can be observed in the bursting forces of the deck and soffit. A slight decrease occurred in the diaphragm transverse bursting forces.

The proposed equations for the transverse bursting forces in the diaphragm, deck and soffit did not include any variable for diaphragm openings. Therefore, the proposed equations develop constant values for the transverse bursting forces with various opening dimensions as shown in Figure 9-13. The openings implemented in all the developed FEMs were centered between web girders. Opening dimensions width and height are labeled as $W_o$ and $H_o$, respectively.

![Figure 9-13: Diaphragm opening verification for transverse bursting forces.](image)

In order to modify the developed equations to accommodate the diaphragm openings in the transverse bursting forces, the strut-and-tie model for transverse bursting forces in
the deck and the diaphragm is updated as shown in Figure 9-14. This figure illustrates the effect of diaphragm opening in shifting the strut extending from the anchor to the diaphragm face, which causes a reduction in the transverse bursting force of the diaphragm \( T_{\text{diaph. transverse}} \) and an amplification in the transverse bursting force of the deck \( T_{\text{deck}} \). This shift develops a reduction in the \( y \) ordinate, which is considered in the modifications for equations 9-6 to accommodate diaphragm openings as follows:

\[
T_{\text{deck}} = \left[ \frac{S}{4} - \frac{b}{4} \right] - y \left( 1 - \frac{W_0}{4S} \right) \times \frac{F_{\text{deck}}}{S} \quad \text{Eq. 9-22}
\]

The same concept is applied to the transverse bursting force in the soffit by implementing a reduction in the \( y \) ordinate in equation 9-7 as follows:

\[
T_{\text{soffit}} = \left[ \frac{S}{4} - \frac{b}{4} \right] - y \left( 1 - \frac{W_0}{4S} \right) \times \frac{F_{\text{soffit}}}{S} \quad \text{Eq. 9-23}
\]

Figure 9-14: Modification of conceptual strut-and-tie model for transverse bursting forces in the deck and the diaphragm due to diaphragm opening.
Modifying both equations 9-6 and 9-7 will consequently update the diaphragm transverse forces obtained from equation 9-8. Using the modified set of proposed equations, the transverse bursting forces showed good agreement with the finite element results as shown in Figure 9-13. The coefficient of determination between the modified equations for transverse bursting forces and the FE results is 0.97 for the diaphragm, 0.92 for the deck and 0.95 for the soffit.

Based on experimental work and FE results, diaphragm openings have no significant effect on the vertical bursting forces of the diaphragm and the web. Using the developed set of proposed equations, the vertical bursting forces showed good agreement with FE results as shown in Figure 9-15.

![Graph showing the percentage of bursting force to design prestressing force vs opening width to girder spacing (Wg/S) and opening height to box girder height (Hv/h).](image)

**Figure 9-15: Diaphragm opening verification for vertical bursting forces.**

However, the even with slight effect of the diaphragm opening on the bursting forces, the reinforcement should be densified around the opening corners to minimize stress concentrations at these zones. The amount of the reinforcement that cannot be placed because of the opening should be placed along each side and on the top/bottom of the opening.
9.4.3 Diaphragm skew angle modification

Skew angle ($\alpha_{\text{skew}}$) is defined as the angle between the axis of support relative to a line normal to the longitudinal axis of the bridge, i.e. a zero-degree skew denotes a rectangular bridge. Diaphragm skew angle was investigated in the FEMs parametric study. Based on this parameter investigated in section 8.5.8, the transverse bursting forces in the diaphragm, deck and soffit are directly proportional with the diaphragm skew angle. However, the vertical bursting forces of the diaphragm and the web are not affected by changing the skew angle.

The proposed equations for the transverse bursting forces in the diaphragm, deck and soffit did not include any variable for diaphragm skew angle. Therefore, the proposed equations develop constant values for the transverse bursting forces with various skew angles as shown in Figure 9-16. However, the transverse bursting forces in the diaphragm, deck and soffit are rapidly increasing with the diaphragm skew angle based on FE results.

![Figure 9-16: Diaphragm skew angle verification for transverse bursting forces.](image)

In order to modify the developed equations to accommodate the diaphragm skew angle ($\alpha_{\text{skew}}$) in the transverse bursting forces, the strut-and-tie model for transverse
bursting forces in the deck and the diaphragm is updated as shown in Figure 9-17. This figure illustrates the effect of diaphragm skew angle in modifying the whole strut-and-tie model for transverse bursting forces. Compression diagonal members (i1-i6 and i6-i7) are implemented in model to maintain stability. The compression forces in these diagonals will be resisted by the concrete compressive strength. The transverse tie in the diaphragm inner face is inclined based on the diaphragm skew angle. This inclination changed the model by shifting node i6 towards the deck and node i3 away from the deck. This shift is calculated as follows:

From $\Delta i3, i4, i5$ (Right angle triangle)

Angle $i4$-$i3$-$i5 = \alpha_{skew}$

Member $i3$-$i4 = i3$-$i2 + i2$-$i4 = y + (b/4)$

So, member $i4$-$i5 = \left(y + \frac{b}{4}\right)\tan\alpha_{skew}$

Distance between nodes $i4$-$i5$ represents the shift of nodes $i3$ and $i6$. This shift develops a reduction in the moment arm for the transverse bursting force of the deck

$T_{deck} \left(\frac{s}{2} - \left[\left(y + \frac{b}{4}\right)\tan\alpha_{skew}\right]\right)$ instead of $\frac{s}{2}$, which is considered in the modifications for equations 9-6 to accommodate diaphragm skew angle as follows:

$$T_{deck} = \frac{s - \frac{b}{4} - y}{\frac{s}{2} - \left[\left(y + \frac{b}{4}\right)\tan\alpha_{skew}\right]} \times \frac{F_{deck}}{2} \quad \text{Eq. 9-24}$$

After simplification

$$T_{deck} = \frac{s - b - 4y}{4s - [(8y + 2b)\tan\alpha_{skew}]} \times F_{deck} \quad \text{Eq. 9-25}$$
The same concept is applied to the transverse bursting force in the soffit by implementing this shift in equation 9-7 as follows:

\[ T_{soffit} = \frac{s-b-4y}{4s-[(8y+2b)\tan \alpha_{skew}]} \times F_{soffit} \quad \text{Eq. 9-26} \]

From both equations for the deck and soffit the transverse bursting force in the interior diaphragm face was extracted using equilibrium of forces as follows:

\[ T_{diaph.\, transverse} = \left[ \left( \frac{F_{deck}+F_{soffit}}{2 \cos \alpha_{skew}} \right) \times \frac{y}{b_{de}-\left[ (\frac{y}{2}^2 + \frac{b}{2}) \tan \alpha_{skew} \right]} \right] - \frac{T_{deck}}{\cos \alpha_{skew}} - \frac{T_{soffit}}{\cos \alpha_{skew}} \quad \text{Eq. 9-27} \]

Several approximations were applied for these three equations 9-25 to 9-27 in order to maintain simplicity of the equations as long as these equations provide good correlation with FE results as presented in Figure 9-16. The coefficient of determination between the modified equations for transverse bursting forces and the FE results is 0.99 for the diaphragm and the deck and 0.97 for the soffit.
Based on field monitoring and FE results, diaphragm openings have no significant effect on the vertical bursting forces of the diaphragm and the web. Using the developed set of proposed equations, the vertical bursting forces showed good agreement with FE results as shown in Figure 9-18.

Figure 9-17: Modification of conceptual strut-and-tie model for transverse bursting forces in the deck and the diaphragm to consider diaphragm skew angle.

Figure 9-18: Diaphragm skew angle verification for vertical bursting forces.
Although, the vertical bursting forces of the diaphragm are not affect by changing the skew angle, the distribution of vertical bursting forces on both acute and obtuse sides of the diaphragm are not the same as shown in Figure 9-19. The vertical bursting force in the obtuse side increases with the diaphragm skew angle, however, the vertical bursting force in the acute side decreases. Consequently, the ratio between vertical bursting forces in the acute side to the force in the obtuse side decreases with the diaphragm skew angle.

Figure 9-19: Verification of ratio of diaphragm vertical bursting force in acute side to obtuse side based on diaphragm skew angle.

Even with the total vertical bursting force not affected by the diaphragm skew angle, its value obtained from equation 9-18, is divided on both sides of the web to \( T_{dv-acute} \) affecting the acute side of the diaphragm and \( T_{dv-obtuse} \) affecting the obtuse side of the diaphragm as follows:

\[
T_{diaph.\,vertical} = T_{dv-acute} + T_{dv-obtuse} \quad \text{Eq. 9-28}
\]

The ratio between those two forces was related to the diaphragm skew angle as follows:

\[
\frac{T_{dv-acute}}{T_{dv-obtuse}} = \cos^2(\alpha_{skew}) \quad \text{Eq. 9-29}
\]
This equation provides good correlation to the FE results as shown in Figure 9-19. In order to obtain the vertical bursting forces applied on both acute and obtuse sides of the diaphragm inner face equations 9-28 and 9-29 can be solved simultaneously.

### 9.4.4 Stressing both bridge ends modification

In case of stressing both bridge ends, bursting strain values increase at the initial stressing end due to stressing the second end. Strain propagation due to stressing the both ends is inversely proportional with friction losses as presented previously in Eq. 5-6 for $\beta_{end}$. This factor shall be considered in the design force for anchorage zone as follows:

In case of stressing both bridge ends use Eq. 5-6

In case of stressing the bridge from one side only

$\beta_{end} = 1.0$  \hspace{1cm} \text{Eq. 9-30}

The ultimate design force for anchorage zone per girder ($P_{du}$) can be calculated as follows:

$P_{du} = 1.2 \times \beta_{end} \times P_{j \text{ per girder}}$  \hspace{1cm} \text{Eq. 9-31}

Where

$P_{j \text{ per girder}}$ is the applied jacking force per girder

1.2 is the load factor for jacking force as stated by AASHTO LRFD Bridge Design Specifications [3], section 3.4.3.2.

### 9.5 Summary of Modified Equations

The summary for modified equations is presented as follows:
9.5.1 Ultimate design force for anchorage zone per girder

Based on equations 5-6, 9-30 and 9-31,

\[ P_{du} = 1.2 \times \beta_{end} \times P_{J\text{ per girder}} \]  
Eq. 9-32

In case of stressing both bridge ends

\[ \beta_{end} = 1.12 - [0.12 \left(1 - e^{-(Kx + \mu \alpha)}\right)] \]  
Eq. 9-33

In case of stressing the bridge from one side only both bridge ends

\[ \beta_{end} = 1.0 \]  
Eq. 9-34

9.5.2 Deck and soffit transverse bursting forces

Based on equations 9-22, 9-23, 9-25 and 9-26, transverse bursting forces in the deck and soffit acting at S/2 from the diaphragm inner face are:

\[ T_{\text{deck}} = \frac{s-b-4\left[y\left(\frac{1-W_o}{4S}\right)\right]}{4S - [(8y+2b)\tan \alpha_{\text{skew}}]} \times F_{\text{deck}} \]  
Eq. 9-35

\[ T_{\text{soffit}} = \frac{s-b-4\left[y\left(\frac{1-W_o}{4S}\right)\right]}{4S - [(8y+2b)\tan \alpha_{\text{skew}}]} \times F_{\text{soffit}} \]  
Eq. 9-36

9.5.3 Diaphragm transverse bursting forces

Based on equation 9-27, diaphragm transverse bursting force acting uniformly along the diaphragm inner face is:

\[ T_{\text{diaph. transverse}} = \left[\frac{F_{\text{deck}} + F_{\text{soffit}}}{2 \cos \alpha_{\text{skew}}}\right] \times \frac{y}{b_{de} - \left[(\frac{y}{2})\tan \alpha_{\text{skew}}\right]} - \frac{T_{\text{deck}}}{\cos \alpha_{\text{skew}}} - \frac{T_{\text{soffit}}}{\cos \alpha_{\text{skew}}} \]  
Eq. 9-37

9.5.4 Diaphragm vertical bursting forces

Based on equation 9-20, 9-28 and 9-29, diaphragm vertical bursting force acting uniformly along the diaphragm inner face is:
\[ T_{\text{diaph. vertical}} = \frac{h' - a'}{2 b_{de}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10 h} \right)^{0.3} \times [1 + |\sin(\alpha_{\text{inc.}})|] \]  
Eq. 9-38

In case of skew diaphragm the force is divided on both acute and obtuse sides as follows:

\[ T_{\text{diaph. vertical}} = T_{dv-acute} + T_{dv-obtuse} \]  
Eq. 9-39

\[ \frac{T_{dv-acute}}{T_{dv-obtuse}} = \cos^2(\alpha_{\text{skew}}) \]  
Eq. 9-40

### 9.5.5 Web vertical bursting forces

Based on equation 9-12, 9-17 and 9-21, web vertical bursting force acting at h/4 from the diaphragm inner face is:

\[ T_{\text{web 1}} = \left( \frac{h - d - t_2}{4} \right) - \frac{a_w}{4} \times \left( \frac{F_{\text{web}}}{2} \right) \]  
Eq. 9-41

\[ T_{\text{web 2}} = \frac{h' - a'}{2} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left[ 1 - \left( \frac{b_{de}}{10 h} \right)^{0.3} \right] \]  
Eq. 9-42

\[ T_{\text{web}} = (T_{\text{web 1}} + T_{\text{web 2}}) \times [1 + |\sin(\alpha_{\text{inc.}})|] \]  
Eq. 9-43

### 9.6 Final Equations Verification

The modified proposed set of equations presented in section 9.5 was verified using the remaining variables of FE parametric study as well as the effect of opening dimensions obtained from experimental results of double girder specimens. The verification included girder spacing to box girder height, deck and soffit thicknesses, web width, number of prestressing anchors, edge eccentricity ratio and diaphragm opening dimension ratios. The results of the modified proposed set of equations are compared to the FE results. These comparisons provide validation for the modified design equations and assure its applicability.
9.6.1 Girder spacing to box girder height

Ratio of girder spacing to box girder height was investigated in the FEMs parametric study. Based on this investigation, all developed bursting forces were directly proportional with girder spacing to box girder height ratio except vertical bursting forces in the web. No significant effect was observed on the web vertical bursting forces.

Comparison between the modified proposed equations and FE results for diaphragm, deck and soffit transverse bursting forces is shown in Figure 9-20. It illustrates the relationship between percentage of bursting force to design prestressing force and the ratio of girder spacing to box girder height. This figure provides very good correlation between the modified proposed equations and the FE results for the transverse bursting forces in the diaphragm, deck and soffit. The coefficient of determination between the results is 0.99 for the diaphragm and the soffit and 0.98 for the deck. This illustrates the adequacy of the modified equations.

![Figure 9-20: Ratio of girder spacing to box girder height verification for transverse bursting forces.](image)

Comparison between the modified proposed equations and FE results for diaphragm and web vertical bursting forces is shown in Figure 9-21. This figure illustrates very good
correlation between the modified proposed equations and the FE results for the vertical bursting forces in the diaphragm and the web. The coefficient of determination between the results is 0.98 for the diaphragm and 0.86 for the web.

![Figure 9-21: Ratio of girder spacing to box girder height verification for vertical bursting forces.](image)

### 9.6.2 Deck and soffit thicknesses

Deck and soffit thicknesses were investigated in the FEMs parametric study. Based on this investigation, all developed bursting forces are directly proportional with the increase in the deck and soffit thicknesses, except the vertical bursting force of the web. Vertical bursting force in the web is inversely proportional with the increase of the deck and soffit thicknesses. This indicates that the thicker the element is the more bursting force it attracts.

Comparison between the modified proposed equations and FE results for diaphragm, deck and soffit transverse bursting forces is shown in Figure 9-22. It illustrates the relationship between the bursting force to design prestressing force and deck thickness on the primary horizontal axis as well as the soffit thickness on the secondary horizontal axis. This figure provides very good correlation between the modified proposed equations and
the FE results for the transverse bursting forces in the diaphragm, deck and soffit. This illustrates the efficiency of developed equations to distribute bursting forces based on element stiffness, as the thicker the element is the more bursting force it attracts. The coefficient of determination between the results is 0.99 for the diaphragm and the deck and 0.93 for the soffit.

Figure 9-22: Deck and soffit thicknesses verification for transverse bursting forces.

Comparison between the modified proposed equations and FE results for diaphragm and web vertical bursting forces is shown in Figure 9-23. This figure illustrates very good correlation between the modified proposed equations and the FE results for the vertical bursting forces in the diaphragm and the web. The coefficient of determination between the results is 0.91 for the diaphragm and 0.99 for the web.
9.6.3 Web width

Web width was investigated in the FEMs parametric study. Based on this investigation, all developed bursting forces were inversely proportional with the increase in web width, except the vertical bursting force of the web. Vertical bursting force in the web was directly proportional with the increase of web width.

Comparison between the modified proposed equations and FE results for diaphragm, deck and soffit transverse bursting forces is shown in Figure 9-24. It illustrates the relationship between the bursting force to design prestressing force and web girder width. This figure provides very good correlation between the modified proposed equations and the FE results for the transverse bursting forces in the diaphragm, deck and soffit. This illustrates the efficiency of developed equations to distribute bursting forces based on element stiffness, as the thicker the element is the more bursting force it attracts. The coefficient of determination between the results is 0.98 for the diaphragm, 0.96 for the deck and 0.99 for the soffit.
Comparison between the modified proposed equations and FE results for diaphragm and web vertical bursting forces is shown in Figure 9-25. This figure illustrates very good correlation between the modified proposed equations and the FE results for the vertical bursting forces in the diaphragm and the web. The coefficient of determination between the results is 0.93 for the diaphragm and 0.88 for the web.

These good correlations approve the efficiency of developed equations to distribute bursting forces based on element stiffness, as the thicker the element is the more bursting force it attracts. This matches the results obtained from the verification of the deck and soffit thicknesses.
9.6.4 Number of prestressing anchors

Number of prestressing anchors was investigated in the FEMs parametric study. Based on this investigation, vertical bursting forces in the web and the diaphragm were inversely proportional with number of anchors. No significant effect was observed on the transverse bursting forces of the diaphragm, deck or soffit.

Comparison between the modified proposed equations and FE results for diaphragm, deck and soffit transverse bursting forces is shown in Figure 9-26. It illustrates the relationship between the bursting force to design prestressing force and number of prestressing anchors. This figure provides very good correlation between the modified proposed equations and the FE results for the transverse bursting forces in the diaphragm, deck and soffit. The coefficient of determination between the results is 0.98 for the diaphragm and 0.97 for the deck and the soffit.

![Figure 9-26: Number of prestressing anchors verification for transverse bursting forces.](image)

Comparison between the modified proposed equations and FE results for diaphragm and web vertical bursting forces is shown in Figure 9-27. This figure illustrates good correlation between the modified proposed equations and the FE results for the vertical
bursting forces in the diaphragm and the web. The coefficient of determination between the results is 0.96 for the diaphragm and 0.93 for the web.

Figure 9-27: Number of prestressing anchors verification for vertical bursting forces.

### 9.6.5 Edge eccentricity ratio

Edge eccentricity was investigated in the FEMs parametric study. Based on this investigation, bursting forces are not affected by edge eccentricity except the transverse bursting forces of the deck and soffit.

Comparison between the modified proposed equations and FE results for diaphragm, deck and soffit transverse bursting forces is shown in Figure 9-28. It illustrates the relationship between the bursting force to design prestressing force and eccentricity ratio. This figure provides very good correlation between the modified proposed equations and the FE results for the transverse bursting forces in the diaphragm, deck and soffit. The coefficient of determination between the results is 0.96 for the diaphragm and 0.99 for the deck and the soffit. Obtaining a set of equations that can capture the changes including the eccentricity of prestressing forces provides confidence in the approaches used to develop this set of equations.
Comparison between the modified proposed equations and FE results for diaphragm and web vertical bursting forces is shown in Figure 9-29. This figure illustrates good correlation between the modified proposed equations and the FE results for the vertical bursting forces in the diaphragm and the web. The coefficient of determination between the results is 0.87 for the diaphragm and 0.89 for the web.

9.6.6 Diaphragm openings experimental results

The results of the modified set of equations are compared to the experimental results of double girder specimens. In this phase of the experimental work, the diaphragm opening
was the major investigated parameter. These comparisons provide validation for the modified design equations and assure its applicability. The comparisons include diaphragm transverse and vertical bursting forces, web vertical bursting force as well as deck and soffit transverse bursting forces.

Comparison between results for the diaphragm transverse bursting forces is shown in Figure 9-30. It illustrates the relationship between percentage of diaphragm transverse bursting force to design prestressing force and opening dimension ratios on the horizontal primary and secondary axes. This comparison includes experimental results, FE results and the modified proposed equation. The proposed equation provides a very good correlation with the FE results neglecting concrete tensile strength. This case represents the ultimate case, where reinforcement bars are responsible for resisting the total bursting forces. The coefficient of determination between the proposed equation and the FE results is 0.97, this illustrates the adequacy of the modified equation.

The experimental results are less than the proposed equation. The reason for this difference is the contribution of concrete section in resisting tensile bursting forces due to concrete tensile capacity and tension stiffening. The plotted experimental bursting force possess the same inversely proportional relationship with opening dimensions ratio.
Verification for the diaphragm vertical bursting forces is shown in Figure 9-31. It illustrates the relationship between percentage of diaphragm vertical bursting force to design prestressing force and opening dimension ratios. The proposed equation provides a good correlation with the FE results. The coefficient of determination between the proposed equation and the FE results is 0.87, which illustrates the adequacy of the proposed equation. Experimental results are less than the proposed equation due to concrete tensile capacity and tension stiffening.
The same good correlation is observed in the comparison between results for the web vertical bursting forces as shown in Figure 9-32. The proposed equation provides a very good correlation with the FE results, where coefficient of determination between results is 0.93.

Experimental results are less than the proposed equation due to concrete tensile capacity and tension stiffening. However, the difference in values between proposed equation and experimental results, the plotted bursting force possess the same constant relationship with opening dimensions ratio as for the experimental results and the proposed equation.

![Figure 9-32: Verification of diaphragm opening experimental results for web vertical bursting forces.](image)

Verification of the deck and soffit transverse bursting forces is shown in Figures 9-33 and 9-34, respectively. These figures illustrate the relationship between percentage of bursting force to design prestressing force and opening dimension ratios. In both figures, the proposed equations provide very good correlation with the FE results. The coefficient of determination between the proposed equations and experimental results is 0.92 for deck and 0.95 for soffit, this illustrates the adequacy of the proposed equations.
Figure 9-33: Verification of diaphragm opening experimental results for deck transverse bursting forces.

Figure 9-34: Verification of diaphragm opening experimental results for soffit transverse bursting forces.

As the diaphragm opening has minimal effect on the vertical as well as transverse bursting forces up to 0.2 opening height to box girder height or opening width to girder spacing, effect of diaphragm opening can be neglected up to this limit.

9.6.7 Combinations of variable parameters

In order to validate the equations with combinations of variable parameters, eight finite elements models were developed. Variable parameters were changed in each of these models including: box girder height, web width, girder spacing, effective diaphragm width,
edge eccentricity, number of anchors, diaphragm skew angle and diaphragm opening dimensions. Configurations of developed finite element models are summarized in Table 3-3.

Table 9-2: Configurations of developed finite element models.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model Number</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Box girder height [h] (ft)</td>
<td></td>
<td>9.75</td>
<td>9.75</td>
<td>3.75</td>
<td>3.75</td>
<td>6.75</td>
<td>6.75</td>
<td>6.75</td>
<td>6.75</td>
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<td>Web width [b_w] (in)</td>
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<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>18</td>
<td>12</td>
<td>12</td>
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<tr>
<td>Girder spacing [S] (ft)</td>
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<td>9.75</td>
<td>7.5</td>
<td>3.75</td>
<td>11.5</td>
<td>8.5</td>
<td>11.5</td>
<td>11.5</td>
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<tr>
<td>Deck thickness [t_d] (in)</td>
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<td>8.5</td>
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<td>8.5</td>
<td>8.5</td>
<td>8.5</td>
<td>8.5</td>
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<tr>
<td>Soffit thickness [t_s] (in)</td>
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<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
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<tr>
<td>Effective diaphragm width [b_de] (in)</td>
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<td>Tendon inclination angle [α_inc.] (degree)</td>
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<td>Height of anchorage [a] (in)</td>
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<td>12</td>
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<tr>
<td>Width of anchorage [b] (in)</td>
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<td>13.5</td>
<td>12</td>
<td>12</td>
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<td>13.5</td>
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<tr>
<td>Diaphragm skew angle [α_skew] (degree)</td>
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<td>Opening width [W_o] (in)</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>40.5</td>
<td>32</td>
</tr>
</tbody>
</table>

The results of the developed models showed good correlation with the proposed equations estimating bursting forces. The variation between results did not exceed 15% as shown in Figure 9-35, where a comparison between FEMs with combinations of variable parameters and the proposed equations is conducted.
Failure mode of general anchorage zone in box girder bridges is web compression failure. This was concluded from longitudinal concrete gauges in the web, which were subjected to high compression values. Also, buckling of longitudinal bars for specimen ends D6T, D5T and D5H was observed. In addition, high compressive stresses were observed in case of diaphragm openings using FEMs. However, no failure occurred in specimens with solid end diaphragm.

Web compression failure governed the safety of the anchorage zone based on comparison between failure loads and ultimate web capacity presented in Table 7-3. In order to assure safety of general anchorage, web nominal capacity shall be higher than the ultimate jacking force in case of diaphragm openings. Also, this shall be satisfied in extreme case, where diaphragm effective width is less than 0.3 of the box girder height. This case was not applicable to be tested experimentally using ½ scaled specimens.
small scale, part of the prestressing anchors will have been inserted into the web, which is not representative for the full prototype bridge.

As the diaphragm opening has minimal effect on the vertical as well as transverse bursting forces up to 0.2 opening height to box girder height or opening width to girder spacing, effect of diaphragm opening can be neglected up to this limits.

The web nominal capacity should be calculated and compared to the ultimate jacking force in the following cases:

- \( \frac{H_o}{h} > 0.2 \) or
- \( \frac{W_o}{S} > 0.2 \) or
- \( b_{de} < 0.4 h \)

The web nominal capacity is calculated based on the AASHTO LRFD Bridge Design Specifications - equation 5.7.4.4-3 [3] as follows:

\[
P_{n-web} = 0.8 \left[ 0.85 f'_{ci} \left( A_{g-web} - A_{st-web} - A_{ducts} \right) + f_y A_{st-web} \right]
\]

Eq. 9-44

Where:

- \( P_{n-web} \) = Nominal axial resistance of the web girder (kip)
- \( f'_{ci} \) = Initial concrete compressive strength at prestressing day (ksi)
- \( A_{g-web} \) = Gross area of the web section \([b_w \times h]\) (in.\(^2\))
- \( A_{st-web} \) = Total area of longitudinal reinforcement included in the web (in.\(^2\))
- \( A_{ducts} \) = Total area of prestressing ducts (in.\(^2\))
- \( f_y \) = Yield strength of reinforcement (ksi)
To maintain safety

\[ 1.2 P_{\text{per girder}} < \phi P_{n-web} \]  

Eq. 9-45

Where \( \phi \) factor is 0.8 based on the AASHTO LRFD Bridge Design Specifications - section 5.5.4.2.1 [3]

By applying this check of web capacity, brittle failure at general anchorage zone is eliminated. The total safety factor to prevent any unexpected failure is satisfied. The developed set of proposed equations provides adequate correlation with all of the investigated parameters including; diaphragm width to box girder height, tendon inclination angle, diaphragm openings, diaphragm skew angle, girder spacing to box girder height, deck and soffit thicknesses, web width, number of prestressing anchors and edge eccentricity. Also, developing a certain check based on the failure mode satisfy the complete safety factor in extreme cases, where failure may occur. The proposed equations provide an optimum method to design general anchorage zone of box girder bridges.

\section*{9.8 Simplified Design Method}

The equations that have been develop permit the engineer to design a general anchorage zone considering many variables. What is often needed is a simple quick estimation of the reinforcement needed in the general anchorage zone. The proposed design equations were used to develop simplified design table. The main geometry parameters affecting the bursting forces of an anchorage zone were diaphragm width to box girder height ratio and girder spacing to box girder height ratio. Slight effect was observed for web width, and deck and soffit thicknesses on bursting forces. Therefore, to provide simplified technique of design, anchorage zone bursting forces were calculated based on
the effective diaphragm width to box girder height ratio as illustrated in Table 9-3. This table is divided into two main sections based on the girder spacing to box girder height.

Several combinations of variables were investigated to develop this simplified methodology including: box girder height, girder spacing, web width, deck and soffit thicknesses, tendon inclination, number of anchors, edge eccentricity, diaphragm skew angle and diaphragm opening dimensions. These variables were implemented in the proposed equations summarized in section 9.5. These studied combinations were used to capture extreme cases, where maximum bursting forces were extracted and summarized in Table 9-3. In order not to provide overestimated values of bursting forces, the use of the simplified method has a few limitations. This design methodology can be applied in case of the following conditions are satisfied:

- \( |\text{Tendon inclination angle} (\alpha_{\text{inc.}}) | < 6^\circ \)
- \( |\text{Eccentricity ratio} (e_{\text{edge}}/h) | < 0.1 \)
- Minimum two anchors are used for stressing
- \( \text{Diaphragm skew angle} (\alpha_{\text{skew}}) < 30^\circ \)
- \( \text{Diaphragm opening width} (W_0) < 0.2 \ S \)
- \( \text{Diaphragm opening height} (H_0) < 0.2 \ h \)
Table 9-3: Simplified design method for anchorage zone bursting forces.

<table>
<thead>
<tr>
<th>Effective diaphragm width to box girder height (b&lt;sub&gt;de/h&lt;/sub&gt;)</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder spacing to box girder height (S/h)</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>8.7</td>
<td>7.1</td>
<td>6.0</td>
<td>5.1</td>
<td>4.4</td>
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<tr>
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<td>12.6</td>
<td>11.1</td>
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<td>9.8</td>
<td>8.9</td>
<td>8.2</td>
<td>7.6</td>
</tr>
<tr>
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<td>7.6</td>
<td>7.1</td>
<td>6.6</td>
<td>6.2</td>
<td>5.9</td>
<td>5.6</td>
</tr>
<tr>
<td>Soffit Transverse</td>
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<td>6.3</td>
<td>5.9</td>
<td>5.6</td>
<td>5.3</td>
<td>5.0</td>
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<tr>
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<td></td>
<td></td>
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</tr>
<tr>
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<td>6.6</td>
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<tr>
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<td>12.0</td>
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<td>9.8</td>
<td>9.0</td>
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<td>9.7</td>
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<td>7.3</td>
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<td>7.2</td>
<td>6.8</td>
<td>6.5</td>
<td>6.2</td>
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</table>

This design methodology provide conservative results as it assumed severe combination between parameters affecting anchorage zone bursting forces. Detailed design example using the design equations and the simplified method is provided in Appendix (E).
Chapter (10) – Summary and Conclusions

10.1 Summary

Anchorage zone research program included four main stages: field monitoring, experimental work, finite element analysis and results implementation. Four box girders end zones were instrumented in the field in California. These four bridges cover a wide variety of anchorage zone configuration including different diaphragm width, number of girders, box girder height and openings in the end diaphragm. Strain gauges were used in order to capture strains in reinforcing bars and within the concrete elements. The field investigation enabled the measurement of the actual flow of strains in the general anchorage zone. Through these strains, the flow of forces was estimated as well as the different parameters affecting force spreading in the general anchorage zone.

Experimental work included two phases. The first phase included two single half-scale I-section girders with variable solid end diaphragms widths. The second phase contained two double girder half-scale box section with different openings in the end diaphragms as well as one solid diaphragm. Loading was applied with post-tensioning tendons to represent different design levels as well as to reach the ultimate load of the anchorage zone.

Finite element models were developed for the experimental specimens using the DIANA finite element package. These models were used to extend the investigated parameters affecting performance of general anchorage zone including: geometry of box girder end zone, edge eccentricity, number of anchors, tendon inclination and skew angle.
Results obtained from field monitoring of bridges, experimental work and finite element modeling were combined using the strut-and-tie methodology to develop a set of design equations for bursting forces of anchorage zone. A simplified design table was developed for limited range of anchorage zone parameters. A safety check was proposed based on the observed failure mode for the experimental specimens, where web axial force from the post-tensioning is limited to the axial compression capacity of the web.

10.2 Findings and Observations

Work done through anchorage zone research program developed several observations, which are classified into four categories. First, common observations obtained from the field monitoring, experimental work and/or finite element analysis are summarized as follows:

- Developed transverse strains on the inner face of the diaphragm always increase towards the web centerline. The maximum strain values occur at the web face, where cracks can occur at the smallest section. Low strain values occur in the middle of diaphragm between web girders.
- Developed transverse strains in the deck and soffit have a constant plateau of maximum strains that occurs at the web. Strain values always decrease from the web face.
- The vertical strain distribution on the inner face of the diaphragm and at the web is always maximum within the middle third of the height.
- Vertical web strains are inversely proportional with the distance from the diaphragm face. These strain values are significantly reduced at 50% of the girder height in the
longitudinal direction, which clarifies that the effective zone of the web is 50% of the box girder height along the bridge length.

- Transverse concrete strains in web girders are low and can be considered negligible.
- The strain levels of the bars ahead of the prestressing block-outs are significantly lower than those implemented in the deck beyond the diaphragm face.
- Bursting forces in different elements are inversely proportional with the effective diaphragm width to box girder height ratio.
- Diaphragm opening size decreases the transverse bursting forces in the diaphragm inner face and increases the transverse bursting forces in the deck and soffit.
- In case of a skew diaphragm, the obtuse side of the angle between the diaphragm and the web always possesses higher strain values than the acute side. The obtuse side has higher potential to cracking. The higher the skew angle, the larger difference in strains between obtuse and acute sides is developed.
- The distribution of vertical bursting forces on both acute and obtuse sides of the diaphragm are affected by the diaphragm skew angle. The vertical bursting force in the obtuse side increases with the diaphragm skew angle, however, the vertical bursting force in the acute side decreases.
- Tensile transverse spalling stresses along post-tensioning edge are very low and occur distant from the anchors. The spalling tensile stresses along the loaded face are totally resisted by tensile strength capacity of concrete in case of solid diaphragm.
- Strains developed in the deck bottom reinforcement layer are slightly higher than those developed in the top layer and strains developed in the soffit top reinforcement layer are slightly higher than those developed in the bottom layer.
Second, observations obtained based on field monitoring are as follows:

- Parameters affecting difference in strains between investigated bridges were level of prestressing force, ratio of diaphragm width to girder height, presence of opening in the diaphragm and concrete contribution in resistance based on strain level reached.

- Bursting forces and strains developed at interior girders are higher than those at exterior ones by at least 15%.

- The low values of the bursting forces in exterior girders is due to the thicker exterior web girder than interior ones for all investigated bridges. The in-plane horizontal curvature of exterior tendons cause transverse compression force in the diaphragm, deck and soffit. The compressive component of the applied prestressing force develops an inward force that decreases the effect of bursting spreading forces.

- Skew angle of the end diaphragm affects the dimensions of post-tensioning block-outs, which leads to reduction in the diaphragm width. Therefore, the net effective thickness of the diaphragm should be considered in design of the anchorage zone of box girder bridges.

- When stressing the tendons from both bridge ends, stressing the second end anchors causes slight increase in strain values for the first end. Increase in strain values were inversely proportional with prestressing friction losses.

- At the service stage, the total strain at a giving anchorage zone is caused by the stressing of the anchorages in that girder as well as the anchorages in other girders.
Three methods for distribution of jacking force between girders in the post-tensioned bridges are compared. Both the tributary area method and equivalent prestressing contribution ratio (EPCR) method are more realistic ways distribute the jacking force rather than equal distribution of forces between girders.

Third, observations obtained based on experimental work are as follows:

- Cracks are observed in experimental specimens at low loading levels, which indicates that most of the constructed bridges reached the cracking limit in the diaphragm inner face just due to prestressing.
- Crack widths in the diaphragm inner face and the web are inversely proportional with the diaphragm width.
- The crack widths in the diaphragm inner face are inversely proportional with the opening dimensions. The larger the opening, the higher the stress concentration at its corners. Crack patterns at the diaphragm opening sides were inclined starting from specimen centroid spreading towards the deck and the soffit of the specimens developing a punching cone.
- Cracks occurred in the web are always adjacent to the prestressing ducts, where reduced concrete section exists at the duct location.
- Existence of higher reinforcement in the diaphragm causes more distributed cracks with smaller width.
- Strain levels of the anchor spiral did not exceed 50% of the yield strain; however, the applied load exceeded the service limit by 20%. This illustrates the adequacy of the local anchorage zone reinforcement.
The maximum vertical bursting forces occurred at \( \frac{1}{4} \) of the girder spacing away from the web centerline, then a plateau of maximum strains developed at the middle between girders.

Fourth, observations based on finite element analysis parametric study are as follows:

- Diaphragm transverse bursting forces are directly proportional with girder spacing to box girder height ratio, deck and soffit thicknesses, tendon inclination and skew angle, while inversely proportional with web width.
- Deck and soffit transverse bursting forces are directly proportional with girder spacing to box girder height ratio, deck and soffit thicknesses and skew angle, while inversely proportional with web width.
- The transverse bursting force of the deck is inversely proportional with the eccentricity ratio and tendon inclination, however, in the soffit, it is directly proportional with the eccentricity ratio and tendon inclination.
- Diaphragm vertical bursting forces are directly proportional with girder spacing to box girder height ratio, deck and soffit thicknesses and tendon inclination, while inversely proportional with web width and number of anchors.
- Web vertical bursting forces are directly proportional with web width and tendon inclination, while inversely proportional with deck and soffit thicknesses and number of anchors.

### 10.3 Conclusions

The main conclusions that could be drawn from the previously reported: field monitoring, experimental work, finite element analysis and results implementation are as follows:
• Based on the developed database of box girder bridges, the diaphragm reinforcement selection varies substantially and seems to be based more on “rules of thumb” depending on a) typical bar diameters used in the end diaphragm, b) maximum spacing between bars and c) previous experience.

• The developed set of proposed equations provides adequate correlation with all of the investigated parameters including: diaphragm width to box girder height, tendon inclination angle, diaphragm openings, diaphragm skew angle, girder spacing to box girder height, deck and soffit thicknesses, web width, number of prestressing anchors and edge eccentricity.

• In cases where a diaphragm opening exists, the strains developed at specimen centerline between girders are low values. This indicated that the optimum location for diaphragm opening is at the middle of the diaphragm.

• Due to stress concentration observed at diaphragm opening corners, reinforcement that would be cut or removed because of the opening should be placed around the sides of the openings.

• Failure occurred in the general anchorage zone due to a compression failure in the web followed by punching failure of the diaphragm. Web compression failure was observed through buckling of longitudinal bars between vertical stirrups of the web and high longitudinal strain values of concrete. Punching failure in the diaphragm was assessed based on the developed punching cones, the observed punching perimeter and penetration of anchorage head plates into the concrete.

• Based on failure mode of anchorage zone in case of diaphragm opening, axial capacity of the web girder must be checked to sustain the whole jacking force.
Based on the transverse strain distribution in the diaphragm inner face, extending transverse reinforcement of the diaphragm inner face is a must to prevent any lap splices at web girders, where maximum transverse strains occur.

The grillage reinforcement around the anchorage device has a limited role in resisting compressive forces developed adjacent to anchors or any spalling stresses.

The simplified design method specifies fixed reinforcement percentages for different areas of the anchorage zone based on range of parameters; this method greatly simplifies the design for a restricted range of parameters within the anchorage zone. The most influence parameter is diaphragm width to box girder height ratio. The simplified design method is applicable for 90% of bridges in the developed database.

### 10.4 Suggestion for Future Work

The following topics are suggested for future research projects relevant to the subject of this investigation:

- Short term creep of concrete affecting anchorage zone failure to maintain safety factor during stressing.
- More experimental work needed to include concrete tensile strength in bursting force resistance.
- Implementing new advanced materials in the general anchorage zones to enhance the performance.
- Implement vertical and transverse prestressing for the diaphragm to eliminate cracking.
References


[17] American Concrete Institute (ACI), Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, Detroit, USA, 2014.


[38] California Department of Transportation (CALTRANS), "UTILITY OPENING BOX GIRDER B7-10," Sacramento, CA, 2009.


[48] MIDAS Information and Technology Co., Ltd., "MIDAS FX+ Version 3.3.0,"
Gyeonggi-do, South Korea., 2013.


Post-Tensioing Institute, 2006.
Appendix (A) Field Monitoring Instrumentation Plans

This appendix includes set of drawings for field instrumentation plans. It contains detailed drawings for the locations and labels of strain gauges. The list of instrumented bridges are:

- Bridge I - Willits Bypass Floodway Viaduct Frame 6
- Bridge II - Willits Bypass Floodway Viaduct Frame 8
- Bridge III - South Stockton Widening Martin Luther King Jr. Blvd. Overcrossing
- Bridge IV - South Stockton Widening Main Street Overcrossing – Replace
BRIDGE CROSS SECTION SHOWING SISTER BARS ON THE END DIAPHRAGM INNER FACE

DETAILS OF ADDED SISTER BARS
(SHOP DRAWINGS)

TYPICAL SECTION SHOWING LOCATION OF ADDED SISTER BARS
### TYPICAL SECTION SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

**LIST OF INSTRUMENTATIONS**

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<th>DECK</th>
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<td>W01/W02</td>
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<td>(PMFL-50-2LT)</td>
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<td>W01/W02</td>
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DECK PLAN SHOWING LOCATION OF ADDED SISTER BARS IN TOP MESH REINFORCEMENT

LONGITUDINAL CROSS SECTION OF INTERIOR GIRDER SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGES

LONGITUDINAL CROSS SECTION OF EXTERIOR GIRDER SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGES
BRIDGE CROSS SECTION SHOWING SISTER BARS ON THE END DIAPHRAGM INNER FACE

END DIAPHRAGM SECTION
TYPICAL SECTION SHOWING LOCATION OF ADDED SISTER BARS

Details of added sister bars
(Shop Drawings)
TYPICAL SECTION SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

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DECK PLAN SHOWING LOCATION OF ADDED SISTER BARS IN TOP MESH REINFORCEMENT

LONGITUDINAL CROSS SECTION OF INTERIOR GIRDER SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

LONGITUDINAL CROSS SECTION OF EXTERIOR GIRDER SHOWING LOCATION OF EMBEDDED IN CONCRETE STRAIN GAUGE
BRIDGE CROSS SECTION SHOWING SISTER BARS ON THE END DIAPHRAGM INNER FACE

DETAILS OF ADDED SISTER BARS

ADDED VERTICAL SISTER BARS

ADDED HORIZONTAL SISTER BARS

END DIAPHRAGM TYPICAL SECTION SHOWING LOCATION OF ADDED SISTER BARS
DECK PLAN SHOWING LOCATION OF ADDED SISTER BARS IN BOTTOM MESH REINFORCEMENT

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LONGITUDINAL CROSS SECTION OF EXTERIOR GIRDER SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE
BRIDGE CROSS SECTION SHOWING SISTER BARS ON THE END DIAPHRAGM INTERIOR SIDE

TYPICAL SECTION SHOWING LOCATION OF INSTALLED SISTER BARS

DETAILS OF ADDED SISTER BARS

3 X BAR #3 TYPE B
12 X BAR #5 TYPE A
2 X BAR #3 TYPE A
2 X BAR #3 TYPE A
4 X BAR #5 TYPE A
8 X BAR #3 TYPE A
8 X BAR #3 TYPE A

PROJECT:
Anchorage Zone Reinforcing of Tensioned Box Girder Bridges

FIELD INSTRUMENTATION DRAWINGS

SOUTH STOCKTON WIDENING
MAIN STREET OC (REPLACE) ABUTMENT (1) (1/3)

DATE: FEB 2, 2010
SHEET: 1 OF 1
SCALE: 1/80
PREPARED BY: [Signature]
APPROVED BY: [Signature]

University of Nevada, Reno
TYPICAL SECTION SHOWING LOCATION OF INSTALLED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

LONGITUDINAL CROSS SECTION OF EXTERIOR GIRDER (A) SHOWING LOCATION OF INSTALLED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

LONGITUDINAL CROSS SECTION OF INTERIOR GIRDER (B) SHOWING LOCATION OF ADDED SISTER BARS AND EMBEDDED IN CONCRETE STRAIN GAUGE

LONGITUDINAL CROSS SECTION OF INTERIOR GIRDER (C) SHOWING LOCATION OF ADDED SISTER BARS

LONGITUDINAL CROSS SECTION OF INTERIOR GIRDER (D) SHOWING LOCATION OF ADDED SISTER BARS
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Appendix (B) Experimental Specimen Structural Design

This appendix contains the structural design for experimental specimen including checking service stresses acting at mid-span of the specimen during testing. The specimen was designed for ultimate shear and flexure to maintain safety during the whole experimental test. It was also designed according to Caltrans Memo to Designer (11-25) - Anchorage Zone Design in order to make sure it satisfies Caltrans requirements for anchorage zones.

B.1 Design Criteria

Design criteria includes used design codes as well as material properties

Design codes:

- AASHTO LRFD Bridge Design Specifications - Sixth Edition.
- Caltrans Anchorage Zone Design - Memo to Designers 11-25.
- ACI-318, Building Code Requirements for Structural Concrete.

Material properties:

- Reinforcement $f_y = 60 \text{ ksi}$
- Concrete $f_c = 5 \text{ ksi}$

B.2 Designed Section

The cross section of test specimen at mid-span is illustrated in Figure B-1. The design concept was conducted to assure that failure will occur at end diaphragm due to prestressing.
Figure B-1: Reinforced cross section of test specimen at mid-span

Section properties

\[ Y_{c.g.} = 20.872 \text{ in} \quad \text{(Distance from centroid to the base)} \]

\[ A_g = 731.5 \text{ in}^2 \quad \text{(Section gross area)} \]

\[ I_x = 188,163.5 \text{ in}^4 \quad \text{(Moment of inertia about X-axis)} \]

B.3 Loading Patterns

The testing protocol included the following load cases:

1. Specimen own weight \((M_{o.w.} = 231.2 \text{ kips.in} \text{ and } V_{o.w.} = 5.44 \text{ kips})\)

2. External loads to compensate the effect of full scale bridge own weight (Two concentrated loads each 52.8 kips spaced by 48 in, \(M_{\text{ext}} = 3214.7 \text{ kips.in} \text{ and } V_{\text{ext}} = 52.7 \text{ kips})\)

3. Prestressing load \((P) = 1793 \text{ kips}, e = 3.3 \text{ in} \text{ and } V_{\text{eq. prestressing}} = 139.2 \text{ kips})\)
B.4 Check Service Stresses

In case of Own weight + External loads + Prestressing

$$F_{top} = - \frac{P_j}{A_g} + \frac{P_j \times e \times (h - Y_{c.g.})}{I_x} - \frac{(M_{o.w.} + M_{ext.}) \times (h - Y_{c.g.})}{I_x}$$

$$F_{top} = - \left[ \frac{1793}{731.5} \right] + \left[ \frac{1793 \times 3.3 \times (40.5 - 20.872)}{188,163.5} \right]$$

$$F_{top} = - \left[ (231.2 + 3241.7) \times (40.5 - 20.872) / 188,163.5 \right]$$

$$F_{top} = - 1.47 \text{ ksi} \quad < 0.6 \cdot f'_{c} = 3 \text{ ksi} \quad \text{OK}$$

$$F_{bottom} = - \frac{P_j}{A_g} - \frac{P_j \times e \times Y_{c.g.}}{I_x} - \frac{(M_{o.w.} + M_{ext.}) \times Y_{c.g.}}{I_x}$$

$$F_{bottom} = - \left[ \frac{1793}{731.5} \right] - \left[ \frac{1793 \times 3.3 \times 20.872}{188,163.5} \right]$$

$$F_{bottom} = - \left[ (231.2 + 3241.7) \times 20.872 / 188,163.5 \right]$$

$$F_{bottom} = - 2.72 \text{ ksi} \quad < 0.6 \cdot f'_{c} = 3 \text{ ksi} \quad \text{OK}$$

In case of Prestressing only (Ignoring own weight and external loads):

$$F_{top} = - \frac{P_j}{A_g} + \frac{P_j \times e \times (h - Y_{c.g.})}{I_x}$$

$$F_{top} = - \left[ \frac{1793}{731.5} \right] + \left[ \frac{1793 \times 3.3 \times (40.5 - 20.872)}{188,163.5} \right]$$

$$F_{top} = - 1.83 \text{ ksi} \quad < 0.6 \cdot f'_{c} = 3 \text{ ksi} \quad \text{OK}$$

$$F_{bottom} = - \frac{P_j}{A_g} - \frac{P_j \times e \times Y_{c.g.}}{I_x}$$

$$F_{bottom} = - \left[ \frac{1793}{731.5} \right] - \left[ \frac{1793 \times 3.3 \times 20.872}{188,163.5} \right]$$
Checking service stresses showed that mid-span section will be safe during all cases of loading.

**B.5 Flexural Ultimate Capacity**

_in case of Own weight + External loads_

Ultimate capacity as non-prestressed section

- Reinforcement at bottom (tension side)
  \[ A_{S1} = 11\#4 = 2.2 \text{ in}^2 \text{ (top longitudinal layer in the soffit)} \]
  \[ A_{S2} = 4\#3 = 0.44 \text{ in}^2 \text{ (bottom longitudinal layer in the soffit)} \]
  \[ f_c' = 5 \text{ ksi} \rightarrow \beta = 0.8 \]

- Equilibrium equation
  \[ C = T \]
  \[ 0.85 \times f_c' \times \beta \times c \times b_f = A_s \times f_y \]
  \[ 0.85 \times 5 \times 0.8 \times c \times 63 = (2.2 + 0.44) \times 60 \]
  \[ c = 0.739 \text{ in} < \text{Deck thickness} \quad \text{OK} \]
  \[ a = \beta \times c = 0.592 \text{ in} \]

- Reinforcement strain = \(0.003 \times (d - c) / c\)
  \[ \text{Reinforcement strain} = 0.003 \times (37.5 - 0.739) / 0.739 \]
  \[ \text{Reinforcement strain} = 0.1492 > [f_y/E_s =0.00207] \quad \text{OK} \]

- \(M_{ult} = 2.2 \times 60 \times [37.5-(0.592/2)] + 0.44 \times 60 \times [39-(0.592/2)]\)
\[ M_{\text{ult}} = 5932.71 \text{ kips.in} > [(M_{\text{o.w.}} + M_{\text{ext.}}) = 3445.9 \text{ kips.in}] \text{ OK} \]

(Factor of Safety \text{F.S.} = 1.72)

**In case of Own weight + External loads + Prestressing**

Ultimate capacity as non-prestressed section (More critical)

- **Reinforcement at top** (tension side)
  \[ A_{S1} = 12 \#4 = 2.4 \text{ in}^2 \text{ (top longitudinal layer in the deck)} \]
  \[ A_{S2} = 10 \#3 = 1.1 \text{ in}^2 \text{ (bottom longitudinal layer in the deck)} \]
  \[ f'_c = 5 \text{ ksi} \rightarrow \beta = 0.8 \]

- **Equilibrium equation**
  \[ C = T \]
  \[ 0.85 \times f'_c \times \beta \times c \times b_t = A_s \times f_y \]
  \[ 0.85 \times 5 \times 0.8 \times c \times 63 = (2.4+1.1) \times 60 \]
  \[ c = 0.98 \text{ in} < \text{Soffit thickness} \text{ OK} \]
  \[ a = \beta \times c = 0.784 \text{ in} \]

- **Reinforcement strain**
  \[ = 0.003 \times (d - c) / c \]
  \[ = 0.003 \times (37.5 - 0.98) / 0.98 \]
  \[ = 0.112 > [f_y/E_s = 0.00207] \text{ OK} \]

- **\[ M_{\text{ult}} = 2.4 \times 60 \times [37.5-(0.784/2)] + 1.1 \times 60 \times [39-(0.784/2)] \]**
  \[ M_{\text{ult}} = 7891.7 \text{ kips.in} > [P] \times e - (M_{\text{o.w.}} + M_{\text{ext.}}) = 2471 \text{ kips.in}] \text{ OK (F.S. = 3.19)} \]
B.6 Ultimate Shear Capacity

**In case of Own weight + External loads**

\[ V_c = 2 \cdot f_{c'}^{1/2} \cdot b_w \cdot d \]

\[ V_c = 2 \cdot (5000)^{1/2} \times 7 \times 39 = 38.6 \text{ kips} \]

\[ V_s = A_v \cdot f_y \cdot d / s \]

\[ V_s = 2 \times 0.11 \times 60 \times 39 / 6 = 85.8 \text{ kips} < 8 \cdot f_{c'}^{1/2} \cdot b_w \cdot d \text{ OK} \]

\[ V_n = V_c + V_s = 124.4 \text{ kips} > [(V_{o.w.} + V_{ext.}) = (5.44 + 52.7) = 58.14 \text{ kips}] \text{ OK} \]

(F.S. = 2.14)

**In case of Own weight + External loads + Prestressing**

\[ V_c = 2 \cdot f_{c'}^{1/2} \cdot b_w \cdot d \]

\[ V_c = 2 \cdot (5000)^{1/2} \times 7 \times 39 = 38.6 \text{ kips} \]

\[ V_s = A_v \cdot f_y \cdot d / s \]

\[ V_s = 2 \times 0.11 \times 60 \times 39 / 6 = 85.8 \text{ kips} << 8 \cdot f_{c'}^{1/2} \cdot b_w \cdot d \text{ OK} \]

\[ V_n = V_c + V_s = 124.4 \text{ kips} > [(V_{eq. \text{ prestressing}} - V_{o.w.} - V_{ext.}) = (139.2 - 5.44 - 52.7) = 81.06 \text{ kips}] \text{ OK} \]

(F.S. = 1.53)
B.7 Check Caltrans Requirements - Memo to Designers (11-25)

The calculation for the Memo to Designer (11-25) was conducted using full scale prototype dimensions shown in Figure B-2, then the developed reinforcement ratios were maintained for the half scaled specimen.

![Figure B-2: Prototype dimensions of interior and exterior girders representing average values extracted from bridge database.](image)

End diaphragm width

\[ T_d > 0.3 \times h \]

Minimum diaphragm width used will be 0.4 \( h \)

Girder stem reinforcement

\[ A_{s1} = 1.33P \times [h-(P/1200)]/(300\times h^2) \quad \text{[Placed within the first h/2]} \]

\[ A_{s2} = 0.67P \times [h-(P/1200)]/(300\times h^2) \quad \text{[Placed within the last h/2]} \]

\[ P = 0.2 \bar{f}_c A_g = 0.2 \times 5 \times 2796 = 2796 \text{ kips} \quad \text{(Average value from bridge database)} \]

\[ h = \frac{81}{12} = 6.75 \text{ ft} \]
\( A_{s1} = 1.2 \text{ in}^2/\text{ft} \)

Reinforcement ratio = \( A_{s1} / (b_w \times 1.0 \text{ ft}) = 1.2 / (13.4 \times 12) = 0.00746 \)

> Existing stirrups #3 @ 6” = 0.22 / (7 \times 6) = 0.00524 \hspace{1cm} \text{NOT OK}

Use #4 @ 6” for the first 3 stirrups with reinforcement ratio = 0.00952

\( A_{s2} = 0.6 \text{ in}^2/\text{ft} \)

Reinforcement ratio = \( 0.6 / (13.4 \times 12) = 0.00373 \)

< Existing stirrups #3 @ 6” = 0.22 / (7 \times 6) = 0.00524 \hspace{1cm} \text{OK}

**Girder stem thickness**

\[
(t_w)_{\text{Required}} = \frac{1000P}{[(P/1200 - 1) \times 18 + 3(T_d - 12)] \times 0.7f'_c}
\]

\( P = 0.2 f'_c A_g = 2796 \text{ kips (Average value from bridge database)} \)

\( T_d = 0.4 \times h = 0.4 \times 81 = 32.5 \text{ in} \)

\( \phi = 0.7 \) (Resistance factor for compression)

\( (t_w)_{\text{Required}} = 13.4 \text{ in} \)

\( t_w \text{ scaled} = 13.4 \times 0.5 = 6.7 \text{ in} < \hspace{1cm} \text{used web thickness} = 7.0 \text{ in} \hspace{1cm} \text{OK} \)
B.8 Specimen Design Level Based on Actual Compressive Strength

End anchorage design level was calculated based on the actual compressive strength of experimental specimens. The concrete compressive strength at test day was 6.49 ksi for single girder specimens and 5.87 ksi for double girder specimens, which was implemented in capacity calculations. The governing limit for anchorage zone capacity was Memo to Designer (11-25).

Based on end diaphragm width

\[ T_d > 0.3 \times h \] (Satisfied) → Not governing in the anchorage capacity.

Based on girder stem reinforcement

Used #4 @6” for the first 3 stirrups with reinforcement ratio = 0.00952

\[ A_{s1} = 1.33P \times \frac{[h-(P/1200)]}{(300 \times h^2)} \] [Placed within the first h/2]

P = 7200 kips (Prototype)

P = 1800 kips (1/2 Scaled) → (1)

Used #3 @ 6” with reinforcement ratio = 0.00524

\[ A_{s2} = 0.67P \times \frac{[h-(P/1200)]}{(300 \times h^2)} \] [Placed within the last h/2]

P = 7200 kips (Prototype)

P = 1800 kips (1/2 Scaled) → (2)
**Based on girder stem thickness**

$t_w = 14$ in

$f'c = 6.49$ ksi \hspace{1cm} (Single girder specimens)

$f'c = 5.87$ ksi \hspace{1cm} (Double girder specimens)

$T_d = 0.4 \times h = 0.4 \times 81 = 32.5$ in

$\phi = 0.7$ (Resistance factor for compression)

\[(t_w)\text{Required} = \frac{1000P}{[(P/1200 - 1) \times 18 + 3(T_d - 12)] \times 0.7f'c}\]

$P = 5400$ kips (Prototype) $\rightarrow$ (Single girder specimens)

$P = 1350$ kips (1/2 Scaled) \hspace{1cm} \rightarrow (3)$

$P = 4420$ kips (Prototype) $\rightarrow$ (Double girder specimens)

$P = 1075$ kips (1/2 Scaled) \hspace{1cm} \rightarrow (4)$

From (1), (2), (3) and (4)

Anchorage zone design capacity of experimental specimens is 1350 kips for single girder specimens and 1075 kips for double girder specimens
Appendix (C) Construction Drawings and Instrumentation Plans for Experimental Specimens

This appendix includes set of construction drawings for experimental specimens as well as instrumentation plans. It contains concrete dimensions and reinforcement details for the experimental specimens. Locations and labels for implemented strain gauges and displacement gauges are shown in instrumentation plans. The drawing set includes:

- Construction drawings for single girder specimens S45 and S56
- Instrumentation plans for single girder specimens S45 and S56
- Construction drawings for double girder specimens DS6 and D55
- Instrumentation plans for double girder specimens DS6 and D55
Anchorage Zone Reinforcing of Post-Tensioned Box Girder Bridges Phase (I) Single Girder Specimens

GENERAL NOTES
- ALL CONSTRUCTION SHALL BE DONE IN ACCORDANCE WITH ACI-318 AND ACI-301.
- ALL DIMENSIONS ARE IN INCHES.
- FOR ANY MISSING DATA OR DIMENSIONS REFER TO THE DESIGNER.
- CONCRETE SHALL HAVE MINIMUM COMpressive STRENGTH $f'c$ AT 28 DAYS OF 5000 Psi.
- STEEL REINFORCING BARS SHALL BE GRADE 60 ACCORDING TO ASTM SPECS WITH MINIMUM PROOF STRENGTH OF 60,0000 Psi.
- CONTRACTOR IS RESPONSIBLE TO PROVIDE THE EXACT DIMENSIONS OF ELEMENT WITH MAXIMUM TOLERANCE OF ±1/8".
- MINIMUM CLEAR COVER SHALL BE 0.5" FOR ALL REINFORCEMENT EXCEPT NOTED OTHERWISE.
CABLE PROFILE

DISTANCE  0   27   47   67   87  107  127  147  174
DUCT (A)  33 7/8"  28 7/16  26 1/2  25 3/8  25  25 3/8  26 1/2  28 7/16  33 7/8"
DUCT (B)  20 7/8"  17 3/16  16 1/8  15 1/2  15 5/16  15 1/2  16 1/8  17 3/16  20 7/8"
DUCT (C)  7 7/8"   6 5/16  6 5/16  6 5/16  6 5/16  6 5/16  6 5/16  6 5/16  3 5/16  7 7/8"

ALL ELEVATIONS ARE MEASURED IN (in) FROM BOTTOM OF DUCT TO FORM WORK (*) MEASURED TO C.G. OF ANCHOR
DISTANCE | 0 | 27 | 47 | 67 | 87 | 107 | 127 | 147 | 174
DUCT (A) | 33 7/8" | 28 7/16 | 26 1/2 | 25 3/8 | 25 | 25 3/8 | 26 1/2 | 28 7/16 | 33 7/8"
DUCT (B) | 20 7/8" | 17 3/16 | 16 1/8 | 15 1/2 | 15 5/16 | 15 1/2 | 16 1/8 | 17 3/16 | 20 7/8"
DUCT (C) | 7 7/8" | 6 5/16 | 6 5/16 | 6 5/16 | 6 5/16 | 6 5/16 | 6 5/16 | 6 5/16 | 3 5/16 | 7 7/8"

ALL ELEVATIONS ARE MEASURED IN (in) FROM BOTTOM OF DUCT TO FORM WORK (*) MEASURED TO C.G. OF ANCHOR

CABLE PROFILE

SECTION (B-B)

SECTION (C-C)
LONGITUDINAL SECTION AT BLOCK-OUTS

(TN) INDICATES STRAIN GAUGES ATTACHED TO PRESTRESSING STRANDS

EXTERIOR FACE OF DIAPHRAGM (S04)

EXTERIOR FACE OF DIAPHRAGM (S05L)
PLAN FOR THE DECK (BOTTOM REINFORCEMENT LAYER)
(*) INDICATES STRAIN GAUGES ATTACHED TO REINFORCING BARS PLACED IN TOP LAYER

PLAN FOR THE SOFFIT (TOP REINFORCEMENT LAYER)
(*) INDICATES STRAIN GAUGES ATTACHED TO REINFORCING BARS PLACED IN BOTTOM LAYER
Anchorage Zone Reinforcing of Post-Tensioned Box Girder Bridges Phase (II) Double Girder Specimens

GENERAL NOTES
- ALL CONSTRUCTION SHALL BE DONE IN ACCORDANCE WITH ACI-318 AND ACI-301.
- ALL DIMENSIONS ARE IN INCHES.
- FOR ANY MISSING DATA OR DIMENSIONS REFER TO THE DESIGNER.
- CONCRETE SHALL HAVE MINIMUM COMpressive STRENGTH $f_c$ AT 28 DAYS OF 4000 Psi.
- STEEL REINFORCING BARS SHALL BE GRADE 60 ACCORDING TO ASTM SPECS WITH MINIMUM PROOF STRENGTH OF 60,000 Psi.
- CONTRACTOR IS RESPONSIBLE TO PROVIDE THE EXACT DIMENSIONS OF ELEMENT WITH MAXIMUM TOLERANCE OF $\pm 1/8"$
- MINIMUM CLEAR COVER SHALL BE 0.5" FOR ALL REINFORCEMENT EXCEPT NOTED OTHERWISE.
<table>
<thead>
<tr>
<th>DISTANCE</th>
<th>0</th>
<th>27</th>
<th>47</th>
<th>67</th>
<th>87</th>
<th>107</th>
<th>127</th>
<th>147</th>
<th>174</th>
</tr>
</thead>
<tbody>
<tr>
<td>DUCT (A)</td>
<td>33 7/8&quot;</td>
<td>28 7/16</td>
<td>26 1/2</td>
<td>25 3/8</td>
<td>25 3/8</td>
<td>26 1/2</td>
<td>28 7/16</td>
<td>33 7/8&quot;</td>
<td></td>
</tr>
<tr>
<td>DUCT (B)</td>
<td>20 7/8&quot;</td>
<td>17 3/16</td>
<td>16 1/8</td>
<td>15 1/2</td>
<td>15 5/16</td>
<td>15 1/2</td>
<td>16 1/8</td>
<td>17 3/16</td>
<td>20 7/8&quot;</td>
</tr>
<tr>
<td>DUCT (C)</td>
<td>7 7/8&quot;</td>
<td>6 5/16</td>
<td>6 5/16</td>
<td>6 5/16</td>
<td>6 5/16</td>
<td>6 5/16</td>
<td>6 5/16</td>
<td>3 5/16</td>
<td>7 7/8&quot;</td>
</tr>
</tbody>
</table>

ALL ELEVATIONS ARE MEASURED IN (in) FROM BOTTOM OF DUCT TO FORM WORK (*) MEASURED TO C.G. OF ANCHOR
PLAN FOR THE DECK (BOTTOM REINFORCEMENT LAYER)
<table>
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<tr>
<th>DISTANCE</th>
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<th>27</th>
<th>47</th>
<th>67</th>
<th>87</th>
<th>107</th>
<th>127</th>
<th>147</th>
<th>174</th>
</tr>
</thead>
<tbody>
<tr>
<td>DUCT (A)</td>
<td>33</td>
<td>28</td>
<td>26</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>26</td>
<td>28</td>
<td>33</td>
</tr>
<tr>
<td>DUCT (B)</td>
<td>20</td>
<td>17</td>
<td>16</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>16</td>
<td>17</td>
<td>20</td>
</tr>
<tr>
<td>DUCT (C)</td>
<td>7</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>6.5</td>
<td>7</td>
</tr>
</tbody>
</table>

ALL ELEVATIONS ARE MEASURED IN (in) FROM BOTTOM OF DUCT TO FORM WORK
(*) MEASURED TO C.G. OF ANCHOR

CABLE PROFILE

SECTION (C-C)
LONGITUDINAL SECTION

LONGITUDINAL SECTION AT BLOCK-OUTS
PLAN FOR THE DECK (BOTTOM REINFORCEMENT LAYER)
INTERIOR FACE OF DIAPHRAGM (D6T)

EXTERIOR FACE OF DIAPHRAGM (D6T)
PLAN FOR THE DECK (BOTTOM REINFORCEMENT LAYER)

(*) INDICATES STRAIN GAUGES ATTACHED TO REINFORCING BARS PLACED IN TOP LAYER

****** LOCATION OF WEB REINFORCEMENT GAUGES
PLAN FOR THE SOFFIT (TOP REINFORCEMENT LAYER)

(*) INDICATES STRAIN GAUGES ATTACHED TO REINFORCING BARS PLACED IN BOTTOM LAYER
******** LOCATION OF WEB REINFORCEMENT GAUGES
LONGITUDINAL SECTION AT BLOCK-OUTS

INTERIOR FACE OF DIAPHRAGM (D5T)

EXTERIOR FACE OF DIAPHRAGM (D5T)

NOTES:
- (T4) INDICATES STRAIN GAUGES ATTACHED TO PRESTRESSING STRANDS
- GAUGES D5T-M01 TO D5T-M08 ARE PLACED ON INTERIOR FACE OF GIRDER (A) (GIRLAGE SIDE)
- GAUGES D5T-M09 TO D5T-M12 ARE PLACED ON EXTERIOR FACE OF GIRDER (B) (WITHOUT GIRLAGE)
- GAUGES D5T-M01 TO D5T-M08 ARE PLACED VERTICALLY IN GIRDER (B) (WITHOUT GIRLAGE)
- GAUGES D5T-M09 TO D5T-M12 ARE PLACED LATERALLY IN GIRDER (A) (GIRLAGE SIDE)

REINFORCING BARS STRAIN GAUGES (1/2"x1/16")
EMBEDDED IN CONCRETE STRAIN GAUGES (1/8"x1/8")

PROJECT:
Anchorage Zone Reinforcing of Post-Tensioned Box Girder Bridges

EXPERIMENTAL SPECIMENS
SPECIMEN (4) - D55 INSTRUMENTATION
(1/5)

DATE: OCTOBER 15, 2016
SCALE: 1:25
PREPARED BY: Ahmed Memari
APPROVED BY: Dr. David Sanders

University of Nevada, Reno
PLAN FOR THE DECK (BOTTOM REINFORCEMENT LAYER)

(*) INDICATES STRAIN GAUGES ATTACHED TO REINFORCING BARS PLACED IN TOP LAYER

********** LOCATION OF WEB REINFORCEMENT GAUGES
Appendix (D) Materials Testing for Experimental Specimens

This appendix contains the test results for concrete and reinforcing bars used in experimental specimens. All performed testing was done according to the following ASTM specifications:

- ASTM C39/C39M – 16, “Compressive Strength of Cylindrical Concrete Specimens”.
- ASTM C496/C496M – 11, “Splitting Tensile Strength of Cylindrical Concrete Specimens”.
- ASTM C78/C78M − 15b, “Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)”.
- ASTM C469/C469M − 14, “Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression”.

D.1 Single Girder Specimens S45 and S56

Concrete compressive strength

Compressive test was done for test cylinder as shown in Figure D-1 based on ASTM C39/C39M – 16. The compressive test results are summarized in Table D-1. It includes the test results after 7, 14 and 34 days from concrete pouring. In order to study the effect of concrete age on compressive strength, the relationship is illustrated in Figure D-2. Average compressive strength at test day was 6.49 ksi.
Figure D-1: Compressive strength test setup and specimens failure.

Table D-1 Compressive test results for single girder specimens.

<table>
<thead>
<tr>
<th>Days after pouring</th>
<th>Compressive strength (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>4.87 4.75 4.86</td>
<td>4.83</td>
</tr>
<tr>
<td>14</td>
<td>5.46 5.78 5.64</td>
<td>5.63</td>
</tr>
<tr>
<td>34*</td>
<td>6.66 6.58 6.23</td>
<td>6.49</td>
</tr>
</tbody>
</table>

(*) Date of experimental testing done for single girder specimens
Concrete tensile strength

Tensile strength was calculated based on splitting tensile strength of cylindrical concrete specimens according to ASTM C496/C496M – 11 and modulus of rupture according to ASTM C78/C78M – 15b as shown in Figure D-3. The test results for both splitting tensile strength and modulus of rupture are summarized in Tables D-2 and D-3, respectively. A comparison between the results is performed as shown in Figure D-4. This comparison illustrates that the splitting tensile strength is higher than modulus of rupture results. The considered value in the finite element models is the splitting tensile strength.
Table D-2 Splitting tensile strength for single girder specimens.

<table>
<thead>
<tr>
<th>Days after pouring</th>
<th>Splitting tensile strength (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>0.465</td>
<td>0.489</td>
</tr>
<tr>
<td>14</td>
<td>0.496</td>
<td>0.522</td>
</tr>
<tr>
<td>34*</td>
<td>0.567</td>
<td>0.588</td>
</tr>
</tbody>
</table>

Table D-3 Modulus of rupture for single girder specimens.

<table>
<thead>
<tr>
<th>Days after pouring</th>
<th>Modulus of rupture (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>0.283</td>
<td>0.303</td>
</tr>
<tr>
<td>14</td>
<td>0.261</td>
<td>0.311</td>
</tr>
<tr>
<td>34</td>
<td>0.317</td>
<td>0.329</td>
</tr>
</tbody>
</table>
Concrete modulus of elasticity

Concrete modulus of elasticity was calculated according to ASTM C469/C469M – 14. Test cylinders were capped with sulphur and prepared for testing as shown in Figure D-5. The test results of experimental specimens on test day are summarized in Table D-4.

Figure D-5: Test setup and specimen for concrete modulus of elasticity
Table D-4 Modulus of elasticity for single girder specimens.

<table>
<thead>
<tr>
<th>Modulus of elasticity (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>2984.58</td>
<td>2866.24</td>
</tr>
<tr>
<td>3092.91</td>
<td>2981.25</td>
</tr>
</tbody>
</table>

Tensile strength and elastic modulus of reinforcing bars

Tensile strength test was done for reinforcing bars used in single girder specimens according to ASTM A370 – 17. The develop stress-strain curves for #3 and #4 rebars used in single girder specimens are shown in Figure D-6. The test results are summarized in Table D-5 including yield strength of reinforcing bars as well as elastic modulus.

![Stress-strain curve for reinforcing bars of single girder specimens.](image)

Figure D-6: Stress-strain curve for reinforcing bars of single girder specimens.
Table D-5 Reinforcement tensile strength test results for single girder specimens.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Bar Size</th>
<th>Results (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength</td>
<td>#3</td>
<td>64.75</td>
<td>64.68</td>
</tr>
<tr>
<td></td>
<td>#4</td>
<td>66.36</td>
<td>66.23</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>#3</td>
<td>29411.82</td>
<td>29398.15</td>
</tr>
<tr>
<td></td>
<td>#4</td>
<td>29893.64</td>
<td>29914.55</td>
</tr>
</tbody>
</table>

D.2 Double Specimens DST and D55

Concrete compressive strength

Two concrete pours were used in casting the double girder specimens. The first one was used for the soffit, web and diaphragms of the double girder specimens. The second concrete pour was used for the deck. Compressive test was done for test cylinder based on ASTM C39/C39M – 16. The compressive test results are summarized in Tables D-6 and D-7 for the first and second concrete pours, respectively. It includes the test results after 7, 14, 28 and 91 days from concrete pouring for the first pour and 7, 14, 28 and 55 for the second pour. In order to study the effect of concrete age on compressive strength, the relationship is illustrated in Figure D-7. Average compressive strength at test day was 5.87 ksi for the first pour and 5.79 for the second pour.
Table D-6 Compressive test results of first concrete pour for double girder specimens.

<table>
<thead>
<tr>
<th>Days after pouring</th>
<th>Compressive strength (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>2.15</td>
<td>2.21</td>
</tr>
<tr>
<td>14</td>
<td>3.60</td>
<td>3.68</td>
</tr>
<tr>
<td>28</td>
<td>4.18</td>
<td>4.29</td>
</tr>
<tr>
<td>91*</td>
<td>5.63</td>
<td>5.94</td>
</tr>
</tbody>
</table>

(*) Date of experimental testing done for double girder specimens

Table D-7 Compressive test results of second concrete pour for double girder specimens.

<table>
<thead>
<tr>
<th>Days after pouring</th>
<th>Compressive strength (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>3.55</td>
<td>3.51</td>
</tr>
<tr>
<td>14</td>
<td>4.10</td>
<td>3.98</td>
</tr>
<tr>
<td>28</td>
<td>5.14</td>
<td>5.05</td>
</tr>
<tr>
<td>55*</td>
<td>5.70</td>
<td>5.74</td>
</tr>
</tbody>
</table>

(*) Date of experimental testing done for double girder specimens

Figure D-7: Effect of concrete age on compressive strength for double girder specimens.
**Concrete tensile strength**

Tensile strength was calculated based on splitting tensile strength of cylindrical concrete specimens according to ASTM C496/C496M – 11. The test results for splitting tensile strength of both concrete pours are summarized in Table D-8.

Table D-8 Splitting tensile strength for double girder specimens.

<table>
<thead>
<tr>
<th>Concrete Pour</th>
<th>Days after pouring</th>
<th>Splitting tensile strength (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour I</td>
<td>28</td>
<td>0.396</td>
<td>0.391</td>
</tr>
<tr>
<td></td>
<td>91</td>
<td>0.580</td>
<td>0.559</td>
</tr>
<tr>
<td>Pour II</td>
<td>28</td>
<td>0.275</td>
<td>0.272</td>
</tr>
<tr>
<td></td>
<td>55</td>
<td>0.378</td>
<td>0.389</td>
</tr>
</tbody>
</table>

**Concrete modulus of elasticity**

Concrete modulus of elasticity was calculated according to ASTM C469/C469M – 14. The test results of experimental specimens on test day are summarized in Table D-9.

Table D-9 Modulus of elasticity for double girder specimens.

<table>
<thead>
<tr>
<th>Concrete Pour</th>
<th>Modulus of elasticity (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour I</td>
<td>3967.45</td>
<td>3958.35</td>
</tr>
<tr>
<td>Pour II</td>
<td>3600.32</td>
<td>3424.85</td>
</tr>
</tbody>
</table>

**Tensile strength and elastic modulus of reinforcing bars**

Tensile strength test was done for reinforcing bars used in double girder specimens according to ASTM A370 – 17. The develop stress-strain curves for #3 and #4 rebars used
in double girder specimens is shown in Figure D-6. The test results are summarized in Table D-10 including yield strength of reinforcing bars as well as elastic modulus.

![Figure D-8: Stress-strain curve for reinforcing bars of double girder specimens.](image)

Table D-10 Reinforcement tensile strength test results for double girder specimens.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Bar Size</th>
<th>Results (ksi)</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td></td>
</tr>
<tr>
<td><strong>Yield Strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#3</td>
<td>65.21</td>
<td>64.64</td>
<td>64.68</td>
</tr>
<tr>
<td>#4</td>
<td>70.09</td>
<td>70.18</td>
<td>70.01</td>
</tr>
<tr>
<td><strong>Elastic Modulus</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#3</td>
<td>29611.59</td>
<td>29389.31</td>
<td>29400.25</td>
</tr>
<tr>
<td>#4</td>
<td>29863.88</td>
<td>29611.15</td>
<td>29776.16</td>
</tr>
</tbody>
</table>
Appendix (E) Design Example

Example (E-1)

Given:
Box girder height \( h = 7' - 3'' = 87'' \)
Girder spacing \( S = 10' - 3'' = 123'' \)
Deck thickness \( t_d = 8.5'' \)
Soffit thickness \( t_s = 7'' \)
Web width \( b_w = 12'' \)
Effective diaphragm width \( b_{de} = 39'' \)
Diaphragm skew angle \( \alpha_{skew} = 0 \)
Tendon inclination angle \( \alpha_{inc.} = 5^\circ \)
Edge eccentricity \( e_{edge} = 0 \)
\( P_{jack} \) per girder = 2210 kips (stressing is performed from one end only)
Web stirrups due to other loads: #5 @ 6"
Deck transverse reinforcement: #5 @ 10" top and bottom
Soffit transverse reinforcement: #4 @ 12" top and bottom
\( f'_{ci} = 4000 \) psi
Steel Grade 60
Method I: Using Proposed Design Equations

Step 1: Ultimate design force for anchorage zone per girder

\[ \beta_{\text{end}} = 1.0 \quad \text{as stressing is performed from one end only} \]

\[ P_{\text{du}} = 1.2 \times \beta_{\text{end}} \times P_{\text{per girder}} \]

\[ P_{\text{du}} = 1.2 \times 1.0 \times 2210 = 2652 \text{ kips} \]

Step 2: Stress distribution along the section at \((h/4 + S/2)/2\) from the diaphragm inner face

Section Properties

![Cross-section of a typical bridge girder.](image)

\[ A = 2764.5 \text{ in}^2 \quad \text{(Area of the section)} \]

\[ Y_{\text{bottom}} = 45.65'' \quad \text{(Distance from section centroid to the bottom)} \]

\[ Y_{\text{top}} = 41.35'' \quad \text{(Distance from section centroid to the top)} \]

\[ I_{\text{x-x}} = 3351261 \text{ in}^4 \quad \text{(Section moment of inertia about X-axis)} \]
Based on section properties $Y_{\text{bottom}}$, $Y_{\text{top}}$, $A$, $I_{x-x}$, tendon inclination angle $\alpha_{\text{inc.}} = 5^\circ$ and ultimate design force $P_{\text{du}}$, the axial stresses acting on the section located at $(h/4 + S/2)/2$ from the diaphragm inner face, see Figure E-2, are calculated as follows:

$f_1 = -0.844$ ksi

$f_2 = -0.868$ ksi

$f_3 = -1.067$ ksi

$f_4 = -1.087$ ksi

**Figure E-2: Longitudinal section illustrating the location of investigated section for general anchorage zone calculation.**

**Step 3: Calculation of longitudinal axial forces in the deck, web and soffit**

\[
F_{\text{deck}} = \frac{f_1 + f_2}{2} \times S \times t_d = \frac{0.844 + 0.868}{2} \times 123 \times 8.5 = 894.7 \text{ kips}
\]

\[
F_{\text{web}} = \frac{f_2 + f_3}{2} \times b_w \times (h - t_s - t_d) = \frac{0.868 + 1.067}{2} \times 12 \times (87 - 7 - 8.5) = 830.0 \text{ kips}
\]

\[
F_{\text{soffit}} = \frac{f_3 + f_4}{2} \times S \times t_s = \frac{1.067 + 1.087}{2} \times 123 \times 7 = 927.3 \text{ kips}
\]
**Step 4: Assumption of anchorage size**

\[ P_{\text{Jack}} = 2210 \text{ kips} \]

Low relax strands \( f_{pu} = 270 \text{ ksi} \)

According to AASHTO table 5.9.3-1

\[
A_{strands} = \frac{2210}{0.75 \times 270} = 10.91 \text{ in}^2
\]

Use 0.6” strands with cross-sectional area of 0.217 in²

No. of strands = 10.91/0.217 = 51 strand

Use 2 anchors each 27-0.6” strand with block-out diameter of 13.5” and minimum spacing of 18”

For anchors group: \( a = 18 + 13.5 = 31.5” \) and \( b = 13.5” \)

**Step 5: Deck transverse bursting forces**

\[
y = \left( \frac{S}{4} - \frac{b}{4} \right) \times \left( \frac{b_{de}}{b_{de} + \frac{S}{2}} \right)^{0.65} = \left( \frac{123}{4} - \frac{13.5}{4} \right) \times \left( \frac{39}{39 + \frac{123}{2}} \right)^{0.65} = 14.8”
\]

\[
T_{\text{deck}} = \frac{S - b - 4 \left[ y \left( 1 - \frac{W_e}{4S} \right) \right]}{4S - [(8y + 2b) \tan \alpha_{\text{skew}}]} \times F_{\text{deck}}
\]

\[
T_{\text{deck}} = \frac{123 - 13.5 - 4 \left[ 14.8 \left( 1 - \frac{0}{4 \times 123} \right) \right]}{4 \times 123 - [(8 \times 14.8 + 2 \times 13.5) \tan 0^\circ]} \times 894.7 = 91.5 \text{ kips}
\]

\[
A_{S-\text{deck}} = \frac{T_{\text{deck}}}{\phi f_y}
\]

\( \phi \) for tension in steel in anchorage zones = 1.0 according to AASHTO section 5.5.4.2.1

\[
A_{S-\text{deck}} = \frac{91.5}{1.0 \times 60} = 1.53 \text{ in}^2
\]

Existing reinforcement within distance \( S \) from the diaphragm face (\( A_{S-\text{deck-existing}} \))
\[ A_{s-deck-existing} = 0.31 \times \frac{123}{10} \times 2 = 7.63 \text{ in}^2 \]

\[ A_{s-deck-existing} \gg A_{s-deck} \quad \text{OK} \]

**Step 6: Soffit transverse bursting forces**

\[
T_{soffit} = \frac{S - b - 4 \left[ y \left( 1 - \frac{W_o}{4S} \right) \right]}{4S - \left( (8y + 2b) \tan \alpha_{skew} \right)} \times F_{soffit}
\]

\[ T_{soffit} = \frac{123 - 13.5 - 4 \left[ 14.8 \left( 1 - \frac{0}{4 \times 123} \right) \right]}{4 \times 123 - \left( (8 \times 14.8 + 2 \times 13.5) \tan 0^\circ \right)} \times 927.3 = 94.8 \text{ kips} \]

\[ A_{s-soffit} = \frac{T_{soffit}}{\phi f_y} = \frac{94.8}{1.0 \times 60} = 1.58 \text{ in}^2 \]

Existing reinforcement within distance S from the diaphragm face (\(A_{s-deck-existing}\))

\[ A_{s-soffit-existing} = 0.2 \times \frac{123}{12} \times 2 = 4.1 \text{ in}^2 \]

\[ A_{s-soffit-existing} \gg A_{s-soffit} \quad \text{OK} \]

**Step 7: Diaphragm transverse bursting forces**

\[
T_{diaph.\ transverse} = \left[ \frac{F_{deck} + F_{soffit}}{2 \cos \alpha_{skew}} \right] \times \frac{y}{b_{de} - \left( \frac{y}{2} + \frac{b}{8} \right) \tan \alpha_{skew}} - \frac{T_{deck}}{\cos \alpha_{skew}} - \frac{T_{soffit}}{\cos \alpha_{skew}}
\]

\[ T_{diaph.\ transverse} = \left[ \frac{894.7 + 927.3}{2 \cos 0^\circ} \right] \times \frac{14.8}{39 - \left( \frac{14.8}{2} + \frac{13.5}{8} \tan 0^\circ \right)} - \frac{91.5}{\cos 0^\circ} - \frac{94.8}{\cos 0^\circ} = 159.3 \text{ kips} \]

\[ A_{s-diaph.\ transverse} = \frac{T_{diaph.\ transverse}}{\phi f_y} = \frac{159.3}{1.0 \times 60} = 2.66 \text{ in}^2 \]

Use 1#6 in the inner face of the diaphragm top and bottom corners and 6 #5 along the side of the inner diaphragm face (#5 @11”)
Step 8: Diaphragm vertical bursting forces

\[
a_d = a \times \frac{F_{\text{deck}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 31.5 \times \frac{894.7}{894.7 + 830.0 + 927.3} = 10.63''
\]

\[
a_w = a \times \frac{F_{\text{web}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 31.5 \times \frac{830.0}{2652.0} = 9.86''
\]

\[
a_s = a \times \frac{F_{\text{soffit}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 31.5 \times \frac{927.3}{2652.0} = 11.01''
\]

\[
a' = a_w + \frac{a_d}{2} + \frac{a_s}{2} = 9.86 + \frac{10.63}{2} + \frac{11.01}{2} = 20.68''
\]

\[
h' = h - \frac{t_d}{2} - \frac{t_s}{2} = 87 - \frac{8.5}{2} - \frac{7}{2} = 79.25''
\]

\[
T_{\text{diaph. vertical}} = \frac{h' - a'}{2 b_{de}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10 h} \right)^{0.3} \times \left[ 1 + |\sin(\alpha_{\text{inc.}})| \right]
\]

\[
T_{\text{diaph. vertical}} = \frac{79.25 - 20.68}{2 \times 39} \times \left( \frac{894.7 + 927.3}{2} \right) \times \left( \frac{39}{10 \times 87} \right)^{0.3} \times \left[ 1 + |\sin(5^\circ)| \right] = 293.0 \text{kips}
\]

\[
A_{s-\text{diaph. vertical}} = \frac{T_{\text{diaph. transverse}}}{\phi f_y} = \frac{293.0}{1.0 \times 60} = 4.88 \text{ in}^2
\]

Use 11 #6 vertical reinforcement in the diaphragm inner face distributed as #6@11”

Step 9: Web vertical bursting forces

\[
T_{\text{web 1}} = \frac{\left( h - \frac{t_d - t_s}{4} \right)}{b_{de} + \frac{h}{4}} \times \left( \frac{F_{\text{web}}}{2} \right)
\]

\[
T_{\text{web 1}} = \frac{\left( 87 - \frac{8.5 - 7}{4} \right)}{39 + \frac{87}{4}} \times \left( \frac{830.0}{2} \right) = 105.27 \text{kips}
\]

\[
T_{\text{web 2}} = \frac{h' - a'}{2 \left( b_{de} + \frac{h}{4} \right) \times \sqrt{\frac{2 h}{b_{de}}}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left[ 1 - \left( \frac{b_{de}}{10 h} \right)^{0.3} \right]
\]
\[ T_{web_2} = \frac{79.25 - 20.68}{2 \left( 39 + \frac{87}{4} \right)} \times \left( \frac{894.7 + 927.3}{2} \right) \times \left[ 1 - \left( \frac{39}{10 \times 87} \right)^{0.3} \right] = 126.0 \text{ kips} \]

\[ T_{web} = (T_{web_1} + T_{web_2}) \times [1 + |\sin(\alpha_{inc.})|] \]
\[ T_{web} = (105.27 + 126.0) \times [1 + |\sin(5\degree)|] = 251.4 \text{ kips} \]

\[ A_{s-web} = \frac{T_{web}}{\phi f_y} = \frac{251.43}{1.0 \times 60} = 4.2 \text{ in}^2 \]

\[ A_{s-web-existing} = 0.31 \times 2 \times \frac{43.5}{6} = 4.43 \text{ in}^2 \]

\[ A_{s-web-existing} > A_{s-web} \quad \text{OK} \]

**Method II: Using Simplified Design Method**

Conditions to use simplified method

- \(|\text{Tendon inclination angle (}\alpha_{inc.}) = 5\degree| < 6\degree\) \quad \text{OK}
- \(|\text{Eccentricity ratio (}\frac{e_{edge}}{h}) = 0| < 0.1\) \quad \text{OK}
- \(\text{Minimum two anchors are used for stressing}\) \quad \text{OK}
- \(\text{Diaphragm skew angle (}\alpha_{skew}) = 0\degree < 30\degree\) \quad \text{OK}
- \(\text{Diaphragm opening width (}\frac{W_o}{0} < 0.2 \text{ S}\) \quad \text{OK}
- \(\text{Diaphragm opening height (}\frac{H_o}{0} < 0.2 \text{ h}\) \quad \text{OK}

In order to use the table \(b_{de}/h = 39/87 = 0.45\), and \(S/h = 123/87 = 1.41\)

Use the upper zone of the simplified design table E-1. Interpolation is done between the columns with \(b_{de}/h = 0.4\) and \(0.5\) as the calculated \(b_{de}/h\) value does not exist in the table.
Table E-1: Simplified design method for anchorage zone bursting forces.

<table>
<thead>
<tr>
<th>Girder spacing to box girder height (S/h)</th>
<th>Effective diaphragm width to box girder height (bde/h)</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 &lt; S/h ≤ 1.6</td>
<td>Diaph. Transverse</td>
<td>11.0</td>
<td>8.7</td>
<td>7.1</td>
<td>6.0</td>
<td>5.1</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>Diaph. Vertical</td>
<td>14.8</td>
<td>12.6</td>
<td>11.1</td>
<td>10.0</td>
<td>9.1</td>
<td>8.4</td>
</tr>
<tr>
<td></td>
<td>Web Vertical</td>
<td>12.2</td>
<td>10.9</td>
<td>9.8</td>
<td>8.9</td>
<td>8.2</td>
<td>7.6</td>
</tr>
<tr>
<td></td>
<td>Deck Transverse</td>
<td>7.6</td>
<td>7.1</td>
<td>6.6</td>
<td>6.2</td>
<td>5.9</td>
<td>5.6</td>
</tr>
<tr>
<td></td>
<td>Soffit Transverse</td>
<td>6.8</td>
<td>6.3</td>
<td>5.9</td>
<td>5.6</td>
<td>5.3</td>
<td>5.0</td>
</tr>
<tr>
<td>1.6 &lt; S/h &lt; 2.1</td>
<td>Diaph. Transverse</td>
<td>15.6</td>
<td>12.5</td>
<td>10.3</td>
<td>8.8</td>
<td>7.6</td>
<td>6.6</td>
</tr>
<tr>
<td></td>
<td>Diaph. Vertical</td>
<td>15.9</td>
<td>13.6</td>
<td>12.0</td>
<td>10.8</td>
<td>9.8</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>Web Vertical</td>
<td>11.9</td>
<td>10.7</td>
<td>9.7</td>
<td>8.8</td>
<td>8.1</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>Deck Transverse</td>
<td>9.0</td>
<td>8.5</td>
<td>8.0</td>
<td>7.6</td>
<td>7.3</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td>Soffit Transverse</td>
<td>8.0</td>
<td>7.6</td>
<td>7.2</td>
<td>6.8</td>
<td>6.5</td>
<td>6.2</td>
</tr>
</tbody>
</table>

\[
T_{\text{diaph. transverse}} = \left( \frac{11.0 + 8.7}{2 \times 100} \right) 2652 = 264.2 \text{ kips}
\]

compared to 159.3 kips from the equations

\[
T_{\text{diaph. vertical}} = \left( \frac{14.8 + 12.6}{2 \times 100} \right) 2652 = 363.3 \text{ kips}
\]

compared to 293.0 kips from the equations

\[
T_{\text{web}} = \left( \frac{12.2 + 10.9}{2 \times 100} \right) 2652 = 306.3 \text{ kips}
\]

compared to 251.4 kips from the equations

\[
T_{\text{deck}} = \left( \frac{7.6 + 7.1}{2 \times 100} \right) 2652 = 194.9 \text{ kips}
\]

compared to 91.5 kips from the equations

\[
T_{\text{soffit}} = \left( \frac{6.8 + 6.3}{2 \times 100} \right) 2652 = 173.7 \text{ kips}
\]

compared to 94.8 kips from the equations
Example (E-2)

Given:
Box girder height \((h) = 8' - 6" = 102"
Girder spacing \((S) = 14' = 168"
Deck thickness \((t_d) = 8.5"
Soffit thickness \((t_s) = 7"
Web width \((b_w) = 12"
Effective diaphragm width \((b_{de}) = 32"
Diaphragm skew angle \((\alpha_{\text{skew}}) = 35^\circ
Tendon inclination angle \((\alpha_{\text{inc.}}) = 5^\circ
Edge eccentricity \((e_{\text{edge}}) = 6"
P_{\text{jack}}\text{ per girder} = 2980 \text{ kips} \quad \text{(stressing is performed from both ends)}
Web stirrups due to other loads: #5 @ 6"
Deck transverse reinforcement: #5 @ 10" top and bottom
Soffit transverse reinforcement: #4 @ 12" top and bottom
\(f'_{\alpha} = 4000 \text{ psi}
Steel Grade 60
Friction losses parameters
- Bridge length \((x) = 250'
- Sum of the absolute values of angular change of prestressing tendon path \((\alpha) = 1.65
- Wobble friction coefficient \((K) = 0.000656 \text{ per ft}
- Coefficient of friction \((\mu) = 0.15
Method I: Using Proposed Design Equations

Step 1: Ultimate design force for anchorage zone per girder

Stressing both bridge ends

\[ \beta_{\text{end}} = 1.12 - [0.12 \left( 1 - e^{-((Kx + \mu a)} \right) ] \]

\[ \beta_{\text{end}} = 1.12 - [0.12 \left( 1 - e^{(-0.00656 \times 250) + (0.15 \times 1.65)}} \right) ] = 1.08 \]

\[ P_{du} = 1.2 \times \beta_{\text{end}} \times P_{\text{J per girder}} \]

\[ P_{du} = 1.2 \times 1.08 \times 2980 = 3862 \text{ kips} \]

Step 2: Assumption of anchorage size

\[ P_{\text{Jack}} = 2980 \text{ kips} \]

Low relax strands \( f_{\text{pu}} = 270 \text{ ksi} \)

According to AASHTO table 5.9.3-1

\[ A_{\text{strands}} = \frac{2980}{0.75 \times 270} = 14.72 \text{ in}^2 \]

Use 0.6” strands with cross-sectional area of 0.217 in²

No. of strands = 14.72/0.217 = 68 strand

Use 3 anchors each 23-0.6” strand with block-out diameter of 13.5” and minimum spacing of 18”

For anchors group: \( a = 18 + 18 + 13.5 = 49.5” \) and \( b = 13.5” \)

Use three corrugated metal ducts each 4.78” diameter

Step 3: Check web axial capacity

Due to small diaphragm width, \( b_{dc} < 0.4 \text{ h} \)

Axial capacity of the web need to be checked
\[ P_{n-web} = 0.8 \left[ 0.85f'_{ci}(A_{g-web} - A_{st-web} - A_{ducts}) + f_yA_{st-web} \right] \]

Small longitudinal reinforcement amount exists in the web so it can be neglected, \( A_{st-web} = 0 \)

\[ P_{n-web} = 0.8 \left[ 0.85 \times 4.0 \left( 12 \times 102 - 0 - 3 \times \frac{\pi \times 4.78^2}{4} \right) + 0 \right] = 3182.8 \text{ kips} \]

\( \phi \) for compression in anchorage zones = 0.8 according to AASHTO section 5.5.4.2.1

\[ \phi P_{n-web} = 0.8 \times 3182.8 = 2546.3 \text{ kips} \]

\( P_{du} > \phi P_{n-web} \)

Increase web width, use web flares \( b_w = 18" \)

\[ P_{n-web} = 0.8 \left[ 0.85 \times 4.0 \left( 18 \times 102 - 0 - 3 \times \frac{\pi \times 4.78^2}{4} \right) + 0 \right] = 4847.5 \text{ kips} \]

\[ \phi P_{n-web} = 0.8 \times 4847.5 = 3878 \text{ kips} \]

\( P_{du} < \phi P_{n-web} \quad \text{OK} \)

**Step 4: Stress distribution along the section at \((h/4 + S/2)/2\) from the diaphragm inner face**

**Section Properties**

\( A = 4161 \text{ in}^2 \) (Area of the section)

\( Y_{\text{bottom}} = 53.34" \) (Distance from section centroid to the bottom)

\( Y_{\text{top}} = 48.66" \) (Distance from section centroid to the top)

\( I_{x-x} = 6736672 \text{ in}^4 \) (Section moment of inertia about X-axis)

Based on section properties, \( Y_{\text{bottom}}, Y_{\text{top}}, A, I_{x-x}, \) tendon inclination angle \( \alpha_{\text{inc}} = 5^\circ, \) edge eccentricity = 6” and ultimate design force \( P_{du}, \) the axial stresses acting on the section located at \((h/4 + S/2)/2\) from the diaphragm inner face, see Figure E-2, are calculated as follows:

\( f_1 = -0.655 \text{ ksi} \)

\( f_2 = -0.703\text{ksi} \)
Step 5: Calculation of longitudinal axial forces in the deck, web and soffit

\[ F_{\text{deck}} = \frac{f_1 + f_2}{2} \times S \times t_d = \frac{0.655 + 0.703}{2} \times 168 \times 8.5 = 969.3 \text{ kips} \]

\[ F_{\text{web}} = \frac{f_2 + f_3}{2} \times b_w \times (h - t_s - t_d) = \frac{0.703 + 1.188}{2} \times 12 \times (102 - 7 - 8.5) = 1472.1 \text{ kips} \]

\[ F_{\text{soffit}} = \frac{f_3 + f_4}{2} \times S \times t_s = \frac{1.188 + 1.228}{2} \times 168 \times 7 = 1420.6 \text{ kips} \]

Step 6: Deck transverse bursting forces

\[ y = \left( \frac{S}{4} - \frac{b}{4} \right) \times \left( \frac{b_{de}}{b_{de} + \frac{S}{2}} \right)^{0.65} = \left( \frac{168}{4} - \frac{13.5}{4} \right) \times \left( \frac{32}{32 + \frac{168}{2}} \right)^{0.65} = 16.72" \]

\[ T_{\text{deck}} = \frac{S - b - 4 \left[ y \left( 1 - \frac{W_0}{4S} \right) \right]}{4S - [(8y + 2b) \tan \alpha_{\text{skew}}]} \times F_{\text{deck}} \]

\[ T_{\text{deck}} = \frac{168 - 13.5 - 4 \left[ 16.72 \left( 1 - \frac{0}{4 \times 168} \right) \right]}{4 \times 168 - \left[ (8 \times 16.72 + 2 \times 13.5) \tan 35^\circ \right]} \times 969.3 = 151.8 \text{ kips} \]

\[ A_{s-\text{deck}} = \frac{T_{\text{deck}}}{\phi f_y} = \frac{151.8}{1.0 \times 60} = 2.53 \text{ in}^2 \]

Existing reinforcement within distance S from the diaphragm face \((A_{s-\text{deck-existing}})\)

\[ A_{s-\text{deck-existing}} = 0.31 \times \frac{168}{10} \times 2 = 10.42 \text{ in}^2 \]

\[ A_{s-\text{deck-existing}} \gg A_{s-\text{deck}} \quad \text{OK} \]
Step 7: Soffit transverse bursting forces

\[ T_{soffit} = \frac{S - b - 4 \left[ y \left( 1 - \frac{W_o}{4S} \right) \right]}{4S - \left[ (8y + 2b) \tan \alpha_{skew} \right]} \times F_{soffit} \]

\[ T_{soffit} = \frac{168 - 13.5 - 4 \left[ 16.72 \left( 1 - \frac{0}{4 \times 168} \right) \right]}{4 \times 168 - \left[ (8 \times 16.72 + 2 \times 13.5) \tan 35^\circ \right]} \times 1420.6 = 222.5 \text{kips} \]

\[ A_{s-soffit} = \frac{T_{soffit}}{\phi f_y} = \frac{222.5}{1.0 \times 60} = 3.71 \text{ in}^2 \]

Existing reinforcement within distance S from the diaphragm face (As-deck-existing)

\[ A_{s-soffit-existing} = 0.2 \times \frac{168}{12} \times 2 = 5.6 \text{ in}^2 \]

\[ A_{s-soffit-existing} \gg A_{s-soffit} \quad \text{OK} \]

Step 8: Diaphragm transverse bursting forces

\[ T_{diaph. \, transverse} = \left[ \frac{F_{deck} + F_{soffit}}{2 \cos \alpha_{skew}} \right] \times \frac{y}{b_{de} - \left[ \left( \frac{y}{2} + \frac{b}{8} \right) \tan \alpha_{skew} \right]} - \frac{T_{deck \, \cos \alpha_{skew}}}{\cos \alpha_{skew}} - \frac{T_{soffit \, \cos \alpha_{skew}}}{\cos \alpha_{skew}} \]

\[ T_{diaph. \, transverse} = \left[ \frac{969.3 + 1420.6}{2 \cos 35^\circ} \right] \times \frac{16.72}{32 - \left[ \left( \frac{16.72}{2} + \frac{135}{8} \right) \tan 35^\circ \right]} - \frac{151.8}{\cos 35^\circ} - \frac{222.5}{\cos 35^\circ} = 520.3 \text{kips} \]

\[ A_{s-diaph. \, transverse} = \frac{T_{diaph. \, transverse}}{\phi f_y} = \frac{520.3}{1.0 \times 60} = 8.67 \text{ in}^2 \]

Use 2#7 in the inner face of the diaphragm top and bottom corners and 15 #6 along the side of the inner diaphragm face (#6 @6”), Reinforcement will increase significantly in case of using thicker diaphragm.
Step 9: Diaphragm vertical bursting forces

\[ a_d = a \times \frac{F_{\text{deck}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 49.5 \times \frac{969.3}{969.3 + 1472.1 + 1420.6} = 12.42" \]

\[ a_w = a \times \frac{F_{\text{web}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 49.5 \times \frac{1472.1}{3862.0} = 18.87" \]

\[ a_s = a \times \frac{F_{\text{soffit}}}{F_{\text{deck}} + F_{\text{web}} + F_{\text{soffit}}} = 49.5 \times \frac{1420.6}{3862.0} = 18.21" \]

\[ a' = a_w + \frac{a_d}{2} + \frac{a_s}{2} = 18.87 + \frac{12.42}{2} + \frac{18.21}{2} = 34.18" \]

\[ h' = h - \frac{t_d - t_s}{2} = 102 - \frac{8.5 - 7}{2} = 94.25" \]

\[ T_{\text{diaph. vertical}} = \frac{h' - a'}{2 b_{de}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left( \frac{b_{de}}{10 h} \right)^{0.3} \times [1 + |\sin(\alpha_{\text{inc.}})|] \]

\[ T_{\text{diaph. vertical}} = \frac{94.25 - 34.18}{2 \times 32} \times \left( \frac{969.3 + 1420.6}{2} \right) \times \left( \frac{32}{10 \times 102} \right)^{0.3} \times [1 + |\sin(5\degree)|] = 431.6 \text{ kips} \]

\[ A_{s-diaph. vertical} = \frac{T_{\text{diaph. transverse}}}{\phi f_y} = \frac{431.6}{1.0 \times 60} = 7.19 \text{ in}^2 \]

Use 17 #6 vertical reinforcement in the diaphragm inner face distributed as #6@10"

Step 10: Web vertical bursting forces

\[ T_{\text{web 1}} = \frac{\left( \frac{h - t_d - t_s}{4} \right) - \frac{a_w}{4}}{b_{de} + \frac{h}{4}} \times \left( \frac{F_{\text{web}}}{2} \right) \]

\[ T_{\text{web 1}} = \frac{\left( \frac{102 - 8.5 - 7}{4} \right) - \frac{18.87}{4}}{32 + \frac{102}{4}} \times \left( \frac{1472.1}{2} \right) = 216.4 \text{ kips} \]

\[ T_{\text{web 2}} = \frac{h' - a'}{2 \left( b_{de} + \frac{h}{4} \right)} \times \sqrt{\frac{2 h}{b_{de}}} \times \left( \frac{F_{\text{deck}} + F_{\text{soffit}}}{2} \right) \times \left[ 1 - \left( \frac{b_{de}}{10 h} \right)^{0.3} \right] \]
\[ T_{\text{web} 2} = \frac{94.25 - 34.18}{2 \left( 32 + \frac{102}{4} \right)} \times \sqrt{\frac{\left( 32 + 102 \right)}{32}} \times \left( \frac{969.3 + 1420.6}{2} \right) \times \left[ 1 - \left( \frac{32}{10 \times 102} \right)^{0.3} \right] = 159.7 \text{kips} \]

\[ T_{\text{web}} = (T_{\text{web} 1} + T_{\text{web} 2}) \times [1 + |\sin(\alpha_{\text{inc.}})|] \]

\[ T_{\text{web}} = (216.4 + 159.7) \times [1 + |\sin(5^\circ)|] = 408.9 \text{kips} \]

\[ A_{s-\text{web}} = \frac{T_{\text{web}}}{\phi f_y} = \frac{408.9}{1 \times 60} = 6.82 \text{in}^2 \]

\[ A_{s-\text{web-existing}} = 0.31 \times 2 \times \frac{102/2}{6} = 5.27 \text{in}^2 \]

\[ A_{s-\text{web-existing}} < A_{s-\text{web}} \]

Use #6@6” extending h/2 in longitudinal direction.

**Method II: Using Simplified Design Method**

Conditions to use simplified method

- | Tendon inclination angle \((\alpha_{\text{inc.}}) = 5^\circ | < 6^\circ | \) OK
- | Eccentricity ratio \((e_{\text{edge}/h}) = 6/102 = 0.06 | < 0.1 | \) OK
- Minimum two anchors are used for stressing \ OK
- Diaphragm skew angle \((\alpha_{\text{skew}}) = 35^\circ | > 30^\circ | \) Not Satisfied
- Diaphragm opening width \((W_o=0) < 0.2 \text{ S} \) \ OK
- Diaphragm opening height \((H_o=0) < 0.2 \text{ h} \) \ OK
- Diaphragm width \((b_{de}) = 32” < 0.4 \text{ h} \) Out of table limit

Therefore, due to diaphragm skew angle and width simplified design table cannot be used.