A Thesis

entitled

Evaluation of Reserve Shear Capacity of Bridge Pier Caps Using the Deep Beam Theory

by

Pappu Baniya

Submitted to the Graduate Faculty as partial fulfillment of the requirements for the

Master of Science Degree in

Civil Engineering

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August 2019

An Abstract of

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Many bridge pier caps are deep due to short shear spans. When analyzed using the sectional method, a large number of pier caps are found to be shear-overloaded even though they don't exhibit any noticeable cracking or signs of distress. AASHTO LRFD 2014 recommends the use of either a strut-and-tie or nonlinear finite element model for the analysis and design of deep members. Both methods are more sophisticated and require more effort than the sectional method. The objective of this study was to simplify the strut-and-tie method for pier caps to obtain larger and less conservative shear capacity predictions. For this purpose, a solution algorithm (through a computer program) was developed based on Section 5.6.3 Strut-and-Tie Model of AASHTO LRFD 2014. The program, named STM-CAP (Strut-and-Tie Method for pier CAPs), is implemented in Microsoft Excel using Visual Basic macro codes. An adaptive graphical solution procedure was employed to minimize the input errors and give the analyst options for optimizing the automatically-generated model.

STM-CAP calculates the utilization ratio for every element, which reflects the condition (overload or reserve capacity percentage) of the pier cap. If overloaded, STM-CAP indicates the calculated

failure mode and its location. Suitable rehabilitation methods and load limits can thus be determined accordingly.

STM-CAP was verified using a general-purpose strut-and-tie software, CAST (Computer Aided Strut-and-Tie) for eight existing pier caps located in Ohio. Numerical response simulation of five bridge pier caps were performed using VecTor2 (a nonlinear finite element analysis software) for in-depth analysis and to compare with the STM. In addition, the sectional method calculations were performed to demonstrate the higher shear capacity predictions obtained from the strut-and-tie method. The strut-and-tie method predicted two to three times higher shear capacities for beam with shear span-to-depth ratio (*a/d*) of 0.50. The predictions by STM-CAP and the sectional method converged as the *a/d* ratio reached 3.0.

The research results have a potential to result in significant cost savings by rehabilitating fewer number of pier caps and reducing the associated construction work and traffic disruption. The developed program STM-CAP can also be used when load rating concrete bridge pier caps.

Acknowledgements

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Chapter 1 Introduction

1.1 Background

The increase in traffic and transport freight over the past decade has significantly increased the loading on bridge structures. The cost to maintain bridges or to repair issues that arise from the increased traffic can be costly. Such a prohibitive cost requires accurate analysis methods to correctly identify the overloaded bridges.

'Pier caps,' or 'bent caps,' transfer the load from bridge girders to columns as shown in **Figure 1-1**. Bridge pier caps are deep structures due to the short shear span over which the girder loads are applied. A beam for which the distance between the applied load and the reaction point is less than about twice the member depth is referred to as a deep beam. Most pier caps are *'deep beams'* and possess additional shear strength due to the formation of the strut action. Unlike slender beams, deep beams transfer shear forces to supports through compressive stresses rather than shear stresses. The diagonal cracks in deep beams eliminate the inclined principal tensile stresses required for beam action and lead to a redistribution of internal stresses so that the beam acts as a tied arch known as strut action. The AASHTO LRFD code began to include the deep beam methods in 1994. Since the average age of the bridges in the United States is over forty years, most in-service bridges were not originally designed considering the deep beam effects and thus possess a hidden reserve shear capacity.

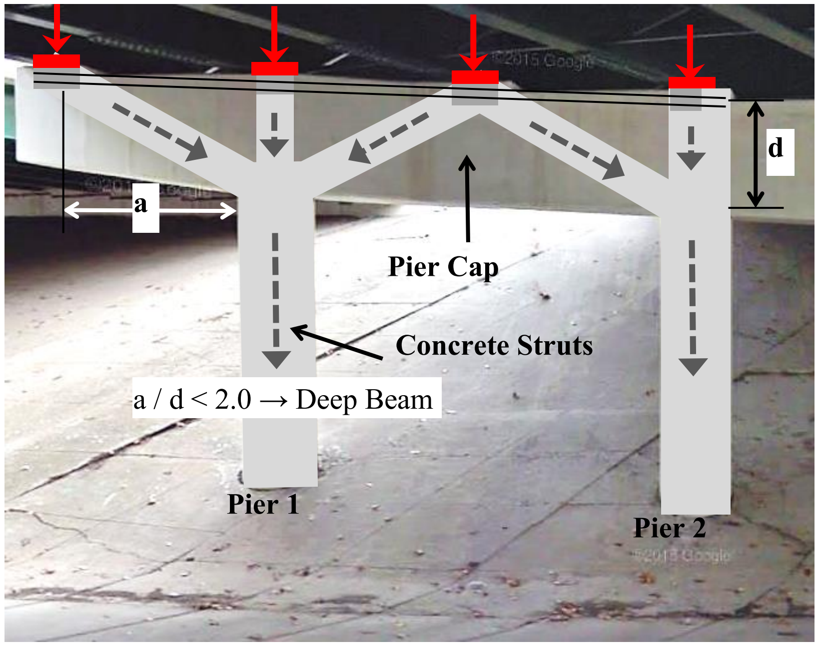


Figure 1-1: A sample pier cap

The AASHTO LRFD (Load and Resistance Factor Design) Bridge Design Specification 2017 contains two main analysis methods for the design of reinforced concrete members: the sectional method and the Strut-and-Tie Method (STM). The sectional method requires checking the shear/moment capacities at critical sections based on the plane-sections-remain-plane hypothesis (i.e., the slender beam theory). STM, on the other hand, does not rely on this hypothesis and is suitable for the analysis of deep beams, which exhibit nonlinear strain gradient. STM is a truss model where the stress field in the structural concrete is equivalent to the hypothetical simple uniaxial truss to give a proper and definite load path. The truss analogy consists of struts, ties, and nodes.

STM is conceptually a simple design methodology. However, its implementation is rather complicated. STM implementation requires iterations and graphical representation for the STM model. A specialized STM knowledge is required for the creation of the STM model. STM requires more effort and experience than the sectional method. In civil engineering practice, the sectional method is the most popular method and is dominantly used for analyzing and load rating existing pier caps, even if they are deep. If a deep beam is analyzed by the sectional method, overly-conservative (i.e., low) shear capacity predictions are obtained. This theory neglects the deep beam action and cannot capture the additional shear capacity. When deep beams are analyzed using the sectional methods, they may be incorrectly found shear overloaded. These beams may in fact have reserve capacities when analyzed by a proper analysis method, such as STM. To reduce rehabilitation costs and to correctly identify the overloaded bridge pier caps, there is a need of practical analysis methods that can account for the deep beam action.

1.2 Research Objectives

Since there is limited public funds for rehabilitation and strengthening of the deficient bridges, it is imperative to use the proper analysis method to correctly identify and rank the overloaded bridges. The main objective of this study was to explore innovative strategies to reduce the complexity of STM to a level comparable to the sectional method for analyzing deep cap beams. It sought to create a computer program (STM-CAP) with strong graphical capabilities to automatically generate efficient STM models while intuitively educating practicing engineers in the correct use of STM. To check the accuracy of STM-CAP, a number of bridge pier caps were modeled using STM-CAP, CAST (Computer Aided Strut-and-Tie), and VecTor2 (a nonlinear finite element analysis method), the latter suitable for a more detailed investigation of pier caps. A secondary objective was to compare the shear strength predictions obtained from the sectional method and learn if the sectional method always underestimates the shear capacities of deep beams, and if so, to what extent and under what conditions.

1.3 Research Approach

To accomplish the research objectives, the following research approaches were undertaken.

1.3.1 Literature Review

A thorough literature review was conducted to assess the current state of the practice in determining the shear capacities of pier caps. This review helped to better understand the current state of the art and challenges that need to be overcome.

1.3.2 Development, Testing, Debugging and Refinement of a Spreadsheet Program

STM requires a graphical representation, and therefore more effort and experience than the sectional method. Multiple STM models can be developed for the same bridge—some being more efficient (and less conservative) than the others. In addition, STM is not typically taught in undergraduate Civil Engineering education and many engineers in practice are not familiar with it. There are over one-million bridge pier caps in the United States alone, and each pier cap analysis takes a significant amount of time. Because of this, an automated computer program is needed.

STM was used to develop a spreadsheet program, Strut-and-Tie Method for pier CAPs (STM-CAP), for the analysis of deep pier caps subjected to girder loads. A major objective was to use a graphical solution approach as part of the analysis process to help the analyst develop a better understanding of STM as well as catch any input mistakes. More than 6,000 lines of Visual Basic (VBA) code was written to provide this graphical capability. STM-CAP was tested using several pier caps and refined accordingly. Various warning and error checks were performed, and relevant messages were printed during an analysis if required.

1.3.3 Numerical Modeling of Pier Caps

ODOT provided original design drawings for thirteen bridges for this study. These bridges are modeled using three methods: **(1)** a strut-and-tie analysis, **(2)** a nonlinear analysis, and (**3)** a linear analysis using the sectional method. This stage included comparing all analysis results, developing load-deflection curves, and calculating the demand-to-capacity ratios. A statistical analysis was then performed to determine the additional shear strengths that may be predicted from STM over those from the sectional method.

1.4 Outline of Thesis

This thesis contains nine Chapters and four Appendices. Chapter 1 introduces the background, research objectives, and the outline of the thesis. Chapter 2 provides literature review. Chapter 3 explains the historical development, theory, and AASHTO formulations behind STM. Chapter 4 introduces the developed spreadsheet STM-CAP (Strut-and-Tie Method for pier CAPs). In this chapter, the theory, formulations, development process, input guidelines, and methodology of STM-CAP is described. Chapter 5 presents the verification of STM-CAP using another strut-and-tie tool, CAST (Computer Aided Strut-and-Tie), by modeling eight pier caps. Chapter 6 deals with the nonlinear modeling of the bridge pier caps using VecTor2 to simulate the global behavior. The results obtained were compared with the STM results to determine how conservative AASHTO 2017 STM provisions are. Chapter 7 compares the shear strength predictions of the sectional method with the STM-CAP results in order to understand under what conditions and to what extent the sectional method underestimates the shear capacities. Chapter 8 explains the updated STM formulations of AASHTO LRFD 2017. Chapter 9 summarizes the research results and conclusions. Appendix A and Appendix B includes printouts of eight solved bridge pier caps using 2014 and 2017 AASHTO provisions, respectively. Appendix C contains the list of checks performed in STM-CAP and corresponding warning and error messages. Appendix D discusses the application and challenges when implementing STM-CAP for the strength evaluations of bridge pier caps in practice.

Chapter 2 Literature Review

2.1 Introduction

A literature search was performed to identify the analytical and experimental investigations that have been conducted concerning analysis and design of bridge pier caps. The summary of key works and finding are presented in this section.

Numerous experimental studies have been carried out for the study of the behavior of bridge pier caps due to their deep nature. Pier caps have short shear span over which the load is applied, which makes them deep beams. Many theories and hypotheses have been proposed to predict the failure modes and capacity of the pier caps. Shear resistance in reinforced concrete beams has been studied for over fifty years and there are many theories concerning the mechanisms of how the beams resist shear and about the prediction of their ultimate shear strength.

2.2 Experimental Investigations

The early work on deep beams includes the work of Kani at the University of Toronto in the 1960s. His work brought light to many factors that influence the resistance of deep reinforced concrete beams. In the experimental test, he found that shear failure was the prominent type of failure in deep beams. However, the failure types differed according to the shear span-to-depth ratio (*a/d*). As shown in **Figure 2-1**, the probability of shear failure was higher for smaller *a/d* ratio. As *a/d* ratio increased, the failure mode gradually shifted from shear to flexure.

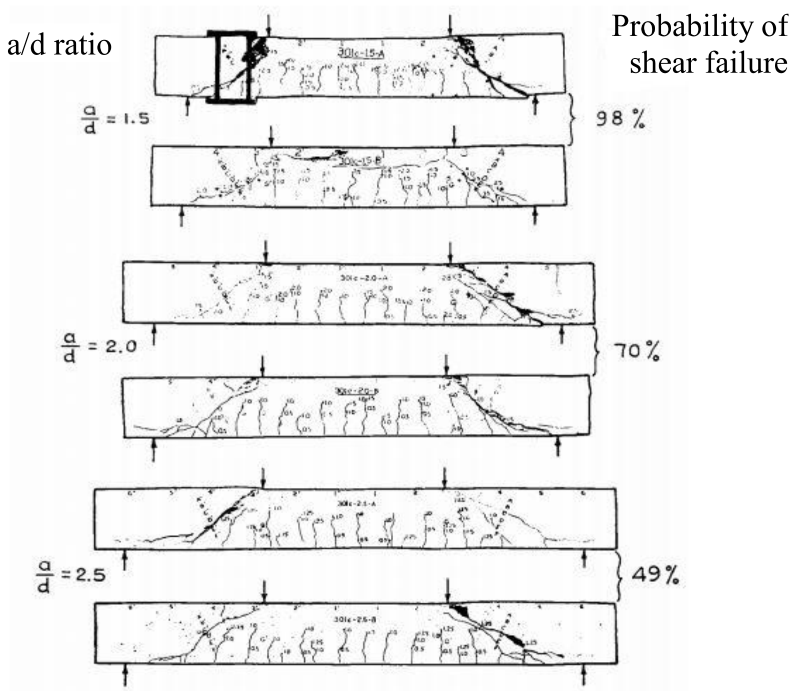


Figure 2-1: Probability of shear failure for corresponding *a/d* ratio (Kani, 1964)

In 1964, Kani performed a series of tests to calculate the load carrying capacity of fourteen reinforced concrete beams with varied *a/d* ratio. All the beams had the same cross section, concrete strength, and reinforcement. The only factor that varied was shear span (*a*).

The results of a test done by Kani on twenty-four-inch-deep beams for different shear span ratio is shown in **Figure 2-2**. The triangular dots are the experimental results and the two curves are the predictions based on different methods. The red curve and blue curve are the predictions based on the sectional analysis and STM respectively. Note that when the *a/d* ratio was less than 2.0 (deep beams), the sectional model became increasingly poor and conservative at predicting the shear strength of the section. Kani inferred that, for these deep beams, a strut-and-tie method was more appropriate since it provided more accurate and less conservative results.

Figure 2-2 shows that STM is better than the sectional method for the analysis and design of deep beams, whereas the sectional method was better at predicting shear strength of slender beams. This work verified that combination of both methods, the sectional method and STM, should be used for the analysis and design of beams. The sectional method should be used for the slender beams (*a/d* ratio > 2.0) and STM should be used for deep beams (*a/d* ratio < 2.0).

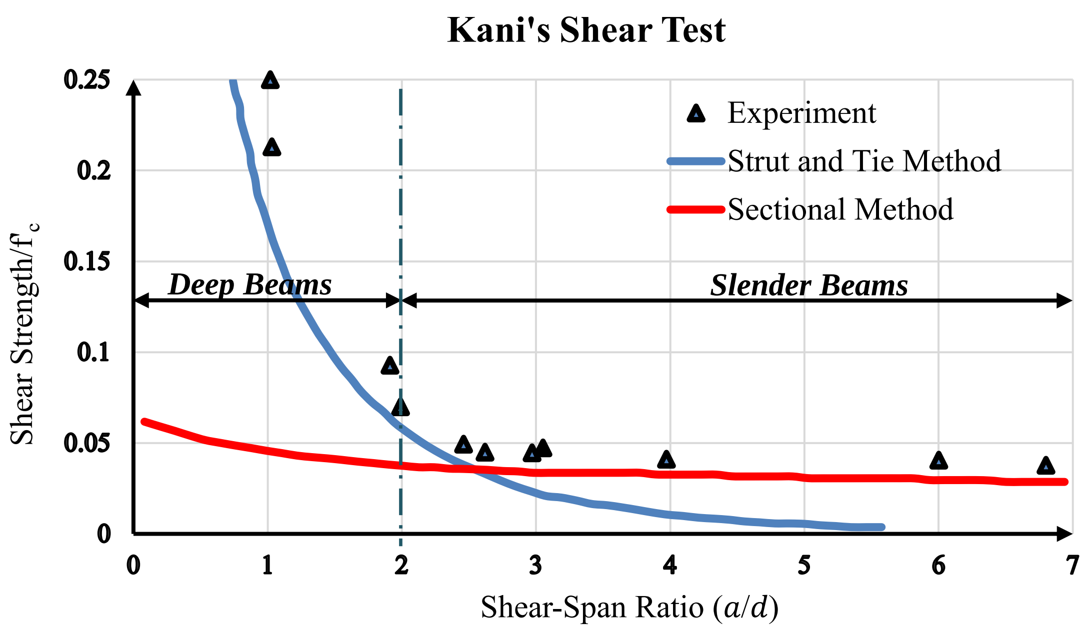


Figure 2-2: Shear strength vs *a/d* ratio (Kani, 1964)

In 1964, at the University of Texas, Ferguson conducted a notable experiment on thirty-six 36-inch deep pier cap overhangs. The variable to be studied were shear span, bar anchorage length, skin reinforcement, grade and area of rebar, amount of shear reinforcement, etc. The test was conducted until failure of the pier cap overhang.

A key finding was within a shear span-to-depth ratio (*a/d*) 0.5 to 1.2, the ultimate shear strength is found to be conservatively higher than the strength calculated by the sectional method. This finding yielded a consistent result to Kani's. Ferguson also found the vertical stirrup used had no significant effect on the shear capacity of overhang. The reason for this was due to the steep angles of cracks which did not cross many stirrups. Ferguson suggested that using the horizontal skin reinforcement improved the shear strength of deep pier cap overhangs and reducing the crack width. It appeared that no bond failure occurred for an end anchorage of fifteen inches for #11 bars and twelve inches for #8 bars.

Later, in 1966, Ferguson and Liao conducted a similar experiment on the pier cap between the columns. The cracks and results were similar to the previous experiment on overhangs. They found the shear failure was uniformly along a direct line from load to the face of the support. The stirrup had more effect for these types of pier caps than the overhang.

Twenty-three reinforced concrete deep beams, which included six simple span beams and seventeen two span-continuous beams, were tested by Rogowsky et al. (1983). The shear span-to-depth ratio (*a/d*) varied from 1 to 2.5. It was found that the beam generally failed in shear depending upon the amount and arrangement of reinforcement and the *a/d* ratio. Ultimate failure was usually due to shear compression or crushing at the end of one of these struts as shown in **Figure 2-3.**

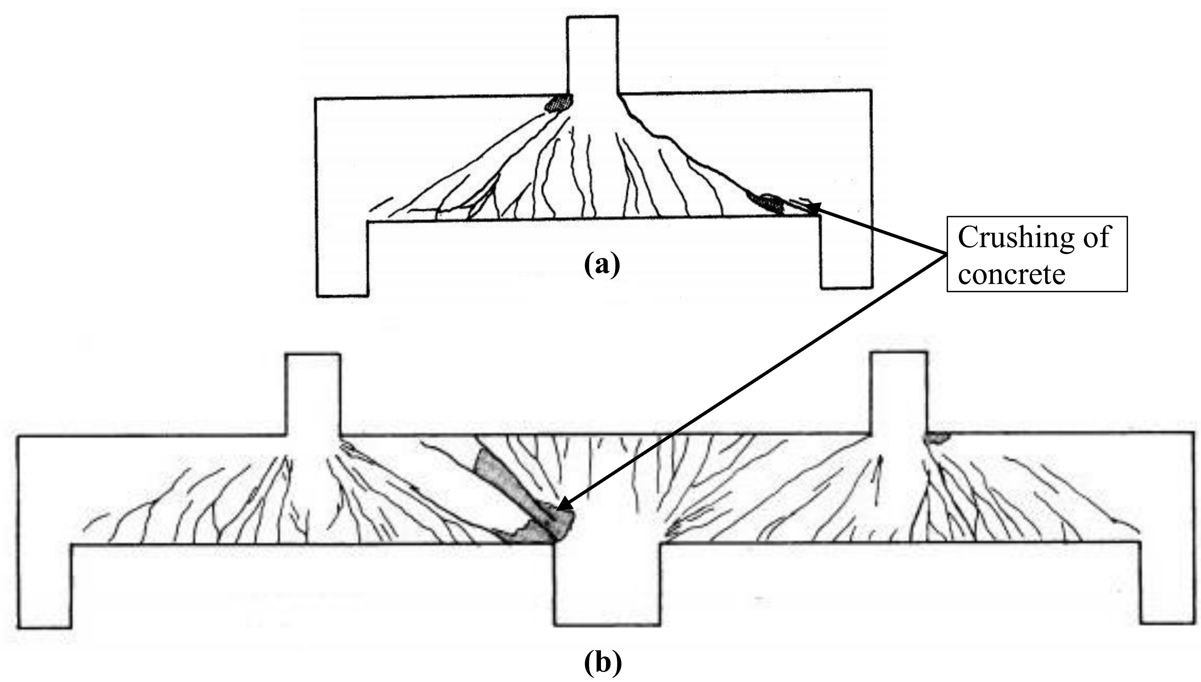


Figure 2-3: Typical failure in **(a)** simply supported; **(b)** continuous cap (Rogowsky, 1983)

Denio et al. (1995) conducted an experiment on six pier cap specimens at 30% scale. Five different reinforcing steel patterns were used to examine the contributions of different reinforcing types to the pier cap strength. These pier caps were loaded to failure under eleven static loads. In all specimens, it was found that the load on the pier caps were primarily carried by the action of the tied arch from the load base plates to the column.

Testing showed that the pier cap resisted loads through a tied arch, which is a stronger load-carrying mechanism than concrete in shear. The strut-and-tie models used were more accurate than conventional design methods in predicting the capacity of the pier caps because they modeled the compression arch action observed during testing. Denio et al. recommended using the strut-and-tie method for design and analysis of pier caps because strut-and-tie analyses gave the best correlation with test results, modeled true behavior, and were still conservative.

Young et al. (2000 and 2002) tested sixteen full-scale bent caps overhang to study the unexpected cracking in the bent cap during 2000's as shown in **Figure 2-4 (a)**. The experimental setup is shown in **Figure 2-4 (b)**. The specimen had a shear span of 54-inches and were 36-inches deep. The different models had varied longitudinal reinforcement, stirrup, and skin reinforcement. The specimens were loaded until the failure of the pier cap. All the specimens failed in shear.

The study found the flexural cracking initiated for rebar stress of four to seven ksi. It was found that the stress in rebar was the primary factor influencing the crack widths. Skin reinforcement had little influence on the crack width. In contrast to P. Ferguson (1964), the increase in vertical stirrup decreased the width of flexure-shear cracks.

The study suggested using the center of column support as the critical section for the design for a/d=1.5. This resulted in additional rebar at the column face thus limiting the rebar stresses. Young et al. suggested increasing the shear strength of the pier cap in order to limit the width of cracks and to prevent shear failure mechanism.

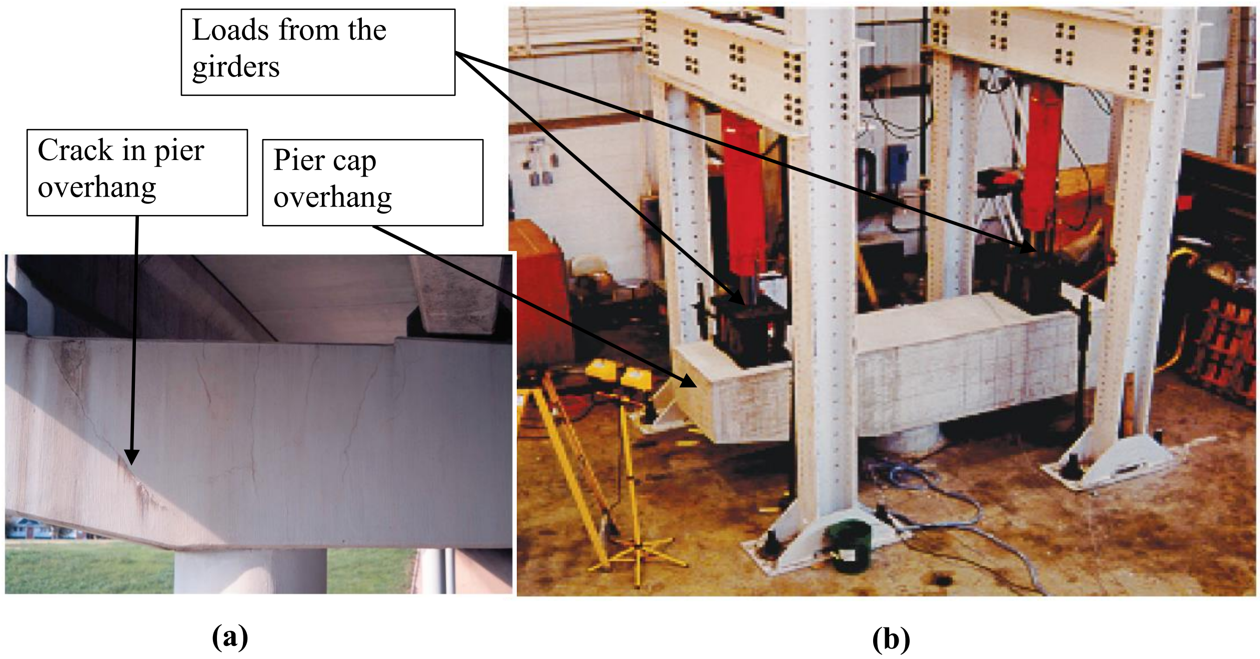


Figure 2-4: **(a**) Crack in pier cap; **(b)** experimental setup (Young et al., 2000 and 2002)

Higgins et al. (2008) conducted an experiment on six full-scale specimens of in-service bent caps for Oregon DOT. These pier caps were detailed with 1950s vintage details including reinforcement details, anchorage bars, etc. and tested subjected to cyclic loading. They observed shear failure, as shown in **Figure 2-5,** with a diagonal crack in all the specimens for *a/d* ratio between 1.0 to 2.5. The report summarized ACI 318-99 deep beam shear design produced conservative results. On the other hand, a more detailed STM model produced relatively better prediction results but were still conservative. Nonlinear finite element software VecTor2 provided one of the best correlations with experimental results in terms of predicted capacities, crack patterns, and the flow of principle stresses.

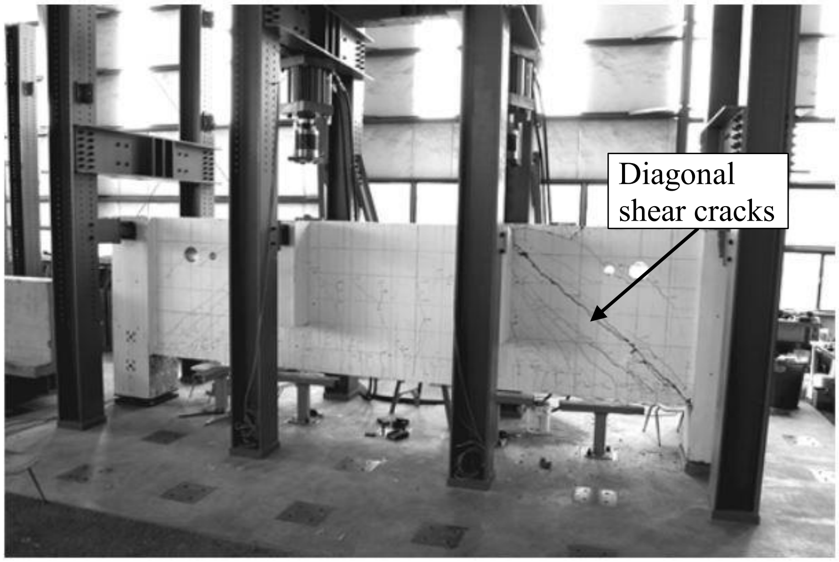


Figure 2-5: Shear crack in bridge pier cap (Higgins et al., 2008)

Dr. Bechtel at the Georgia Institute of Technology conducted full-scale testing of seven pier caps typical to the State of Georgia and showed the suitability of the strut-and-tie method (Bechtel, 2012).

2.3 Analytical Investigations

Cunningham, L.S. (2000) performed an analysis of deep beams and outlined that the main stress path represented a strut-and-tie model. In his work, the assumption of strut-and-tie model elements, i.e. struts, ties, and nodes, were verified using the non-linear analysis approach. Cunningham proposed the strut-and-tie model as a procedure to define major stress paths in a deep beam.

He summarized that when a deep beam is loaded, there are regions with high stress and low stress. He found that the parts of structures that are lowly stressed can be rejected/removed without affecting the overall strength. This rejection of the lowly stressed region led to verification of STM. He introduced the ratio of rejection of low-stress field that is referred to as reduction ratio (rr).

A simply supported beam of *a/d* ratio 1.67 is shown in **Figure 2-6** with a vertical load at center, and cantilever beam of *a/d* ratio 1.0 is shown in **Figure 2-7**. In both cases, a clear strut-and-tie model is shown at a reduction ratio (rr) of about 20-25%.

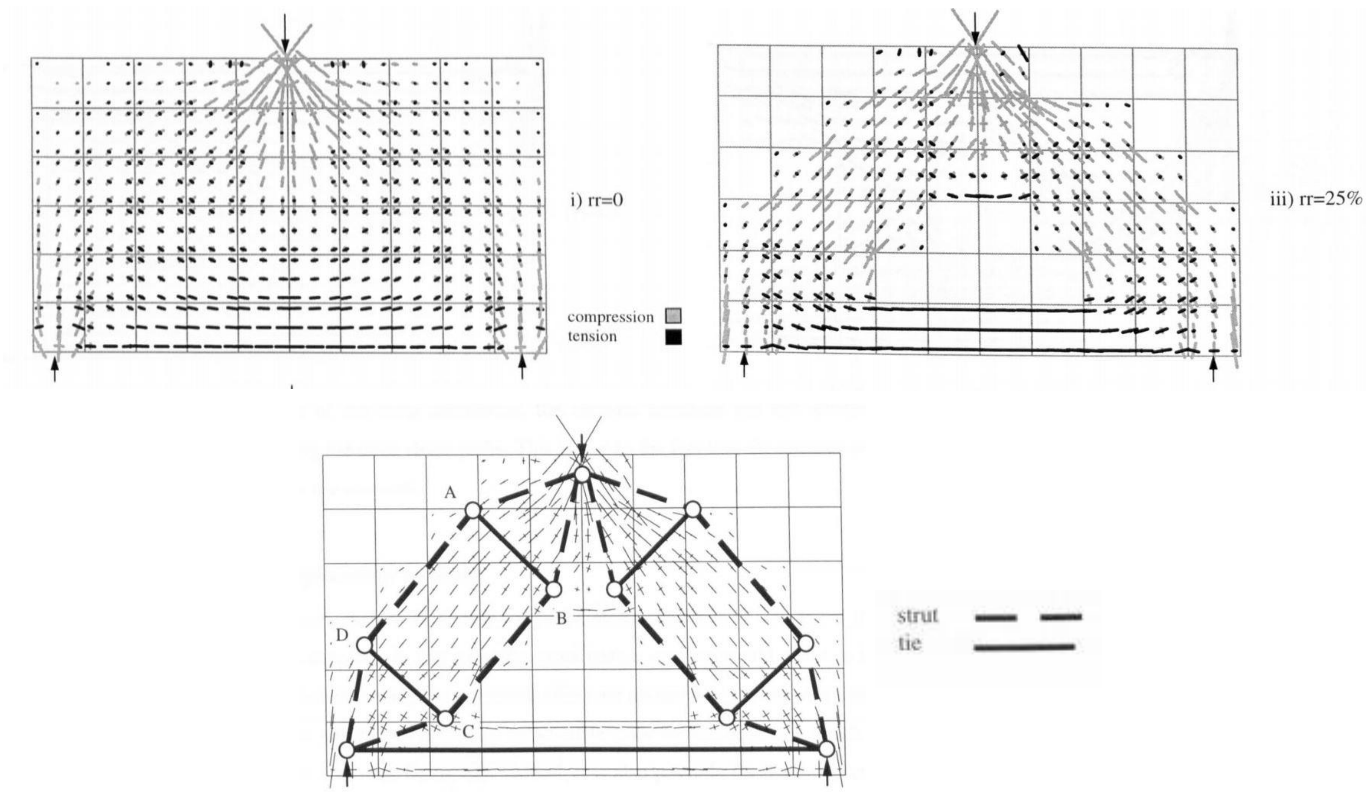


Figure 2-6: Deep simply supported beam of *a/d*=1.67 visualizing STM model

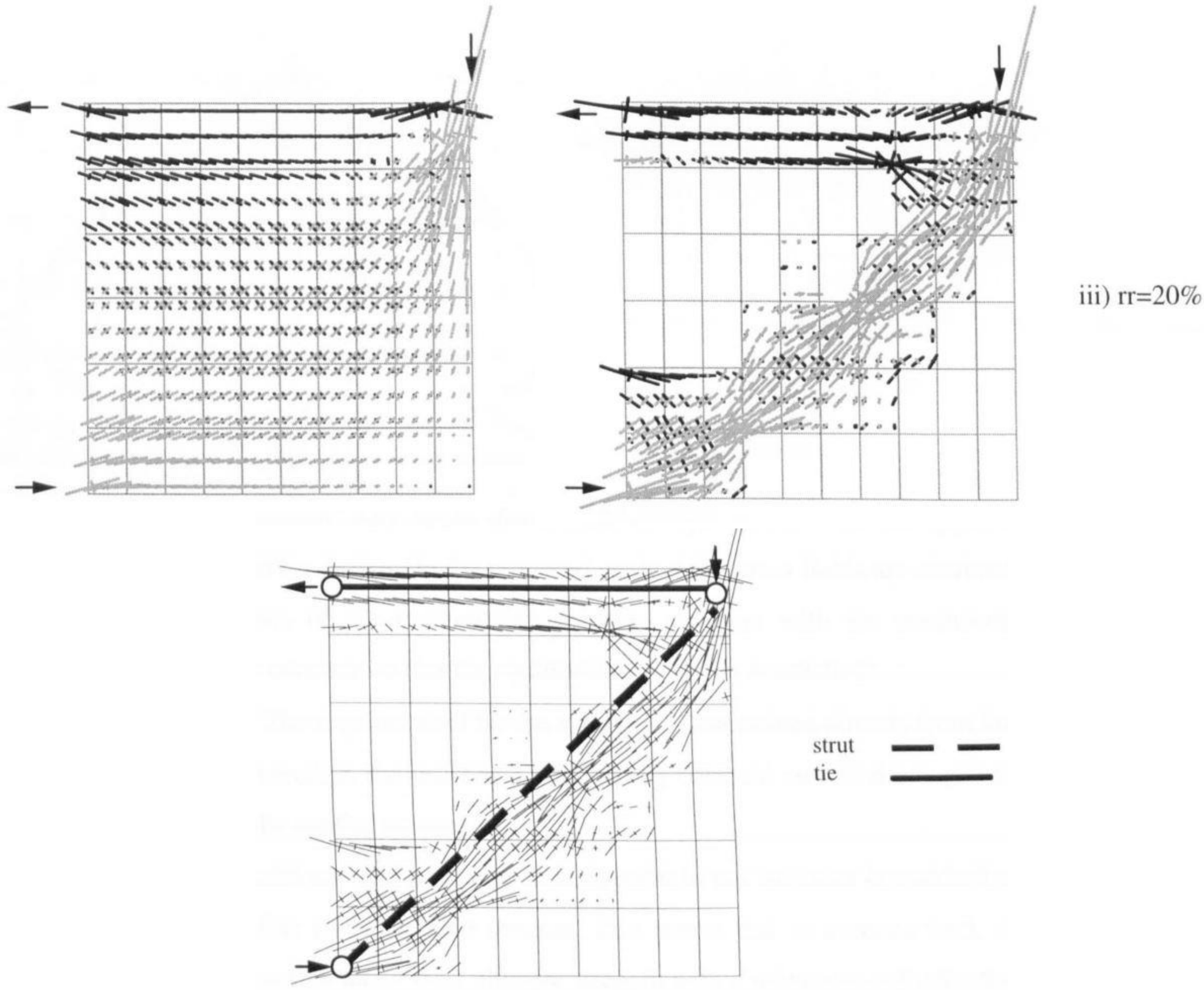


Figure 2-7: Deep cantilever beam of *a/d* ratio = 1.00 visualizing STM model

This process of removing low-stress paths led to the isolation of the main stress paths within the structures and to the identification of suitable strut-and-tie models for a given load case. This verified the strut-and-tie method and concluded that accurate shear strength of deep beam can be estimated from STM.

A report on retrofitting shear cracks in pier caps by Milde et al. (2005) for the Minnesota Department of Transportation, summarized that a cracked but undeteriorated pier cap overhang might not be prone to structural failure. It showed an AASHTO-based strut-and-tie method provision provided adequate prediction of the ultimate capacity of pier cap overhangs. They found that crack width in pier caps can be limited by designing the pier cap at the center of the column, rather than the face of the column, to increase the tensile reinforcement.

Several factors influencing the behavior of deep beams were studied by Lafta et al. (2016). It was found that factors such as loading and supporting conditions, horizontal and vertical web reinforcement, shear span-to-depth ratio, load and support bearing plates, tension reinforcement, and compressive strength are the main influencing factors. The report concluded deep beams with high compressive strength exhibit higher shear capacity. Unlike Ferguson (1964), the study found the increase in vertical web reinforcement amount increased ultimate load capacity and restrained the diagonal crack. Horizontal web reinforcement had less effect on ultimate capacity than vertical reinforcement. The study proved the elastic theory of bending is not appropriate to problems including deep beams. The stress pattern is nonlinear and deviates considerably from the elastic theory of bending. Therefore, the strength of such beams must be estimated using the non-linear analysis. These beams have high shear strength due to tied arch action—the behavior of strut-and-tie which transmits the load directly to the support through concrete compression struts. The strain distribution across the depth of the beam for different shear-span-ratio is shown in **Figure 2-6**. It was found the shear span-to-depth (*a/d*) ratio was the main influencing factor for shear strength of deep beams.

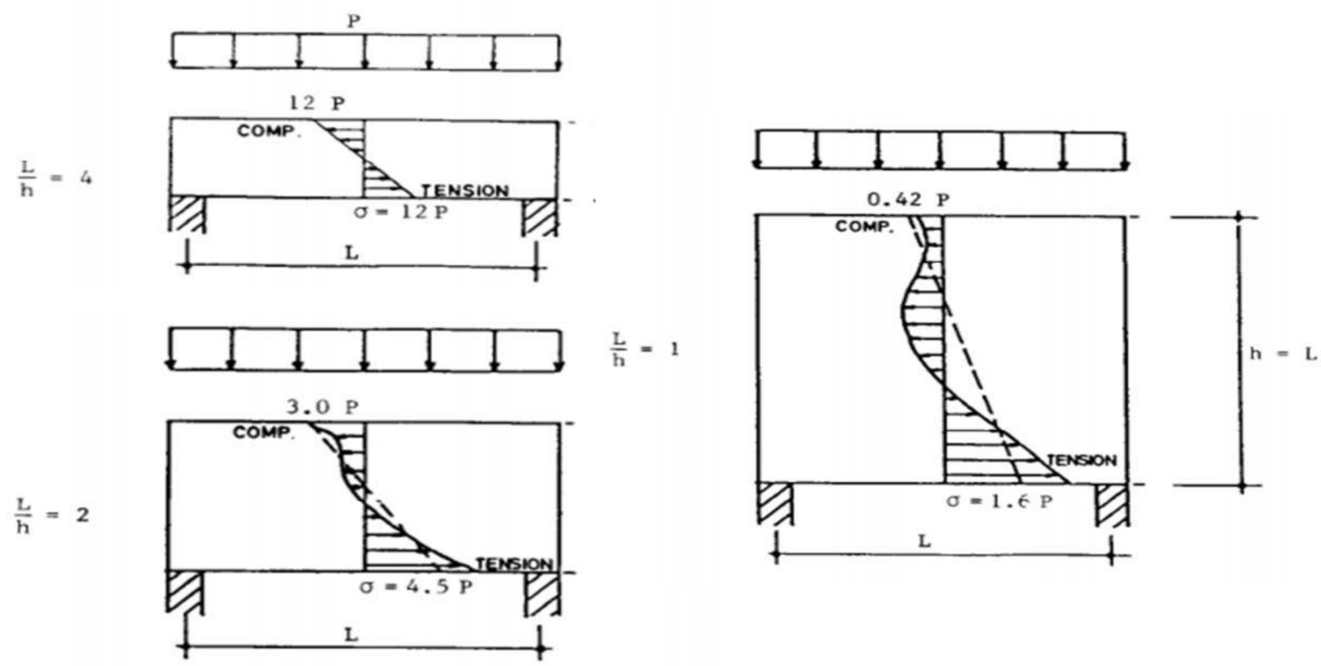


Figure 2-8: Nonlinear strain distribution for different *a/d* ratio (Lafta et al., 2016)

2.4 Summary

In this chapter, previous work done on pier caps was discussed, including experimental work as well as analytical work. The problems, and the solutions to the problems, in pier cap were discussed.

The literature reviews highlighted shear failure as prominent types of failure in deep beams. Tests included in references such as Kani (1964), Rogowsky et al. (1983), Denio et al. (1995), Young et al. (2000 and 2002), Higgins et al. (2008), etc. showed that deep beams fail in shear. Different analytical methods were used to predict the ultimate capacity of the beams. As discussed by Lafta et al. (2016), the strain distribution in deep beams are nonlinear and thus the strength of such beams should be estimated using the strut and tie methododlogy. Even for deep beam, the sectional method uses effective depth, M/V_d as important parameters affecting the ultimate capacity whereas it is found that *a/d* ratio is more important parameters for deep beams.

Different analytical methods were compared to estimate the ultimate capacity of deep beams and pier caps as shown by Kani (1964), Denio et al. (1995), etc. It was found that STM is better at predicting ultimate capacity. Other methods yielded highly conservative results (by a factor of three to four) and thus are not applicable methods for the analysis of deep beams.

As bridge pier caps have a short shear span and a small value for the shear span-to-depth ratio (*a/d*), the bridge pier caps behave as deep beams. Thus, under AASHTO provisions, the method that should be used for the analysis of pier caps is a strut-and-tie method or finite element method. STM implementation requires iterations and graphical representation for the model. A specialized knowledge is required for the improvement of STM. STM will be used for the development of a solution algorithm for analysis of bridge pier caps in the next chapter.

Chapter 3 The Deep Beam Theory: Strut-and-Tie Method

3.1 Introduction

The Strut-and-Tie Method (STM) is an analysis and design method where the internal stress distribution in a structure is idealized by a truss model. The truss model is known as a strut-and-tie model. STM is a generalization of the truss model (Schlaich et al, 1987). The use of strut-and-tie models was introduced, and has been used, since the nineteenth century. STM became popular and accepted for application after the work of Schlaich et al. (1987) although many researchers have contributed to the development and improvement of STM. Although STM has been included in AASHTO LRFD Bridge Specifications since 1994, it is a new concept to many structural engineers. AASHTO LRFD 2014 states that STM analysis and design is highly recommended for deep beams such as dapped beams, pile caps, pier caps, etc. Because of this, interest among engineers has grown, but due to the complexity of analysis and design, many designers and DOTs (Department of Transportation) are still not following STM.

The base theory for the in-practice analysis method, the sectional method, is the Euler-Bernoulli hypothesis, "Plane sections remain plane after bending." Deep beams are disturbed region (D-Region) and have nonlinear strain distribution. For D-region, deep beams, beams with openings, load discontinuity, etc., sectional design approaches are not valid. The STM method has been proposed for the analysis and design of deep reinforced concrete members where the sectional method provides conservative results. However, most bridge designers have not embraced the strut-and-tie model due to unfamiliarity with the design procedure, the inability to check the truss model's validity (without laboratory tests or a finite element model), the graphical representations, iterations, and the time it takes to complete each strut-and-tie model analysis and design (Nicholas et al, 2011).

STM is a truss model, and the truss analogy consists of struts, ties, and nodes. Ties represent the tension truss element; struts represent the compressive truss element; and the nodes are the connection of the truss analogy. AASHTO LRFD Bridge Design Specifications 2014 provides empirical formulations for determining the strength of the STM members. AASHTO checks nodal capacity and bearing capacity for truss nodes. AASHTO also requires the reinforcement development checks to ensure the tension rebar and stirrups are adequately developed affecting the strength of the STM member.

3.2 History of Strut-and-Tie Model

The idea of using truss models for the design of reinforced concrete beams was first proposed by Ritter (1899). He introduced the idea that beams can be analyzed as truss where rebar carries tension force and the concrete carries compressive force. Morsch (1909) used this idea to determine the shear reinforcement required for beams in flexure. A space truss model was used to analyze and design beams subjected to combined torsion and bending by Lampert et al. (1971).

The strut-and-tie method was globally recognized after the work of Schlaich et al. (1987). Their extensive research work presented a global set of procedures and rules for analyzing the truss model. Thurliman et al., Kong et al., Rogowsky, etc., contributed to the development and verification of STM. Adebar et al. expanded the application of STM to the pile cap 3D model. The strut-and-tie method was also adopted by Canadian Code for deep beams after the Collins and Mitchell work in 1986.

In 1994, STM was introduced into the AASHTO LRFD Bridge Design Specifications as an appendix for design and analysis of concrete members under certain conditions. In 2007, AASHTO made STM a recommended analysis methodology for D-Region and deep members. This developed an interest in the application of the method for design of new structures as well as analysis of existing structural members.

3.3 Applicability of STM

The Euler-Bernoulli hypothesis states that "Plane sections remain plane after bending." This hypothesis facilitates the flexural design of reinforced concrete structures by allowing a linear strain distribution of all loading stages, including ultimate flexural capacity.

St. Venant's principle states that "The localized effects caused by any load acting on the body will dissipate or smooth out within regions that are sufficiently away from the location of the load," as shown in **Figure 3-1**. St. Venant's principle suggests that the localized effect of disturbance such as a concentrated load or reaction will dissipate within one beam depth from the point of the disturbance. On this basis, disturbed regions (or D- regions) are assumed to extend one member-depth each way from the discontinuity.

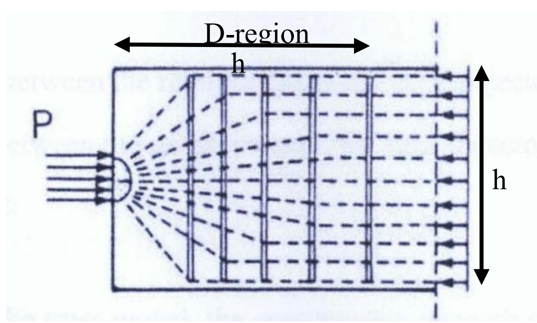


Figure 3-1: St. Venant's principle

It is found that the localized stresses and strains will dissipate enough at a distance equal to the height or depth of the beam (h). As shown in **Figure 3-2**, the D-region was approximated to extend a distance equal to the height of the beam (h) at the location of the supports. If the span of the beam is reduced such that the distance between the applied load and the end reaction is less than 2h, the disturbed regions overlap. Hence, the entire beam will be considered a D-region and the behavior of the beam will be strongly influenced by the disturbed flow of stresses.

In **Figure 3-2**, the light shaded portion represents the D-region corresponding to the support reaction, while the red arrow line indicates the extension of D-region due to load. Because of the intersection of D-region from support and the load, the entire beam is a D-region. For this case, the strut-and-tie method approach would be appropriate for the design. In some cases, the entire beam may not be a D-region as there could be a small section of Bernoulli region (or B region). For B-regions, the AASHTO LRFD Specifications permit the use of either the sectional method or the strut-and-tie method.

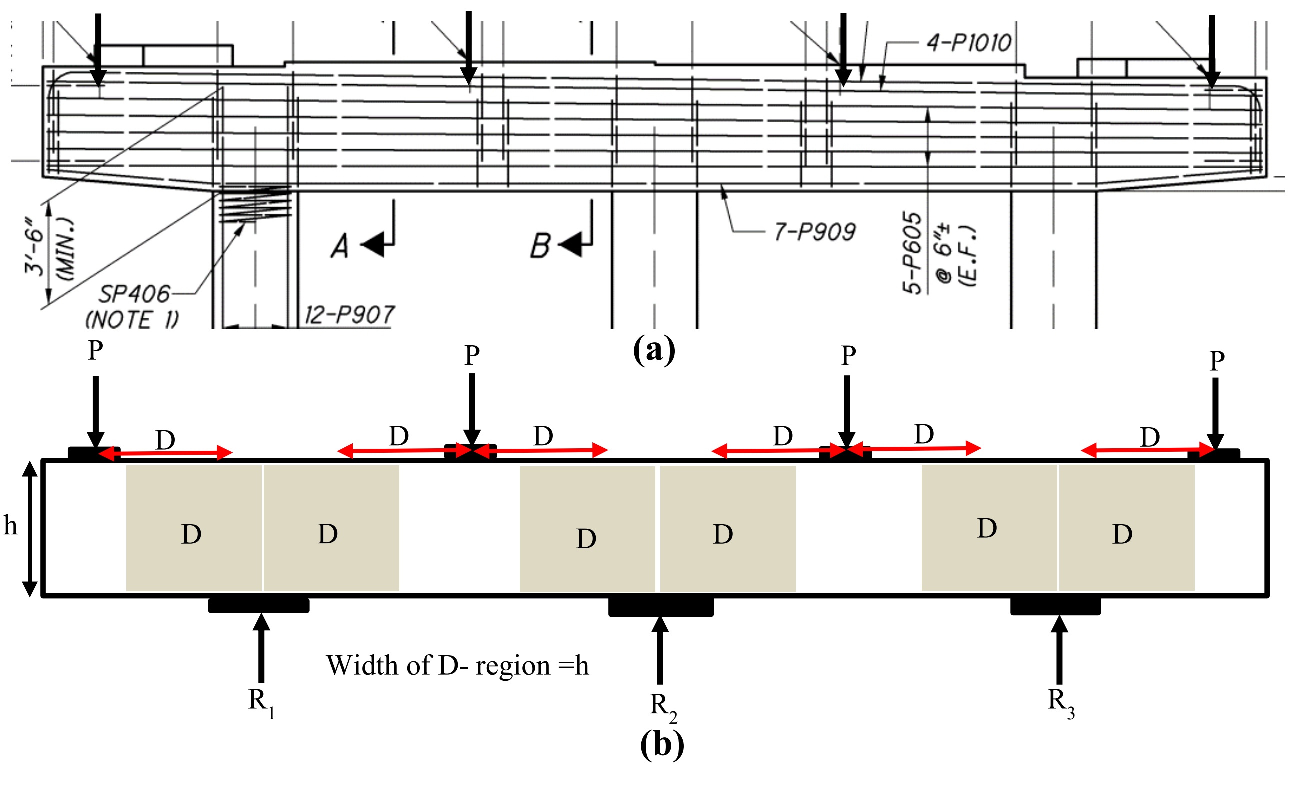


Figure 3-2: Illustration of D-region and B-region on pier cap of a typical cap

The design and analysis of flexural members can be done from the sectional method using Whitney's stress block based on the Euler-Bernoulli hypothesis. This is also valid for B-region. However, the region in which the Bernoulli hypothesis does not hold true (i.e., the strain distribution is nonlinear) is known as D-region. For long, slender beams such as girders, D-region has a small influence on the behavior of the beam and is ignored. Therefore, for such beams, STM is not needed. Girders can be analyzed using the sectional method. But for deep beams such as bridge pier caps, there is a high influence of D-region and STM is the recommended method. In reference to the various experiments conducted on the comparison of different analysis methods, STM provided the better prediction for deep members. AASHTO code states that STM is an appropriate analysis method used for D-region or any beams for which *a/d* ratio is less than or equal to two.

3.4 Elements of the Strut-and-Tie Model

The strut-and-tie method is a conceptual framework in which the internal stress distribution in a structure is idealized by a truss model. It uses a hypothetical equivalent simple uniaxial truss to represent the load path in the structural concrete member. Each uniaxial load path is considered a member of the truss model of STM. This truss analogy consists of a system of struts, ties, and connecting nodes.

Figure 3-3 shows a simple strut-and-tie model applied to a simply supported deep beam. It shows a load (P) applied to the beam where R_1 and R_2 are the corresponding reaction on two column bearing supports. The load can be assumed to directly transfer to the point of application to the support. In this figure, the lighter shaded regions represent concrete compressive struts, the steel reinforcing bar represents tensile tie, and the dark shaded regions represent nodal zones.

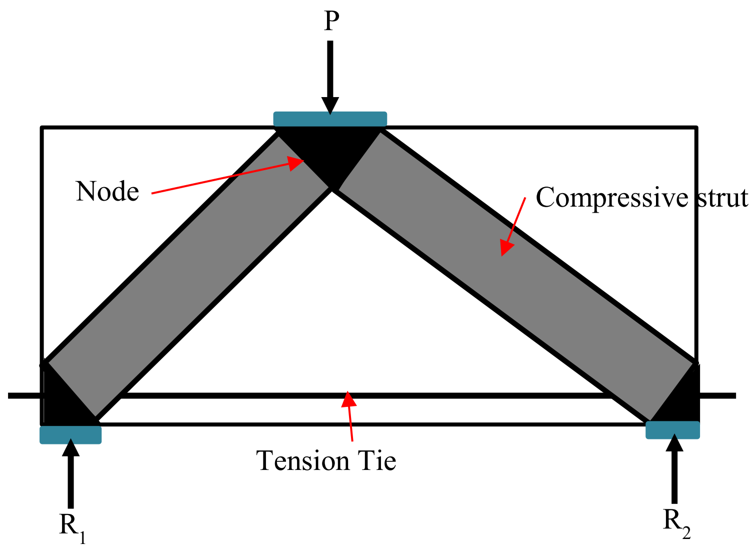


Figure 3-3: Strut-and-tie model in a beam

The applied forces are in equilibrium with a system of forces existing in concrete compression struts and steel tension ties. STM is based on the lower-bound theorem of plasticity. A lower-bound theorem of plasticity states "a load system based on statically admissible field which nowhere violates the yield condition of lower-bound to the collapse load" (Muttoni, Schwartz and Thuerlimann, 1997). A strut-and-tie model is implemented by laying out a truss that carries the applied load to the supports. Struts and ties meet at nodal regions. The sizes of the struts, ties, and nodal regions are based on equilibrium with the applied loads and the size of the bearings at the nodal regions.

Figure 3-4 shows a pier cap that is idealized by truss model. In this figure, the loads are transferred from girders to the columns (piers). The truss element in magenta shows the tension tie, whereas the blue represents the strut. The node at the connection of struts and ties is shown in black.

The girder load can either be directly transferred to the column or with the help of a vertical tie. Both mechanisms are shown in the figure below. The mid located girder loads are directly transferred to the column on one side, while on the other side they are transferred with the help of vertical tie. An effective truss model indirectly depends upon the angle of inclination of struts. Therefore, if an angle is too low, a vertical tie should be used to transmit the load to increase the angle. If the angle is not too low, the load should be directly transferred to the column.

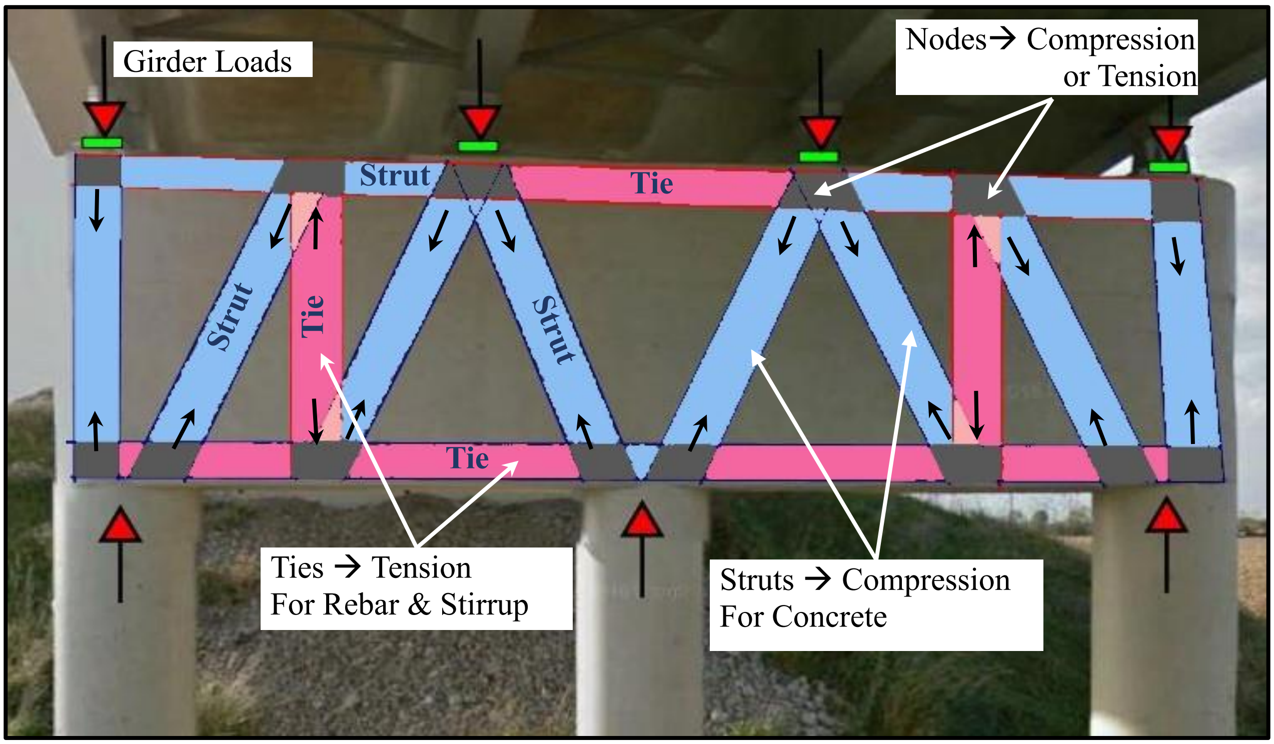


Figure 3-4: Strut-and-tie model idealization in real-life scenario

Table 3-1: Strut-and-tie model elements and their representation

Element	Nature	Represents
Struts	Compression member	Concrete
Ties	Tension member	Rebar and stirrup
Nodes	Connection (joint)	Concrete

3.4.1 Struts

The element of the strut-and-tie truss model which represents the uniaxial compressive stress is known as a strut. Struts are the compression members and represent concrete stress fields whose principal compressive stresses are predominantly along the centerline of the strut. The diagonal struts represent the line of cracking in the concrete.

For the analysis and design of pier caps, the compressive forces in strut-and-tie model are carried by concrete struts. Struts which have a steel reinforcement are reinforced struts. These reinforced struts are mainly found when the struts are in the line of horizontal ties that represents longitudinal steel.

3.4.2 Ties

The element of the strut-and-tie truss model which represents the uniaxial tensile stress is known as ties. The tensile forces in the strut-and-tie model are carried by longitudinal and transverse reinforcement or any special detail reinforcement. They occasionally represent prestressing tendons or concrete fields with principal tension. Ties are the tensile members and represent one or several layers of flexural reinforcement in the deep section. The locations of the tension ties normally are defined at the centroid of the reinforcement.

3.4.3 Nodes

Nodes are the region of connections or joints of struts and ties in a valid strut-and-tie model. Nodal zones are formed where the ties, struts, and exterior loads or reaction intersect. The node indicates a change in the direction of forces meeting at a point.

Different types of nodes as shown in **Figure 3-5,** depending upon the combination of the number of compressive struts (C) and tensile ties (T) meeting at the node. The nodes can be categorized as:

CCC-nodes: This occurs where there is an intersection of three compressive struts.

CCT-nodes: This occurs at the intersection of two or more compressive struts and a tension tie.

CTT-nodes: This occurs at the intersection of a compressive strut and two or more tension ties.

TTT-nodes: This occurs where there is an intersection of three or more tensile ties.

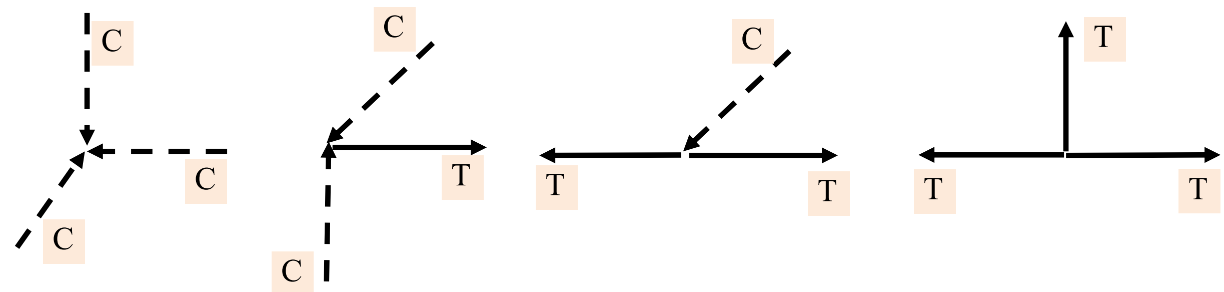


Figure 3-5: Types of nodes

3.5 Provision of AASHTO code for STM

From Section 5.6.3.1 of AASHTO LRFD 2014: "The strut-and-tie model should be considered for the design of deep footing and pile caps or other situations in which the distance between the centers of applied load and the supporting reactions is less than about twice the member thickness."

The factored resistance, P_r of struts and ties are

$P_r = \phi P_n$ …………..AASHTO LRFD 2014, Equation 5.6.3.2-1

where:

P_n = nominal resistance of strut or tie

ϕ = resistance factor for tension or compression

=0.9 for tension-controlled concrete sections

=0.7 for compression-controlled concrete sections

3.5.1 Strength of Struts

The nominal resistance of an unreinforced compressive strut (AASHTO 2014, Section 5.6.3.3) shall be taken as:

$P_n = f_{cu} A_{cs}$ ………….. AASHTO LRFD 2014, Equation 5.6.3.3.1-1

where:

P_n = nominal resistance of a compressive strut (kips)

f_{cu} = limiting compressive stress (ksi)

A_{cs} = effective cross-sectional area of strut (in.^{2}) as shown in **Figure 3-6**.

The effective cross-sectional area of strut depends upon the anchorage condition and size of the bearing. It can be calculated as the width of the strut times the thickness of pier cap for the strut anchored by bearing and reinforcement and the strut anchored by bearing and strut.

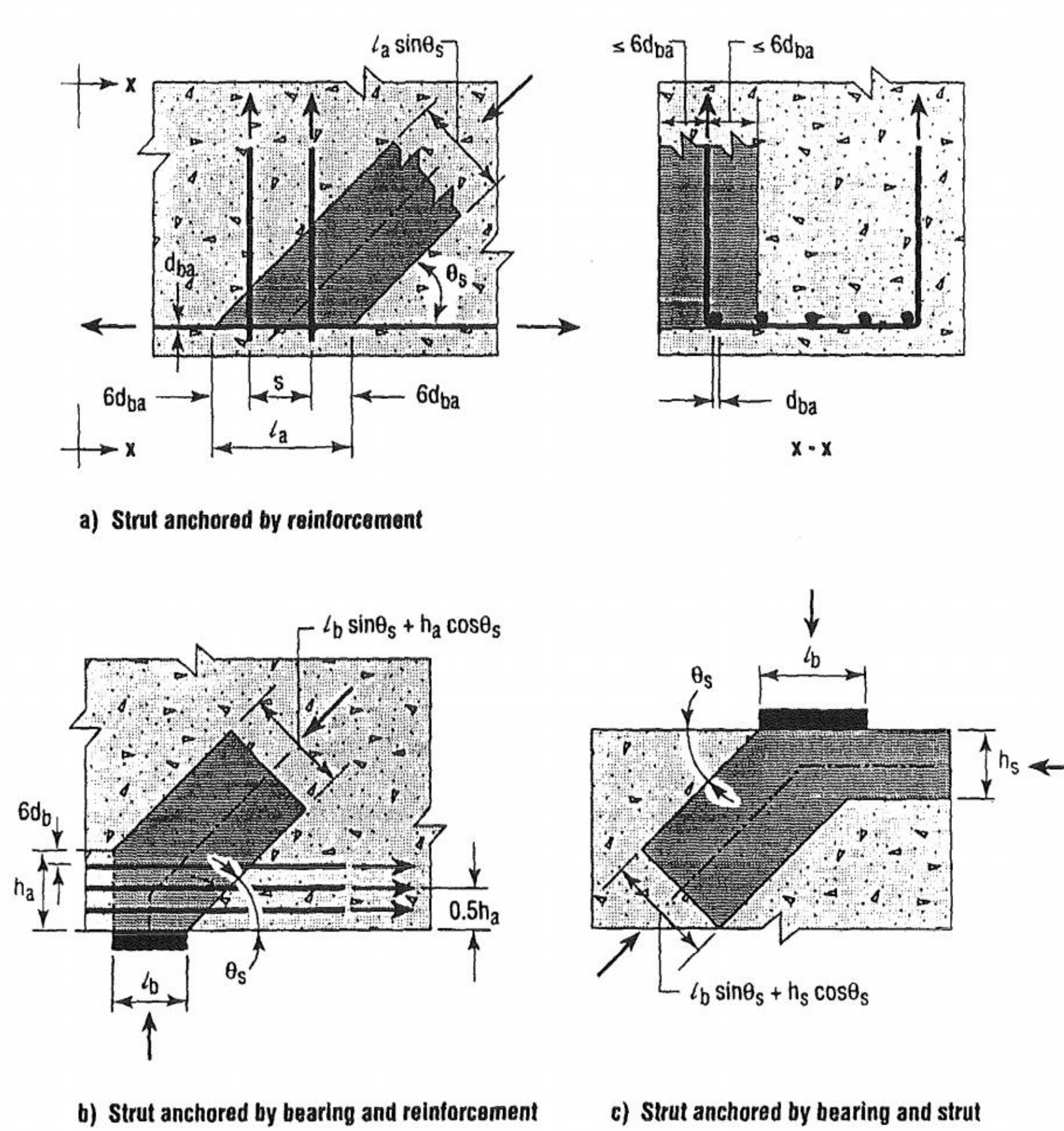


Figure 3-6: Effective area of struts. (AASHTO Figure 5.6.3.3.2-1)

For a strut anchored by reinforcement, the effective concrete area may be considered to extend up to a distance of six bar diameters from the anchored bar, as shown in **Figure 3-6**.

Limiting compressive stress, f_{cu} is calculated as:

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_l} \leq 0.85 f_c'$$ ………….. AASHTO LRFD 2014, Equation 5.6.3.3.3-1

in which,

$\varepsilon_l = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s$ ………….. AASHTO LRFD 2014, Equation 5.6.3.3.3-2

α_s =smallest angle between the compressive strut and adjoining tension ties

For a reinforced strut, the nominal resistance of a reinforced compressive strut (AASHTO 2014, Section 5.6.3.3.4) shall be calculated as:

$P_n = f_{cu}A_{cs} + f_y A_{ss}$ ………….. AASHTO LRFD 2014, Equation 5.6.3.3.4-1

where:

f_y = yield strength of longitudinal rebar (ksi)

A_{ss} = area of rebar in (in.^{2})

3.5.2 Strength of Ties

Proper anchorage should be provided from the inner face of the nodal zone. The nominal strength of the tension tie (AASHTO 2014, Section 5.6.3.4.1) is calculated as:

$P_n = f_y A_{st} + A_{ps}\left[f_{pe} + f_y\right]$ ………….. AASHTO LRFD 2014, Equation 5.6.3.4.1-1

where:

A_{st} = total area of longitudinal mild steel reinforcement in the tie (in.^{2})

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

f_y = yield strength of mild steel (ksi)

f_{pe} =stress in prestressing steel after losses (ksi)

3.5.3 Strength of Node Regions

The concrete compressive stress in the node regions of the strut (AASHTO 2014, Section 5.6.3.5) should not exceed:

For node regions bounded by compressive struts and bearing areas (CCC Nodes): $0.85\Phi f'_c$

For node regions anchoring a one-direction tension tie (CCT): $0.75\Phi f'_c$

For node regions anchoring tension ties in more than one direction (CTT or TTT Nodes): $0.65\Phi f'_c$

3.5.4 Development Length Requirements

The end tie in a strut-and-tie model should be anchored properly in order to develop the tensile stress/force in the tie. The main longitudinal rebar must be developed/anchored a specific length beyond the nodal point. The available development length is measured from the inner junction of strut and tie width. It can also be measured at the centroid of reinforcement where the tie leaves the intersection of effective strut width and the effective tie width. The required development length is calculated in reference to AASHTO code. The development length calculation for straight bar follows Section 5.11.2.1 of AASHTO LRFD 2014, while the development length for hook is based on Section 5.11.2.4 of the same code. The development length is equal to basic development length times the modification factors.

In cases where the anchorage is not properly provided, the strength of tie reduces by a factor of deficient to full anchorage to the required length of anchorage.

In the analysis of the pier cap, it is assumed that adequate development length is provided while lapping of rebar in the midsection. The only point to be checked is the ties at the end of the beam. Development length will be discussed in the next chapter.

3.5.5 Serviceability Requirements

In the AASHTO LRFD Bridge Design Specifications 2014, crack control reinforcement is required if a strut-and-tie method is used. The reinforcement ratio in both the longitudinal and transverse direction must be at least 0.003. These details are meant to improve the serviceability of members designed using a strut-and-tie analysis to limit the crack width and to ensure a minimum ductility for the member. The crack control reinforcement follows Section 5.6.3.6 of AASHTO 2014.

$$\frac{A_v}{b_w s_v} \geq 0.003$$ ………….. AASHTO LRFD 2014, Equation 5.6.3.6-1

$$\frac{A_h}{b_w s_h} \geq 0.003$$ ………….. AASHTO LRFD 2014, Equation 5.6.3.6-2

where:

A_h and A_v = Total area of horizontal and vertical crack control reinforcement, respectively (in.^{2}) as shown in **Figure 3-7.**

b_w = width of member's web (in.) as in **Figure 3-7.**

s_h and s_v = spacing of horizontal and vertical crack control reinforcement, respectively (in.) as shown in **Figure 3-7.**

The maximum spacing of the bars in these girds (horizontal and vertical) should not exceed the smaller of d/4 and 12.0 inches.

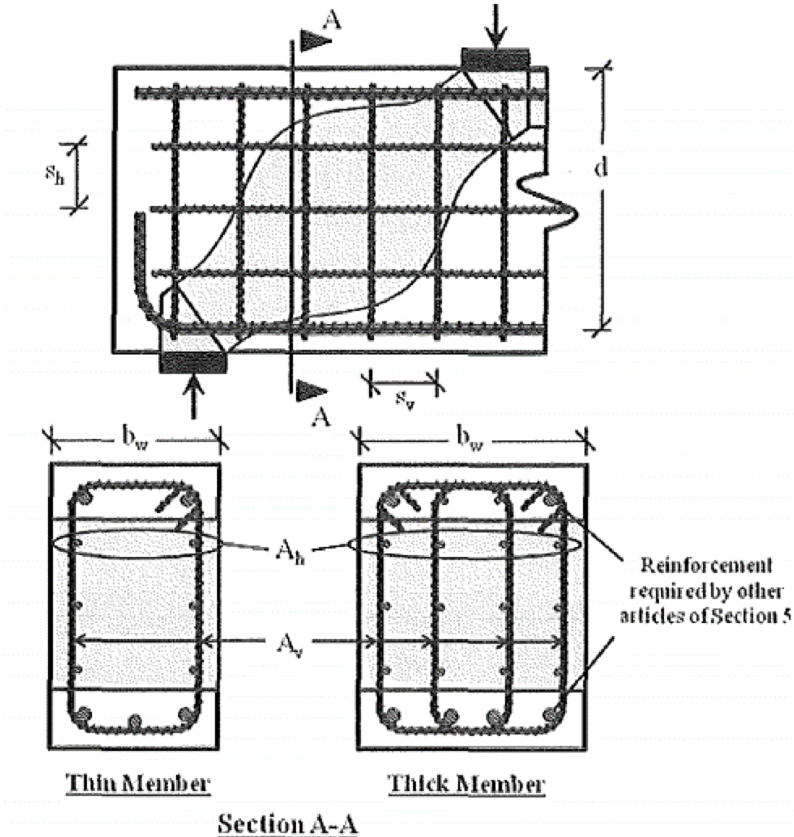


Figure 3-7: Distribution of crack control reinforcement (AASHTO Figure C5.6.3.6-1)

In the analysis of pier caps, which is not design work, crack control also known as skin reinforcement percentage is checked. In reference to AASHTO LRFD 2014, the crack control reinforcement is referred to as a secondary check which does not affect any strength calculations of STM elements. This requirement is important in the design of pier caps using the STM towards redistribution of stresses.

Chapter 4 Development of Spreadsheet, STM-CAP

4.1 Introduction

The Strut-and-Tie Method (STM) was used to develop a spreadsheet program named STM-CAP or Strut-and-Tie Method for pier CAPs. This chapter covers the theory, formulations, development process, and guidelines for using STM-CAP.

STM-CAP is a spreadsheet program for the analysis of deep pier caps subjected to the static girder loads. It is divided into several sections. The initial sections include the input parameters while the subsequent sections present the analysis results. One of the major objectives was to use graphical representation as a part of the analysis process to assist the analyst in understanding the system and be able to identify potential errors. Various error messages are displayed based on checks and conditions listed in Appendix C. The input, calculation details, and the output are explained in this chapter in sequential order.

STM-CAP is designed to analyze both symmetrical and asymmetrical pier caps. For symmetrical pier caps, it can analyze pier caps with up to eight columns. It can analyze asymmetrical pier caps with up to four columns. For symmetrical pier caps, the input and output of the analysis is limited to the centerline, while for asymmetrical pier caps, the analysis shows the full pier cap details.

It is recommended that the strut-and-tie method be used for deep members. Therefore, STM-CAP initially determines if a pier cap is deep in order to check applicability of the STM for the selected pier cap. Based on the load and geometry inputs, STM-CAP calculates the shear span-to-depth ratio for every region and if the ratio is less than 2.0, it is considered deep region. If the beam is deep, further inputs are made. If the beam is not deep, the user has the choice to stop or continue the analysis procedure.

STM-CAP models the pier cap with a truss model consisting of ties, struts, and nodes. It considers two types of ties—horizontal ties for main bars and vertical ties for stirrup ties. If required, the truss model can be adjusted with a combination of vertical ties. Inclined struts and horizontal reinforced struts, if applicable, are considered for modeling concrete compression. STM-CAP determines the load, capacity and utilization ratio (ratio of load to capacity) for each element of STM. Using the utilization ratio, overloaded bridges can be categorized, and limited strengthening funds can be directed to the caps with the largest utilization ratios. If overloaded or failed, STM-CAP indicates the governing failure mode and location of the failure. This will facilitate strengthening cap beams at the correct locations.

STM-CAP performs the reinforcement development checks to ensure that the longitudinal bars are adequately developed. Suitable reductions are made for the tension tie capacity. It also performs bearing checks, nodal checks, and secondary checks, e.g., crack control reinforcement checks as required by AASHTO.

STM-CAP uses factored loads and factored material resistances and preforms an LFRD analysis. A utilization ratio 1.0 indicates that the cap has a sufficient factor of safety as per the LFRD method. Although not the main intention, STM-CAP may also be used, at the discretion of the user, to conduct a nominal analysis by adjusting resistance factors and the load input to un-factored values.

4.2 General Information

Input and Output Cells: The input and output cells are color coded. As shown in **Figure 4-1,** orange represents the input cells while the gray background with bold letter represents the output cell.

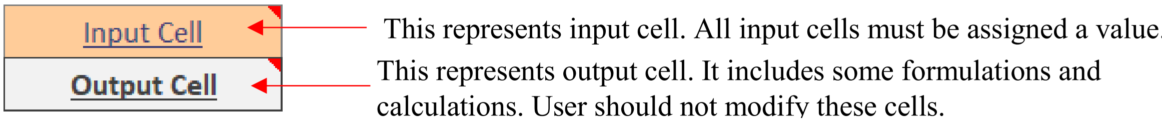


Figure 4-1: Input and output cells

Reset All Data: This button makes all input *'zero'* and deletes all input and output graphics. It is recommended to click on '*Reset All Data*' whenever a new pier cap is being analyzed. The button is shown in **Figure 4-2.**

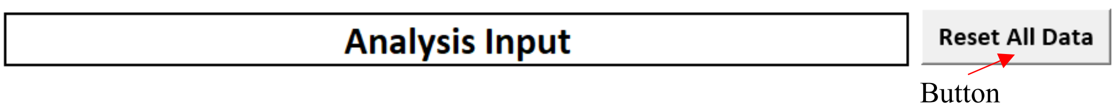


Figure 4-2: '*Reset All Data*' button

Bridge Details: This input includes basic information about the bridge to be analyzed. All input in this section can be found in the template of engineering drawings. The input for STM-CAP are shown in **Figure 4-3.**

Bridge Details:

Bridge Name:	MOT-075-XXX	Pier Number:	XXXX
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XX-XX-XX

Figure 4-3: Bridge details screenshot of the STM-CAP

Input Guidelines

Bridge Name: A unique name is assigned to each bridge.

SFN Number: This stands for the *Structure File Number*. It is a unique number provided for every bridge.

PID Number: PID number is assigned to each bridge.

Pier Number: While there may be many piers for a single bridge, only one pier can be analyzed at a time. Therefore, each pier cap should be given a meaningful name.

Designer: Input the name of the person analyzing the pier cap.

Date: Input the date on which the analysis is performed.

4.3 Section 1: Total Number of Columns (i.e., Piers)

In this section, the total number of supporting columns for a specified *Pier Number* is given. If the pier cap being analyzed is asymmetrical, a check mark is provided. A pier cap may be asymmetrical due to geometry, load positioning, load values, reinforcement, etc. Several visual basic application (VBA) codes and calculations depend upon the input of the total number of piers. A sample input for Section 1 is shown in **Figure 4-4**.

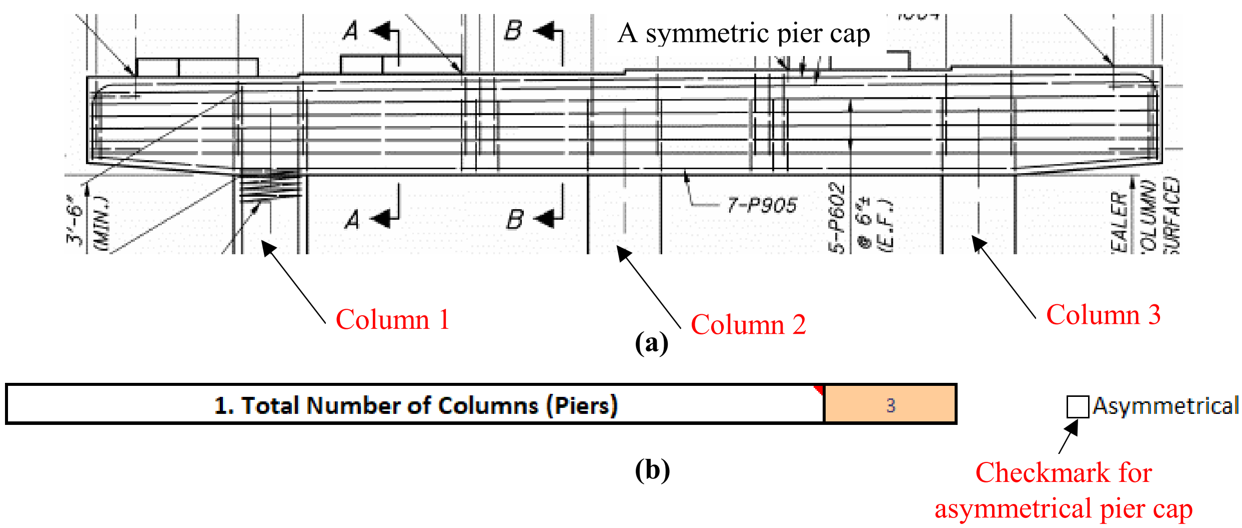
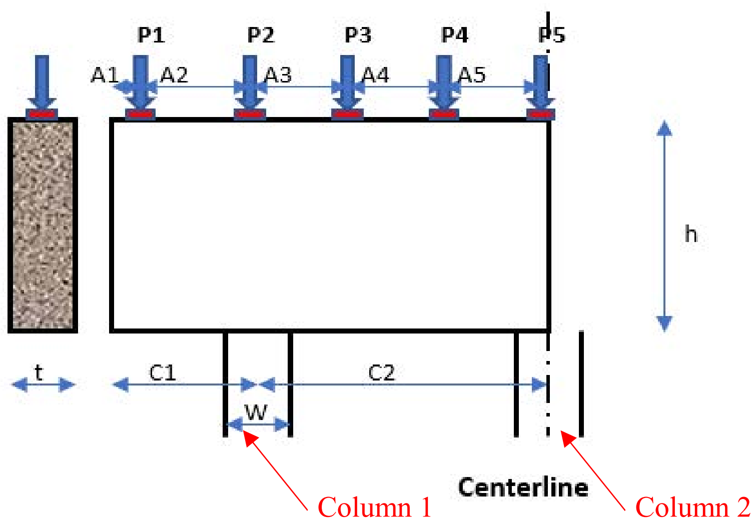


Figure 4-4: Total number of columns

4.4 Section 2: Generate

'2. Generate' is a form control button which runs VBA codes in the background to generate a sketch depending upon the number of piers. The drawing is sketched by inserting and formatting different shapes with the help of VBA. The further input in Section 3 and Section 4 are based on the generated sketch. Also, the input fields are highlighted depending upon the number of piers. For a higher number of piers, there is more input to be made and vice versa. A sample Section 2 is shown in **Figure 4-5.**

(a)



(b)

Figure 4-5: (a) *'2. Generate'* button; **(b)** automated drawing showing the input variables

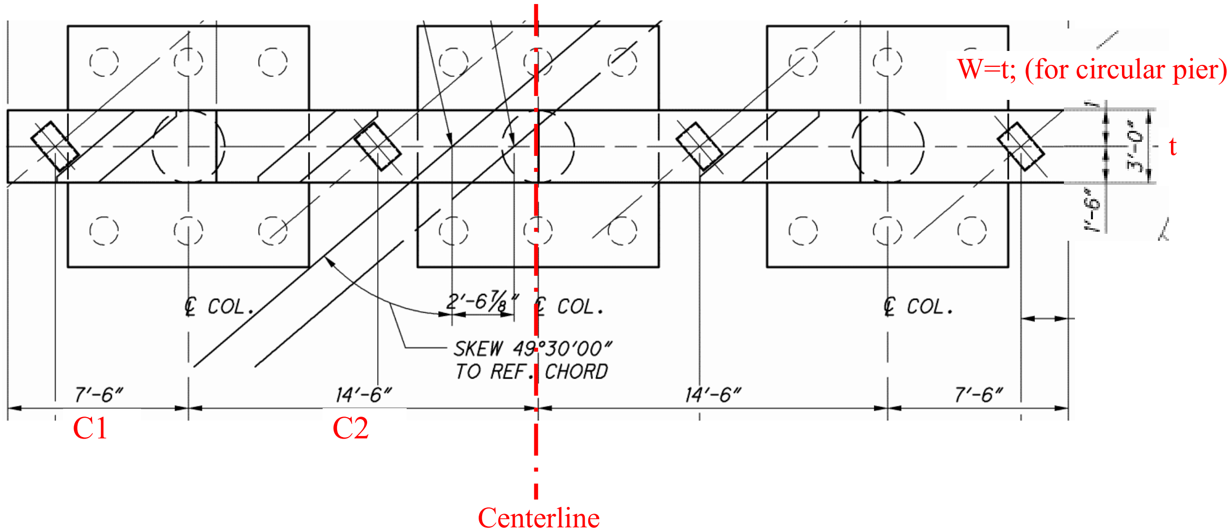
4.5 Section 3: Geometry Details

This section deals with the geometrical details of the pier cap. For symmetrical pier caps, the input variables are shown only up to centerline, but for asymmetrical pier caps, full pier cap input variable is shown. The variables have been explained in the text as shown in **Figure 4-6** as well as labeled in the previous generated drawing in **Figure 4-5**. All the variables can be found in the design drawings. A sample input for Section 3 is shown in **Figure 4-6.**

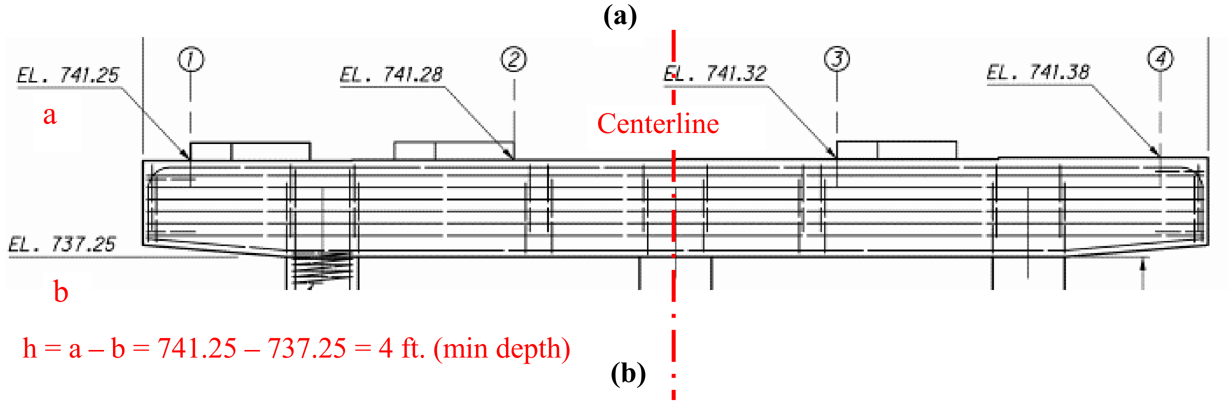
4.5.1 Input Guidelines

As stated earlier, this is a symmetrical pier cap and only a half section of the pier cap is analyzed. Thus, the geometry details input is limited to the centerline of the bridge pier cap. The variables *C1, C2, C3, C4, C5, W, h, t,* etc., that need to be input is shown in **Figure 4-5 (b).** These values can be found in the engineering drawings.

Figure 4-6 (a) and **(b)** shows the screenshots of engineering drawings that provide important input parameters for the geometry details. The input requires the centerline distances (e.g. *C1, C2, C3,* etc.) in feet and inches; other geometry details are input in inches. The STM-CAP input screen is shown in **Figure 4-6(c)**. The pier type is selected from the drop-down list as either circular or rectangular.



(a)



(b)

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	7 ft	6 in	90 in
Distance from center of first column to center of second column (C2)	14 ft	6 in	174 in
Column width (W)	36 in	Circular	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

(c)

Figure 4-6: (a), **(b)** Engineering drawings capture; **(c)** STM-CAP input for geometry details

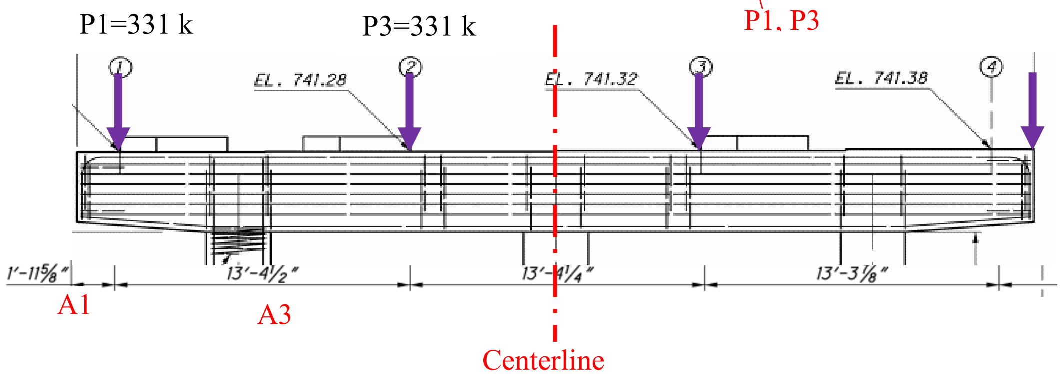
4.6 Section 4: Factored Loads and their Position

Section 4 deals with the input of girder loads that are transferred to the cap beam and the spacing between each girder load. The load corresponds to the ultimate (factored) load based on LRFD principles. The self-weight of the pier cap is not automatically considered by STM-CAP. It can be added manually to the applied loads. Applying the entire self-weight from the top of the beam may be conservative. Only half may be applied and the other half (representing the lower half of the beam) may be assumed to directly go into the column supports without stressing the struts and ties on the condition that there are no bearing overloads at the top of column supports. In **Figure 4.7 (a)**, it is considered that the total load (331.3 kips) is the factored load obtained by summing up the given factored dead load (215.5 kips) and factored live load (115.8 kips). In cases where these loads are unfactored, suitable load factored as per AASHTO LRFD should be used.

There are three options available for Section 4 input:

Option 1: If each factored girder loads and spacing between girders is the same, very few inputs are to be made as shown in **Figure 4-7 (b).** Input the first three fields, the edge distance, the girder spacing, and the factored load, and then press *'Generate Load Table'* as shown in **Figure 4-7**. This will populate the table as shown in **Figure 4-7(c).**

BEARING LOCATION	BEARING TYPE	NO. REQ'D. PER LOCATION	DEAD LOAD (KIPS)	LIVE LOAD (KIPS)	TOTAL LOAD (DL+LL) (KIPS)	Lb (inches)	Wb (inches)	tb (inches)
REAR ABUTMENT	EXP.	8	107.1	98.1	205.2	11	18	1½
PIER 1	EXP.	8	216.5	114.2	330.7	13	21	2 1/16
PIER 2	EXP.	8	215.5	115.8	331.3	13	21	2 1/16
PIER 3	EXP.	8	152.9	101.3	254.2	13	18	2 1/16
FORWARD ABUTMENT	EXP.	8	90.9	95.5	186.4	11	18	1½



(a)

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	0 in	24 in
Spacing between the girders	13 ft	4 in	160 in
Factored Load	331 k		

(b)

Generate Load Table

Factored Load		Distance			
P1	331 k	2 ft	0.0 in	24.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	331 k	13 ft	4.0 in	160.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5

(c)

Figure 4-7: (a) Engineering drawings capture; **(b)** option 1 input **(c)** populated table

Option 2: If the spacing between the girders is the same but the factored girder loads are different, the process is the same as *Option 1.* After the table is populated, change the load values in the table **Figure 4-7(c)** to the new factored girder loads.

Option 3: If the spacing between the girders is different, then the input should be done manually based on regions as shown in **Figure 4-8**. For each individual column, there are three regions: left, center, and right. The left regions start either from the edge of a pier cap (e.g., R1) or from the mid-point of a span (e.g., R4) to the first edge of the column. The center regions are the regions just above the columns (e.g., R2, R5). The right regions range from the end of the column (end of the center region) to the mid-point of the next span. The load in Region 1 is referred as P1 and so forth. If any of the regions do not have a load, input a value of zero. Also, the corresponding distance (An) should be input as zero.

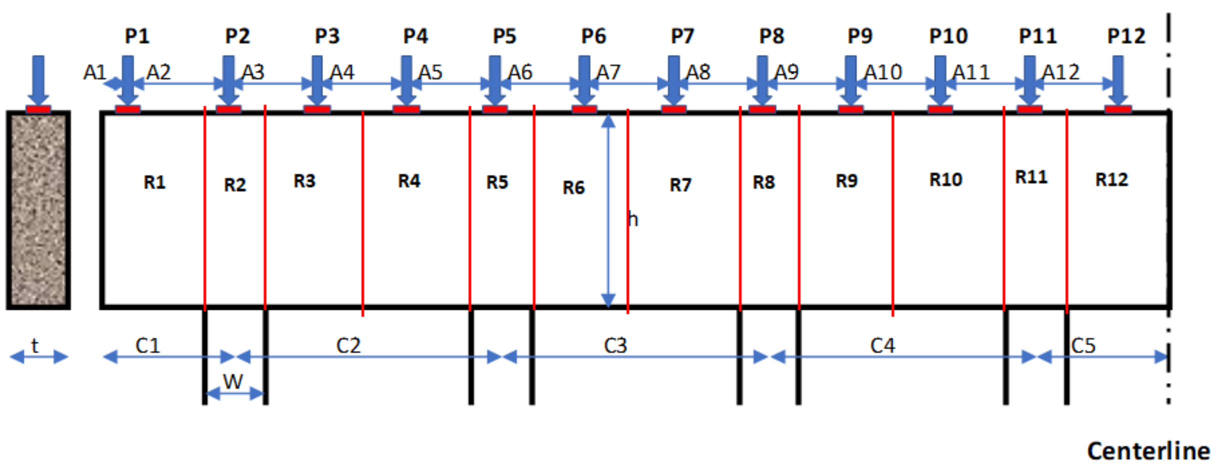
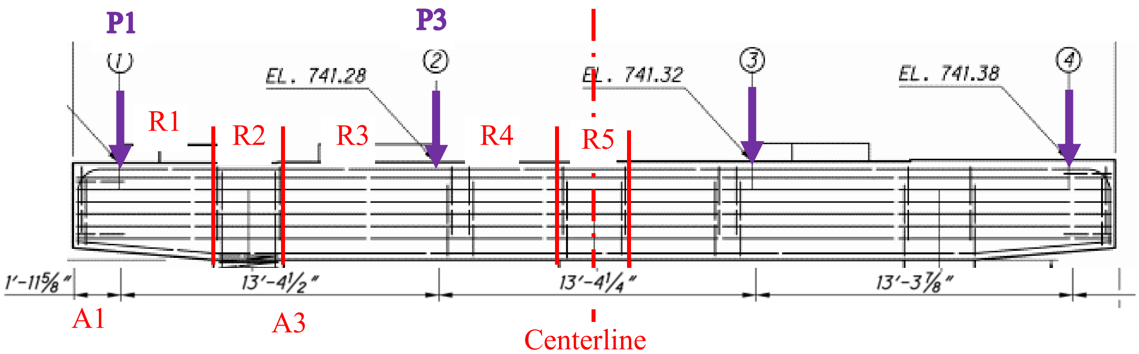


Figure 4-8: Pier cap showing the regions



Factored Load		Distance			
P1	331 k	2 ft	0.0 in	24.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	331 k	13 ft	4.0 in	160.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5

Figure 4-9: STM-CAP input for Section 4

In **Figure 4-9**, there is no load in Region 2; therefore, P2 = 0 and A2 = 0. Similarly, P5 and A5 equal zero and so on. Region 3 and Region 4 might be difficult to differentiate; therefore, the load can be input in either Region 3 or Region 4.

The distances (*A1, A2, A3*, etc.) are between the loads. As stated earlier, if any load (Pn) is zero, then the corresponding distance (An) should also be input as 'zero'. The first distance would be the distance from the end of the cantilever to the girder load; other distances are the spacing between the girders, which can be referred to as girder spacing.

4.7 Section 5: Generate

'5. Generate' is a form control button, which runs VBA codes in the background to generate a sketch depending upon the input made for geometry details and load details. The drawing is sketched by inserting and formatting different shapes with the help of VBA based on the input of Section 3 and Section 4. A message is displayed to confirm the accuracy of the generated drawing.

If there is a mistake, re-input of Section 3 and Section 4 is required and generate again. A sample *'2. Generate'* button and informatory sketch depending upon input made in previous sections is shown in **Figure 4-10.**

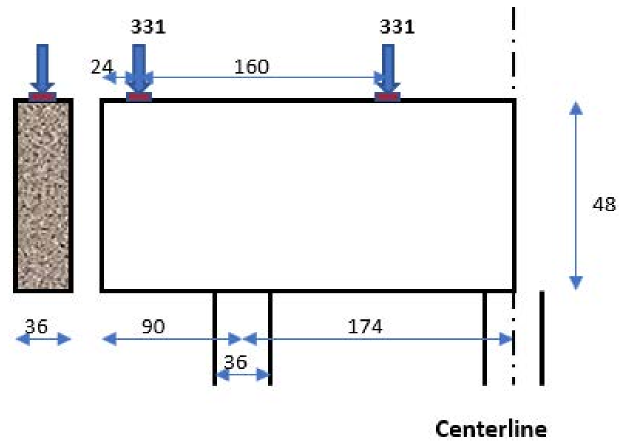


Figure 4-10: Generated sketch based on input

4.8 Section 6: Check whether the Pier Cap is Deep

This output section indicates if the pier cap is deep or not. If the shear span-to-depth ratio (*a/d*) is less than 2.0, the region is referred to as a deep region, as per AASHTO LRFD 2017 Bridge Design Specifications. Every region can be *Deep*, *Slender,* or *Zero* depending on the load positions and the geometry details.

The input made in Section 3 and Section 4 is used to calculate the shear span (*a*), which is the distance of the load to its corresponding reaction. The depth in the shear span-to-depth ratio is the effective depth, which is assumed to be 0.9 times the total depth. Even if most regions are slender, the pier cap can be treated as deep pier cap given that the deep beam analysis tends to give conservative results for slender regions. To check if the whole pier cap is deep, the *a/d* of the pier cap is calculated as mean of *a/d* ratios of individual regions. Instructions are displayed if the pier cap is deep; otherwise, the sectional analysis methods should be used for slender cap beams. A sample deep beam check is shown in **Figure 4-11.**

6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	60.33 in	1.40	Deep Region
R2	0.00 in	0.00	Zero Region
R3	81.67 in	1.89	Deep Region
R4	71.00 in	1.64	Deep Region
R5	0.00 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

Instruction

Figure 4-11: STM-CAP results for deep regions

4.9 Section 7: Material Properties

These fields are required while calculating the capacity of the pier cap. A sample input for Section 7 is shown in **Figure 4-12 (c).**

4.9.1 Input Guidelines

This section includes the input of properties of concrete and rebar. These properties can be found in the engineering drawings. The material properties from drawings and their corresponding STM-CAP input screenshot is shown in **Figure 4-12**.

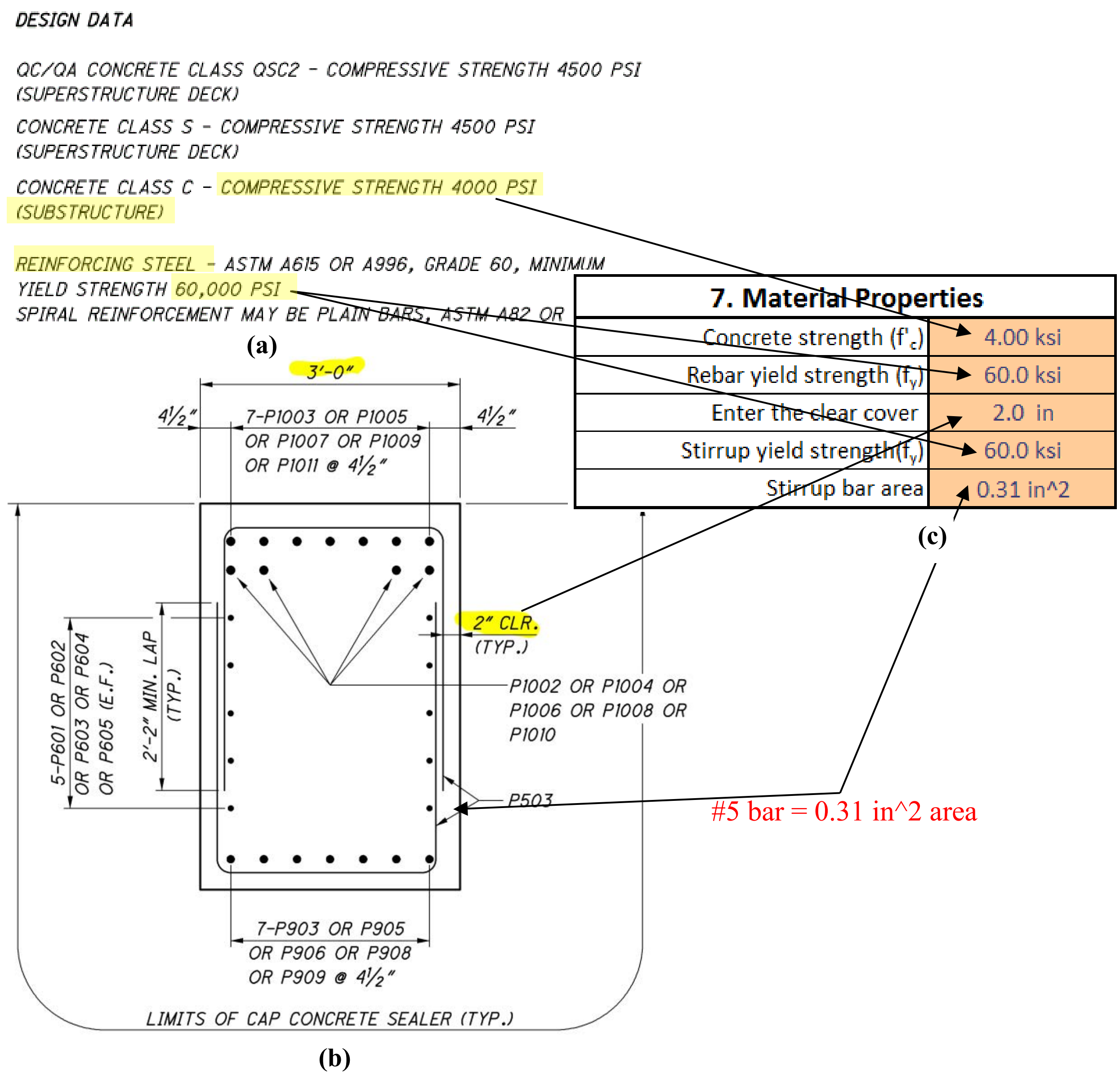


Figure 4-12: (a) Material properties; **(b)** cross-sectional view; **(c)** STM-CAP input

4.10 Section 8: Resistance Factors

The resistance factor input is the material resistance factors and the nodal capacity multipliers. These factors are used for the determination of the factored capacities of the STM members and nodes. The resistance factors are based on Section 5.5.4.2 of AASHTO LRFD 2017. Similarly, the nodal compressive strength multipliers are based on Section 5.8.2.5.3 of AASHTO LRFD 2017. A sample of currently used resistance factors is shown in **Figure 4-13**. These factors can be modified if the new editions of the code requires different values. Alternately, the concrete and rebar resistance factors may be input as 1 to determine the nominal capacity of the pier cap.

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebar	0.9
For stirrup	0.9
CCC v-factor for the bearing and back face	0.85
CCT v-factor for the bearing and back face	0.7
CTT v-factor for the bearing and back face	**0.65**

Figure 4-13: Sample resistance factor from AASHTO, 2017

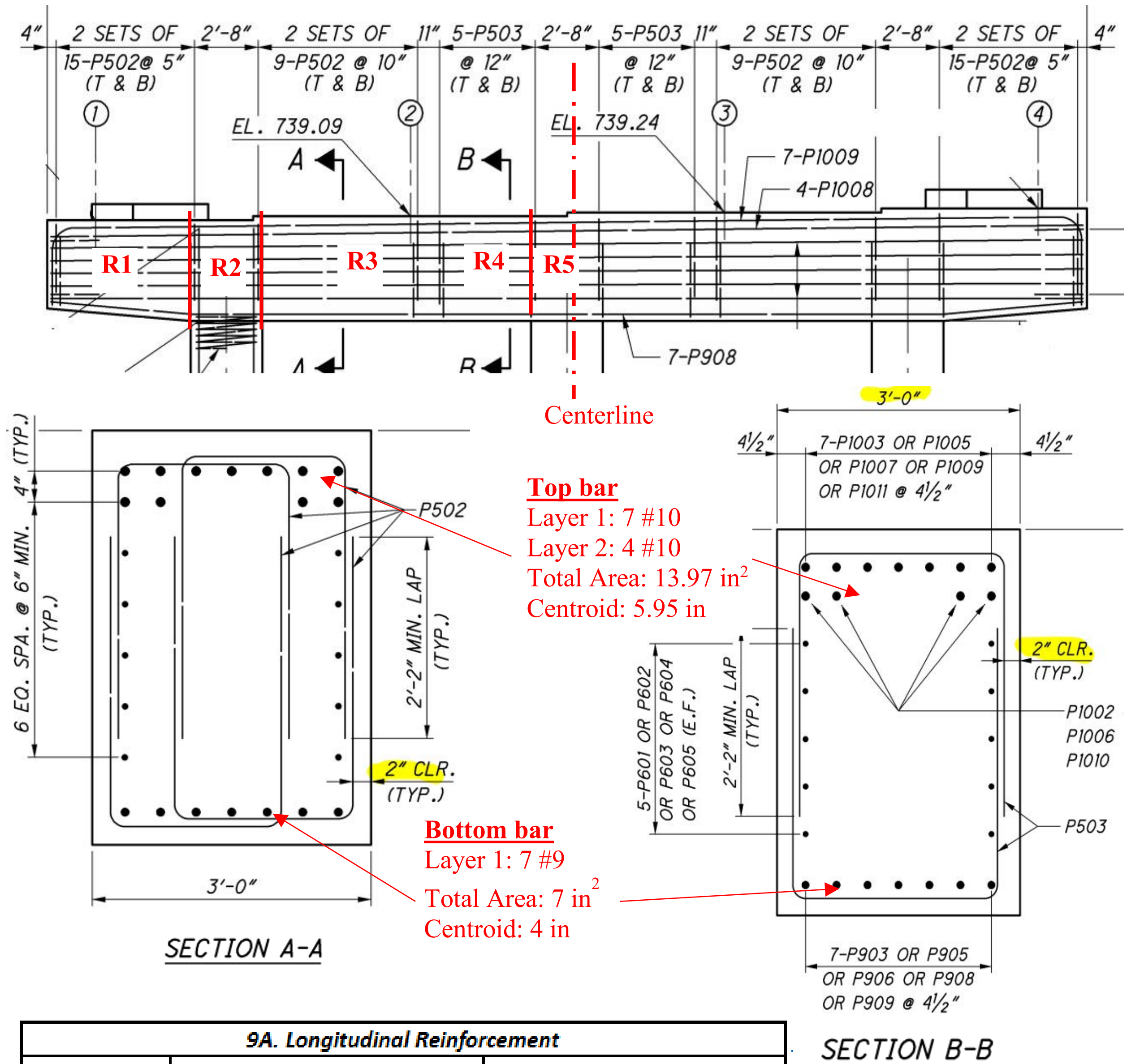
4.11 Section 9: Reinforcement Details

The reinforcement input provides the capacity for the STM elements. Two types of reinforcement input are possible:

1. Longitudinal Reinforcement

The total area and centroid of the top and bottom longitudinal reinforcement are input based on regions. The rebar may be constant throughout or might change at certain locations. Therefore, it is necessary to pay attention to the engineering drawings for rebar change sections.

In most pier caps, the rebar is provided in multiple layers. The total area of rebar should be concentrated at the centroid of the top and bottom sections. The position (centroid) of the horizontal reinforcement determines the position of the truss elements for STM. The area of longitudinal reinforcement is required when calculating the capacity of horizontal ties and horizontal reinforced struts.



9A. Longitudinal Reinforcement				
Region	**Top Steel (in^2, in)**		**Bottom Steel (in^2, in)**	
	Total Area (A_t)	**Centroid (C_t)**	**Total Area (A_b)**	**Centroid (C_b)**
R1	13.97	5.95	7	4
R2	13.97	5.95	7	4
R3	13.97	5.95	7	4
R4	13.97	5.95	7	4
R5	13.97	5.95	7	4

Figure 4-14: Sample longitudinal reinforcement input

A sample input for longitudinal reinforcement is shown in **Figure 4-14.**

2. Transverse Reinforcement (Stirrup)

The input for transverse reinforcement should be conducted in each region. The number of legs and the spacing of the stirrups is input for each region. The total area of the stirrups is calculated from spacing, number of legs, and length of each region by STM-CAP. The total area is used to calculate the capacity of vertical ties. The total area of the stirrup is concentrated at the center of each region, which determines the position of the vertical ties. A sample input for transverse reinforcement is shown in **Figure 4-15.** The vertical tie is only used in regions where the user selects to use it in Section 13.

4.11.1 Input Guidelines

From **Figure 4-14**, the number of legs and the spacing can be determined as below:

Region 1: four legged from Section A-A, and five-inch spacing from the design drawing elevation**.**

Region 2: no stirrups.

Region 3: four legged from Section A-A, and ten-inch spacing from the design drawing elevation**.**

Region 4: two legged from the Section A-A, and twelve-inch spacing from the design drawing elevation**.**

Region 5: no stirrups.

9B. Transverse Reinforcement		
Region	**No. of Legs**	**Stirrup Spacing**
R1	4	5.0 in
R2	0	0.0 in
R3	4	10.0 in
R4	2	12.0 in
R5	0	0.0 in

Figure 4-15: Sample input for transverse reinforcement

3. Horizontal Crack Control Reinforcement

Horizontal crack control reinforcement, as shown in **Figure 4-16,** is based on Section 5.8.2.6 of the AASHTO LRFD 2017 code. This reinforcement is intended to control the width of the cracks and to ensure a minimum ductility for the member, so that, if required, significant redistribution of internal stresses is possible. If no crack control reinforcement is provided, stress redistribution will not be possible. This Section is used to limit the diagonal crack width as a design requirement of strength and serviceability. For an existing cap beam which performs satisfactorily for a long duration of time with no visible cracking, whether to enforce this check is at the discretion of the engineer. A sample horizontal crack control reinforcement is shown in **Figure 4-17.** The horizontal crack control reinforcement is normally constant throughout the section of the pier cap and hence only one check is performed.

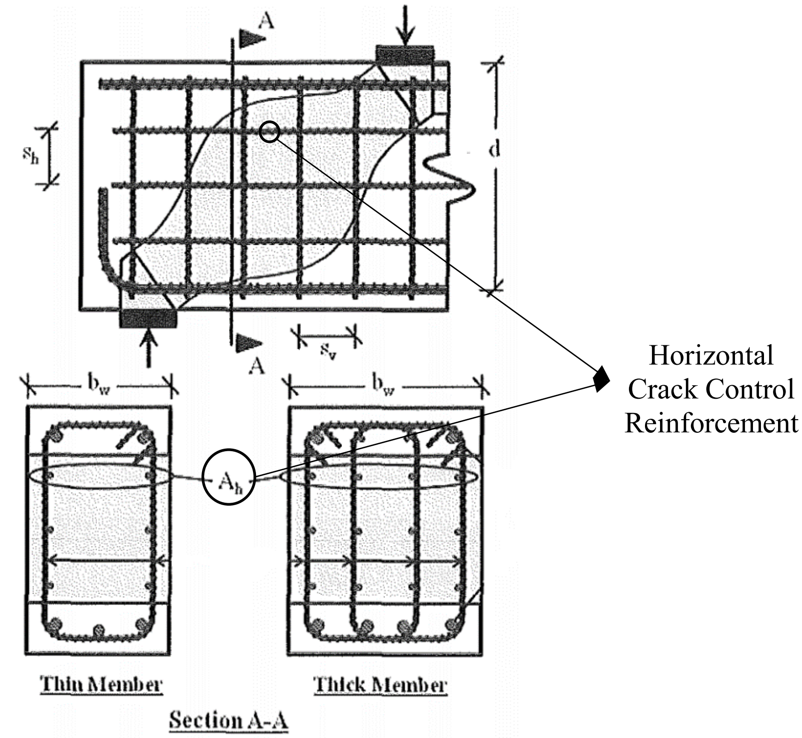


Figure C5.6.3.6-1—Distribution of Crack Control Reinforcement in Compression Strut

Figure 4-16: Crack control Reinforcement (AASHTO)

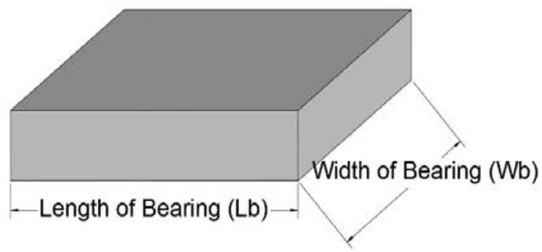
9C. Min Horizontal Crack Control Reinforcement	
Crack control rebar area (in^2)	0.44
Spacing (in)	11.0
No of layers of crack control rebar	2
Provided crack control reinforcement	0.22%
Code required crack control reinforcement	0.30%

Figure 4-17: Sample crack control reinforcement input

4.12 Section 10: Base Plate Details

This section requires the input of the length (L_b) and the width (W_b) of the base plate. The length of the base plate is required when determining the width of the inclined struts. The L_b and W_b are required for bearing check when determining the bearing capacity and utilization ratio of the concrete under the bearing plate. The bearing check is shown in Section 13.

A sample input for Section 9 is shown in **Figure 4-18**.



BEARING LOCATION	BEARING TYPE	NO. REQ'D. PER LOCATION	DEAD LOAD (KIPS)	LIVE LOAD (KIPS)	TOTAL LOAD (DL+LL) (KIPS)	Lb (inches)	Wb (inches)	tb (inches)
REAR ABUTMENT	EXP.	8	107.1	98.1	205.2	11	18	1½
PIER 1	EXP.	8	216.5	114.2	330.7	13	21	2 1/16
PIER 2	EXP.	8	215.5	115.8	331.3	13	21	2 1/16
PIER 3	EXP.	8	152.9	101.3	254.2	13	18	2 1/16
FORWARD ABUTMENT	EXP.	8	90.9	95.5	186.4	11	18	1½

L_b W_b

(a)

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	21.0 in
Base plate width perpendicular to the pier cap (W_b)	13.0 in

(b)

Figure 4-18: Sample **(a)** Bearing details; **(b)** input for bearing details

4.13 Section 11: Reinforcement Development

If a tension bar is not adequately developed, it will have a reduced capacity in the strut-and-tie analysis. This check assumes that the internal tension bars are fully developed. If not developed adequately, users are required to make suitable reductions based on personal experience. In STM-CAP, the reinforcement development is only checked for the end tension bars. The tension may be at the top or at the bottom of the beam. In the case of cantilever pier cap, the tension and the check, occurs at the top. In the case of a non-cantilever pier cap, the tension and the check, is usually at the bottom.

The development length is calculated using the AASHTO LRFD 2017 code. The development length calculation for straight bar follows Section 5.10.8.2.1 while the development length for hook is based on Section 5.10.8.2.4. The development length is equal to basic development length times the modification factor. **Figure 4-19** and **Figure 4-20** shows the calculation procedure for the development length.

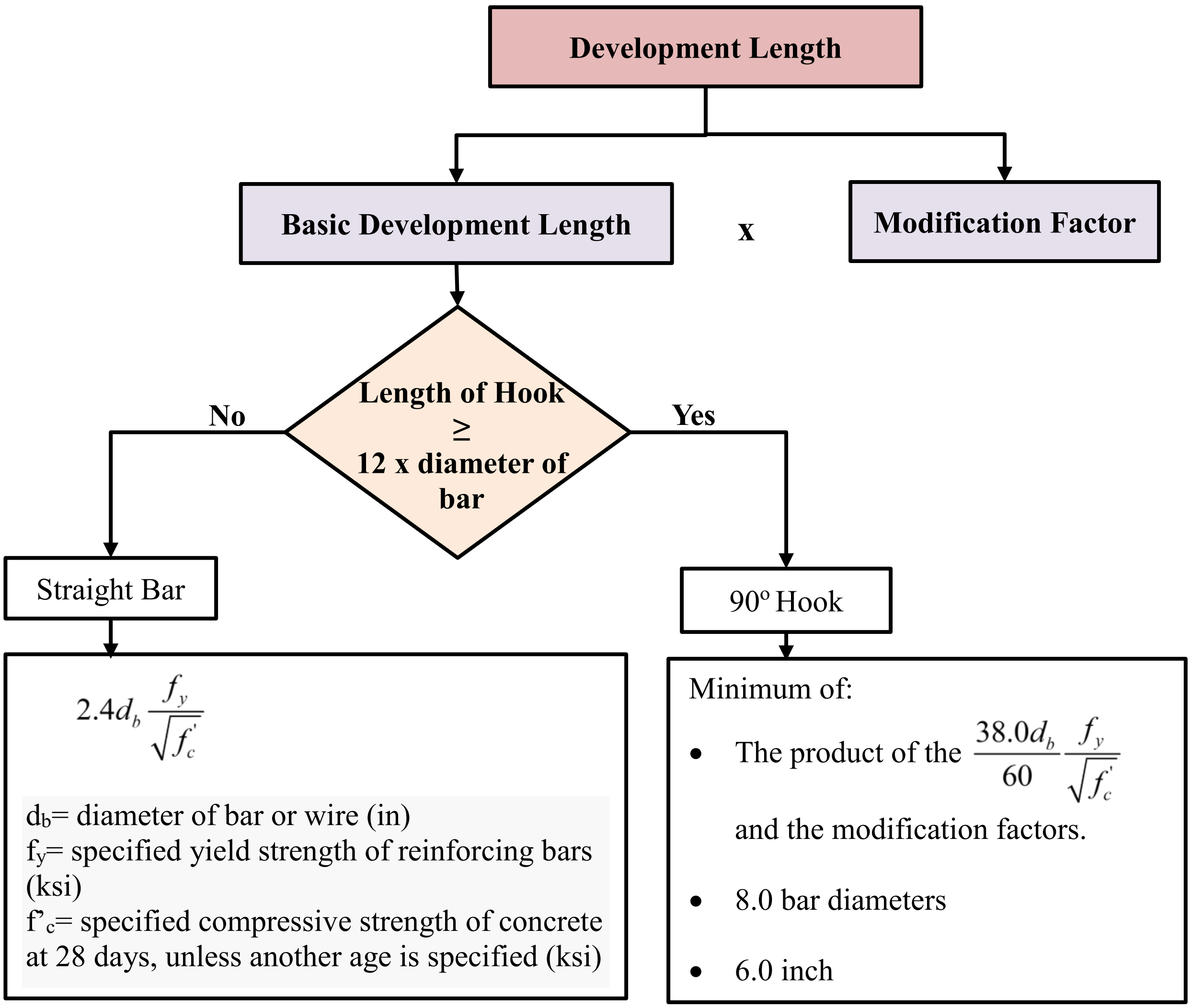


Figure 4-19: Flowchart for reinforcement development (AASHTO, 2017)

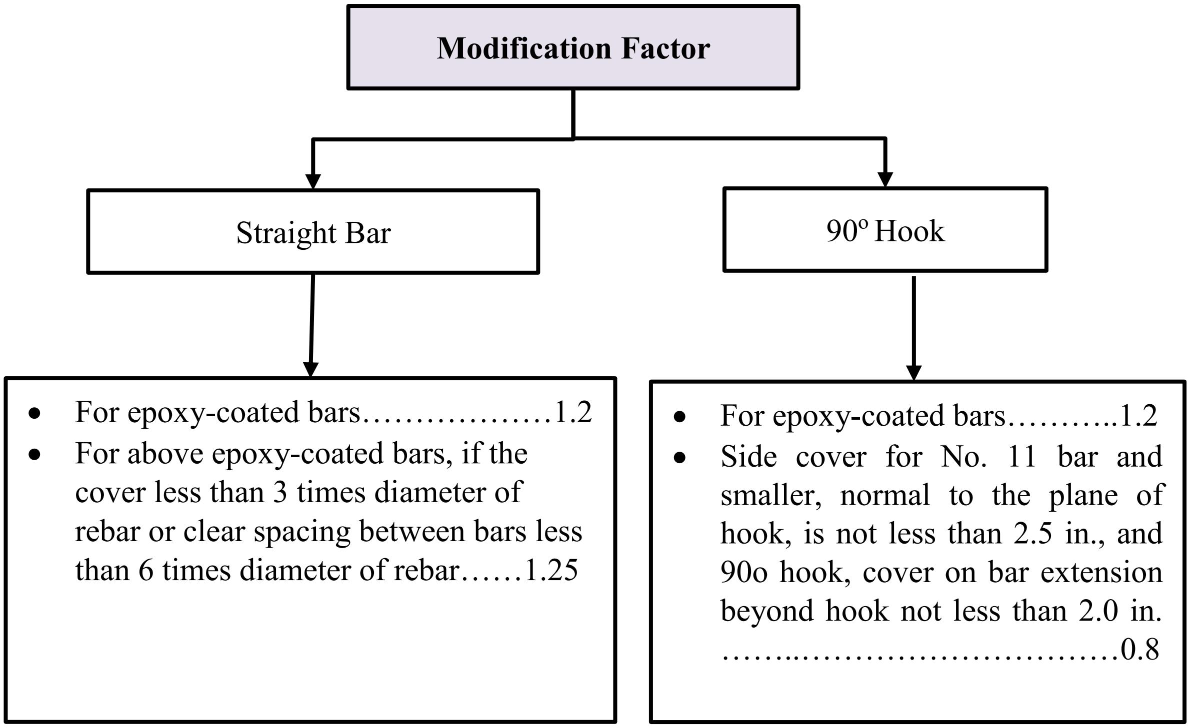


Figure 4-20: Modification factors for reinforcement development (AASHTO, 2017)

The above flowchart is incorporated in Section 11 of the STM-CAP. **Figure 4-21** shows a sample reinforcement development check.

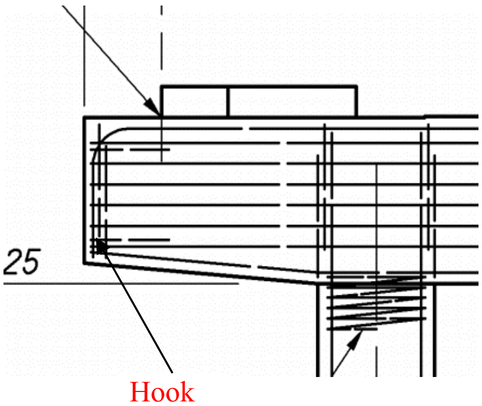
4.13.1 Input Guidelines

In this sample, a cantilever part causes tension on top and compression at the bottom of the cantilever. Therefore, it is necessary to check the development length of top longitudinal rebar on the cantilever portion of the beam.

In **Figure 4-21**, the *Horizontal length available* is an output section that calculates the total development length provided for the rebar based on the clear cover, the position of the end bearing, and other variables.

The next input asks for the diameter of top rebar (the biggest bar diameter if different rebar sizes are present), e.g., 1.27 in. for #10 bars. The next input asks the hook length to determine if the bar qualifies for 90º hook. The result is shown in red. In the example, the hook is provided throughout the depth of the beam excluding the clear cover on the top and bottom. Therefore, the hook length can be takes as approximately thirty inches. Consequently, the basic development length is found to be twenty-four inches and the hook qualifies for 90º hook.

Next, the basic development length must be multiplied with the modification factors to calculate the required development length. For the modification factors, select *Yes* or *No* from the drop-down lists. For this example, the required developed length is found to be twenty-four inches and the available development length is thirty-four inches. Therefore, the development length is adequately provided, and the *Reinforcement Capacity Multiplier* is calculated as 1.0. In the contrary, a multiplier less than 1.0 will appear in the box.



(a)

11. Reinforcement Development

Horizontal length available (L_d)	33 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	24 in

It qualifies for 90° hook.

Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	29 in
Available development length (L_d)	33 in

Reinforcement Capacity Multiplier:	1.00

(b)

Figure 4-21: Sample reinforcement development input

4.14 Section 12: Generate Output Model

'12. Generate Output Model' is a form control button which runs VBA codes in the background to generate an output model sketch with utilization ratios (ratio of member forces to member capacity). This is an important section as it shows the analysis results. A sample output sketch is shown in **Figure 4-22.** The pier cap with truss model is generated.

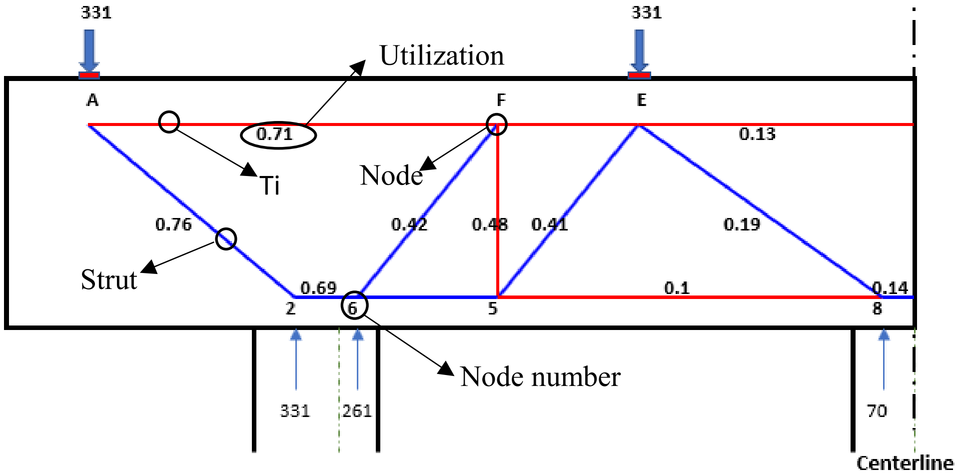
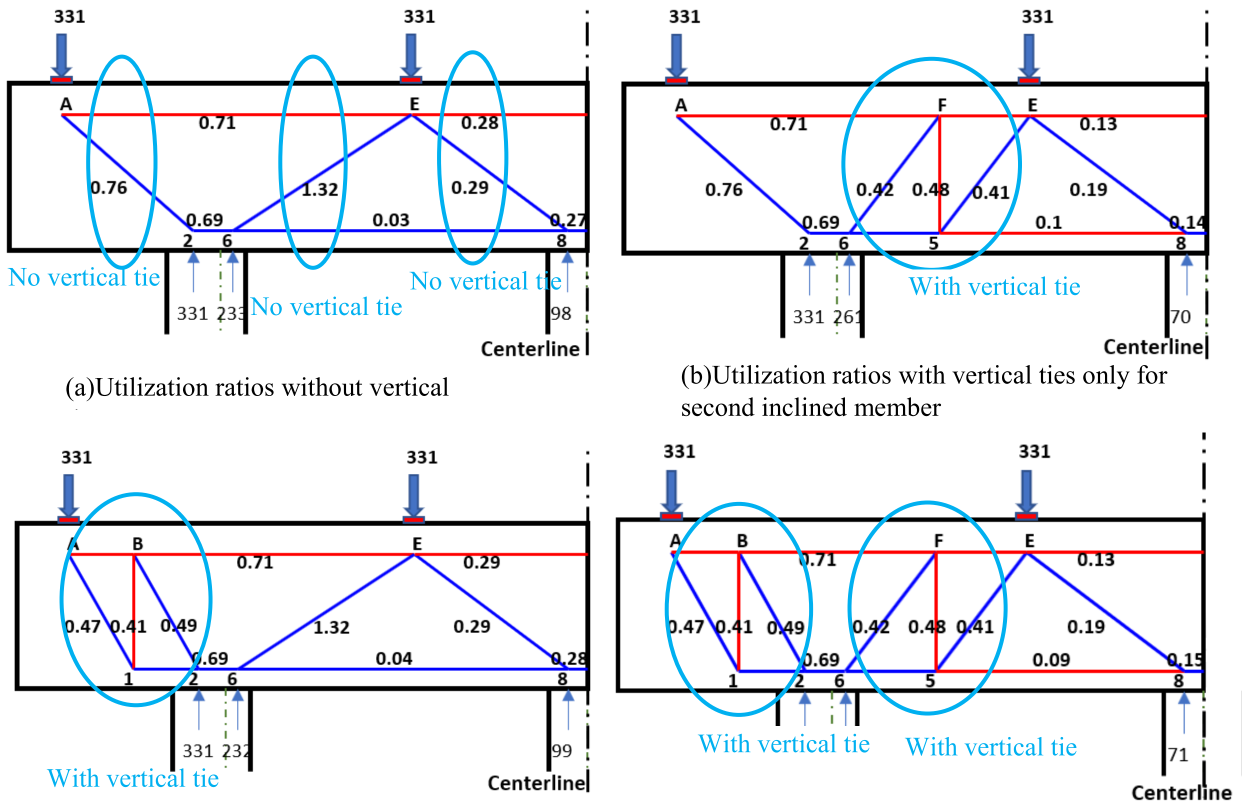


Figure 4-22: Sample output model with utilization ratios

The model shown is color-coded; *'red'* represents *'ties,'* *'blue'* represents *'struts,'* and the *'intersections'* represents the *'nodes'*. The nodes are labeled. The utilization ratio is shown for main struts and ties (not for every element) in the model. These utilization ratios are taken from Section 13.

The pier cap can be modeled with a variety of STM truss models. The truss model should represent the load path or the internal distribution of the stress. "The truss model should follow the most direct load path through the D (Disturbed) region," Martin (2007). In contrast, "refined strut-and-tie model should be used for more realistic flow of stresses in D-regions," Mitchell and Collins (2013). Therefore, the truss model can have a direct path using inclined members or can be refined using vertical ties to support the inclined members. However, the refinement of the truss model is based on Schlaich et al. (1987), who proposed the best truss model is that resulting from the use of minimum strain energy.

The capacity of the inclined member depends upon the angle of incline with respect to the horizontal from the AASHTO formulations. With a higher angle of incline, the inclined member capacity is higher and vice versa. Hence, the STM model is selected to obtain the maximum capacity from the pier cap. This process of obtaining maximum shear capacity or minimum utilization ratio is known as optimization of the model and the model is known as the efficient model. In STM-CAP, a truss model is developed upon generation of an output model that may be an optimized or unoptimized truss model. The truss model can be further adjusted by the user with a combination of vertical ties by toggling between truss model without vertical ties and truss model with vertical ties or a combination of both from Section 13. If the inclined members are failing, the user can activate the vertical tie option by entering 1 adjacent to the inclined member. Even though the inclined member is not failing, the vertical tie can be activated to optimize the truss model and get lower utilization ratios. The outcome of this action is dependent on the quantity of the ties present in the pier cap. If the quantity is too low, activation will result in tie overloads. In this case, the user should select the model with the smaller overload ratio. The utilization ratios are updated along with the updated model, which gives the confirmation for the efficient truss model. **Figure 4-23** shows the different combinations of direct strut and two-panel model to obtain the efficient truss model. In this example, the truss model (d) would be the best model for the analysis of this sample pier cap.



(a)Utilization ratios without vertical

(b)Utilization ratios with vertical ties only for second inclined member

(c)Utilization ratios with vertical ties only for first inclined member

(d)Utilization ratios with vertical ties for first and second inclined member

Figure 4-23: Optimization of utilization ratios with various truss models.

4.15 Section 13: Strut and Tie Output Summary

Section 13 summarizes all the results from the calculations of struts, ties, nodes, and bearing checks. The struts, ties, and nodes capacities are calculated in Section 15 onward.

In **Figure 4-22**, the struts, ties, and nodes are labeled. For each STM element, the member force is calculated using the matrix stiffness method for the statically indeterminate truss analogy sketched as explained in Section 17. The member forces are summarized in **Figure 4-24.**

The capacity of each STM element is calculated based on AASHTO 2017, Bridge Design Specification. The nodal capacity is incorporated while calculating these capacities. The element capacity is summarized in **Figure 4-24**, calculation details of which will be explained in Section 16. The utilization ratio for each member is the load-to-capacity ratio of each member and is calculated in **Figure 4-24.**

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down

Option	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-F	533	754	0.71	PASS	Top Members
	E-K	95	754	0.13	PASS	
	2-6	-533	-771	0.69	PASS	Bottom Members
	5-8	37	378	0.10	PASS	
	8-12	-95	-680	0.14	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	261	547	0.48	PASS	
	H-7	-	-	0.00	-	
0	A-2	-627	-826	0.76	PASS	Inclined Members
1	F-6	-386	-923	0.42	PASS	
	E-5	-386	-937	0.41	PASS	
0	E-8	-149	-782	0.19	PASS	
Bearing Areas Nodes at ⇉	A	331	573	0.58	PASS	1
	E	331	497	0.67	PASS	
	2	331	1723	0.19	PASS	2
	6	261	1361	0.19	PASS	
	8	70	1361	0.05	PASS	

Figure 4-24: Summary output of STM-CAP

If utilization ratio is less than 1.0, there is a reserve capacity. A utilization ratio of 0.71 means that 71% of total capacity is used, and there is approximately 29% reserve capacity. If force (load) is greater than capacity, or the utilization ratio is greater than 1.0, then the member is overloaded.

An overload in the horizontal ties from the top and bottom member exhibits flexure overload; an overload in the horizontal strut indicates a compression overload; and an overload in the vertical ties and inclined struts indicates shear overload. The type of overload in the member can be determined from the summary. The member also denotes position in the pier cap. Therefore, from the STM output summary, the failure mode and location can be determined.

The bearing check is performed for top bearing on pier cap and bottom bearing on the column. The top and bottom bearing checks are the bearing checks on the pier cap concrete under base plate and bearing on the column concrete under the pier cap respectively, as indicated by 1 and 2 in **Figure 4-25**. The load at the top bearing is the applied girder load. The capacity is calculated using AASHTO specification.

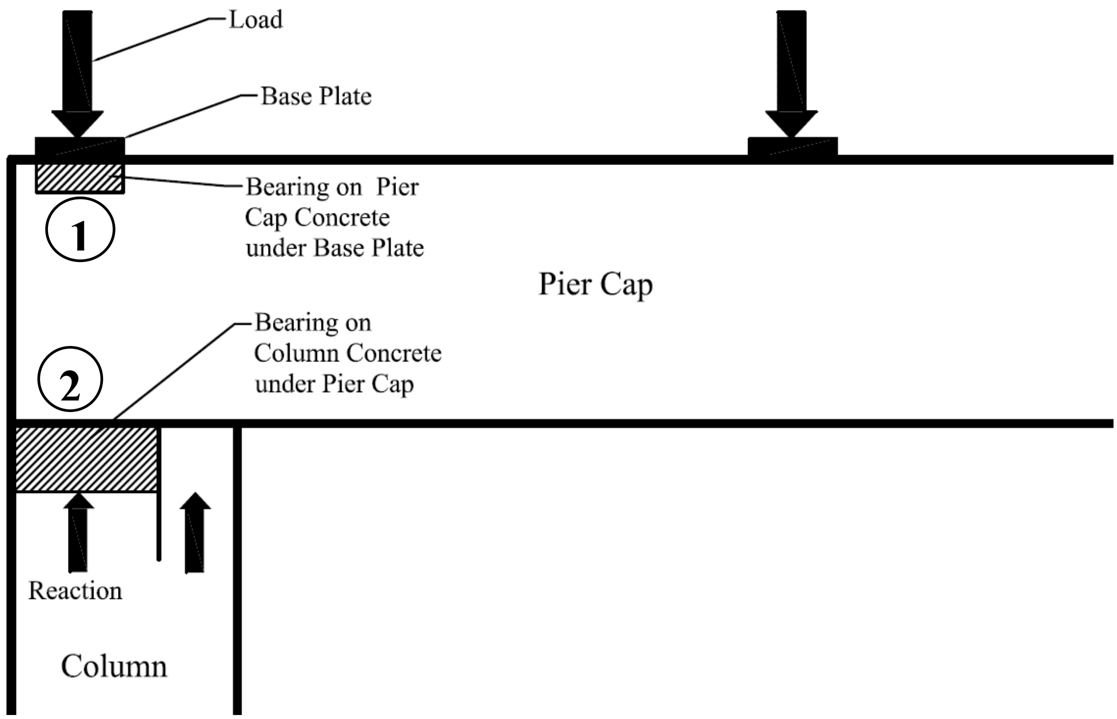


Figure 4-25: Bearing check concrete areas

A sample calculation for the bearing check at node 1 is shown below.

Load at node 1 =331 kips

Node 1 type = CCT

Nodal compressive strength = $0.70*f'c$

Bearing capacity= concrete resistance factor* nodal compressive strength*area of the base plate

$$= \Phi*0.70*f'c*L_b*W_b$$

=0.7*0.70*4 ksi *21 in*13 in

=535 kips

Bearing utilization ratio= Load/Capacity =331/535 =0.62

If the utilization ratio of the top bearing is greater than 1.0, it exhibits bearing overloads and means that the bearing size is not adequate.

Similarly, the load at the bottom bearing is the reaction on the column concrete under the pier cap. The capacity is calculated similarly, using the AASHTO LRFD 2017 specification. If the bearing is being overloaded, a larger column is needed.

4.16 Calculation Details

The sections from this point are hidden and not available to the user by default. There is a button to hide or show calculation details if the user is interested in the calculation details. Upon activation, the following sections can be seen:

Section 15 and Section 16 calculates the capacity for each STM member and Section 17 calculates the STM member forces (loads).

4.17 Section 15: Nodal Compressive Strength

From 5.8.2.5.3 of AASHTO LRFD 2017, the concrete compressive stress in the node regions of the strut are calculated for the bearing and the back face.

For node regions bounded by compressive struts and bearing areas (CCC Nodes) $\rightarrow 0.85\Phi f'_c$

For node regions anchoring a one-direction tension tie (CCT) $\rightarrow 0.75\Phi f'_c$

For node regions anchoring tension ties in more than one direction (CTT Nodes) → $(0.85 - f'_c / 20)\Phi f'_c$

The types of nodes are identified from the loads or forces in the truss model. Based on the node type, the compressive stresses of the nodes are calculated.

4.18 Section 16: Strut, Tie, and Nodal Capacities

The formulations for obtaining the capacities of struts, ties, and nodes are explained in Chapter 3. A sample application is shown below:

1. Tie Capacity

   AASHTO 2017 Section 5.8.2.4.1 calculates the nominal strength of the tie as:

   $$P_n = f_y A_{st} + A_{ps}\left[f_{pe} + f_y\right]$$

   For non-prestressed reinforcement, the nominal strength can be calculated as:

   $$P_n = f_y A_{st}$$

   A sample capacity calculation for the horizontal tie is shown below:

   <u>Capacity of the horzontal tie,</u>

   Area of the horizontal reinforcement (A_{st}) = 13.97 in^2

   Rebar yield strength(f_y) = 60 ksi

   Nominal capacity of the tie A-F, $P_n = f_y A_{st}$ =60 ksi*13.97 in^2 = 838.2 kips

   Factored capacity of the tie, ΦP_n = 0.9*838.2 kips = 754 kips

   <u>Capacity of the node face,</u>

   Nodal compressive strength =CTT node = $(0.85 - f'_c / 20) f'_c$

Factored capacity of the node = $\Phi\ (0.85 - f'_c / 20) f'_c$ *width of the node *thickness of pier cap

$= 0.65\Phi f'_c$ *width of the node *thickness of pier cap

= 0.65 * 0.7 * 4 ksi * 12 in * 36 in

= 786 kips

Capacity of the tie = min of (capacity of the horizontal tie, capacity of the node face)

= min (754 kips, 786 kips) = 754 kips

A sample calculation for the vertical tie is shown below. For the vertical tie, the only difference while calculating the capacity is the process for calculation of the area of reinforcement and the available length of region.

A sample capacity calculation for the vertical tie is shown below:

Capacity of the vertical tie,

Area of the vertical reinforcement,

$$A_{st} = \frac{\text{stirrup bar area * no. of legs *Available length of region}}{\text{stirrup spacing}} \text{ in}^2$$

$$A_{st} = \frac{0.31\text{in}^2 * 4 * 81.67}{10} = 10.12 \text{ in}^2$$

Rebar yield strength(f_y) = 60 ksi

Nominal capacity of the tie, $P_n = f_y A_{st}$ =60 ksi*10.12 in^2 = 608 kips

Factored capacity of the tie, ΦP_n = 0.9*608 kips = 547 kips

2. Strut-to-Node Face Capacity

A horizontal node face can be a reinforced strut due to the presence of horizontal compressive rebar parallel to the strut. The developed program STM-CAP considers the reinforced strut action, whereas some other STM research software, e.g., CAST, etc., omits the strength of rebar in struts. AASHTO 2017 Section 5.8.2.5.1 calculates the strength of the reinforced node face.

Nominal capacity, $P_n = f_{cu} A_{cs} + f_y A_{ss}$

A sample capacity calculation for the horizontal reinforced node face is shown below:

<u>Capacity of the horizontal node face (CCT),</u>

Area of the compressive horizontal reinforcement (A_{ss}) = 7 in^2

Rebar yield strength(f_y) = 60 ksi

Strut compressive stress, $f_{cu} = mvf_c^{'}$ $f_{cu} = 0.85 f_c^{'}; m = 1 \,\&\, v = 0.85$ (CCT Node)

=0.85*4 ksi= 3.4 ksi

Factored capacity of the node face, $P = \Phi f_{cu} A_{cs} + \Phi f_y A_{ss}$

= 0.7 * 3.4 ksi * width of node face* thickness of pier cap + 0.9 * 60 ksi * 7 in^2

= 0.7 * 3.4 ksi * 9 in * 36 in + 0.9 * 60 ksi * 7 in^2

= 1149 kips

A sample calculation for the inclined strut-to-node face is shown below:

<u>Capacity of the inclined strut-to-node face (any node type)</u>

The nominal capacity for the strut-to-node face, $P_n = f_{cu} A_{cs}$

$f_{cu} = mvf_c^{'}$

$f_{cu} = (0.85 - f'_c / 20) f'_c$; (Assume; m=1)

$f_{cu} = 0.65 f'_c = 0.65 * 4 = 2.6 ksi$

Area of the node face, A_{cs} = width of the strut * thickness of pier cap

Width of node face =min (width from the top bearing, width from the bottom bearing)

=min (13* sin31.9 + 12 cos31.9, 25* sin31.9 + 9*cos31.9)

=min (17.1 in, 20.9 in) = 17.1 in.

A_{cs} = 17.1 in * 36 in =616 in^2

Nominal strength of the strut-to-node face, $P_n = f_{cu} A_{cs} = 2.6ksi * 616in^2 = 1602kips$

Factored capacity of the strut-to-node face = 0.7*1602 kips = 1121 kips

4.19 Section 17: Strut and Tie Member Force

The matrix stiffness method is an exact method which performs a truss analysis and calculates the forces in each truss element. This method is particularly suited for the computer-automated analysis of complex and indeterminate structure. The VBA code that generates the input for the matrix stiffness method is explained below using the previous input made by the user. The VBA code for matrix stiffness method analyzes an indeterminate truss structure to determine the member forces.

VBA generated input for matrix stiffness method:

The VBA generated input labels the STM truss members, assigning a number to each member and each node as shown in **Figure 4-26**. The blue numerals are the names for each member and the black numerals are the names for each node.

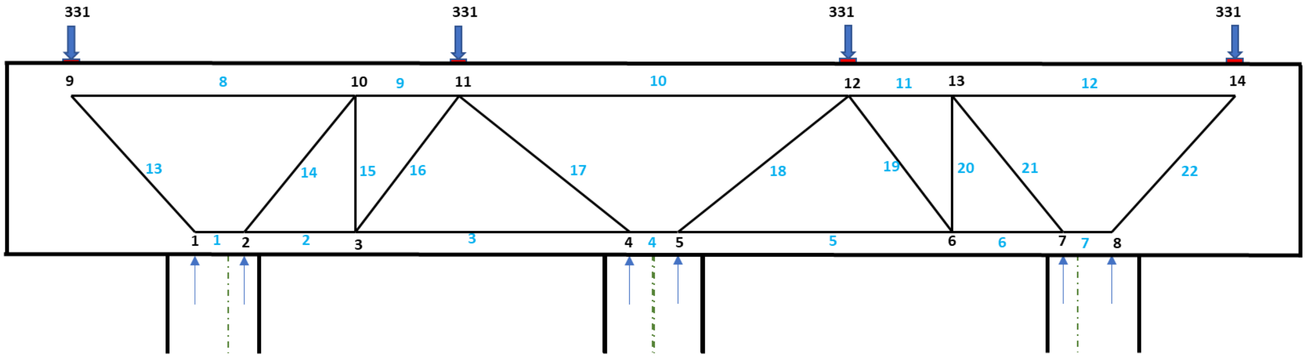


Figure 4-26: Assumed labeling for matrix stiffness method

The connectivity nodes for each member is determined. For example, for Member 1 the connectivity nodes are 1 and 2. The connectivity nodes for every member determined is shown in **Figure 4-27.** The nodal coordinate (in inches) for each node is calculated using the VBA code as shown in **Figure 4-27**, assuming the left-bottom corner of pier cap as origin.

Member Connectivity			***Nodal Coordinate***		
Bar	***Start***	***End***	***Nodes***	***X***	***Y***
1	1	2	1	84.3	4.5
2	2	3	2	102.3	4.5
3	3	4	3	143.0	4.5
4	4	5	4	255.0	4.5
5	5	6	5	273.0	4.5
6	6	7	6	385.0	4.5
7	7	8	7	425.7	4.5
8	9	10	8	443.7	4.5
9	10	11	9	24.0	42.0
10	11	12	10	143.0	42.0
11	12	13	11	184.0	42.0
12	13	14	12	344.0	42.0
13	9	1	13	385.0	42.0
14	10	2	14	504.0	42.0
15	10	3			
16	11	3			
17	11	4			
18	12	5			
19	12	6			
20	13	6			
21	13	7			
22	14	8			

Figure 4-27: Connectivity and nodal coordinates for STM truss model

The VBA is used to determine the nodes at which the point loads are applied and the corresponding load values (either in X-direction or Y-direction) as shown in **Figure 4-28**. It also determines the restraint conditions of each node as shown in **Figure 4-28**.

Point Loads			***Support Condition***		
Nodes	***X***	***Y***	***Nodes***	***X Restraint***	***Y Restraint***
1	0	0	1	1	1
2	0	0	2	0	1
3	0	0	3	0	0
4	0	0	4	0	1
5	0	0	5	0	1
6	0	0	6	0	0
7	0	0	7	0	1
8	0	0	8	0	1
9	0	-331	9	0	0
10	0	0	10	0	0
11	0	-331	11	0	0
12	0	-331	12	0	0
13	0	0	13	0	0
14	0	-331	14	0	0

Figure 4-28: Point loads and restraints for STM truss model

All input is used by the matrix stiffness method to analyze the truss and to determine the support reactions and the member forces as analysis output as shown in **Figure 4-29**. For example, at node 1, the horizontal reaction is negligible and the vertical reaction on the column is 331 kips. The reactions are the support reactions at each node of the STM model of **Figure 4-26**. The member forces are the loads/forces in the truss model of **Figure 4-26**.

Output				
Reaction			***Member Forces***	
Node	***X***	***Y***	***Bar***	***Force***
1	6.6E-13	331.0	1	-533
2		261.4	2	-249
3			3	37
4		69.6	4	-95
5		69.6	5	37
6			6	-249
7		261.4	7	-533
8		331.0	8	533
9			9	249
10			10	95
11			11	249
12			12	533
13			13	-627
14			14	-386
			15	261
			16	-387
			17	-149
			18	-149
			19	-387
			20	261
			21	-386
			22	-627

Figure 4-29: Reaction and member forces output from matrix stiffness method

Following all the procedures, from input to the output, STM-CAP calculates the utilization ratio for each STM member. Suitable conclusion, used capacity, reserve capacity, failure modes, etc., can be drawn from determining the utilization ratios.

Chapter 5 Verification of STM by CAST Software

5.1 Introduction

A total of thirteen pier caps were received from ODOT. Out of these thirteen pier caps, four were rehabilitated and one had missing information (e.g., girder loads and reinforcement details). Consequently, eight pier caps were modeled in this verification study.

The caps were modeled using three methods: STM-CAP, CAST software, and VecTor2 software. The results from each method were compared to assess the accuracy and validate the calculations of the STM-CAP. CAST is a general-purpose, linear-elastic strut-and-tie modeling software used for the analysis and design of disturbed regions. CAST is mainly used for research purposes and is primarily based on ACI codes. CAST was customized with manually calculated factors to work with AASHTO provisions. VecTor2 is a nonlinear finite element analysis method for the analysis of two-dimensional reinforced concrete members.

In this chapter, STM-CAP and CAST are used to model the bridge pier cap. In STM-CAP, a truss model is generated which may be an optimized or unoptimized model. The truss model can be further adjusted by the user to get an optimized model. In this chapter, the truss model comparison includes the direct truss model from STM-CAP without any further optimization to check the suitability for each case with CAST. Since STM-CAP and CAST work on the same principal of strut-and-tie, the comparison with any model (optimized or unoptimized) selection is valid. But for VecTor2 and the sectional method, the results should be compared with optimized STM model because it predicts the maximum capacity and should be compared with optimized STM-CAP model to get maximum capacity.

The analysis result of the eight modeled bridge pier caps using STM-CAP and CAST is summarized in **Table 5-1**, where the utilization ratios (i.e., the ratio of the demand (load) to the capacity) are listed for the strut and tie elements. The nodal capacities are considered while calculating the capacities of the strut and tie elements. A utilization ratio of 0.80, for example, indicates that the pier cap has 80% of its capacity in use and has approximately 20% reserve capacity remaining. The maximum utilization ratio of tension ties, horizontal struts, and inclined struts are compared. The largest utilization ratio value governs the cap behavior, with horizontal ties indicating a flexural failure mode, and vertical ties and diagonal struts indicating a shear failure.

Table 5-1: Bridge pier cap max utilization ratios (unoptimized) summary table

Bridge Name	Pier Cap	Model	STM-CAP	CAST
Bridge 1	Pier 2-Left	Tension Ties	0.71	0.70
		Horizontal Struts	0.69	0.69
		Inclined Struts	0.76	0.75
Bridge 2	Pier 2-Left	Tension Ties	1.02	1.00
		Horizontal Struts	0.83	0.80
		Inclined Struts	0.35	0.34
Bridge 3	North pier cap	Tension Ties	0.51	0.51
		Horizontal Struts	0.35	0.35
		Inclined Struts	0.75	0.74
Bridge 4	Any	Tension Ties	0.50	0.50
		Horizontal Struts	0.32	0.31
		Inclined Struts	0.54	0.54
Bridge 5	Any	Tension Ties	0.47	0.47
		Horizontal Struts	0.32	0.31
		Inclined Struts	0.78	0.78
Bridge 6	Pier 2-Left	Tension Ties	0.37	0.37
		Horizontal Struts	0.52	0.52
		Inclined Struts	0.57	0.57
Bridge 7	Southbound-Left	Tension Ties	0.33	0.34
		Horizontal Struts	0.25	0.25
		Inclined Struts	0.39	0.39
Bridge 8		Tension Ties	0.40	0.40

	Southbound-Right	Horizontal Struts	0.34	0.30
		Inclined Struts	0.48	0.48

CAST verifies the results from the STM-CAP for the eight pier caps modeled and proves its validity for the application of the analysis of pier caps. The utilization ratios compared are similar for each pier cap. In case of some discrepancy, the utilization ratios of the STM-CAP are more accurate than the utilization ratios of CAST verified by hand calculations.

5.2 Detailed Analysis Results

The detailed analysis results of the above bridge pier caps are presented individually from topic 5.3 onwards. STM-CAP detail analysis includes the key inputs and key outputs for the pier caps. The detailed analysis using CAST software demonstrates the input and output analysis model used.

5.3 Bridge 1

There are three pier-lines with two pier caps in each pier-line for a total of six cap beams as shown in **Figure 5-1.** By geometry and loading conditions, all the pier caps are similar. Therefore, similar results can be expected from each of them. For modeling, the left pier cap of pier-line 2 of the bridge was selected because it has the largest magnitude of girder load. The selected pier cap is a symmetrical, cantilever pier cap. It has three columns and two clear spans with cantilever span on both sides, supporting a total of four girder loads.

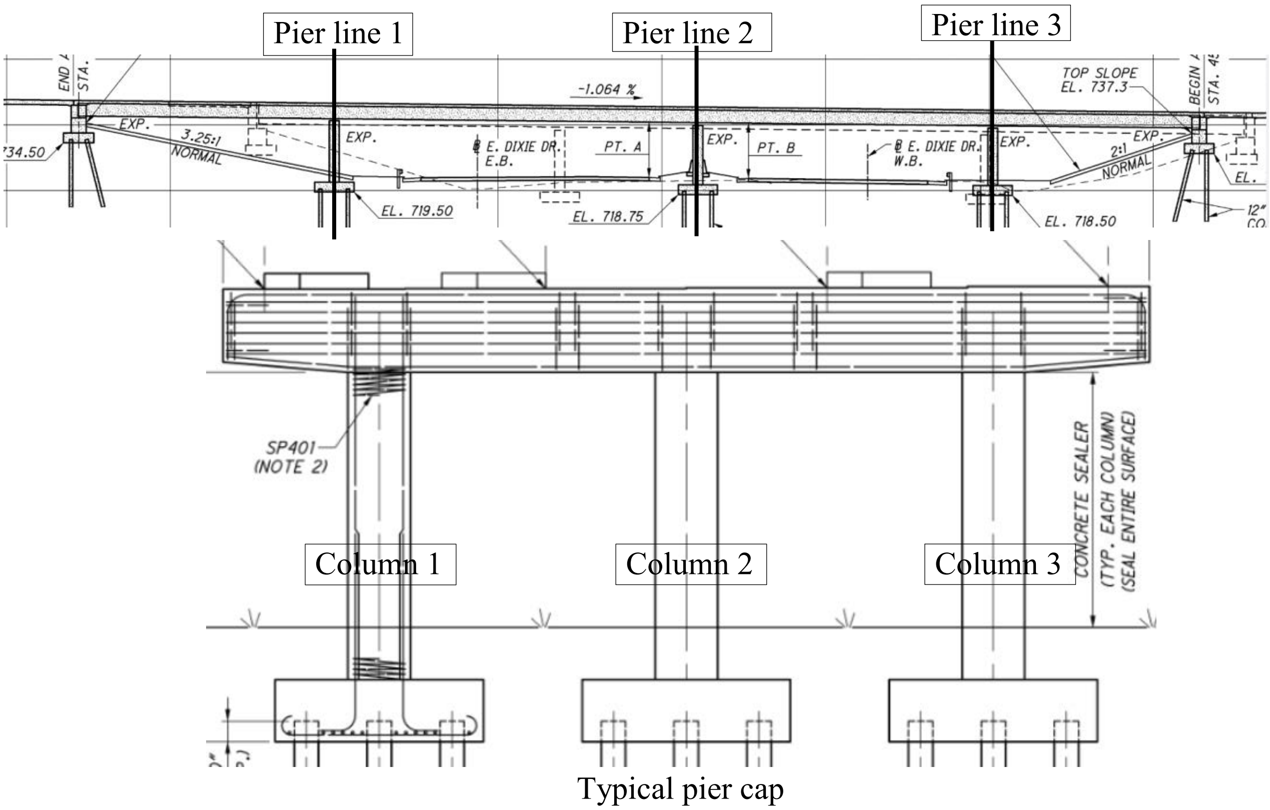
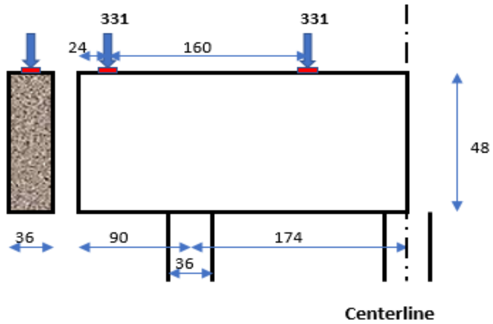


Figure 5-1: Pier cap details for Bridge 1

5.3.1 Modeling with STM-CAP

The STM-CAP was used to model the left pier cap. The load and geometry details are input to assess whether the pier cap is deep and STM-CAP is applicable. As seen in **Figure 5-2**, all regions are deep, and it is appropriate to use STM-CAP. The next input includes the material properties—longitudinal reinforcement, transverse reinforcement, and reinforcement development details.



6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	60.33 in	1.40	Deep Region
R2	0.00 in	0.00	Zero Region
R3	81.67 in	1.89	Deep Region
R4	71.00 in	1.64	Deep Region
R5	0.00 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

All regions are deep.

Figure 5-2: STM-CAP deep beam check (all units are in inches) for Bridge 1

After all inputs are made, the loads and capacity of each STM member are calculated and the output model and summary results are generated. **Figure 5-3** and **Figure 5-4** show the output model with the utilization ratios and the results summary table.

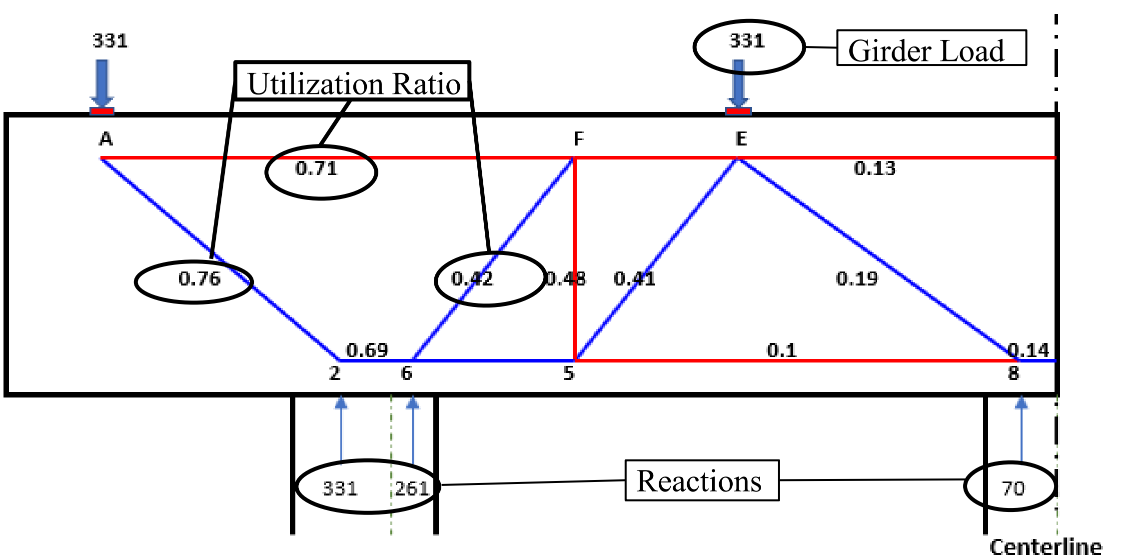


Figure 5-3: STM-CAP output model with utilization ratios for Bridge 1

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down

Option	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-F	533	754	0.71	PASS	Top Members
	E-K	95	754	0.13	PASS	
	2-6	-533	-771	0.69	PASS	Bottom Members
	5-8	37	378	0.10	PASS	
	8-12	-95	-680	0.14	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	261	547	0.48	PASS	
	H-7	-	-	0.00	-	
0	A-2	-627	-826	0.76	PASS	Inclined Members
1	F-6	-386	-923	0.42	PASS	
	E-5	-386	-937	0.41	PASS	
0	E-8	-149	-782	0.19	PASS	

Figure 5-4: STM-CAP output summary forces and utilization ratios for Bridge 1

5.3.2 Modeling with CAST

The analysis process using CAST first requires defining the material properties, thickness, and boundaries. The strut-and-tie model is sketched, and the ultimate girder loads and support conditions for the given pier cap are applied as shown in **Figure 5-5(a).** The truss model is then solved to get the strut and tie member forces as shown in **Figure 5-5(b).**

The strut types, the tie types, and the node types are defined and assigned to each strut, tie and node created. The analysis model is *'Run'* to get the analysis result. The member forces, utilization ratios, girder loads, support reactions, etc., are analysis output as shown in **Figure 5-6** for the snapped half pier cap.

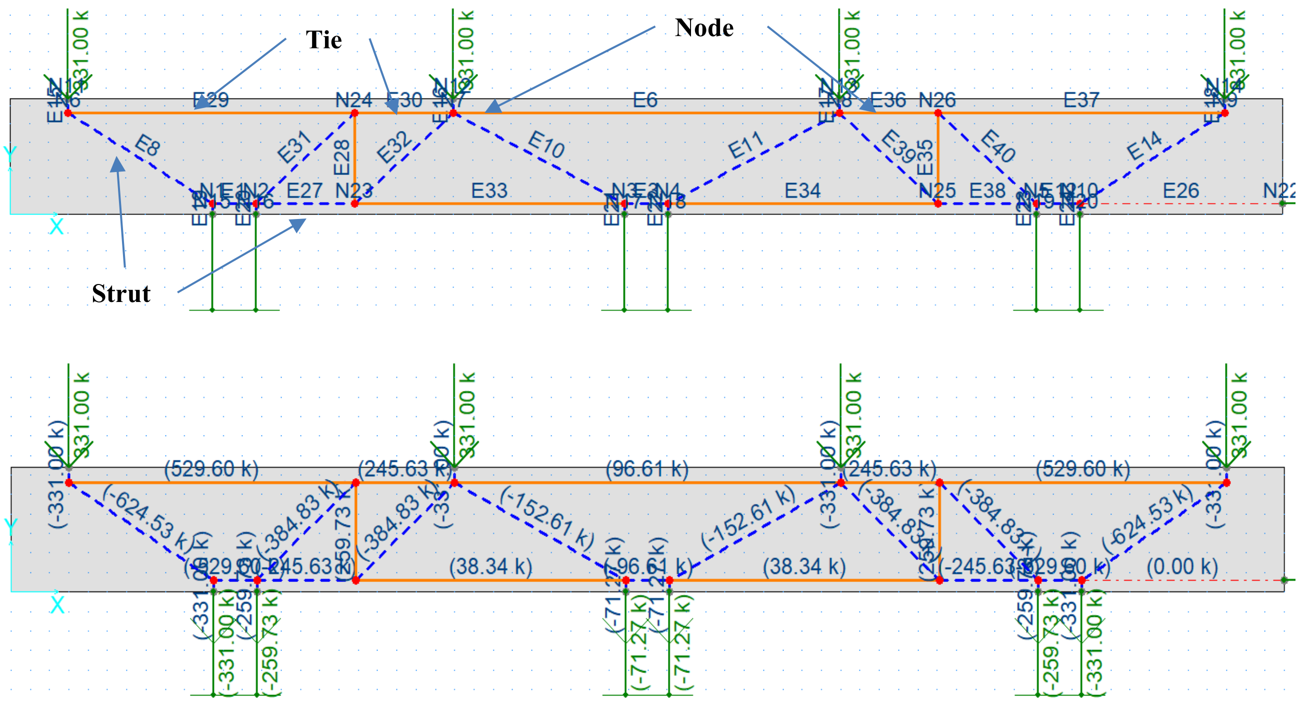


Figure 5-5: CAST model for strut-and-tie method for Bridge 1

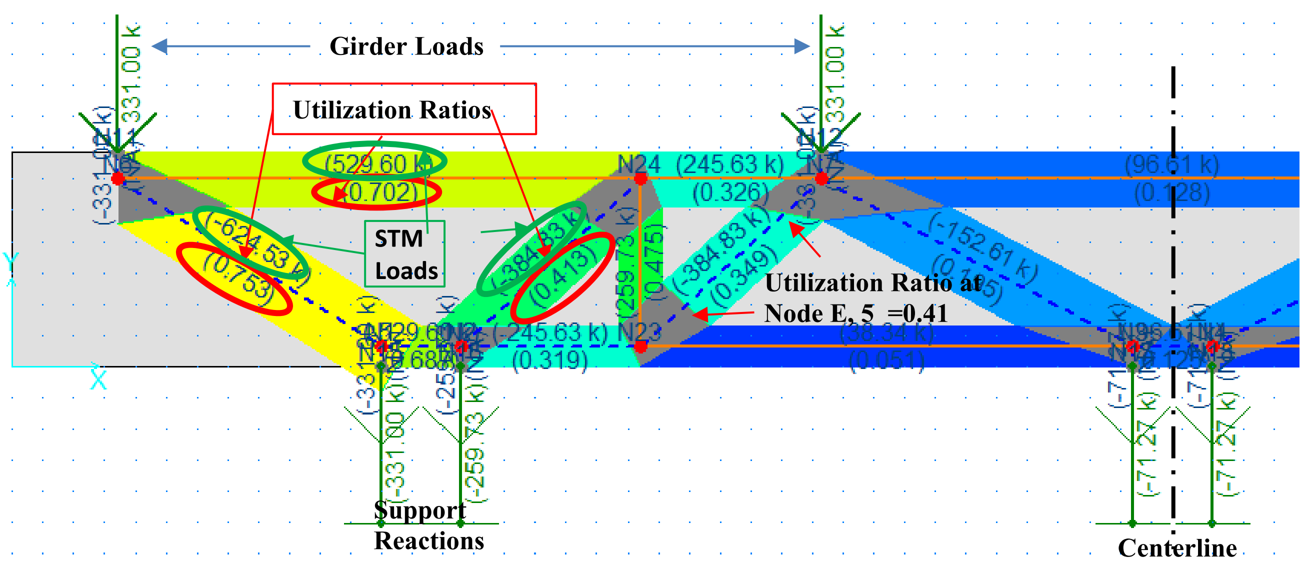


Figure 5-6: CAST analysis output for Bridge 1 for Bridge 1

5.3.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared as shown in **Table 5-2.** For STM-CAP, the nodal capacities are considered while calculating the capacities of the strut and tie elements. The capacity is the minimum of STM element capacity and nodal capacity. The utilization ratio is the maximum of STM element utilization ratio and nodal utilization ratio. For CAST the utilization ratios are separately calculated for STM elements and the nodes. The utilization ratio for CAST is taken manually as maximum of the element and nodal utilization ratio.

Table 5-2: Comparison of the results from the STM-CAP and CAST model for Bridge 1

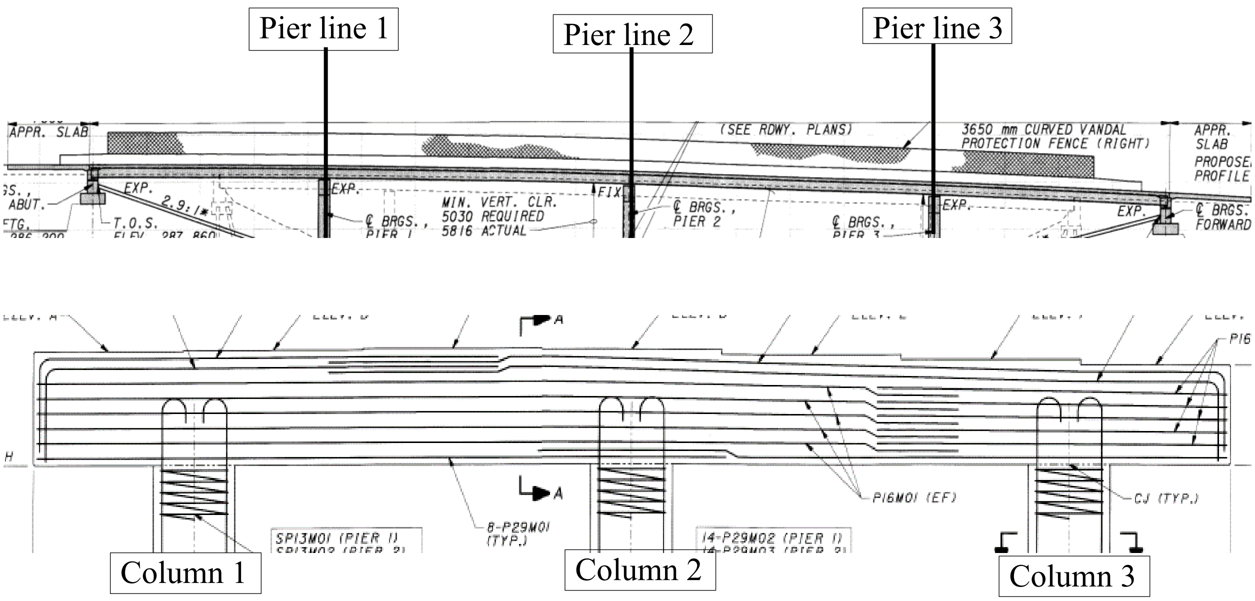
Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-F	533	529	0.71	0.70
F-K	95	96	0.13	0.13
2-6	-533	-529	0.69	0.69
5-8	37	38	0.10	0.10
8-12	-95	-96	0.14	0.14
F-5	261	259	0.48	0.48
A-2	-627	-624	0.76	0.75
F-6	-386	-384	0.42	0.41
E-5	-386	-384	0.41	0.41
E-8	-149	-152	0.19	0.20

The results for each method are similar. Thus, the CAST model verifies the STM-CAP and proves its validity.

Using the process described above, the remaining seven pier caps are modeled. The model details and analysis results are presented below.

5.4 Bridge 2

There are a total of six cap beams in each pier-line as shown in **Figure 5-7.** By geometry and loading conditions, all the pier caps are similar. Therefore, similar results can be expected from each of them. For modeling, pier cap of the pier-line 2 of the bridge was selected because it has the largest magnitude of girder load. The selected pier cap is a symmetrical, cantilever pier cap. It has three columns and two clear spans with cantilever span on both sides, supporting a total of seven girder loads.

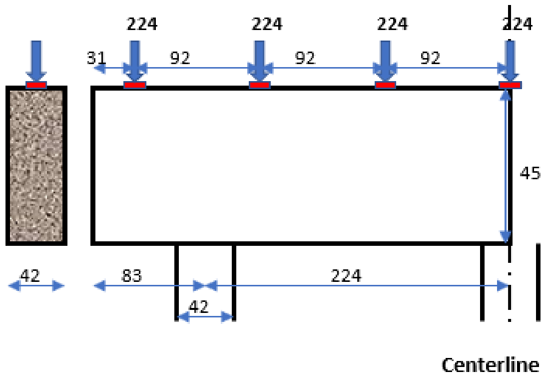


Typical pier cap

Figure 5-7: Pier cap details for Bridge 2

5.4.1 Modeling with STM-CAP

The analysis results are presented below with the help of self-explanatory figures. **Figure 5-8** shows the check for the deep beam. **Figure 5-9** and **Figure 5-10** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	40.41 in	1.00	Deep Region
R2	0.00 in	0.00	Zero Region
R3	30.59 in	0.76	Deep Region
R4	77.36 in	1.91	Deep Region
R5	4.14 in	0.10	Deep Region

This pier cap is deep.
Please continue with Section 7.

All regions are deep.

Figure 5-8: STM-CAP deep beam check (all units are in inches) for Bridge 2

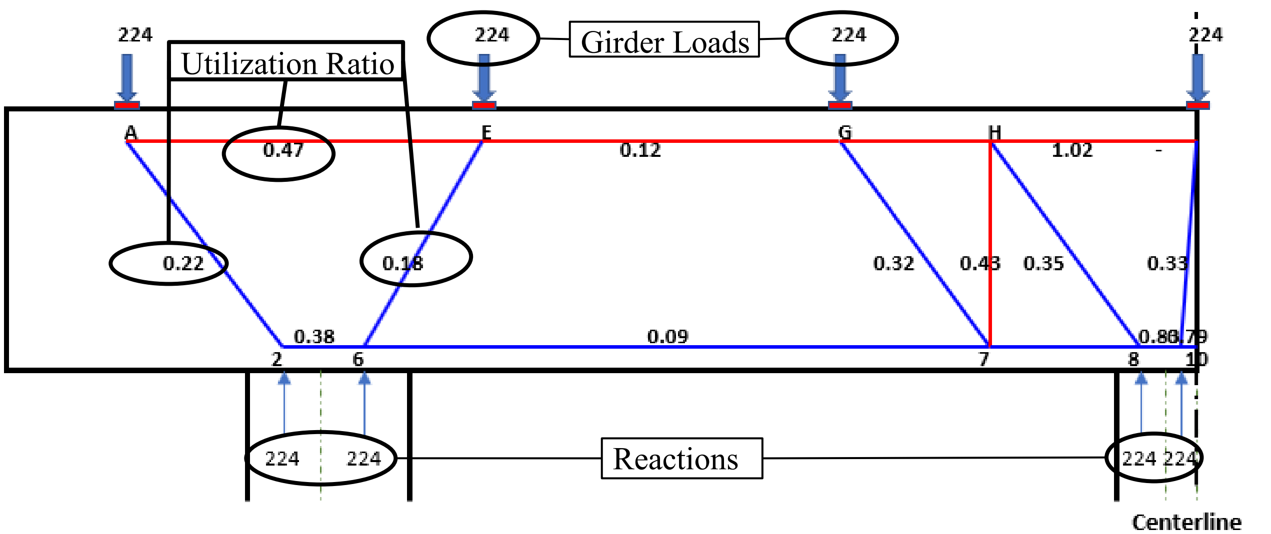


Figure 5-9: STM-CAP output model with utilization ratios for Bridge 2

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary					
	Member Code	*Load (k)*	*Capacity (k)*	*Utilization Ratio*	*Result*	
	A-E	242	511	0.47	PASS	Top Members
	E-G	59	511	0.12	PASS	
	H-I	522	511	1.02	Flexure Overloaded	
	2-6	-242	-630	0.38	PASS	Bottom Members
	6-7	-59	-630	0.09	PASS	
	8-10	-522	-630	0.83	PASS	
	10-12	-497	-630	0.79	PASS	
0	B-1	-	-	0.00	-	Vertical Members
0	F-5	-	-	0.00	-	
1	H-7	224	518	0.43	PASS	
0	J-9	-	-	0.00	-	
	A-2	-330	-1506	0.22	PASS	Inclined Members
	E-6	-289	-1614	0.18	PASS	
	G-7	-322	-1022	0.32	PASS	
	H-8	-322	-933	0.35	PASS	
	I-10	-225	-682	0.33	PASS	

Figure 5-10: STM-CAP output summary forces and utilization ratios for Bridge 2

5.4.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-11** shows the strut-and-tie model and the member forces. **Figure 5-12** shows the utilization ratios for each STM member.

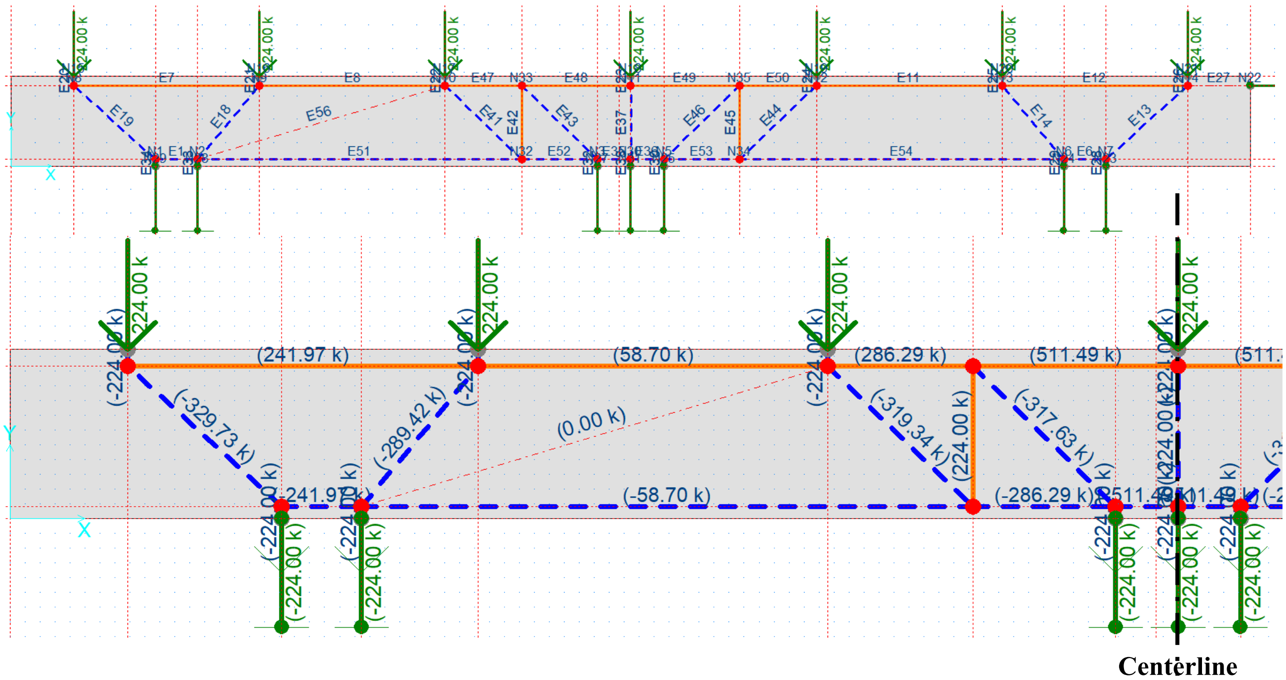


Figure 5-11: CAST model for the strut-and-tie method for Bridge 2

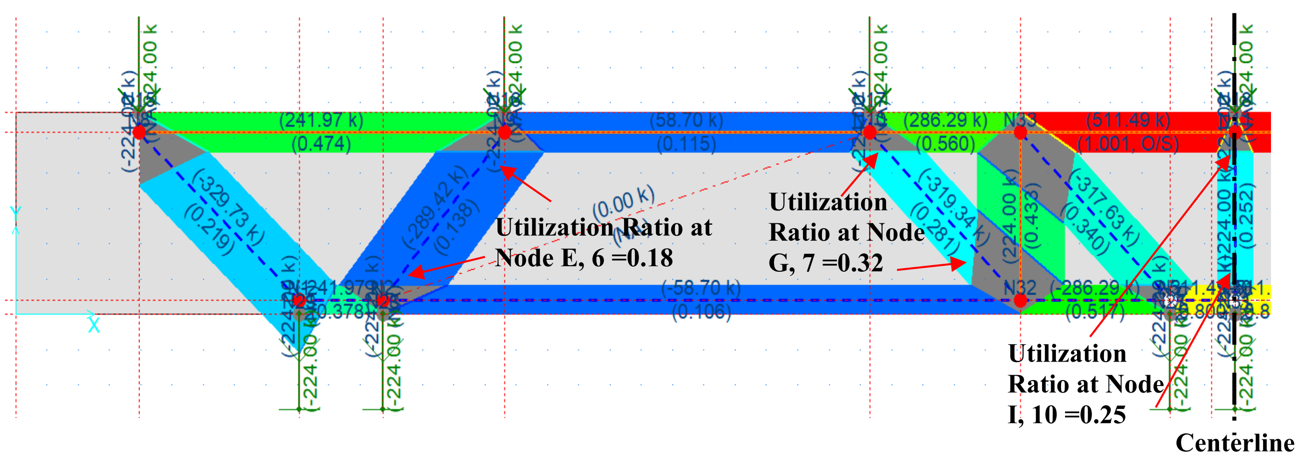


Figure 5-12: CAST analysis output for Bridge 2

5.4.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-3.**

Table 5-3: Comparison of the results from the STM-CAP and CAST model for Bridge 2

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	**CAST**	**STM-CAP**	**CAST**
A-E	242	242	0.47	0.47
E-G	59	59	0.12	0.12
H+	522	522	1.02	1.00
2-6	-242	-242	0.38	0.38
6-7	-59	-59	0.09	0.11
8-10	-522	-512	0.83	0.80
H-7	224	224	0.43	0.43
A-2	-330	-330	0.22	0.22
E-6	-289	-289	0.18	0.18
G-7	-322	-319	0.32	0.32
H-8	-322	-317	0.35	0.34
I-10	-225	-224	0.33	0.33

For Bridge 2, the utilization ratio of Member 6-7 using STM-CAP is lower than the value obtained using CAST. The reason for this is that STM-CAP uses reinforced struts and CAST does not. Reinforced struts consider the contribution of compression rebar to the concrete capacity. In all other aspects, the results for each method are similar. Consequently, the CAST model verifies the STM-CAP and proves its validity.

5.5 Bridge 3

The north pier cap of Bridge 3 was modeled for which the elevation is shown in **Figure 5-13.** The analysis result summary is explained below.

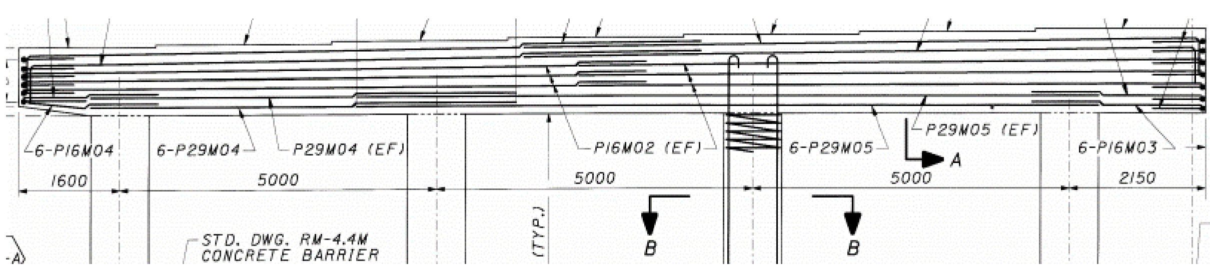
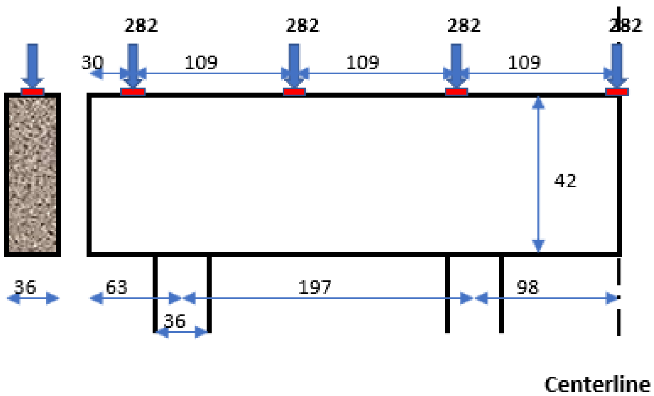


Figure 5-13: Pier cap elevation for Bridge 3

5.5.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-14** shows the check for the deep beam. **Figure 5-15** and **Figure 5-16** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	26.15 in	0.69	Deep Region
R2	0.00 in	0.00	Zero Region
R3	64.85 in	1.72	Deep Region
R4	105.91 in	2.80	Slender Region
R5	7.37 in	0.19	Deep Region
R6	87 in	2.29	Deep Region

This pier cap is deep.
Please continue with Section 7.

Most regions are deep.

Figure 5-14: STM-CAP deep beam check (all units are in inches) for Bridge 3

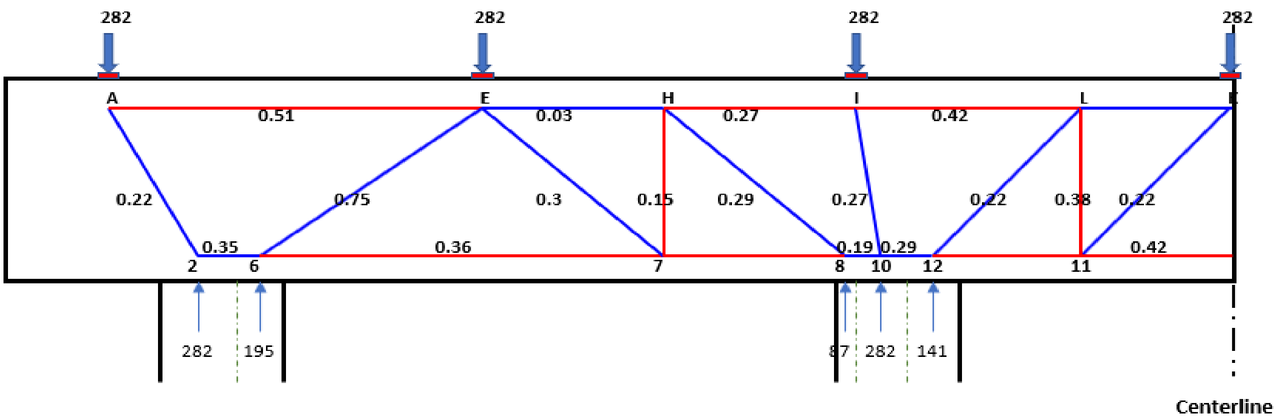


Figure 5-15: STM-CAP output model with utilization ratios for Bridge 3

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	A-E	218	432	0.51	PASS	Top Members
	E-H	-19	-537	0.03	PASS	
	H-I	118	432	0.27	PASS	
	I-L	179	432	0.42	PASS	
	K-M	-182	-703	0.26	PASS	
	2-6	-218	-620	0.35	PASS	Bottom Members
	6-7	155	432	0.36	PASS	
	8-10	-118	-620	0.19	PASS	
	10-12	-179	-620	0.29	PASS	
	11-14	182	432	0.42	PASS	
0	B-1	-	-	0.00	-	Vertical Members
0	F-5	-	-	0.00	-	
1	H-7	87	591	0.15	PASS	
0	J-9	-	-	0.00	-	
1	L-11	141	374	0.38	PASS	
	A-2	-357	-1635	0.22	PASS	Inclined Members
	E-6	-421	-565	0.75	PASS	
	E-7	-162	-546	0.30	PASS	
	H-8	-162	-568	0.29	PASS	
	I-10	-289	-1080	0.27	PASS	
	L-12	-229	-1032	0.22	PASS	
	K-11	-229	-1032	0.22	PASS	

Figure 5-16: STM-CAP output summary forces and utilization ratios for Bridge 3

5.5.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-17** shows the strut-and-tie model and the member forces. **Figure 5-18** shows the utilization ratios for each STM members.

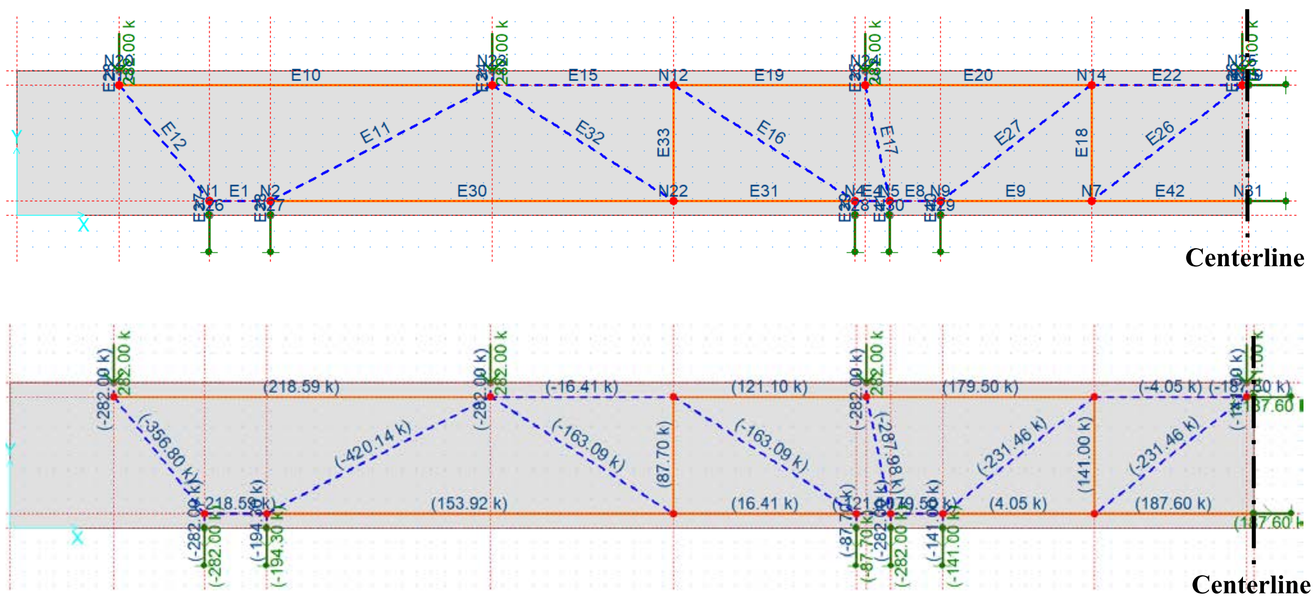


Figure 5-17: CAST model for the strut-and-tie method for Bridge 3

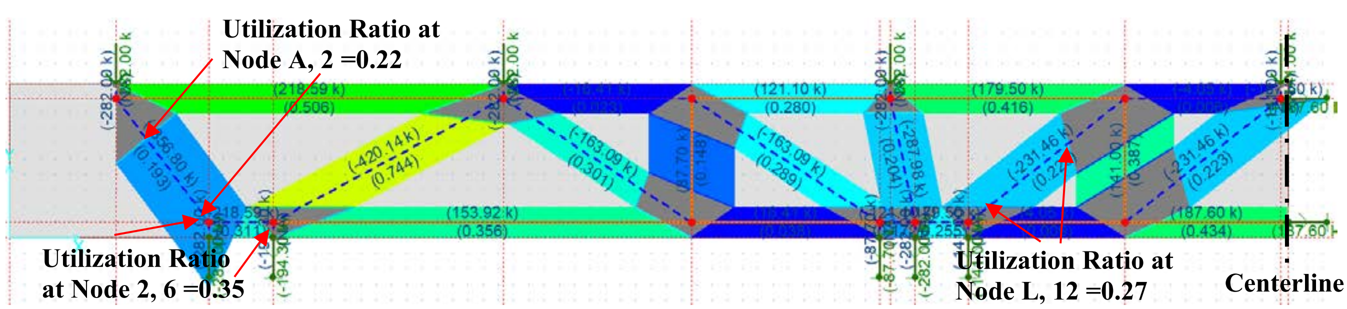


Figure 5-18: CAST analysis output for Bridge 3

5.5.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-4.**

Table 5-4: Comparison of the results from the STM-CAP and CAST model for Bridge 3

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-E	218	219	0.51	0.51
E-H	-19	-16	0.03	0.02
H-I	118	121	0.27	0.28
I-L	179	180	0.42	0.42
2-6	-218	-218	0.35	0.35
6-7	155	154	0.36	0.36
8-10	-118	-121	0.19	0.17
10-12	-179	-179	0.29	0.26
11+	182	188	0.42	0.43
H-7	87	88	0.15	0.15
L-11	141	141	0.38	0.39
A-2	-357	-357	0.22	0.22
E-6	-421	-420	0.75	0.74
E-7	-162	-163	0.30	030
H-8	-162	-163	0.29	0.29
I-10	-289	-288	0.27	0.27
L-12	-229	-231	0.22	0.22
K-11	-229	-231	0.22	0.22

The results for each method are similar. Thus, The CAST model verifies the STM-CAP and proves its validity.

5.6 Bridge 4

There are three similar pier caps for Bridge 4. Pier 1 was modeled for which the elevation is shown in **Figure 5-19.** The analysis result summary is explained below.

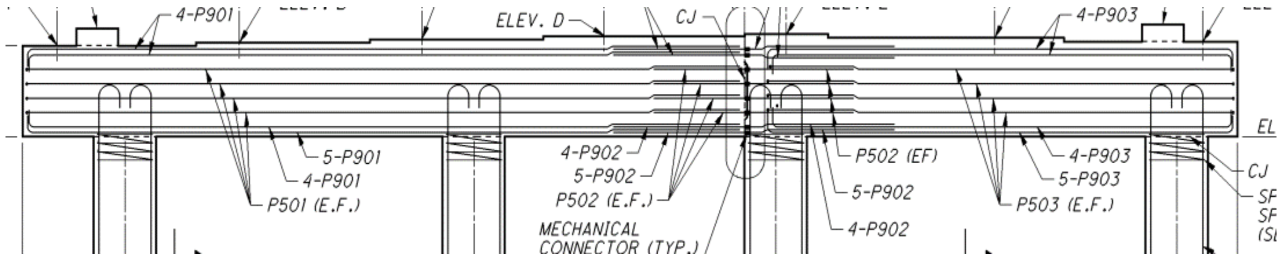
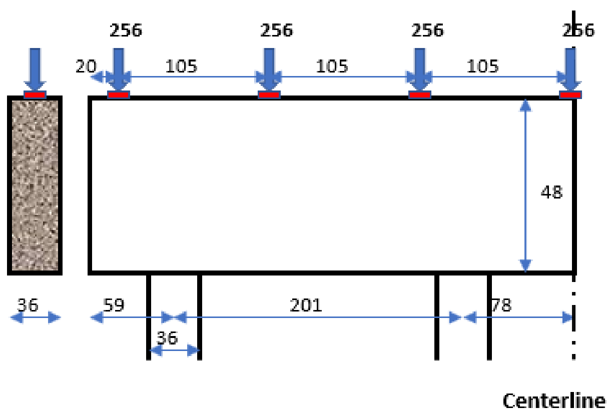


Figure 5-19: Pier cap elevation for Bridge 4

5.6.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-20** shows the check for the deep beam. **Figure 5-21** and **Figure 5-22** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	**Shear span (a)**	**a/d ratio:**	**Result**
R1	30.89 in	0.71	Deep Region
R2	0.00 in	0.00	Zero Region
R3	56.11 in	1.30	Deep Region
R4	21.74 in	0.50	Deep Region
R5	0.00 in	0.00	Zero Region
R6	65 in	1.51	Deep Region

This pier cap is deep.
Please continue with Section 7.

All regions are deep.

Figure 5-20: STM-CAP deep beam check (all units are in inches) for Bridge 4

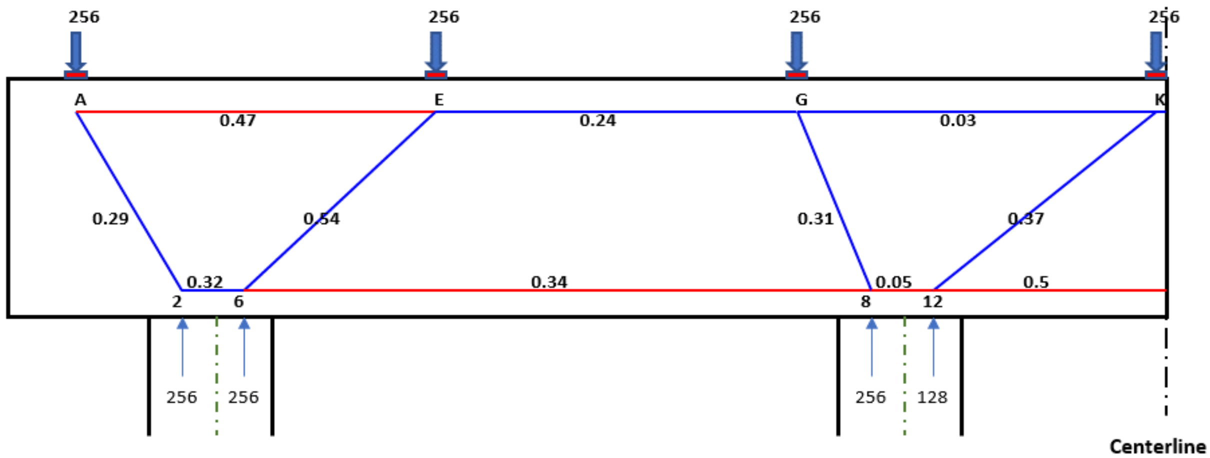


Figure 5-21: STM-CAP output model with utilization ratios for Bridge 4

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here

Option	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-E	201	432	0.47	PASS	Top Members
	E-G	-164	-680	0.24	PASS	
	G-K	-23	-771	0.03	PASS	
	K-M	-235	-771	0.31	PASS	
	2-6	-201	-635	0.32	PASS	Bottom Members
	6-8	164	486	0.34	PASS	
	8-12	23	486	0.05	PASS	
	12-14	245	486	0.50	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	-	-	0.00	-	
	H-7	-	-	0.00	-	
	L-11	-	-	0.00	-	
0	A-2	-326	-1104	0.29	PASS	Inclined Members
0	E-6	-446	-820	0.54	PASS	
0	G-8	-293	-945	0.31	PASS	
0	K-12	-248	-670	0.37	PASS	

Figure 5-22: STM-CAP output summary forces and utilization ratios for Bridge 4

5.6.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-23** shows the strut-and-tie model and the member forces. **Figure 5-24** shows the utilization ratios for each STM members.

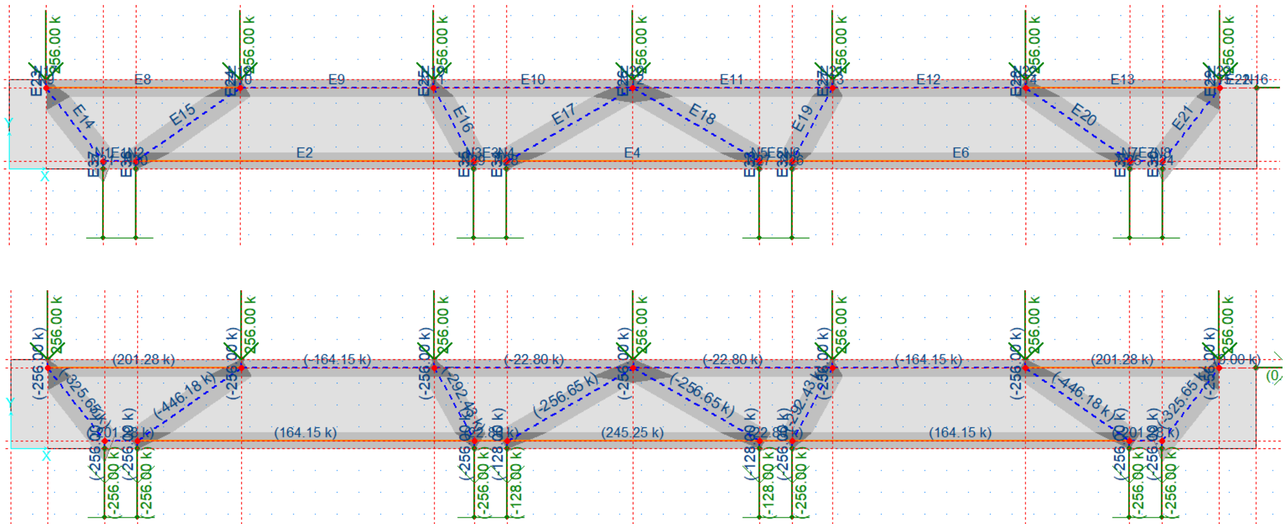


Figure 5-23: CAST model for the strut-and-tie method for Bridge 4

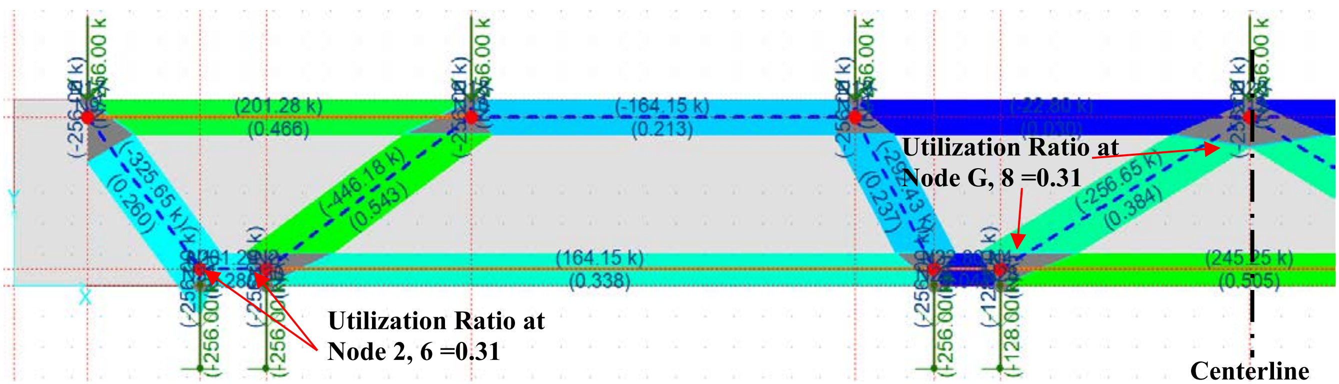


Figure 5-24: CAST analysis output for Bridge 4

5.6.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-5.** The results for each method are similar. Thus, the CAST model verifies the STM-CAP and proves its validity.

Table 5-5: Comparison of the results from the STM-CAP and CAST model for Bridge 4

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-E	201	201	0.47	0.47
E-G	-164	-164	0.24	0.21
G-K	-23	-23	0.03	0.03
2-6	-201	-201	0.32	0.31
6-8	164	164	0.34	0.34
8-12	23	23	0.05	0.05
12+	235	245	0.48	0.50
A-2	-326	-326	0.29	0.26
E-6	-446	-446	0.54	0.54
G-8	-293	-292	0.31	0.31
K-12	-248	-257	0.37	0.38

5.7 Bridge 5

There are five similar pier caps for Bridge 5. Pier 1 was modeled for which the elevation is shown in **Figure 5-25.** The analysis result summary is explained below.

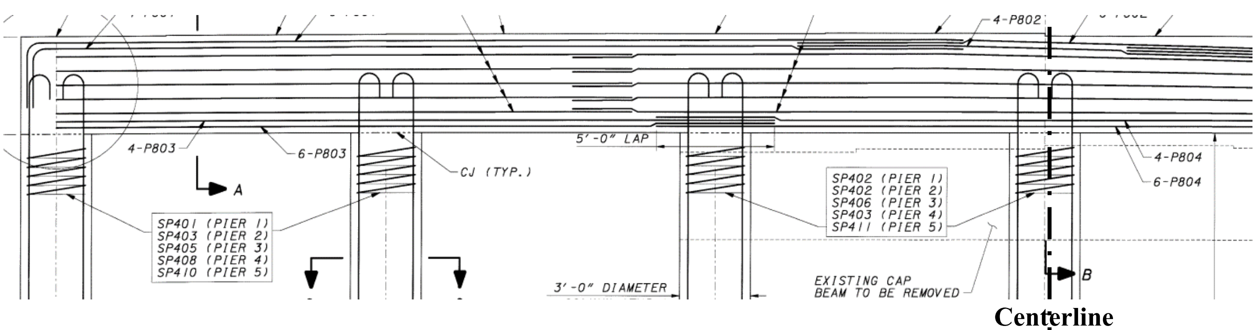
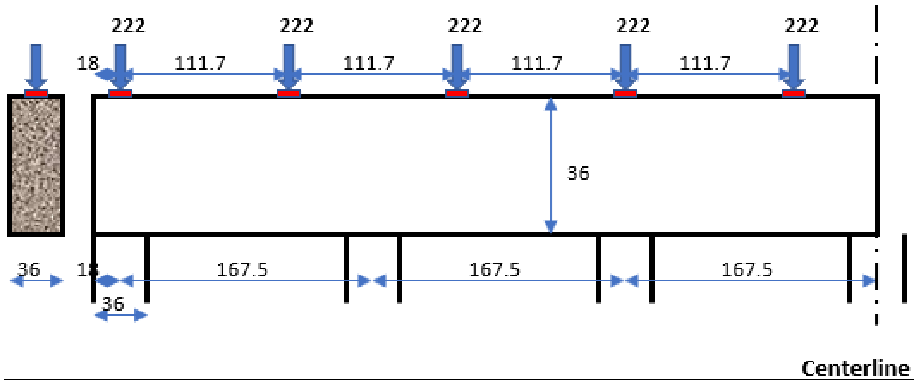


Figure 5-25: Pier cap elevation for Bridge 5

5.7.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-26** shows the check for the deep beam. **Figure 5-27** and **Figure 5-28** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	0.00 in	0.00	Zero Region
R2	4.50 in	0.14	Deep Region
R3	98.20 in	3.03	Slender Region
R4	46.80 in	1.44	Deep Region
R5	0.00 in	0.00	Zero Region
R6	47 in	1.45	Deep Region
R7	97 in	3.00	Slender Region
R8	0 in	0.00	Deep Region
R9	97 in	3.01	Slender Region
R10	47 in	1.44	Deep Region
R11	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

Most regions are deep.

Figure 5-26: STM-CAP deep beam check (all units are in inches) for Bridge 5

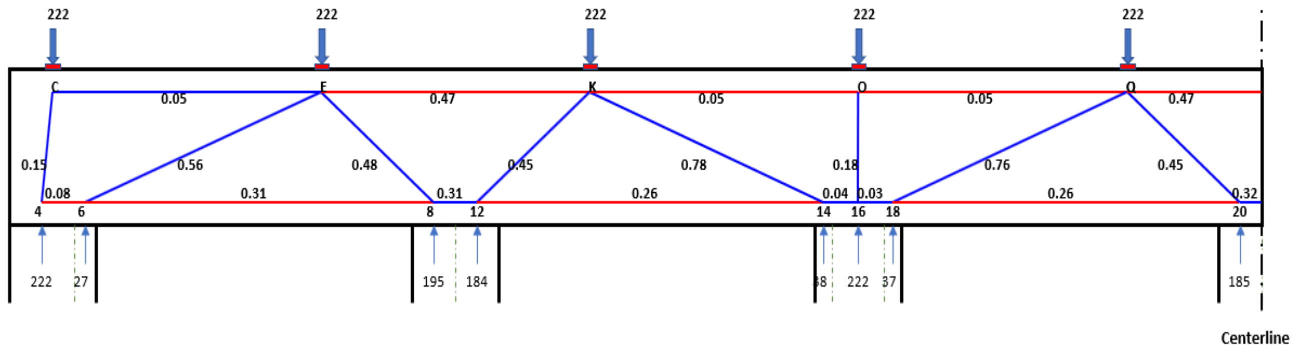


Figure 5-27: STM-CAP output model with utilization ratios for Bridge 5

<u>STM Members</u>

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Tie Option	Summary					
	Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***	
	C-E	-36	-720	0.05	PASS	Top Members
	E-K	199	427	0.47	PASS	
	K-N	-45	-550	0.08	PASS	
	K-O	22	427	0.05	PASS	
	O-Q	22	427	0.05	PASS	
	Q-S	46	427	0.11	PASS	
	Q-W	202	427	0.47	PASS	
	4-6	36	427	0.08	PASS	Bottom Members
	6-8	132	427	0.31	PASS	
	8-12	-199	-635	0.31	PASS	
	12-14	113	427	0.26	PASS	
	14-16	-22	-635	0.04	PASS	
	16-18	-22	-635	0.03	PASS	
	18-20	110	427	0.26	PASS	
	20-24	-202	-635	0.32	PASS	
0	F-5	-	-	0.00	-	Vertical Members
0	H-7	-	-	0.00	-	
0	L-11	-	-	0.00	-	
0	N-13	-	-	0.00	-	
0	R-17	-	-	0.00	-	
0	T-19	-	-	0.00	-	
	C-4	-225	-1520	0.15	PASS	Inclined Members
	E-6	-99	-178	0.56	PASS	
	E-8	-384	-800	0.48	PASS	
	K-12	-362	-798	0.45	PASS	
	K-14	-141	-182	0.78	PASS	
	O-16	-222	-1247	0.18	PASS	
	Q-18	-137	-181	0.76	PASS	
	Q-20	-363	-801	0.45	PASS	

Figure 5-28: STM-CAP output summary forces and utilization ratios for Bridge 5

5.7.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-29** shows the strut-and-tie method and the member forces. **Figure 5-30** shows the utilization ratios for each STM member.

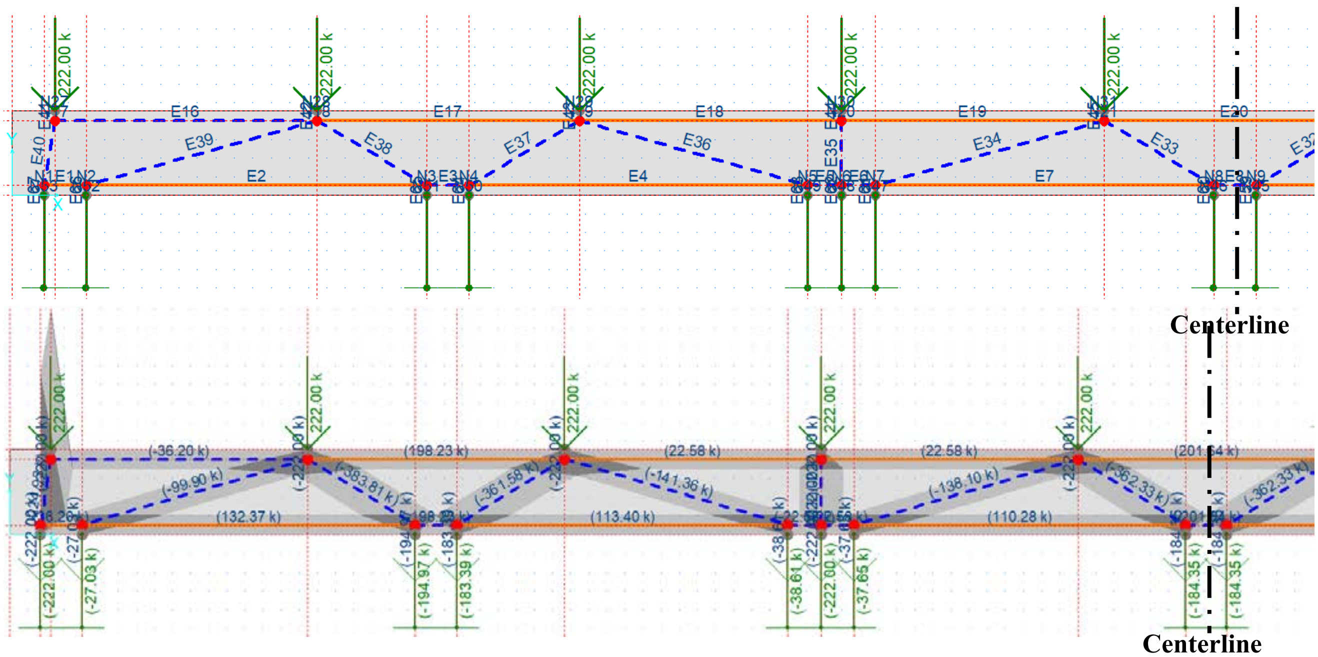


Figure 5-29: CAST model for the strut-and-tie method for Bridge 5

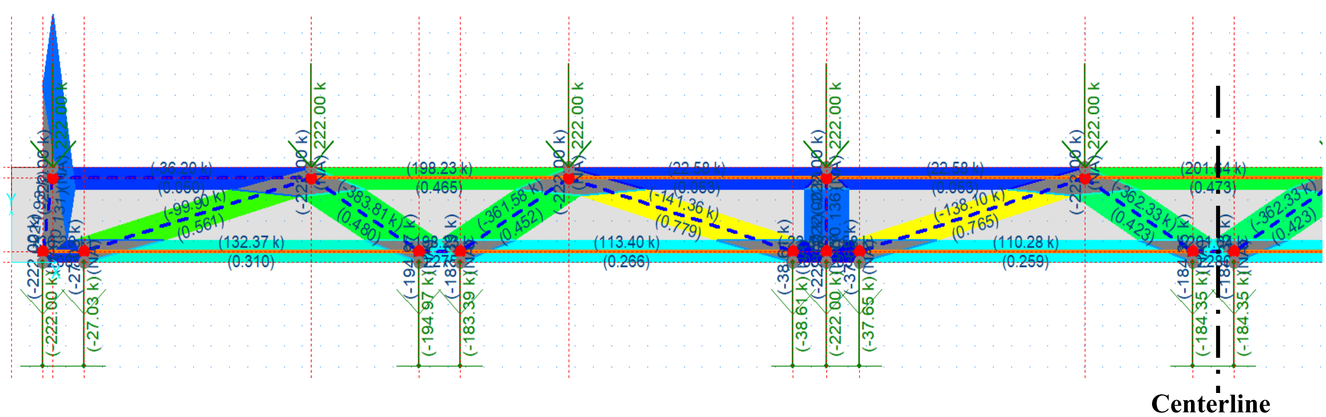


Figure 5-30: CAST analysis output for Bridge 5

5.7.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-6.** The results for each method are similar. Thus, The CAST model verifies the STM-CAP and proves its validity.

Table 5-6: Comparison of the results from the STM-CAP and CAST model for Bridge 5

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
C-E	-36	-36	0.05	0.05
E-K	199	198	0.47	0.47
K-O	22	23	0.05	0.05
O-Q	22	23	0.05	0.05
Q+	202	202	0.47	0.47
4-6	36	36	0.08	0.09
6-8	132	132	0.31	0.31
8-12	-199	-198	0.31	0.31
12-14	113	113	0.26	0.27
14-16	-22	-23	0.04	0.03
16-18	-22	-23	0.03	0.03
18-20	110	110	0.26	0.26
20+	-202	-202	0.32	0.31
C-4	-225	-225	0.15	0.13
E-6	-99	-100	0.56	0.56
E-8	-384	-384	0.48	0.48
K-12	-362	-362	0.45	0.45
K-14	-141	-141	0.78	0.78
O-16	-222	-222	0.18	0.18
Q-18	-137	-138	0.76	0.77
Q-20	-363	-362	0.45	0.42

5.8 Bridge 6

The left pier cap of pier-line 2 was modeled for Bridge 6, for which the elevation is shown in **Figure 5-31**. This is an asymmetrical pier cap with four columns. For the asymmetrical pier cap, a full pier cap is modeled. The analysis result summary is explained below.

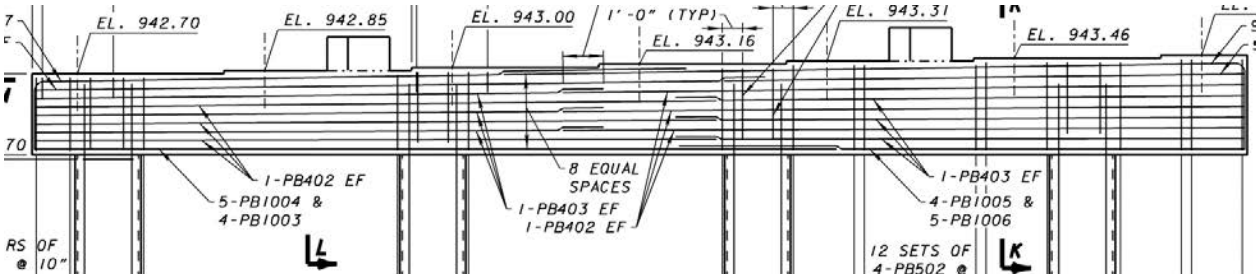
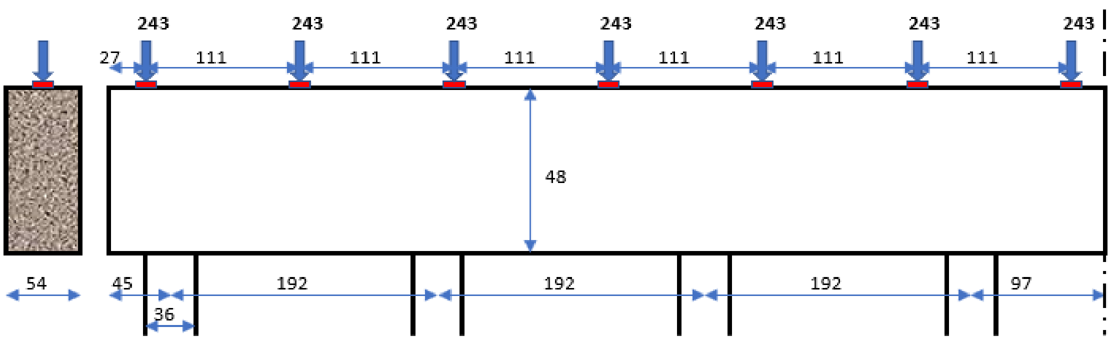


Figure 5-31: Pier cap elevation for Bridge 6

5.8.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-32** shows the check for the deep beam. **Figure 5-33** and **Figure 5-34** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio***	***Result***
R1	11.88 in	0.27	Deep Region
R2	0.00 in	0.00	Zero Region
R3	81.12 in	1.88	Deep Region
R4	85.73 in	1.98	Deep Region
R5	10.78 in	0.25	Deep Region
R6	109 in	2.51	Slender Region
R7	58 in	1.34	Deep Region
R8	0 in	0.00	Zero Region
R9	17 in	0.39	Deep Region
R10	30 in	0.70	Deep Region
R11	0 in	0.00	Zero Region
R12	45 in	1.04	Deep Region

This pier cap is deep.
Please continue with Section 7.

Most regions are deep.

Figure 5-32: STM-CAP deep beam check (all units are in inches) for Bridge 6

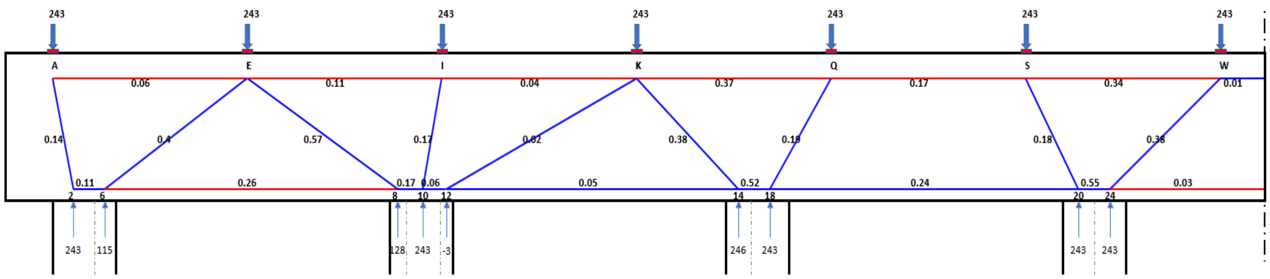


Figure 5-33: STM-CAP output model with utilization ratios for Bridge 6

<u>STM Members</u>

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-E	73	1234	0.06	PASS	Top Members
	E-H	-25	-1081	0.02	PASS	
	E-I	114	1081	0.11	PASS	
	I-K	48	1081	0.04	PASS	
	K-Q	401	1081	0.37	PASS	
	Q-S	187	1081	0.17	PASS	
	S-W	372	1081	0.34	PASS	
	W+	-16	-1247	0.01	PASS	
	2-6	-73	-680	0.11	PASS	Bottom Members
	6-8	164	617	0.26	PASS	
	8-10	-114	-680	0.17	PASS	
	10-12	-48	-771	0.06	PASS	
	12-14	-40	-771	0.05	PASS	
	14-18	-401	-771	0.52	PASS	
	18-20	-187	-771	0.24	PASS	
	20-24	-372	-680	0.55	PASS	
	24+	16	590	0.03	PASS	
0	B-1	-	-	0.00	-	Vertical Members
0	F-5	-	-	0.00	-	
0	H-7	-	-	0.00	-	
0	J-9	-	-	0.00	-	
0	L-11	-	-	0.00	-	
0	N-13	-	-	0.00	-	
0	R-17	-	-	0.00	-	
0	T-19	-	-	0.00	-	
0	X-23	-	-	0.00	-	
	A-2	-254	-1771	0.14	PASS	Inclined Members
	E-6	-263	-663	0.40	PASS	
	E-8	-305	-539	0.57	PASS	
	I-10	-252	-1517	0.17	PASS	
	K-12	-8	-351	0.02	PASS	
	K-14	-437	-1141	0.38	PASS	
	Q-18	-324	-1673	0.19	PASS	
	S-20	-305	-1671	0.18	PASS	
	W-24	-457	-1203	0.38	PASS	

Figure 5-34: STM-CAP output summary forces and utilization ratios for Bridge 6

5.8.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-35** shows the strut-and-tie model and the member forces. **Figure 5-36** shows the utilization ratios for each STM members.

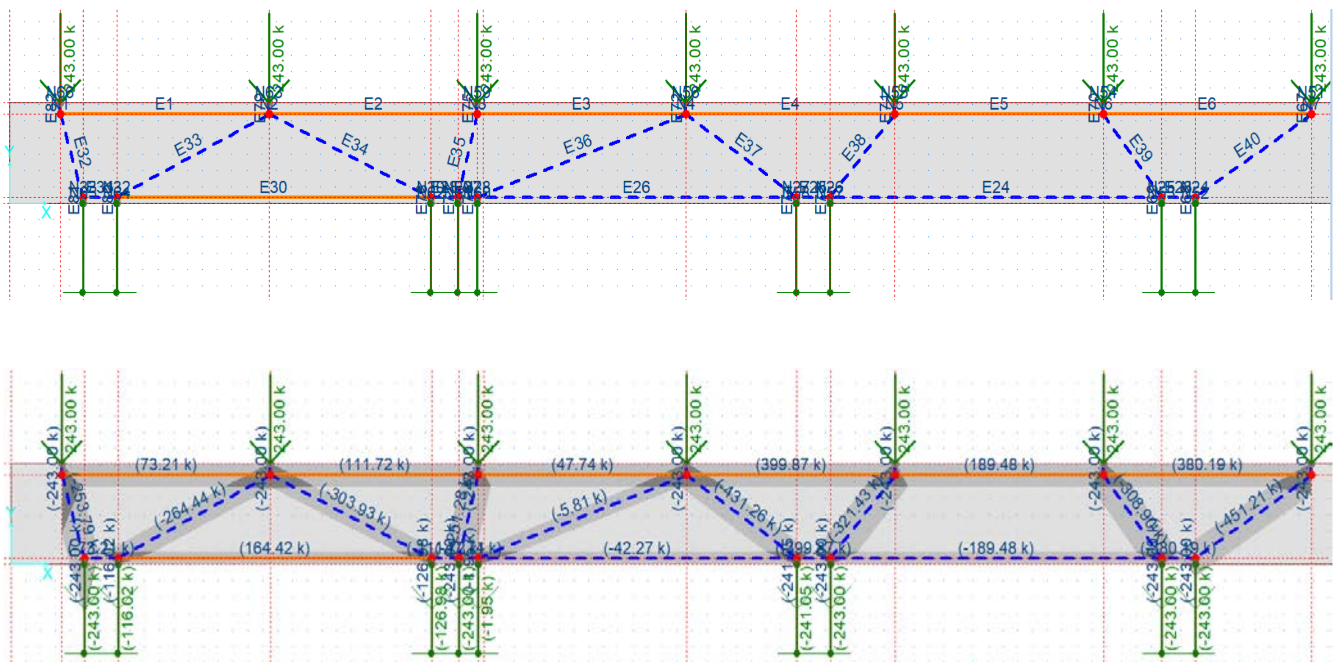


Figure 5-35: CAST model for the strut-and-tie method for Bridge 6

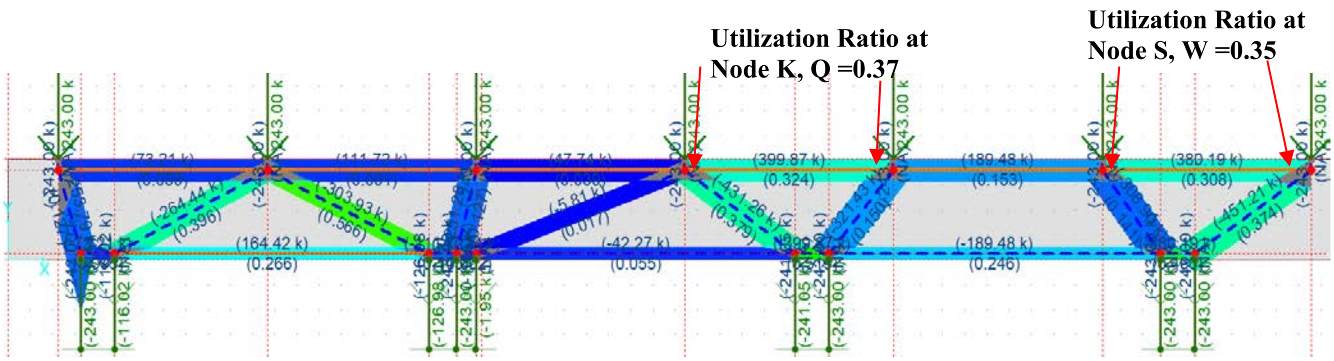


Figure 5-36: CAST analysis output for Bridge 6

5.8.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-7.** The results for each method are similar. Thus, The CAST model verifies the STM-CAP and proves its validity.

Table 5-7: Comparison of the results from the STM-CAP and CAST model for Bridge 6

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-E	73	73	0.06	0.06
E-I	114	112	0.11	0.09
I-K	48	48	0.04	0.04
K-Q	401	400	0.37	0.37
Q-S	187	190	0.17	0.15
S-W	372	380	0.34	0.35
2-6	-73	-73	0.11	0.10
6-8	164	164	0.26	0.27
8-10	-114	-112	0.17	0.15
10-12	-48	-47	0.06	0.06
12-14	-40	-42	0.05	0.05
14-18	-401	-400	0.52	0.52
18-20	-187	-189	0.24	0.25
20-24	-372	-380	0.55	0.49
A-2	-254	-254	0.14	0.13
E-6	-263	-264	0.40	0.40

E-8	-305	-304	0.57	0.57
I-10	-252	-251	0.17	0.17
K-12	-8	-6	0.02	0.02
K-14	-437	-431	0.38	0.38
Q-18	-324	-321	0.19	0.19
S-20	-305	-309	0.18	0.18
W-24	-457	-451	0.38	0.37

5.9 Bridge 7

The left pier cap of the southbound cap was modeled for Bridge 7 for which the elevation is shown in **Figure 5-37.** This pier cap is slightly asymmetrical but modeled as a symmetrical pier cap. The analysis result summary is explained below.

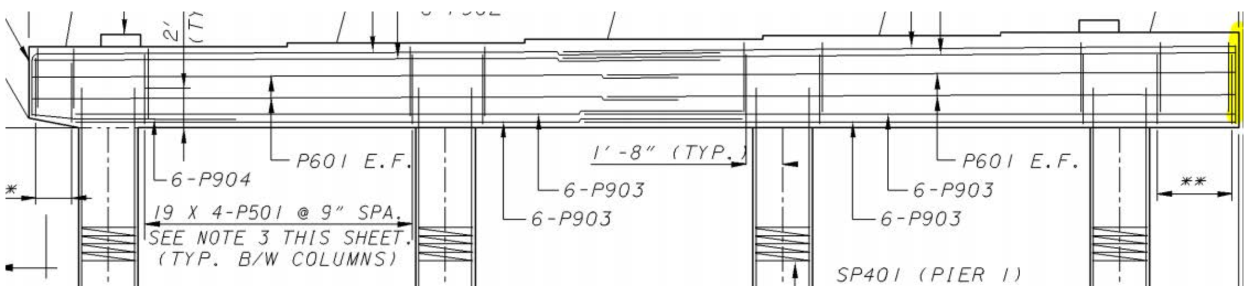
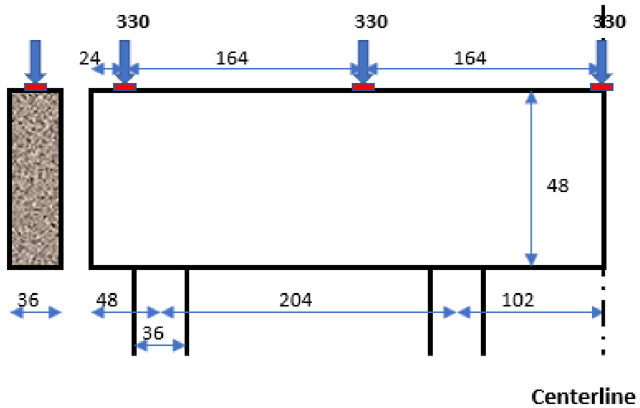


Figure 5-37: Pier cap elevation for Bridge 7

5.9.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-38** shows the check for the deep beam. **Figure 5-39** and **Figure 5-40** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	19.70 in	0.46	Deep Region
R2	0.00 in	0.00	Zero Region
R3	126.30 in	2.92	Slender Region
R4	53.33 in	1.23	Deep Region
R5	0.00 in	0.00	Zero Region
R6	93 in	2.15	Deep Region

This pier cap is deep.
Please continue with Section 7.

Most regions are deep.

Figure 5-38: STM-CAP deep beam check (all units are in inches) for Bridge 7

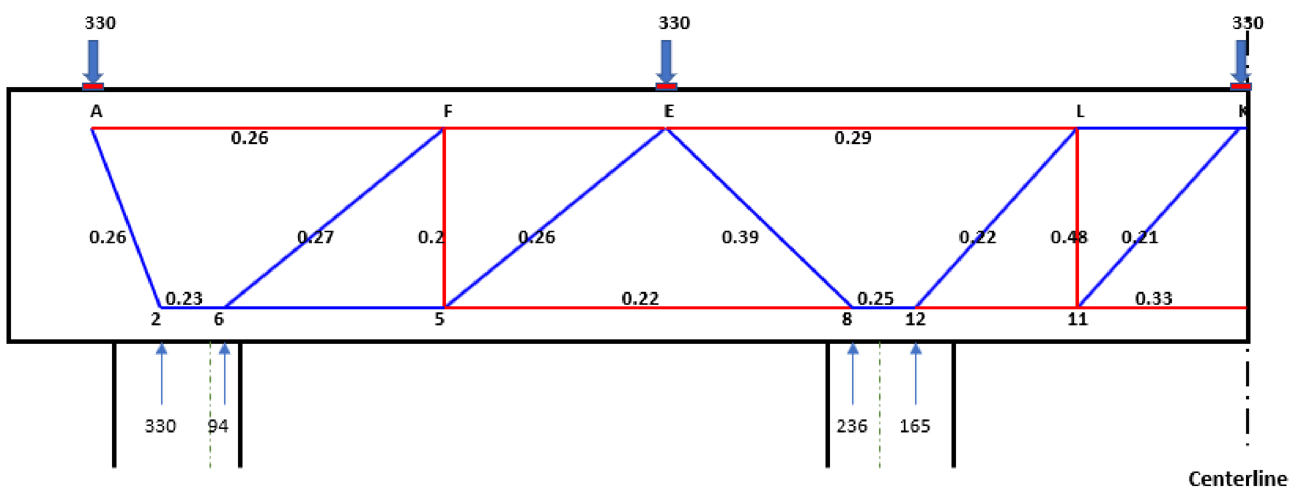


Figure 5-39: STM-CAP output model with utilization ratios for Bridge 7

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
			Summary			
	A-F	171	648	0.26	PASS	Top Members
	E-L	189	648	0.29	PASS	
	K-M	-214	-857	0.25	PASS	
	2-6	-171	-756	0.23	PASS	Bottom Members
	5-8	142	648	0.22	PASS	
	8-12	-189	-756	0.25	PASS	
	11-14	214	648	0.33	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	94	470	0.20	PASS	
	H-7	-	-	0.00	-	
	L-11	165	345	0.48	PASS	
0	A-2	-372	-1422	0.26	PASS	Inclined Members
1	F-6	-183	-686	0.27	PASS	
	E-5	-183	-701	0.26	PASS	
0	E-8	-406	-1044	0.39	PASS	
1	L-12	-260	-1171	0.22	PASS	
	K-11	-260	-1246	0.21	PASS	

Figure 5-40: STM-CAP output summary forces and utilization ratios for Bridge 7

5.9.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-41** shows the strut-and-tie model and the member forces. **Figure 5-42** shows the utilization ratios for each STM members.

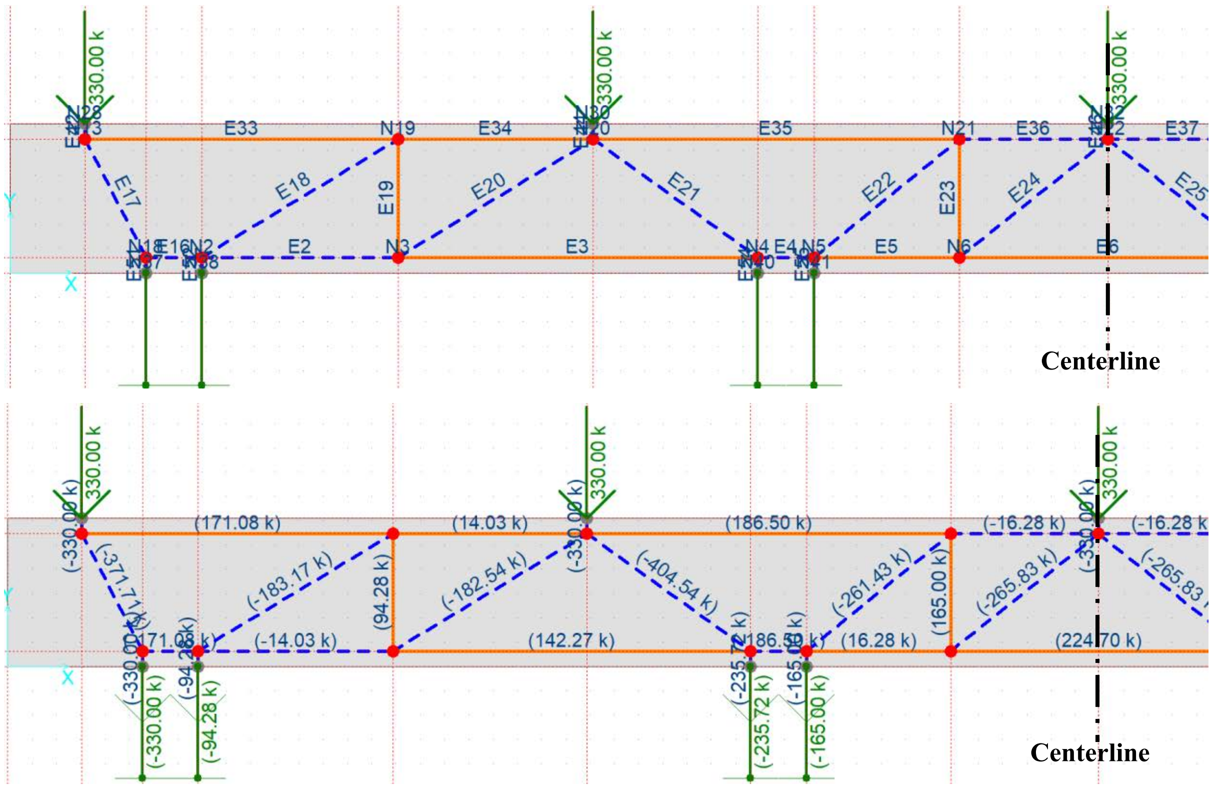


Figure 5-41: CAST model for the strut-and-tie method for Bridge 7

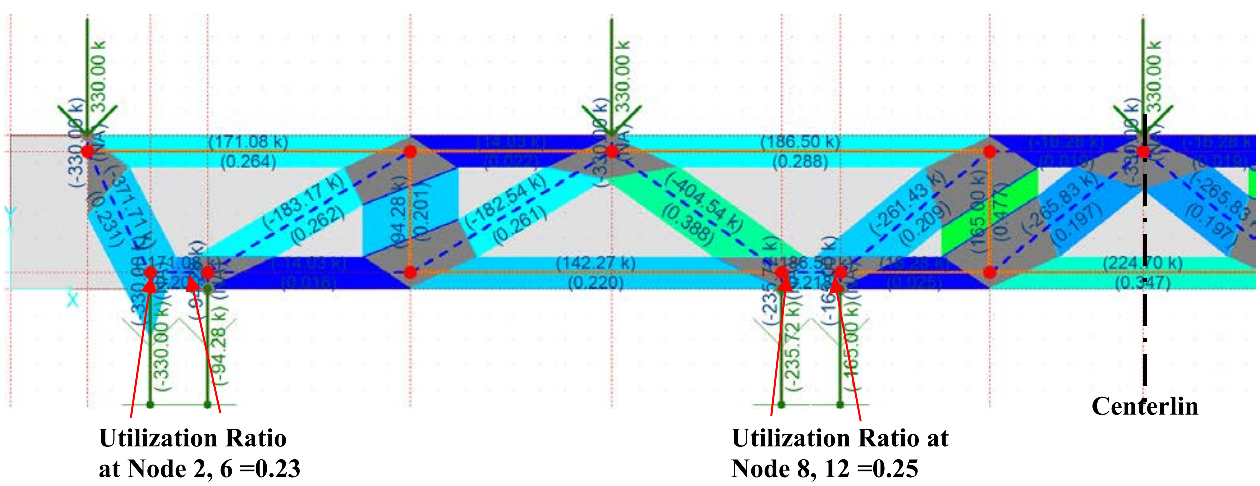


Figure 5-42: CAST analysis output for Bridge 7

5.9.3 Results and Discussions

The member forces and the utilization ratios from STM-CAP and the CAST model are compared in **Table 5-8.** The results for each method are similar. Thus, The CAST model verifies the STM-CAP and proves its validity.

Table 5-8: Comparison of the results from the STM-CAP and CAST model for Bridge 7

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-F	171	171	0.26	0.26
E-L	189	187	0.29	0.29
2-6	-171	-171	0.23	0.23
5-8	142	142	0.22	0.22
8-12	-189	-187	0.25	0.25
11+	214	225	0.33	0.34
F-5	94	94	0.20	0.20
L-11	165	165	0.48	0.48
A-2	-372	-372	0.26	0.23
F-6	-183	-183	0.27	0.26
E-5	-183	-183	0.26	0.26
E-8	-406	-405	0.39	0.39
L-12	-260	-261	0.22	0.21
K-11	-260	-266	0.21	0.20

5.10 Bridge 8

The right pier cap of the southbound cap was modeled for Bridge 8 for which the elevation is shown in **Figure 5-43.** This pier cap is asymmetrical and therefore modeled as Bridge 8 pier cap as explained above. The analysis result summary is explained below.

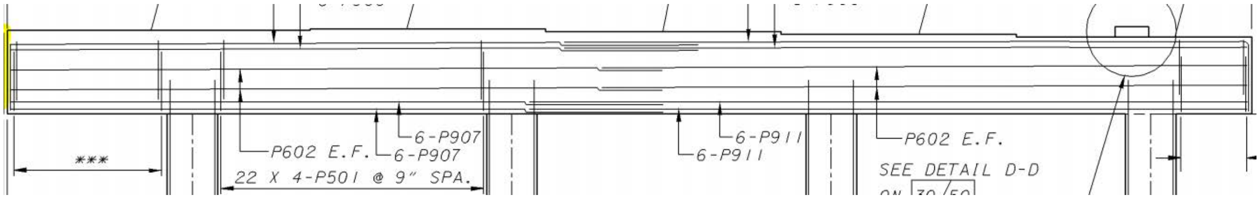
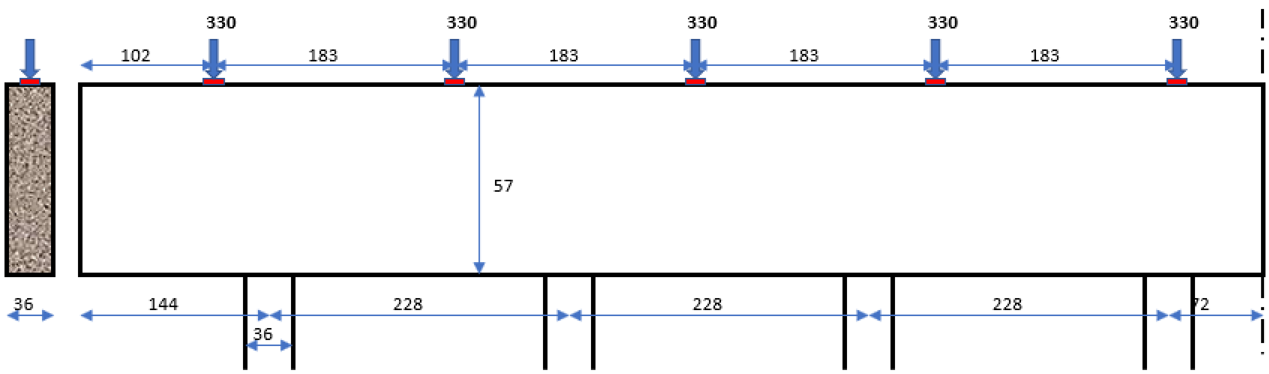


Figure 5-43: Pier cap elevation for Bridge 8

5.10.1 Modeling with STM-CAP

The analysis results are presented below with the help of different figures. **Figure 5-44** shows the check for the deep beam. **Figure 5-45** and **Figure 5-46** show the strut-and-tie method utilization ratios and results summary table.



6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	37.03 in	0.72	Deep Region
R2	0.00 in	0.00	Zero Region
R3	127.97 in	2.49	Deep Region
R4	78.30 in	1.53	Deep Region
R5	0.00 in	0.00	Zero Region
R6	87 in	1.69	Deep Region
R7	120 in	2.35	Deep Region
R8	0 in	0.00	Zero Region
R9	45 in	0.87	Deep Region
R10	162 in	3.16	Slender Region
R11	3 in	0.05	Deep Region
R12	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

Most regions are deep.

Figure 5-44: STM-CAP deep beam check (all units are in inches) for Bridge 8

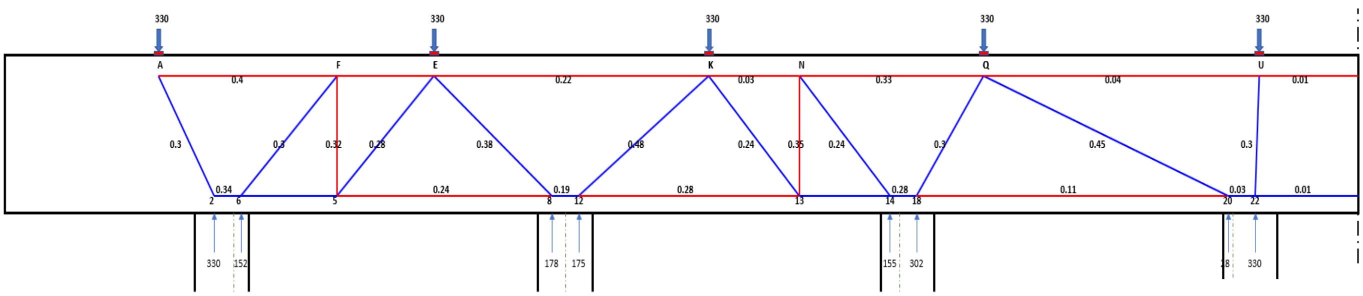


Figure 5-45: STM-CAP output model with utilization ratios for Bridge 8

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	A-F	260	648	0.40	PASS	Top Members
	E-K	142	648	0.22	PASS	
	K-N	17	648	0.03	PASS	
	N-Q	215	648	0.33	PASS	
	Q-S	-24	-655	0.04	PASS	
	Q-U	25	648	0.04	PASS	
	U-W	6	648	0.01	PASS	
	2-6	-260	-756	0.34	PASS	Bottom Members
	5-8	154	648	0.24	PASS	
	8-12	-142	-756	0.19	PASS	
	12-13	181	648	0.28	PASS	
	14-18	-215	-756	0.28	PASS	
	18-20	72	648	0.11	PASS	
	20-22	-25	-756	0.03	PASS	
	22-24	-6	-857	0.01	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	152	476	0.32	PASS	
	H-7	-	-	0.00	-	
	L-11	-	-	0.00	-	
	N-13	155	448	0.35	PASS	
	R-17	-	-	0.00	-	
	T-19	-	-	0.00	-	
	V-21	-	-	0.00	-	
0	A-2	-420	-1418	0.30	PASS	Inclined Members
1	F-6	-257	-853	0.30	PASS	
	E-5	-257	-914	0.28	PASS	
0	E-8	-346	-899	0.38	PASS	
0	K-12	-368	-769	0.48	PASS	
1	K-13	-251	-1027	0.24	PASS	
	N-14	-251	-1027	0.24	PASS	
0	Q-18	-417	-1398	0.30	PASS	
0	Q-20	-101	-227	0.45	PASS	
0	U-22	-331	-1084	0.30	PASS	

Figure 5-46: STM-CAP output summary forces and utilization ratios for Bridge 8

5.10.2 Modeling with CAST

The CAST verification results are explained with the help of figures. **Figure 5-47** shows the strut-and-tie model and the member forces. **Figure 5-48** shows the utilization ratios for each STM members.

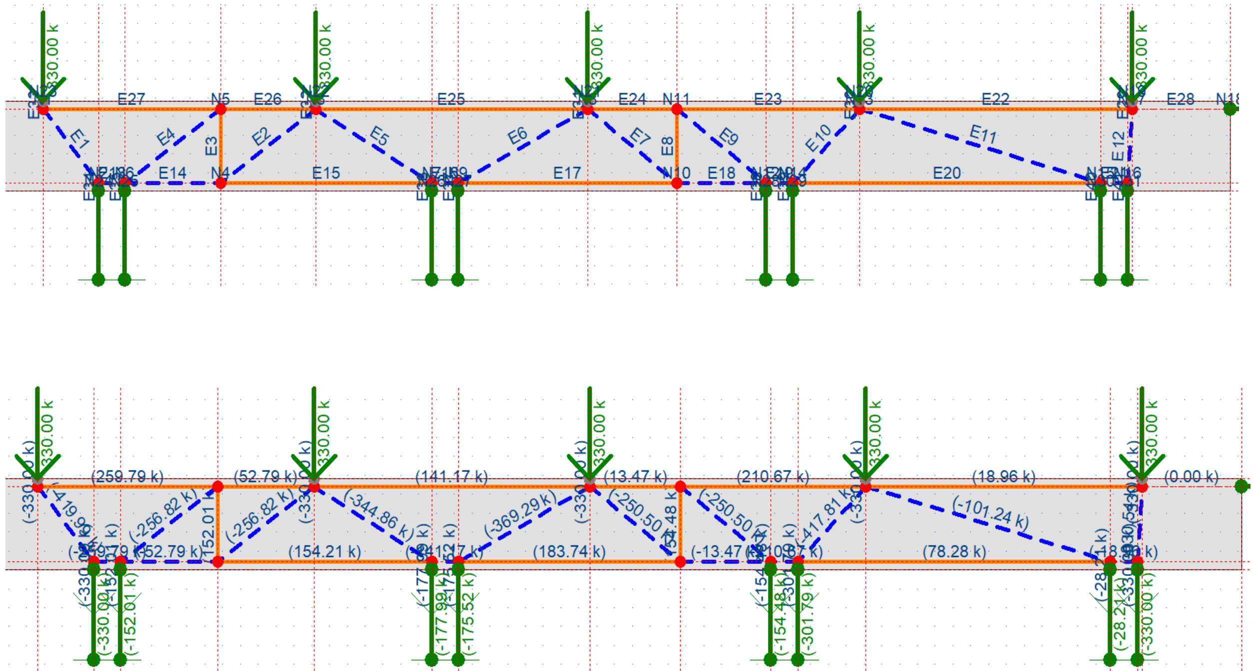


Figure 5-47: CAST model for the strut-and-tie method for Bridge 8

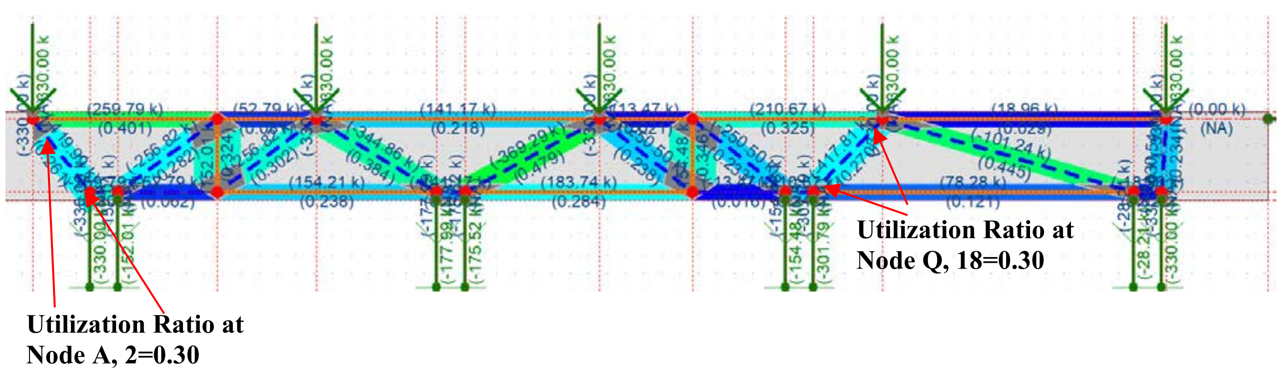


Figure 5-48: CAST analysis output for Bridge 8

5.10.3 Results and Discussions

The member forces and the utilization ratios from the STM-CAP and the CAST model are compared in **Table 5-9.** The results for each method are similar. The CAST model verifies the STM-CAP and proves its validity.

Table 5-9: Comparison of the results from the STM-CAP and CAST model for Bridge 8

Member	Member Forces (kips)		Utilization Ratio	
	STM-CAP	CAST	STM-CAP	CAST
A-F	260	260	0.40	0.40
E-K	142	141	0.22	0.22
K-N	17	13	0.03	0.02
N-Q	215	211	0.33	0.33
Q-U	25	19	0.04	0.03
2-6	-260	-260	0.34	0.30
5-8	154	154	0.24	0.24
8-12	-142	-141	0.19	0.17
12-13	181	184	0.28	0.28
14-18	-215	-211	0.28	0.25
18-20	72	78	0.11	0.12
20-22	-25	-19	0.03	0.02
F-5	152	152	0.32	0.32
N-13	155	154	0.35	0.35
A-2	-420	-420	0.30	0.30
F-6	-257	-257	0.30	0.28
E-5	-257	-257	0.28	0.30
E-8	-346	-345	0.38	0.38
K-12	-368	-369	0.48	0.48
K-13	-251	-251	0.24	0.24
N-14	-251	-251	0.24	0.23
Q-18	-417	-418	0.30	0.30
Q-20	-101	-101	0.45	0.45
U-22	-331	-331	0.30	0.27

5.11 Conclusions

This chapter presented the verification of the STM-CAP with the CAST software. Based on the numerical modeling of the existing bridge pier caps in Ohio, STM-CAP provided identical results to CAST in most of cases because both programs work using the same principles of the strut-and-tie conceptualization. In cases of discrepancy, the difference in the utilization ratios between these two methods was under 5%. One reason for these discrepancies is related to the geometrical simplifications made in CAST which uses a grid with constant spacing. STM-CAP permits more accurate input of the bridge geometry (e.g., a girder spacing of 13' and 11.5''). The other reason may involve round off errors. Verification with the hand calculations indicated that STM-CAP is more accurate in such cases.

In some rare cases, STM-CAP provided lower utilization ratios than CAST for the horizontal struts. For example, for Member 6-7 of Bridge 2, STM-CAP provided 18% lower utilization ratio due to higher capacity predictions than those obtained from CAST. The reason for this is that STM-CAP considers reinforced struts in horizontal directions while CAST neglects it. Reinforced struts account for the embedded rebar in compression zones. They rarely govern the capacities because the nodes usually have lower capacities and thus govern more often. In the cases where the reinforced struts govern, STM-CAP provides more accurate and smaller utilization ratios.

Chapter 6 Nonlinear Finite Element Analysis using program VecTor2

6.1 Introduction

This chapter is written by Mr. Anish Sharma, who is a MS candidate at The University of Toledo. For the completeness of the analysis result and conclusion, this chapter has been included in this thesis.

AASHTO LFRD (2014) requires the use of either a strut-and-tie or a nonlinear finite element analysis for deep beams. The objective of this chapter is to accurately simulate the behavior of the deep cap beams using a nonlinear finite element analysis method, VecTor2, and compare the results with the strut and tie method based on AASHTO LRFD (abbreviated as STM-AASHTO).

Five pier caps (four with cantilever and one without cantilever spans) of existing bridges in Ohio are modeled using nonlinear finite element analysis method VecTor2, which is a non-linear finite element analysis method for two-dimensional structures and is based on the Modified Compression Field Theory (Vecchio and Collins, 1986). The crack patterns and stresses distribution of concrete and reinforcement at the failure and factored loads are presented. The comparison of the STM-AASHTO results with the stress distribution from the nonlinear FEM is performed based on the concept of utilization ratio, which is the ratio of stresses at factored loads divided by the strength of the material. As such, a ratio of, for example, 60%, indicates that the beam has 40% reserve capacity. The utilization ratios are calculated and compared with STM-AASHTO results for concrete and main rebar components with an exception for vertical ties. The reasons for this are: 1) VecTor2 inherently represents the strut behavior due to the deep beam action more accurately, and 2) a single concentrated vertical tie is considered in STM-AASHTO, which gives high-stress ratios, whereas VecTor2 uses a uniform spacing of stirrups, which calculates more distributed and lower stress ratios. As an alternative, the pier cap of Bridge 1 is also modeled with Method 2 to match the STM-AASHTO detailing with a single discrete stirrup band. There was no significant difference in the utilization ratios of the vertical ties while significant difference was found for the governing behavior while using Method 2. Consequently, Method 1 is adopted for the remaining cap beams, which matches the governing member and the mode of failure with STM-AASHTO. In addition, the nonlinear load-displacement response was used to obtain the global capacity of the pier caps.

The maximum utilization ratio of tension ties, horizontal struts, and inclined struts are summarized in **Table 6-1** from three methods: STM-AASHTO, CAST software, and the nonlinear FEM. The governing behavior and the mode of failure matches for the pier caps. The maximum utilization ratio from all three methods, which govern the failure, is found in the same member for the majority of the cases. The utilization ratios from the nonlinear FEM are calculated to be 40% on average of those from STM-AASHTO.

The shear span-to-depth ratios are also compared with the utilization ratios, which is shown in **Figure 6-1**. For the same *a/d* ratio, the utilization ratio is consistently less from the nonlinear FEM than STM-AASHTO. The nonlinear FEM predicts higher capacity, as expected for the deep, as well as slender regions, than those from STM-AASHTO although STM-AASHTO is recommended to be used for deep regions only. The utilization ratios from the nonlinear FEM are consistent in almost every region. Three outliers between *a/d* ratios 1.4 and 2.0, which have a higher utilization ratio in the nonlinear FEM, are the results in the cantilever span of the beam. For *a/d* ratios between 1.5 and 2.0, the nonlinear FEM calculates lower utilization ratios and up to two times higher shear capacity predictions than STM-AASHTO. With the decrease in *a/d* ratio, the discrepancy between the nonlinear FEM and STM-AASHTO decreases and both curves converge at *a/d* ratios less than 0.2.

Table 6-1: Bridge pier cap max utilization ratios summary table

Bridge Name	Pier Cap	Model	Utilization ratios			Nonlinear FEM/ STM-AASHTO
			STM-AASHTO	CAST	Nonlinear FEM	
Bridge 1	Pier 2-Left	Tension Ties	0.71	0.70	0.37	0.52
		Horizontal Struts	0.69	0.69	0.39	0.57
		Inclined Struts	0.49	0.49	0.39	0.80
Bridge 2	Pier 2-Left	Tension Ties	1.02	1.00	0.09	0.09
		Horizontal Struts	0.83	0.80	0.15	0.18
		Inclined Struts	0.35	0.34	0.15	0.43
Bridge 3	North pier cap	Tension Ties	0.51	0.51	0.15	0.29
		Horizontal Struts	0.31	0.31	0.15	0.48
		Inclined Struts	0.55	0.55	0.26	0.47
Bridge 4	Any	Tension Ties	0.48	0.50	0.13	0.27
		Horizontal Struts	0.32	0.31	0.19	0.59
		Inclined Struts	0.54	0.54	0.21	0.39
Bridge 5	Any	Tension Ties	0.34	0.34	0.09	0.26
		Horizontal Struts	0.05	0.05	0.02	0.20
		Inclined Struts	0.44	0.44	0.17	0.39
Average						**0.40**

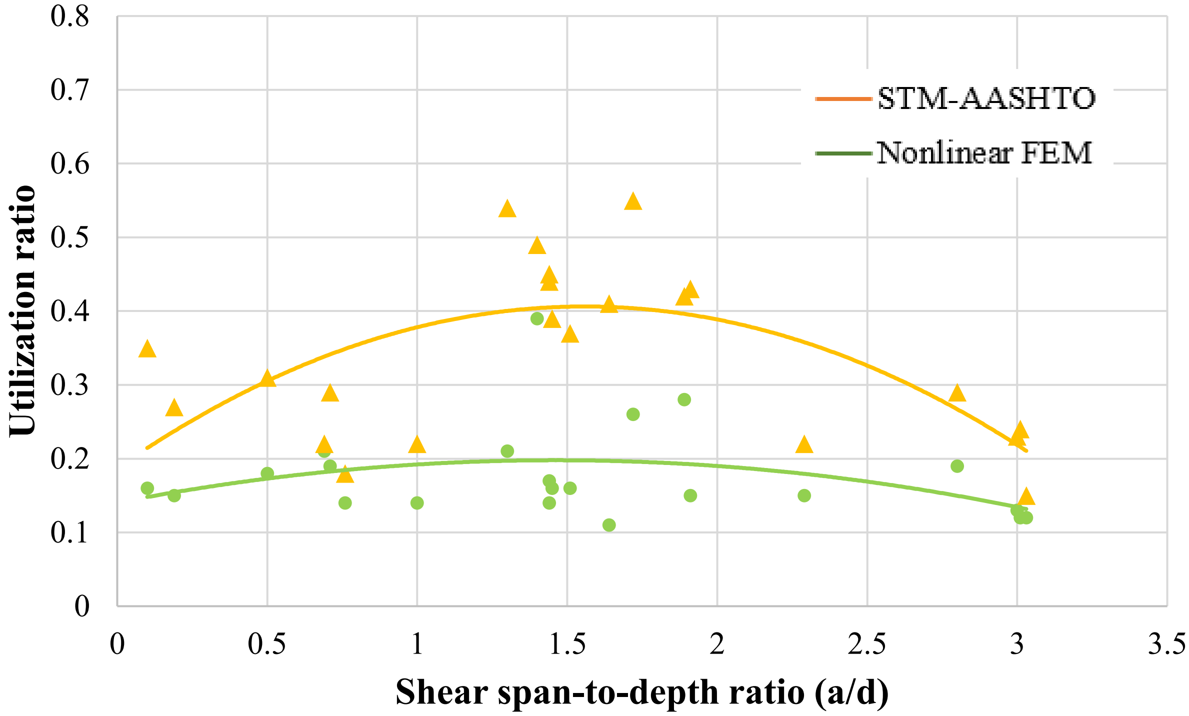


Figure 6-1: Utilization ratio from STM-AASHTO and Nonlinear FEM vs *a/d* ratio

6.2 Bridge 1

Pier-line 2 of the bridge is selected for analysis. This pier cap is symmetrical with cantilever ends and three supporting columns (see **Figure 6-2).** The reinforcement details for the beam at Section A-A and B-B is shown in **Figure 6-3.**

The pier cap for Bridge 1 is modeled with two methods. Method 1 uses smeared stirrups to represent the actual conditions while Method 2 uses a discrete stirrup band to match the STM-AASHTO detailing.

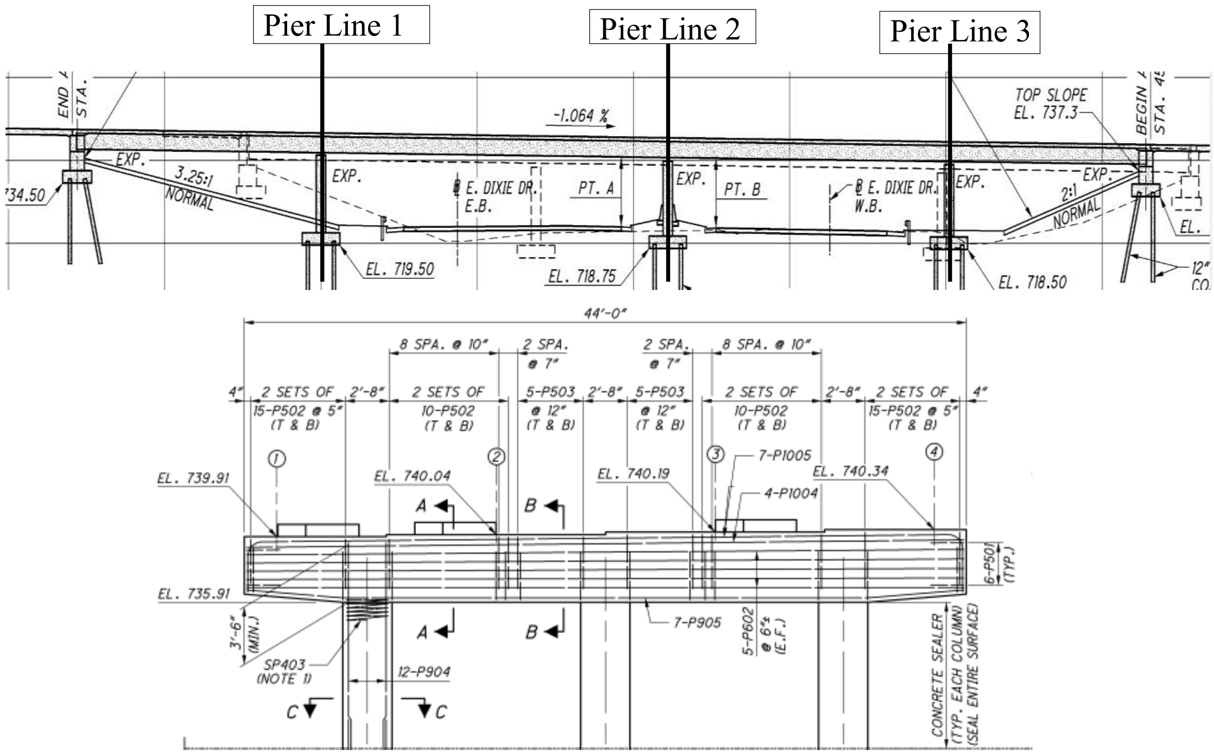


Figure 6-2: Pier cap details for Bridge 1

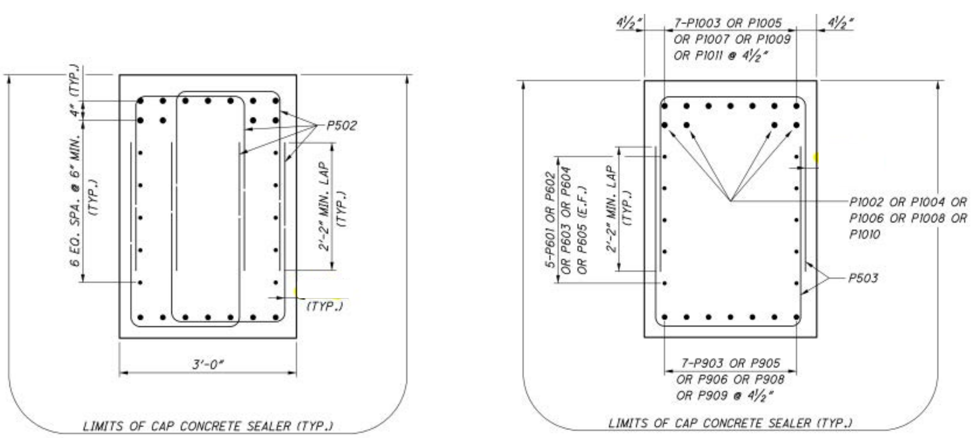


Figure 6-3: Section A-A and Section B-B for Bridge 1

6.2.1 Method 1

Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (415 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are shown in the equations below:

$$f_c = 0.7 f_{ck}$$

$$f_s = 0.9 f_y$$

The material property used in nonlinear FEM is shown in **Figure 6-4.** The ultimate strength of reinforcement (374 MPa) is assumed as nearly the same as yield strength of reinforcement. All dimensions are in SI units.

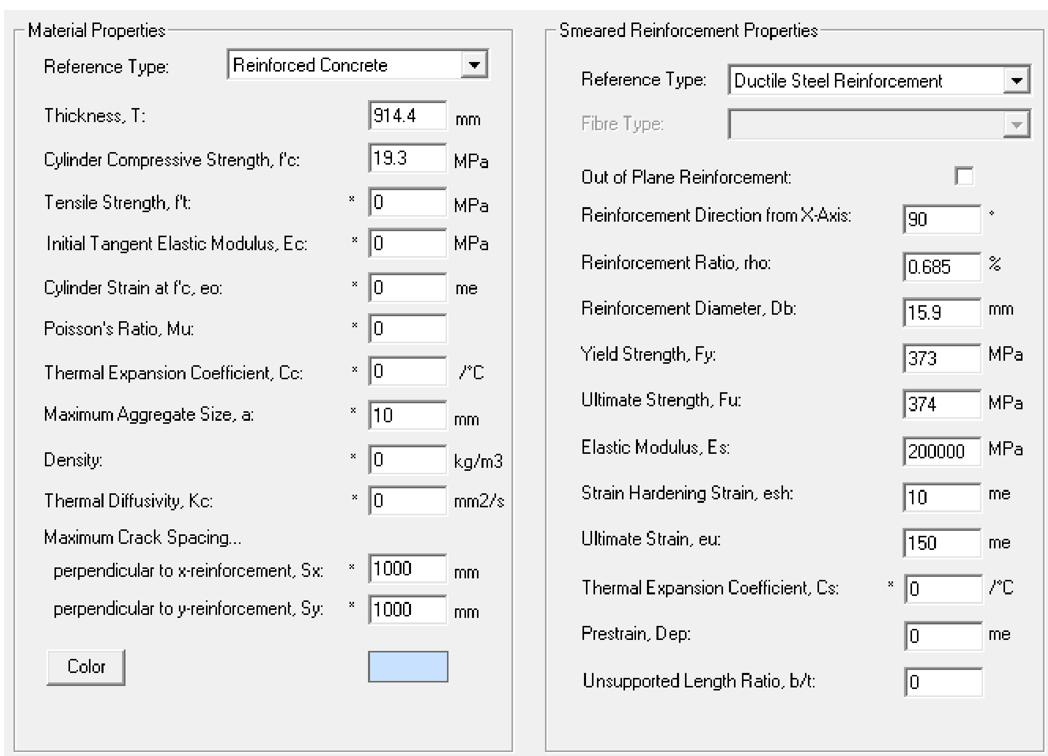


Figure 6-4: Material property for concrete and smeared reinforcement (Beam 1) for Bridge 1

Five different regions were created to represent different smeared reinforcement conditions as shown in **Table 6-2.** Beam-I is the cantilever region of Section A-A (**Figure 6-3**)**.** Beam-II and Beam-III are the mid-region of the cap beam of Section A-A and Section B-B (**Figure 6-3**). The size of the concrete cover is given as two inches (50 mm). The reinforcement ratio for each region, having a cross-sectional area of out of plane reinforcement (*Ab*), spacing (*St*), and width of the cross-section (*Wc*), are calculated as:

$$\rho t = \frac{4\ x\ Ab}{St\ x\ Wc}$$

Table 6-2: Continuum region properties for Bridge 1

Region	Description	Color	f'c (MPa)	Reinforcement ratio
1	Beam-I		19.30	0.69%
2	Beam-II		19.30	0.35%
3	Beam-III		19.30	0.14%
4	Column		19.30	1.18% and 0.367%
5	Concrete cover		19.30	0.00%

The truss bar properties of reinforcement in the cap beam is shown in **Table 6-3.** Top Bar I represent seven reinforcing bars of diameter 32.3 mm; Top Bar II represents four reinforcing bars of diameter 32.3 mm. The side bar is two reinforcing bars of diameter 19.1 mm and the Bottom Bar is seven reinforcing bars of diameter 28.7 mm.

Table 6-3: Truss bar properties for Bridge 1

Truss	Description	Color	Area(mm^2)	fy (MPa)	Diameter(mm)
1	Top Bar-I (7-P10)		5733	373	32.3
2	Top Bar-II (4-P10)		3276	373	32.3
3	Side Bar (2-P6)		568	373	19.1
4	Bottom Bar (7-P9)		4515	373	28.7

The symmetry of the structure allows for modeling one-half of the beam. The finite element model of the cap beam developed in VecTor2 is shown in **Figure 6-5**. Pin supports are defined at the lowermost ends of the columns while rollers are defined at the axis of symmetry. The finite element mesh size of 50 x 50 mm is used.

Factored dead and live loads of 215 kips and 116 kips, are applied on the beam. The load is applied as four-point loads (245 kN, 491 kN, 491 kN, 245 kN) as shown in **Figure 6-6,** where P is a total load of 331 kips (1473 kN). The load is applied in 400 mm, corresponding to bearing pad dimension. The load is applied monotonically at an increment of 10% up to the failure.

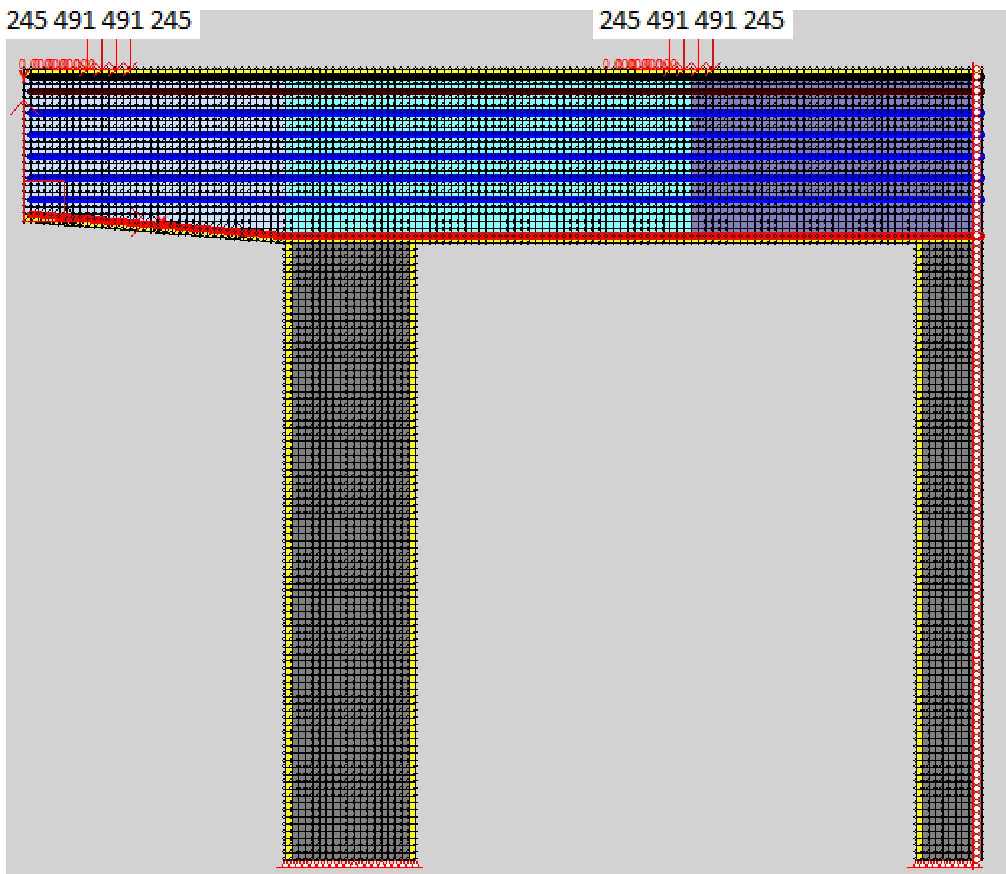


Figure 6-5: Cap beam model in VecTor2 for Bridge 1

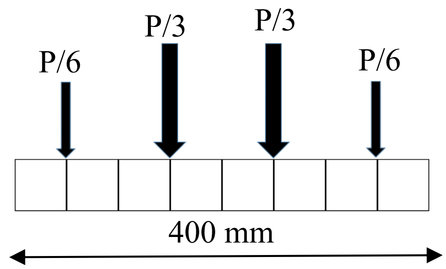
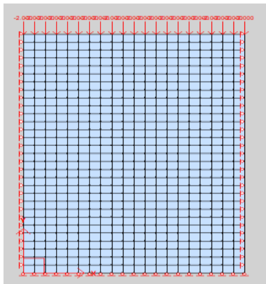


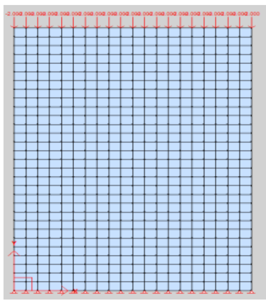
Figure 6-6: Loading condition for Bridge 1

Due to stirrups, there is a confinement effect, which increases the compressive strength of concrete. The confined compressive strength of the concrete is calculated by creating a model of a square cross-section 1000 x 1000 mm with a height of 1500 mm and a compressive strength of 19.3 MPa. The models for the confined and unconfined conditions are shown in **Figure 6-7** (**a**) and **(b).** Pin supports are defined at the lowermost ends and at the side for the confined concrete model. Pin supports are defined at the lowermost ends of the concrete block for the unconfined model. The model is loaded with 2 mm of displacement vertically.

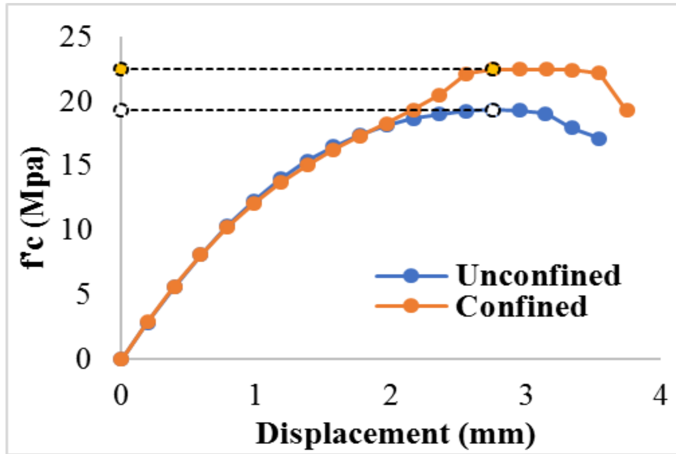
The response of stress and displacement for the confined and unconfined model is shown in **Figure 6-7** (**c**)**.** The confined compressive strength of the concrete is found as 22.5 MPa.



(a) Confined model



(**b**) Unconfined model



(**c**) Strength-displacement response

Figure 6-7: Confined compressive strength of concrete for Bridge 1

Determination of Utilization Ratios

The utilization ratio is the ratio of stress in member (region or element) at factored load divided by the capacity. The utilization ratio for the concrete member is calculated considering the average of highly stressed regions and the reinforcement member is calculated considering the highest stressed single element. The concrete member and reinforcement member are compared with the strut-and-tie members from STM-AASHTO.

Concrete Elements

The failure in concrete occurs over a region. The beam is loaded to failure and the most stressed regions are marked as R1-R9 (**Figure 6-8**). The critical elements with high stress in those regions are found and the average stress presented by them at factored load (**Figure 6-9**) is divided by the confined concrete compressive strength (22.5 MPa) to calculate the utilization ratio. The average stresses and utilization ratio calculated for each region at the factored load are shown in **Figure 6-9**.

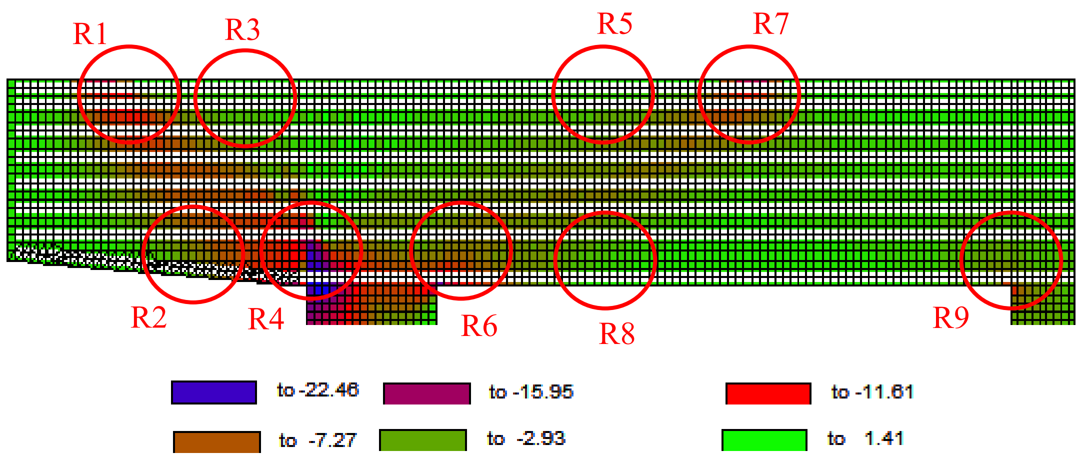


Figure 6-8: Concrete stress at failure for Bridge 1

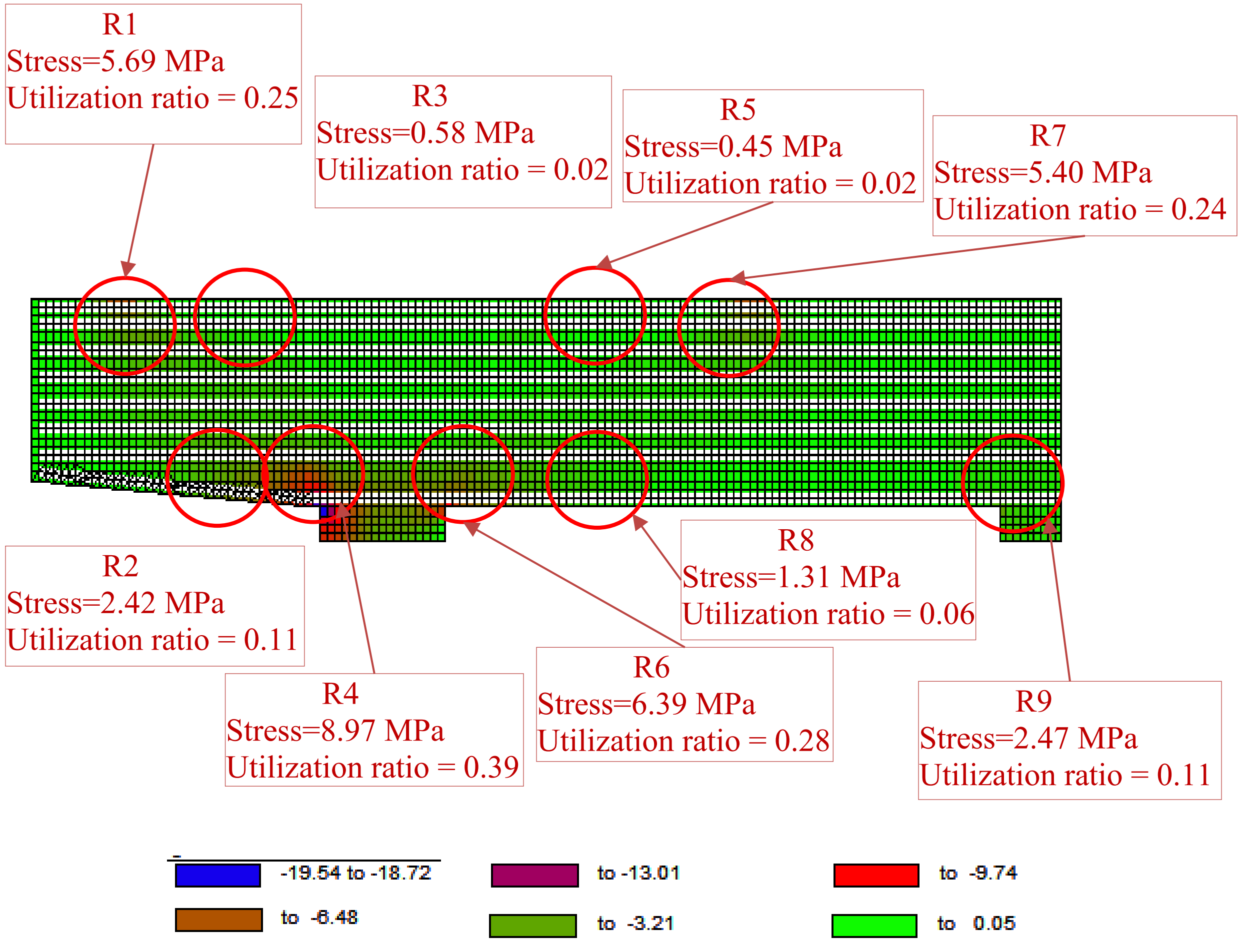


Figure 6-9: Concrete stress at factored load for Bridge 1

The failure is governed by the region with the highest utilization ratio. For example, the utilization ratio of Member A-1 (**Figure 6-14**), which is a concrete member in the cantilever span, is calculated as the highest in Regions R1 and R2. Thus, the utilization ratio of Member A-1 is the utilization ratio of R1 which is 0.25.

The highest utilization ratio between R3 and R4 give the utilization ratio of Member B-2, which is 0.39. For Member F-6 and Member E-5, the utilization ratio is 0.28 and 0.24. For Member E-8, the utilization ratio is the utilization ratio of R9, which is 0.11.

Main Reinforcement Bars

The utilization ratio for the main reinforcing bar is calculated considering only the highest element stress because, unlike the concrete in which the failure happens over an area, the reinforcement failure occurs due to the rupture of a single element.

The average reinforcement stresses at failure load are shown in **Figure 6-10.** The element with the highest stress is identified at failure load (**Figure 6-10)**. The average reinforcement stresses at the factored load with the stress and utilization ratio of the highest stress element are shown in **Figure 6-11.**

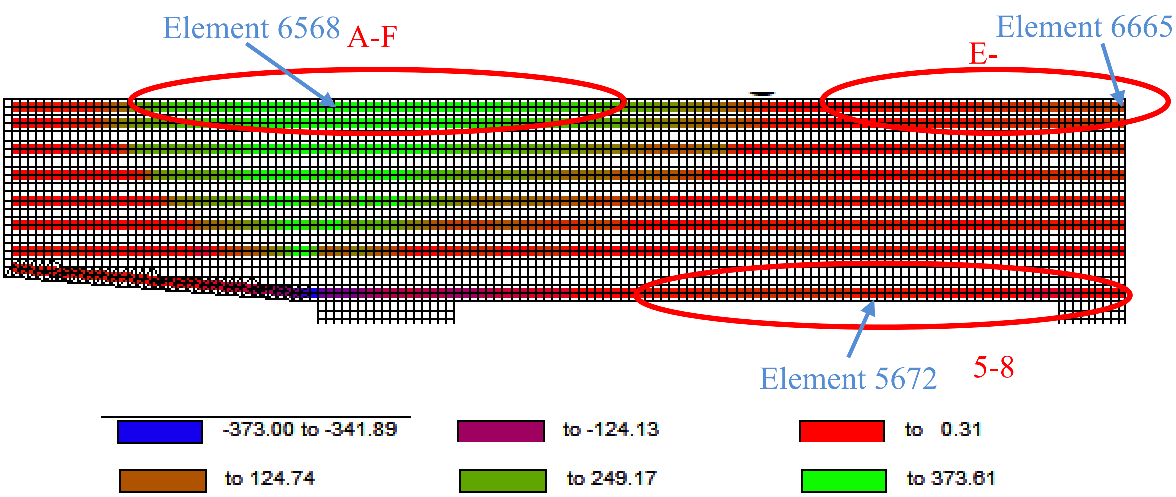


Figure 6-10: Average Reinforcement stresses at failure load for Bridge 1

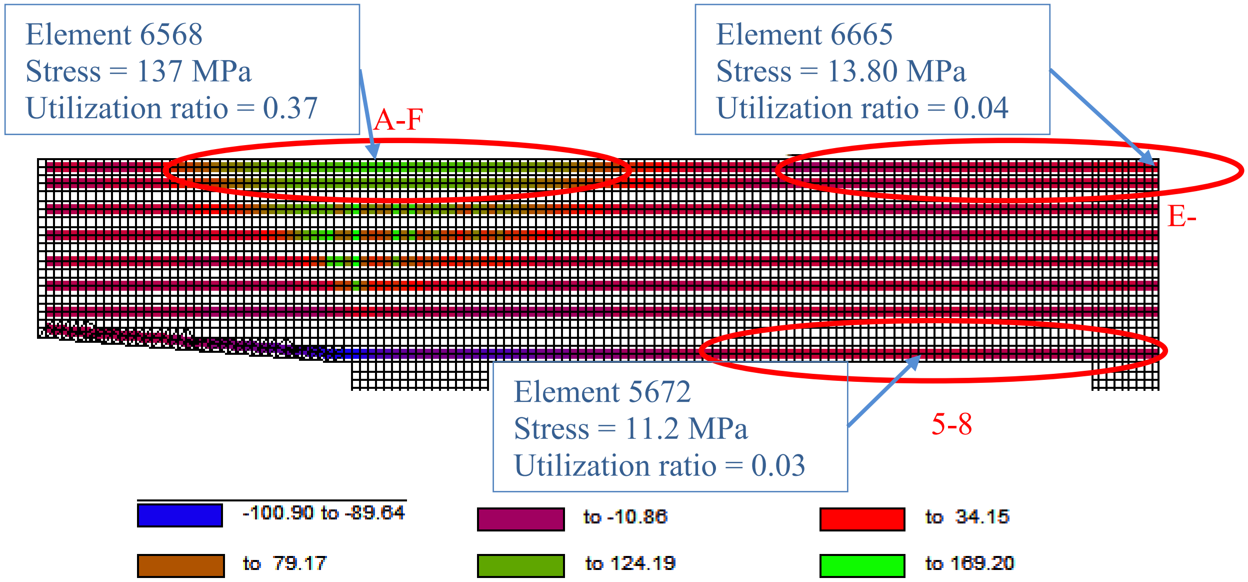


Figure 6-11: Average Reinforcement stresses at factored load for Bridge 1

The element with the maximum stress and strain in Region A-F (top rebar) at failure load is identified as Element 6568 (**Figure 6-10**). At failure load, Element 6568 has the stress of 373 MPa and an average strain of 9.4x10^{-3} m/m. The Element 6568 at factored load has the stress of 137 MPa, which gives a utilization ratio of 0.37. Similarly, for the highest stress element in each region, stress and utilization ratio are calculated as shown in **Figure 6-11.**

Smeared Reinforcement

The average stresses of smeared reinforcement at failure and factored loads are shown in **Figure 6-12** and **Figure 6-13**. Line B-1 and Line F-5 are the location of the vertical tie from the STM-AASHTO. Element 1386 has the highest stress in Line B-1 (**Figure 6-12**) at failure load and 3.90 MPa at factored load, which is a utilization ratio of 0.01. Similarly, Element 2709 has the highest stress in Line F-5 (**Figure 6-12**) and 0.5 MPa at factored load, which gives a utilization ratio of 0.001. The high-stress zones are Region S1 and Region S2.

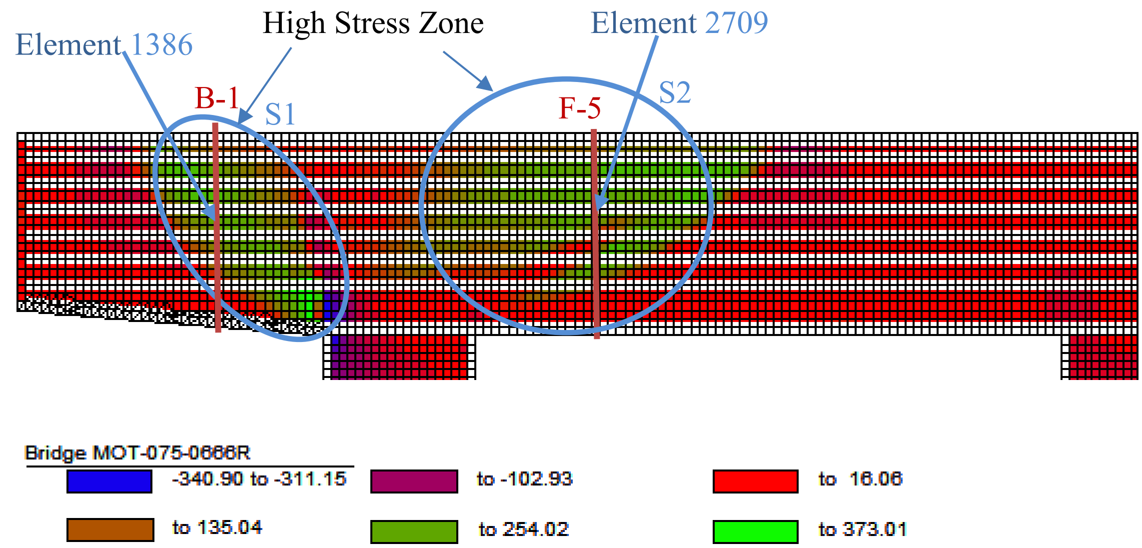


Figure 6-12: Smeared reinforcement stress at failure load for Bridge 1

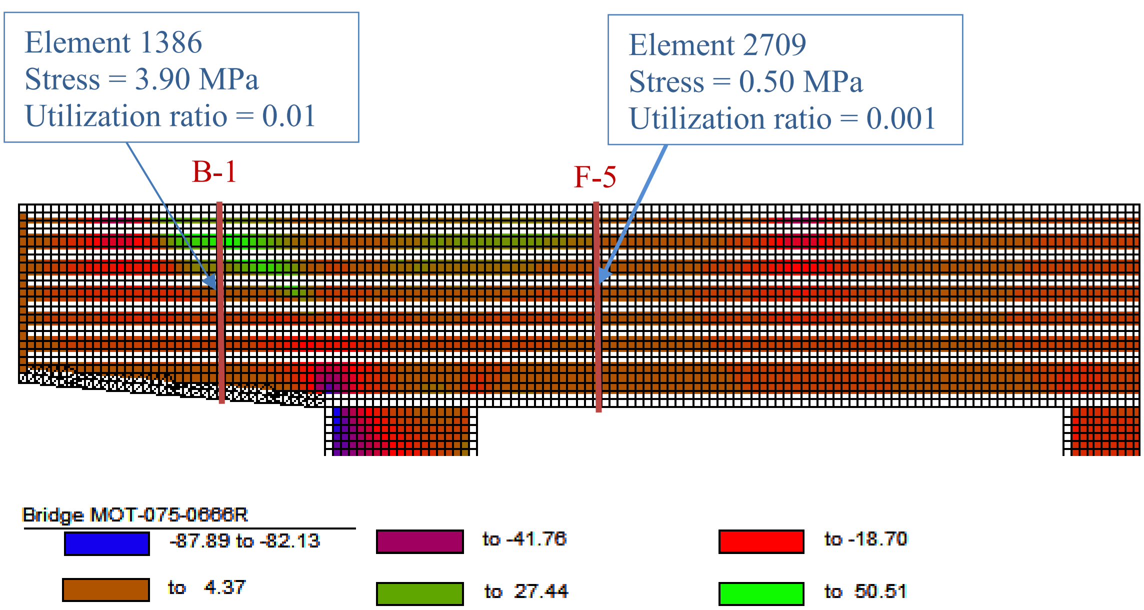


Figure 6-13: Smeared reinforcement stress at factored load for Bridge 1

Comparison of Utilization Ratios with STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses to show contrast to the ratios obtained from STM-AASHTO in **Figure 6-14**. The utilization ratios from the nonlinear FEM were calculated to be 49% on average of those from STM-AASHTO.

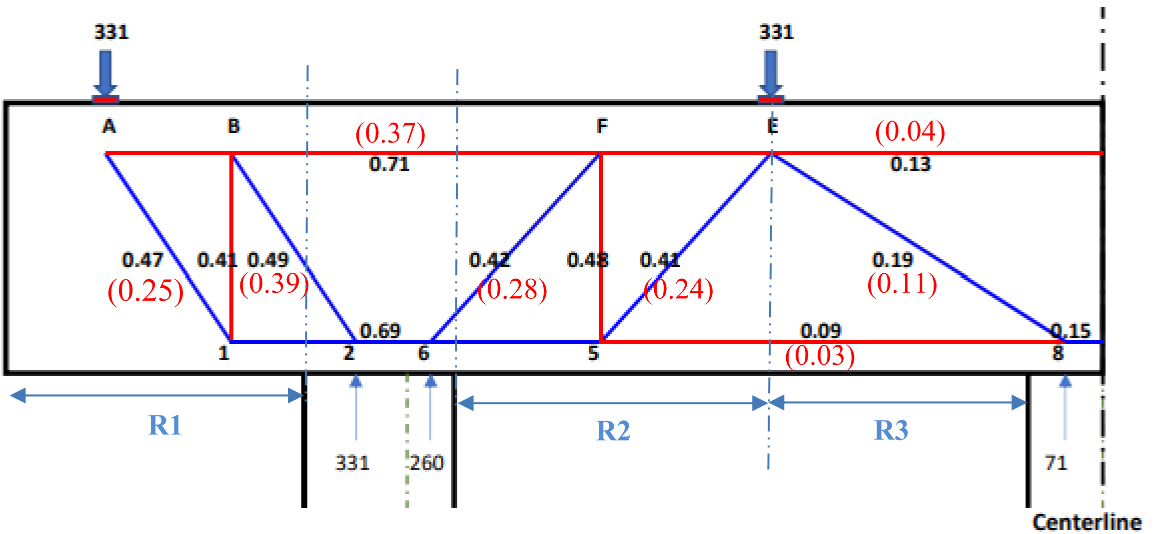


Figure 6-14: Utilization ratio from STM-AASHTO and nonlinear FEM for Bridge 1

Concrete Elements

The utilization ratios are consistently 53% and 80% at the lowest and highest extremes, and 63% on average of the utilization ratios from STM-AASHTO for concrete members. The maximum utilization was predicted for concrete members in the cantilever span, which matched with Member B-2 (**Figure 6-14**) in STM-AASHTO. Concrete in cantilever span governs the failure in nonlinear FEM. The concrete struts (Member A-1 and Member B-2) in the cantilever span of the beam are also highly stressed in STM-AASHTO analysis.

Main Reinforcement Bars

The utilization ratios of the highest stress element calculated for main reinforcement bars are consistently 30% and 53% at the lowest and highest extremes, and 38% on average of STM-AASHTO. Member A-F (**Figure 6-14**), which is the top rebar above the column, has high utilization from both analyses and governs the failure in STM-AASHTO analysis. Low utilization ratio was calculated in at mid-region rebar and bottom rebar from both analyses.

Smeared Reinforcement

The utilization ratio for smeared reinforcement is found to be very low from the nonlinear FEM at locations B-1 and F-5(**Figure 6-14**). In STM-AASHTO, a single vertical tie at location F-5 takes stirrup stresses. However, this stress is distributed along a span of the beam in the nonlinear FEM using VecTor2, thus, it has a very low utilization ratio at location F-5. The high-stress zones are Region S1 and Region S2 as shown in **Figure 6-12.** Line B-1 and Line F-5 calculated from STM-AASHTO fall in these regions.

Effect of a/d Ratio in Utilization Ratios

The trend of the utilization ratio based on the shear span to depth (*a/d*) ratios for the critical concrete members in the regions (**Figure 6-14)** is shown in **Table 6-4.** The depth of beam is forty-three inches. The nonlinear FEM results follow the same trend as STM-AASHTO. For *a/d* ratio of 1.40, the utilization ratio is maximum from both STM-AASHTO and the nonlinear FEM and is minimum from both analyses for *a/d* ratio of 1.64.

Table 6-4: Comparison of utilization ratios with *a/d* ratio for Bridge 1

Shear span (a) (in.)	a/d	Utilization ratios		Remarks
		STM-AASHTO	Nonlinear FEM	
60.00	1.40	0.49	0.39	R1
82.00	1.89	0.42	0.28	R2
71.00	1.64	0.19	0.11	R3

Global Response Result

The cracking pattern of the cap beam at the failure load is shown in **Figure 6-15,** and at the factored load in **Figure 6-16.** The inclined red lines indicate shear failure and vertical red lines indicate flexure failure. Failure occurred by the shear stresses on the cantilever span. The flexural failure of the top reinforcement indicated by vertical cracks occurred above columns.

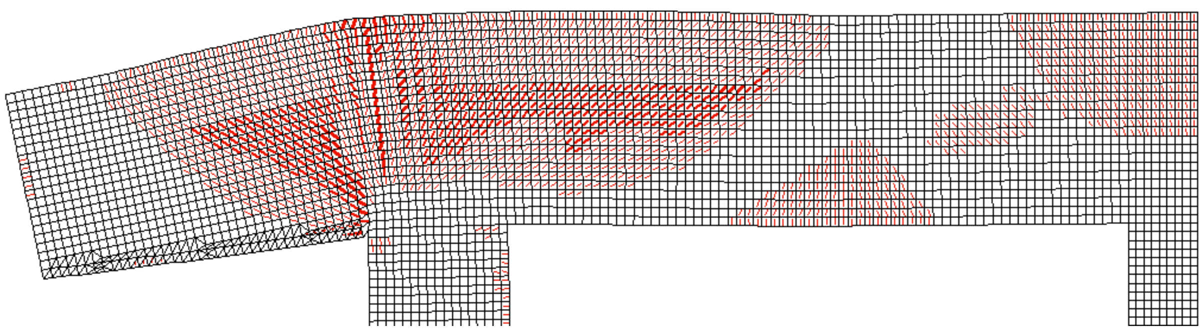


Figure 6-15: Cracking pattern at failure load for Bridge 1

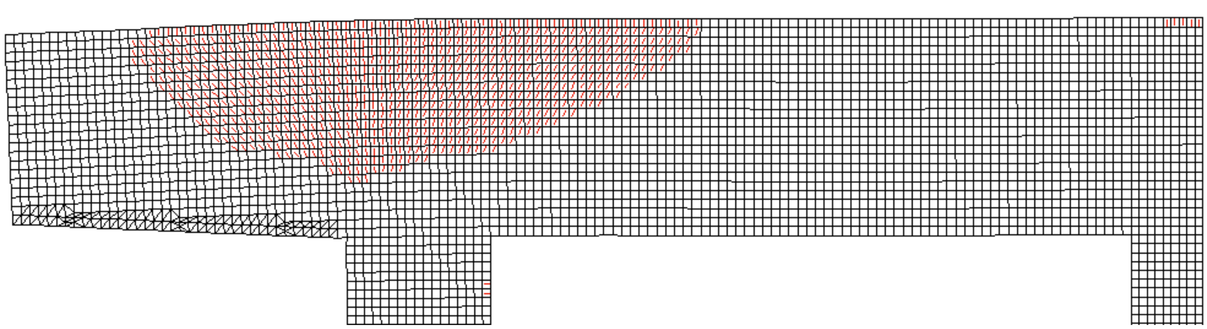


Figure 6-16: Cracking pattern at factored load for Bridge 1

The load-displacement curve is shown in **Figure 6-17.** Load corresponding to 662 kips is factored load acting on the pier cap. Failure occurred at Load Stage 23 with a convergence factor of 1.024, corresponding to a load of 1456 kips. The capacity of pier cap predicted from the STM-AASHTO is 932 kips. The nonlinear FEM is 1.5 times higher capacity prediction than the STM-AASHTO.

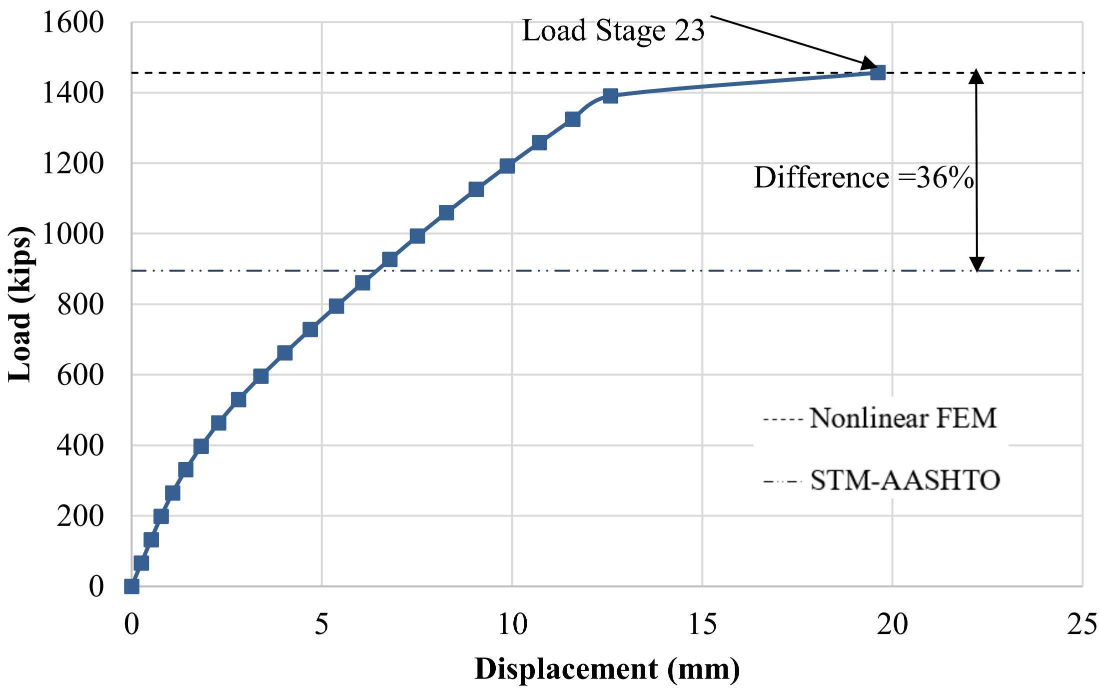


Figure 6-17: Load vs displacement response of cap beam for Bridge 1

6.2.2 Method 2

A new model is developed by replacing the stirrups in the cap beam with the single discrete stirrup band to match the STM-AASHTO detailing.

Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (415 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are used.

The finite element model of the cap beam developed is shown in **Figure 5-18.** The area of the new truss member introduced is equal to the area of nine stirrups between the column and load point. The geometry, restraint condition, and load are the same as the previous model except the stirrups detailing in the beam is replaced by the single vertical ties. In this case, there is no issue with the confinement for the cap beam.

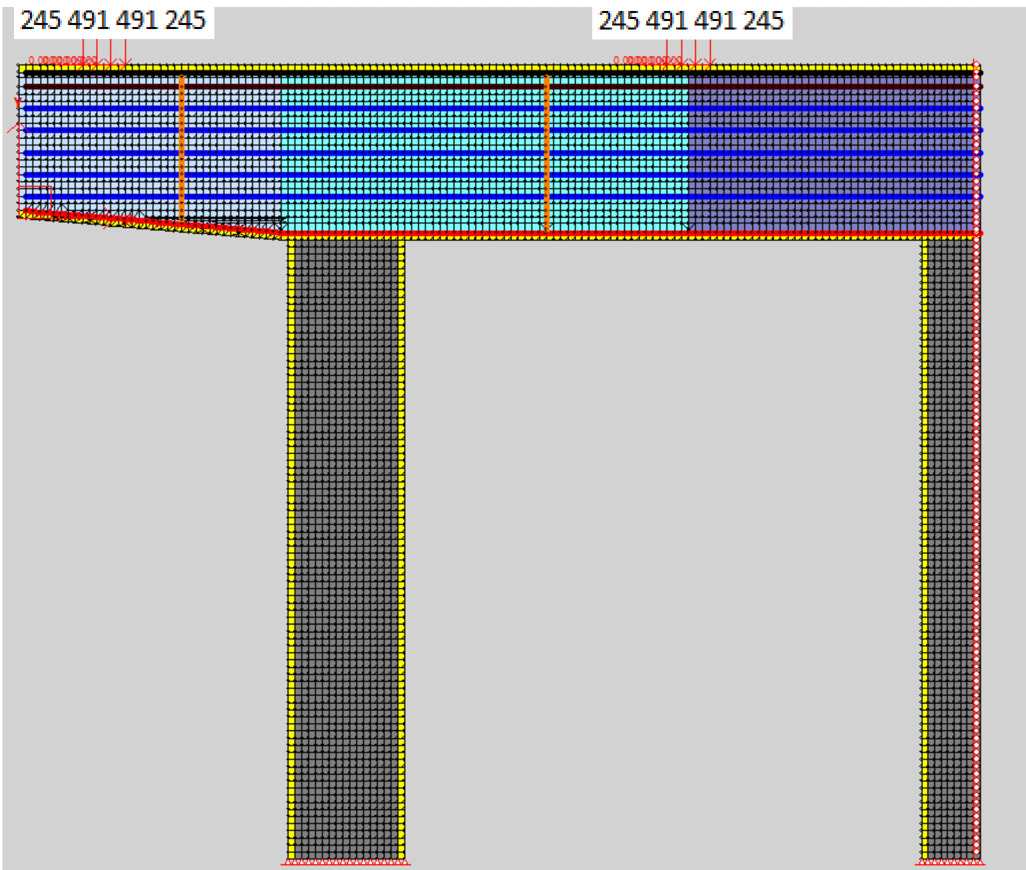


Figure 6-18: Cap beam model in VecTor2 for Bridge 1

Determination of Utilization Ratios

The utilization ratio is the ratio of stress in member (region or element) at factored load divided by the capacity. The utilization ratio for the concrete member is calculated considering the highly stressed regions and the reinforcement member is calculated considering the highest stressed single element.

Concrete Elements

The beam is loaded to failure and the most stressed regions are marked as R1-R8 (**Figure 6-19**). The critical elements with high stress in those regions are found and the average stress presented at factored load (**Figure 6-20**) is divided by the concrete compressive strength (19.3 MPa) to calculate the utilization ratio. The average concrete stresses and utilization ratio calculated for each region at the factored load are shown in **Figure 6-20.**

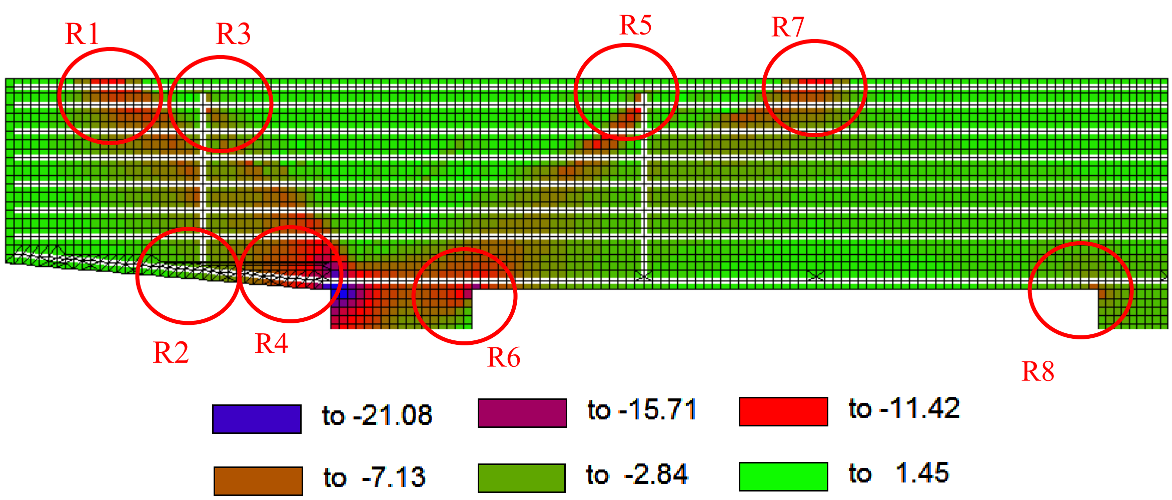


Figure 6-19: Concrete stress at failure for Bridge 1

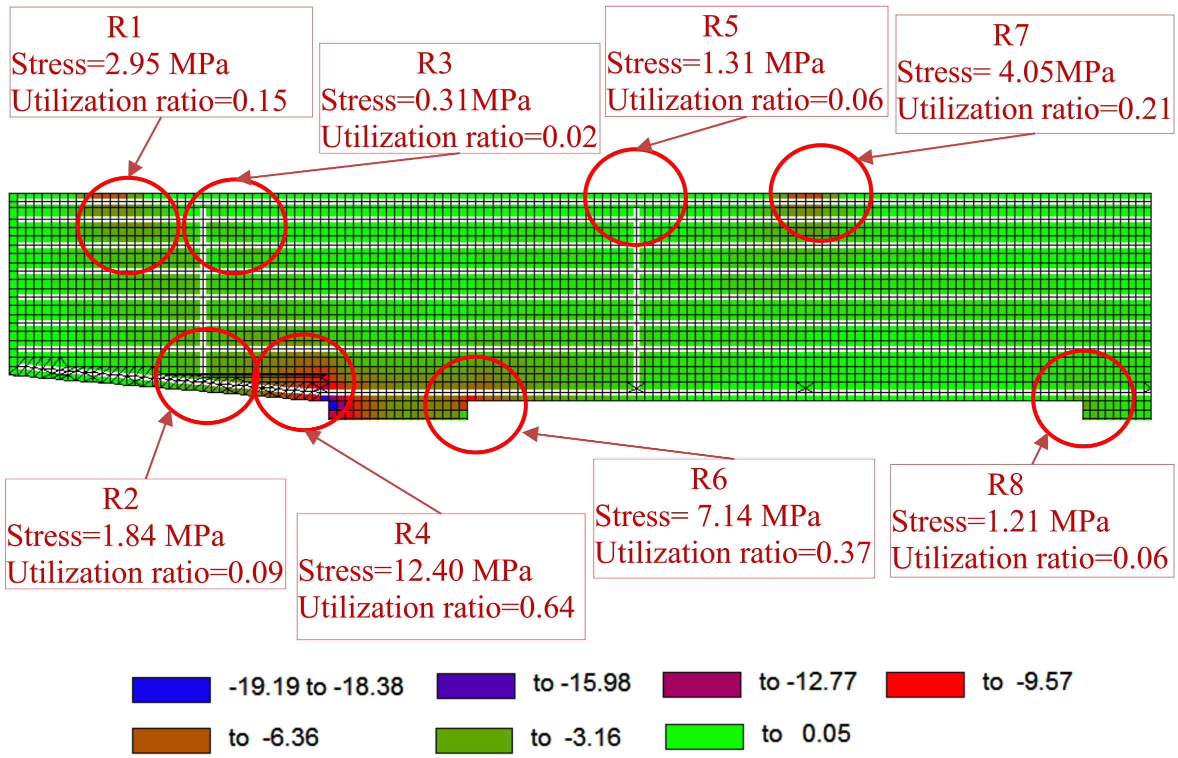


Figure 6-20: Concrete stress at factored load for Bridge 1

The utilization ratio of Member A-1 (**Figure 6-23)**, which is a concrete member in the cantilever span, is calculated as the highest in Regions R1 and R2. Thus, the utilization ratio of Member A-1 is a utilization ratio of R1, which is 0.15.

Similarly, the highest utilization ratio between R3 and R4 give the utilization ratio of Member B-2, which is 0.64. For Member F-6 and Member E-5, utilization ratio is 0.37 and 0.21. For Member E-8, utilization ratio is the utilization ratio of R8 which is 0.06.

Main Reinforcement Bars

The average reinforcement stresses at failure load are shown in **Figure 6-21.** The element with the highest stress is identified at failure load (**Figure 6-21)**. The average reinforcement stresses at factored load with the stress and utilization ratio of the highest stress element are shown in **Figure 6-22.**

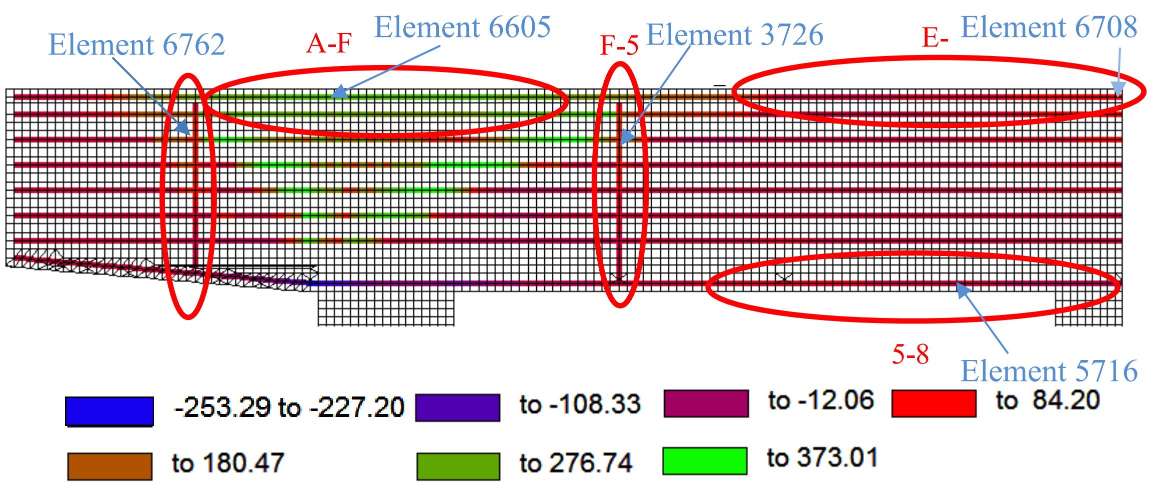


Figure 6-21: Average Reinforcement stresses at failure load for Bridge 1

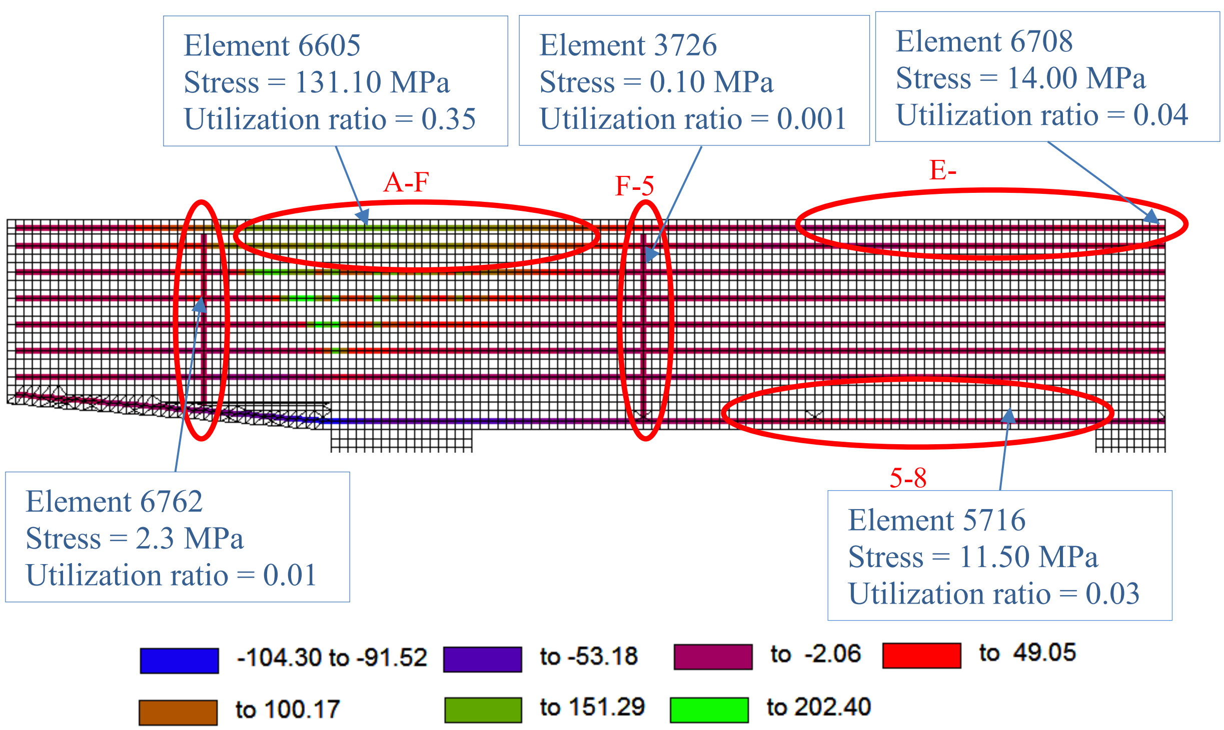


Figure 6-22: Average reinforcement stresses at factored load for Bridge 1

The element with maximum stress and strain in Region A-F indicating top rebar at failure load is identified as Element 6605 **(Figure 6-21).** At failure load, Element 6605 has the stress of 237.70 MPa and an average strain of 1.88×10^{-3} m/m. The stress in Element 6605 at factored load is 131.10 MPa, which is utilization ratio of 0.35. Similarly, stress and utilization ratio for the highest stress element in each region is shown in **Figure 6-22**. There was no significant difference in the utilization ratio of the vertical tie.

Comparison of Utilization Ratios with STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses in contrast to the ratios obtained from the STM-AASHTO in **Figure 6-23**. The utilization ratios from the nonlinear FEM are calculated to be 55% on average of those from the STM-AASHTO.

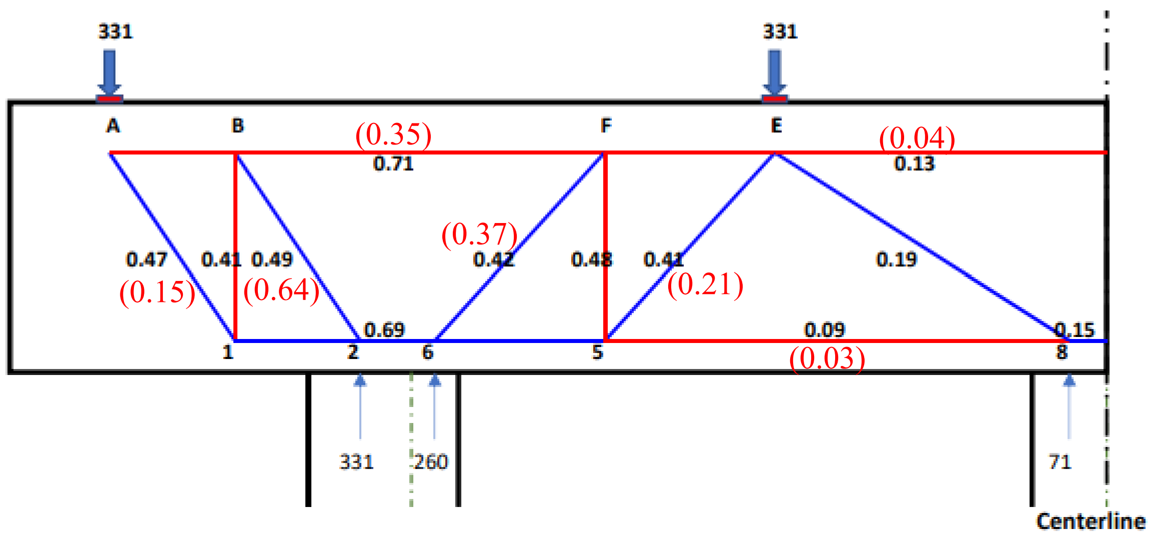


Figure 6-23: Utilization ratio from the STM-AASHTO and nonlinear FEM for Bridge 1

Concrete Elements

The stresses in concrete members are very high for this model. The maximum utilization is calculated for Member B-2(**Figure 6-23**) and it governs the failure for nonlinear FEM. Member F-6 has high utilization ratio. Lack of stirrups and introduction of single stirrup ties creates high stress in the concrete which is indicated by Member B-2 (0.64) and Member E-5 (0.21) with high utilization ratio.

Main Reinforcement Bars

The utilization ratios of the highest stress element for the main reinforcement bars are consistently 51% and 70% of the STM-AASHTO. Member A-F (**Figure 6-23**), which is the top rebar above the column, has high utilization from both analyses. The stress on rebar is less than Method 1. There is no yielding of the top rebar at failure load. The utilization ratio of the truss element used to represent the stirrups gives less utilization ratio. Discrete stirrup band doesn't give the utilization ratio as in the vertical tie from the STM-AASHTO.

Effect of a/d Ratio in Utilization Ratios

The comparison of utilization ratios from the STM-AASHTO and the nonlinear FEM with respect to the shear span and depth (*a/d*) ratio is shown in **Table 6-5.** The depth of beam is forty-three inches. With the change in *a/d* ratio, the utilization ratios from both the STM-AASHTO and nonlinear FEM show the same behavior. For *a/d* ratio of 1.40, the utilization ratio is maximum and for *a/d* ratio of 1.64, the utilization ratio is minimum in both the STM-AASHTO and the nonlinear FEM.

Table 6-5: Comparison of utilization ratios with *a/d* ratio for Bridge 1

Shear span (a) (in.)	a/d	Utilization ratios		Remarks
		STM-AASHTO	Nonlinear FEM	
60.00	1.40	0.49	0.64	B-2
81.00	1.89	0.42	0.37	F-6
71.00	1.64	0.41	0.21	E-5

Global response result

The cracking pattern of the cap beam at failure is shown in **Figure 6-24** and at factored load is shown in **Figure 6-25.** The cracking pattern at failure is different from the previous model. More shear cracks are seen in the beam. Failure occurs in shear due to crushing of concrete in cantilever span, which is same as the previous model. Flexural cracks are less than the previous model.

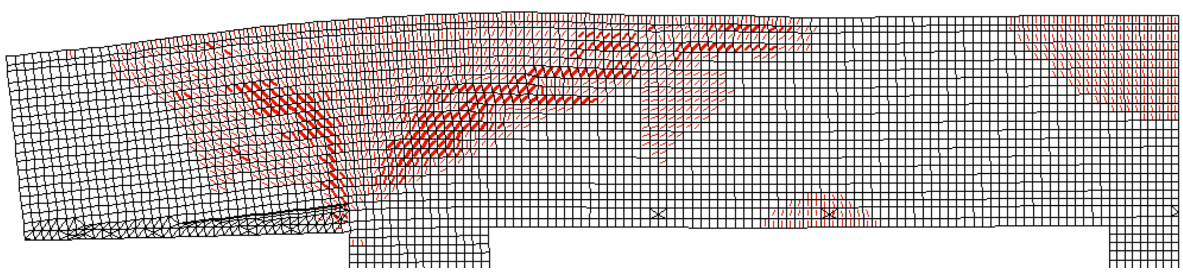


Figure 6-24: Cracking pattern at failure load for Bridge 1

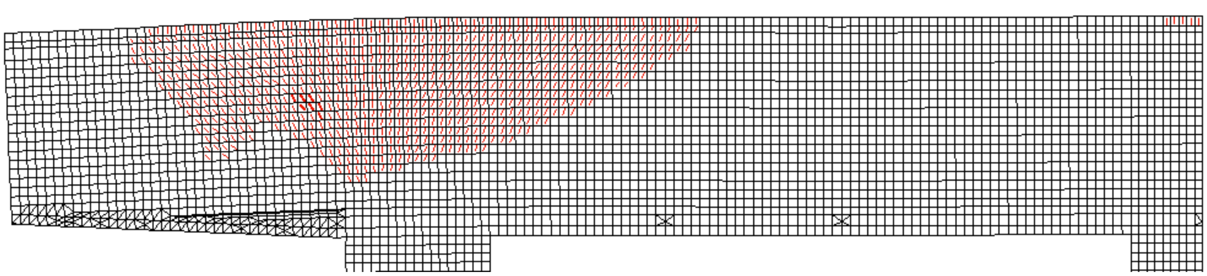


Figure 6-25: Cracking pattern at factored load for Bridge 1

The load-displacement curve is shown in **Figure 6-26.** Load corresponding to 662 kips is the factored load acting on the pier cap. Failure occurs at a load of 1125 kips. The capacity of pier caps predicted from the STM-AASHTO is 895 kips, which is 17% less than the nonlinear FEM. The nonlinear FEM is 1.25 times higher capacity prediction than the STM-AASHTO.

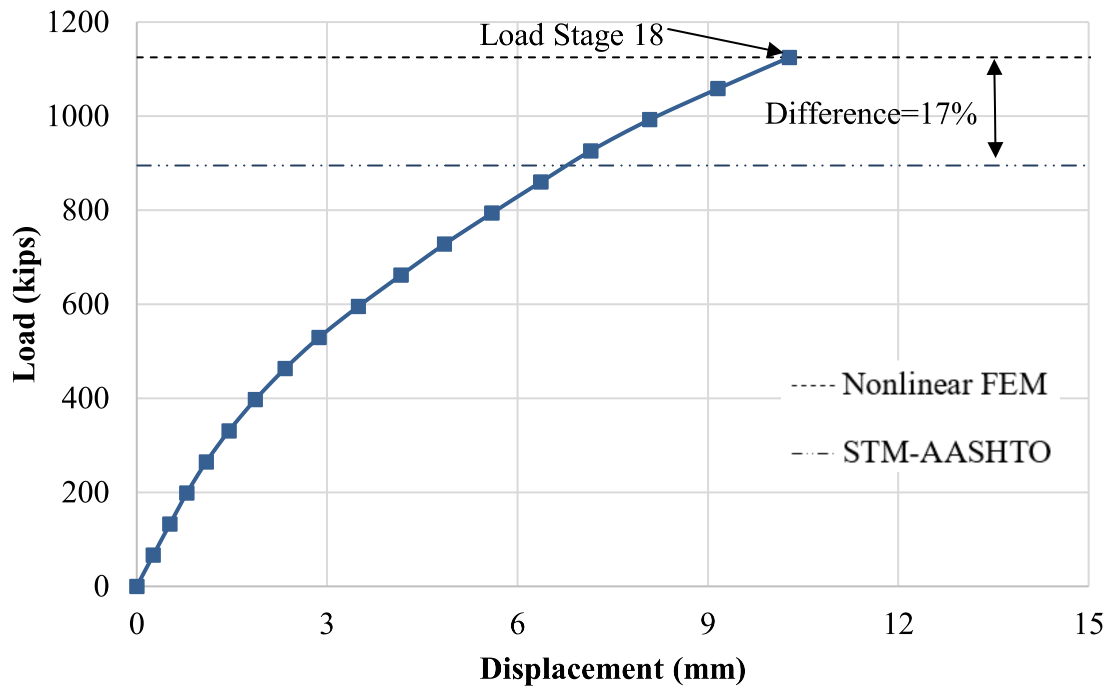


Figure 6-26: Load vs displacement response of cap beam for Bridge 1

6.2.3 Discussions

The mode of failure is shear failure of the cantilever span for Method 1. Top rebar yields, and high compression strain develops at the cantilever-column interface which causes the crushing of concrete. For Method 2, there are significant differences between the governing behaviors. Cracks form in the concrete and the utilization ratio for reinforcement members is less. The issue with the utilization ratios of vertical ties is not significantly improved and the governing behavior changes, hence Method 2 is not used for the remaining pier caps.

In Method 1, the highest utilization ratios are found in concrete in the cantilever span (0.39) and top rebar above column in the cantilever span (0.37), which matches Members B-2 (0.49) and A-F (0.71) from the STM-AASHTO results with the highest utilization ratios. The highest utilized member is the concrete element in the nonlinear FEM and the top rebar in the STM-AASHTO. The failure occurs in the same region (cantilever span) in both analyses. The utilization ratios from the nonlinear FEM (Method 1) are calculated to be 53% on average of those from the STM-AASHTO. Change in shear span to depth ratios show a similar trend in the utilization ratios from the STM-AASHTO and the nonlinear FEM. The capacity of pier cap predicted from the STM-AASHTO is 932 kips. The nonlinear FEM determined 1.5 times higher capacity prediction than the STM-AASHTO.

6.3 Bridge 2

Pier-line 2 of the bridge was selected for analysis. The pier cap is symmetrical with cantilever ends and three supporting columns as shown in **Figure 6-27.** The reinforcement details for the beam at Section A-A is shown in **Figure 6-28.**

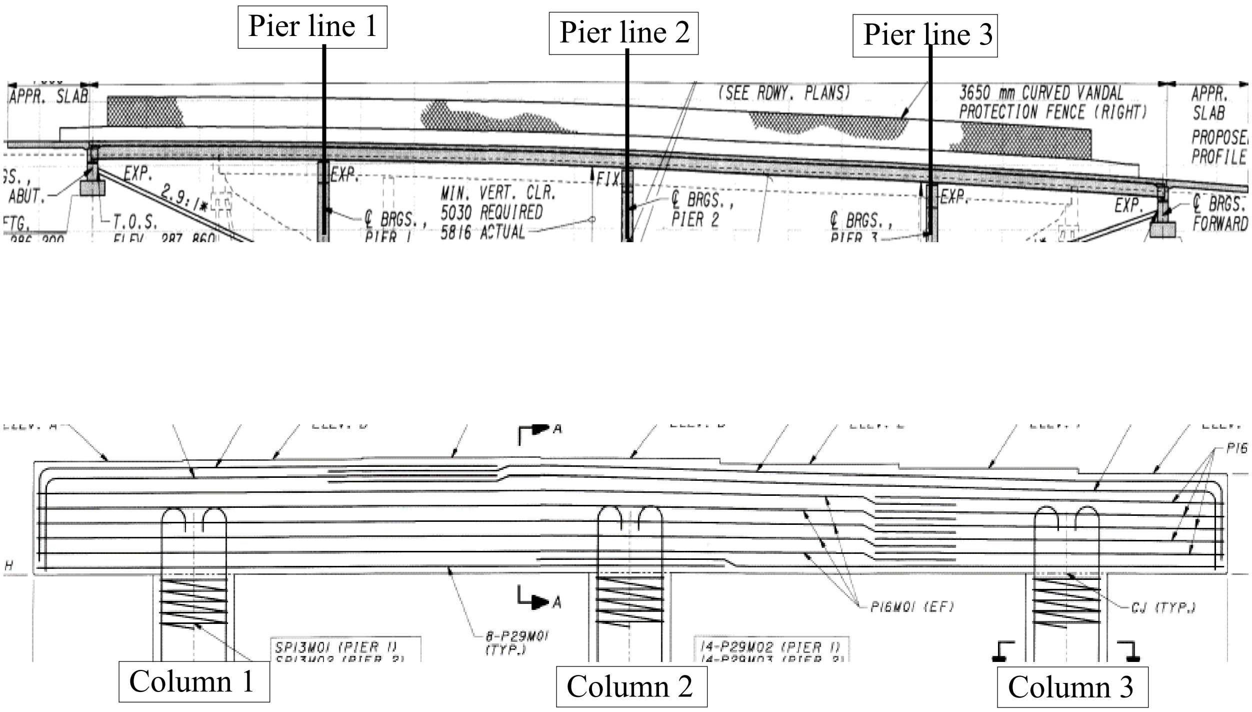


Figure 6-27: Pier cap details for Bridge 2

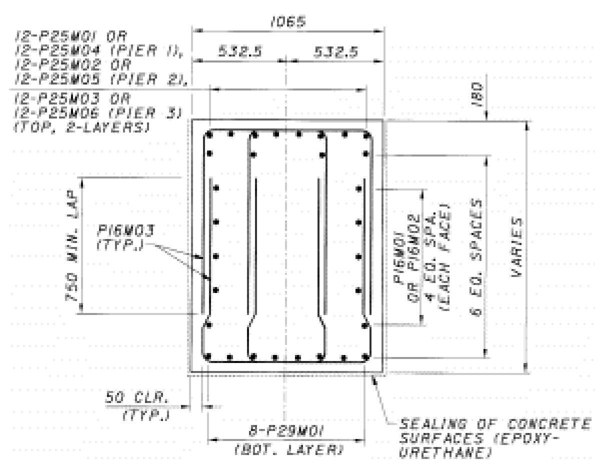


Figure 6-28: Section A-A for Bridge 2

6.3.1 Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (420 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are used. Three different regions are created to represent different smeared reinforcement conditions as shown in **Table 6-6.** Beam-I is the region of Section A-A **(Figure 6-28).** The size of the concrete cover is given as 2 inches (50 mm).

Table 6-6: Continuum region properties for Bridge 2

Region	Description	Color	f'c (MPa)	Reinforcement ratio
1	Beam-I		19.30	0.30%
2	Column		19.30	1.10% and 0.15%
3	Concrete cover		19.30	0.00%

The truss bar properties of reinforcement in cap beam is shown in **Table 6-7.**

Table 6-7: Truss bar properties for Bridge 2

Truss	Description	Color	Area(mm^2)	fy (MPa)	Diameter(mm)
1	Top Bar-I (8-P25)		4080	378	25.4
2	Top Bar-II (4-P25)		2040	378	25.4
3	Side Bar (2-P16)		398	378	15.9
4	Bottom Bar (8-P29)		5160	378	28.7

The symmetry of the structure allows for modeling one-half of the beam. The finite element model of the cap beam developed in VecTor2 is shown in **Figure 6-29**. Pin supports are defined at the lowermost ends of the columns while rollers are defined at the axis of symmetry. The finite element mesh size of 50 x 50 mm is used.

Factored dead and live loads of 665 kN and 331 kN, respectively are applied as four-point loads on the beam, the same as in Bridge 1. The load is applied monotonically at an increment of 10% up to the failure.

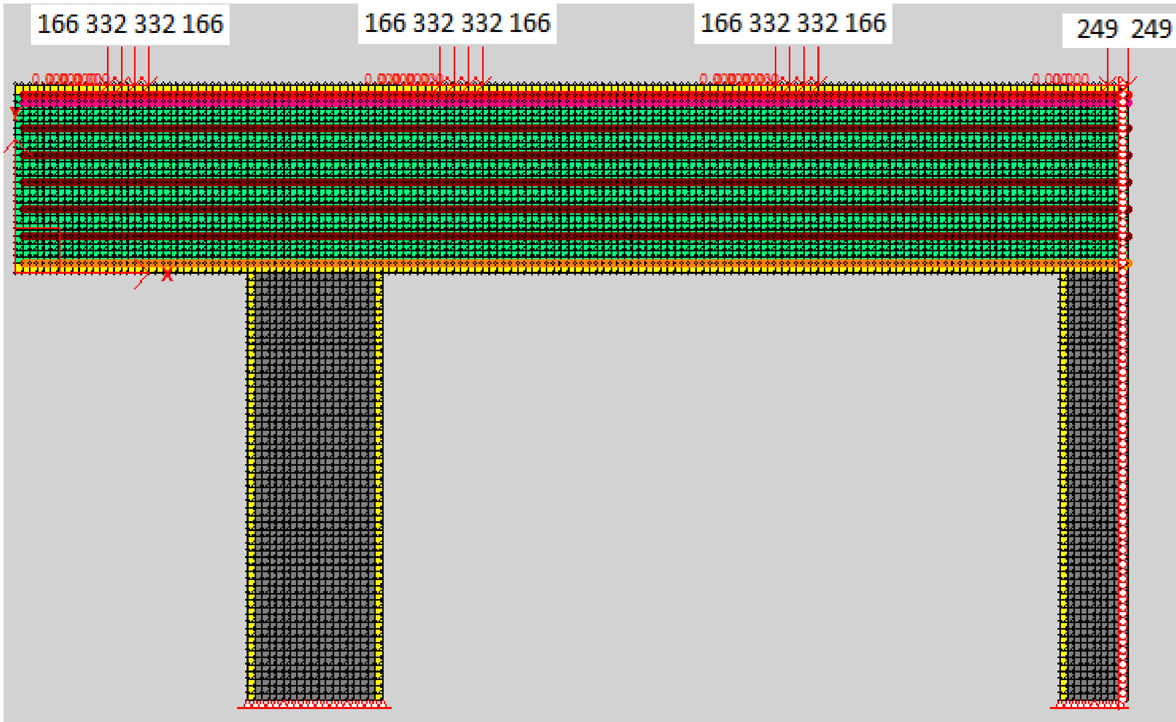


Figure 6-29: Cap beam model in VecTor2 for Bridge 2

6.3.2 Determination of Utilization Ratios

The same procedure as in Bridge 1 is followed to calculate utilization ratio for concrete and reinforcement. The stresses and utilization ratio at the factored load for concrete, main bars, and smeared reinforcement is shown in **Figure 6-30, Figure 6-31,** and **Figure 6-32**. The failure is governed by the region with the highest utilization ratio.

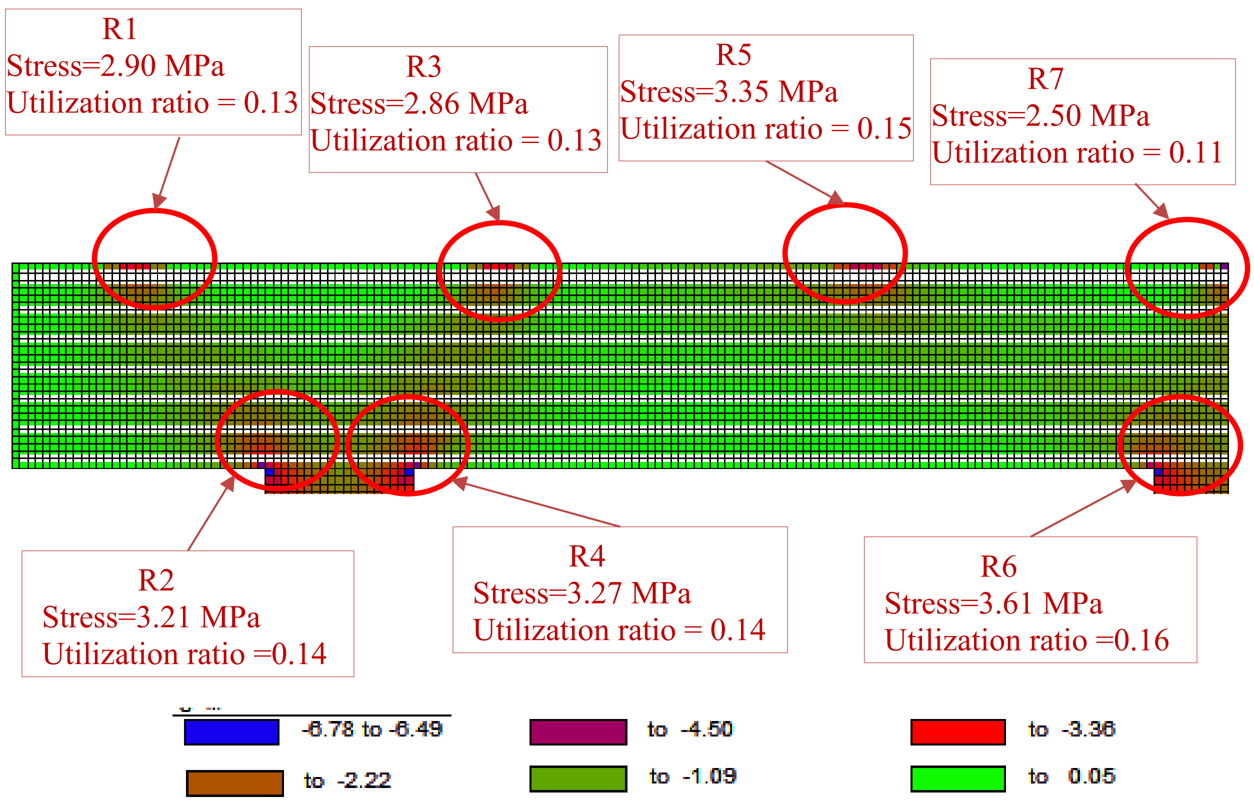


Figure 6-30: Concrete stress and utilization ratio at factored load for Bridge 2

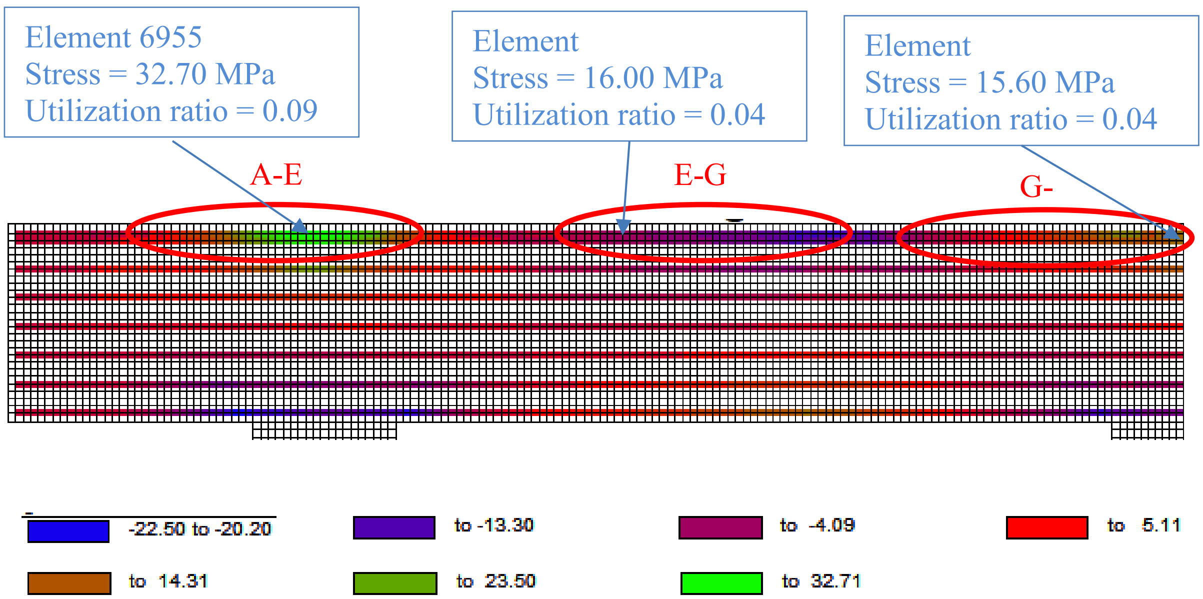


Figure 6-31: Average reinforcement stresses and utilization ratio at factored load, Bridge 2

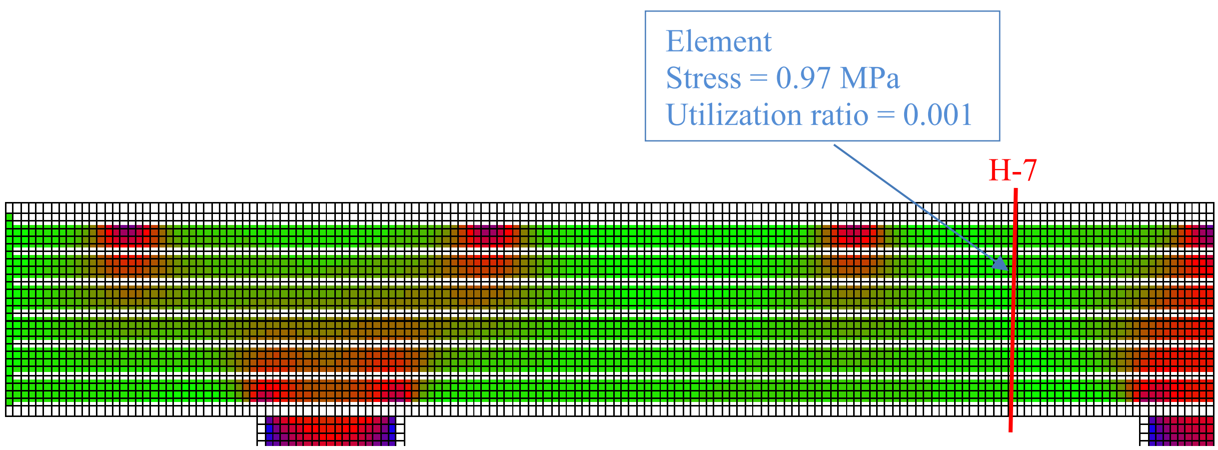


Figure 6-32: Smeared reinforcement stresses and utilization ratio at factored load, Bridge 2

6.3.3 Comparison of Utilization ratios with the STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses in contrast to the ratios obtained from the STM-AASHTO in **Figure 6-33**. The utilization ratios are consistently

39% and 78% at the lowest and highest extremes, and 46% on average of the STM-AASHTO for concrete members.

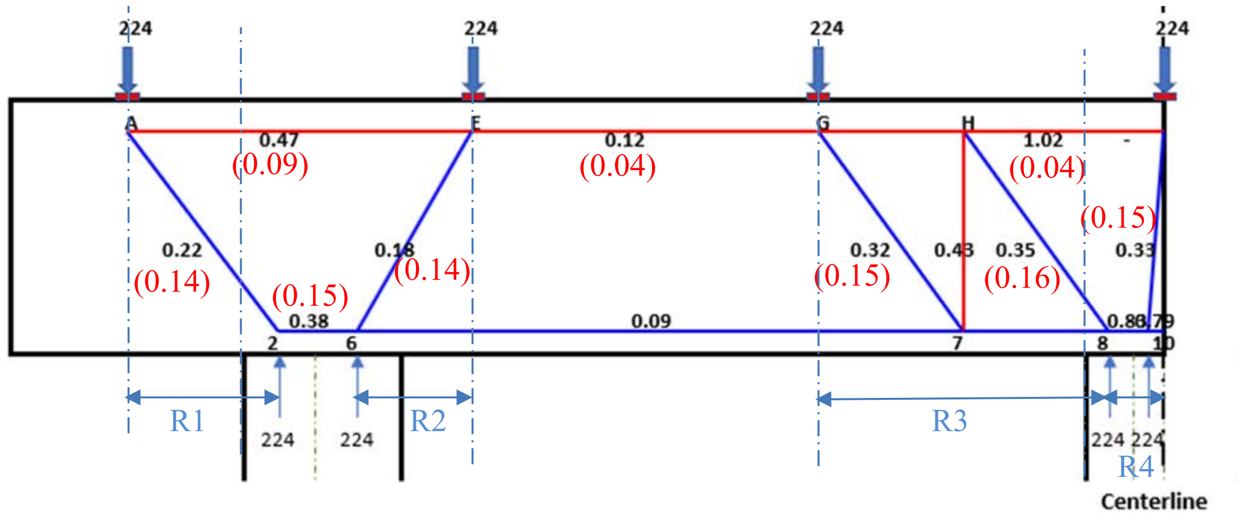


Figure 6-33: Utilization ratio from the STM-AASHTO and nonlinear FEM for Bridge 2

At the factored load levels, there is no yielding of reinforcement, but shear cracks develop in concrete. This gives a higher utilization for the concrete member and a lower utilization for the rebar member. On further increasing of the load, the top rebar starts yielding but the beam doesn't fail. The nonlinear FEM determines the failure mode to be the crushing of the concrete caused by shear, which occurs after yielding of the tensile reinforcement. The STM-AASHTO, on other hand, which is based on the lower-bound theorem, considered yielding of rebar as failure and terminates the analysis once rebar yields. Hence, as expected, STM-AASHTO determined the failure as flexural due to the yielding of top rebar.

The comparison of utilization ratios from the STM-AASHTO and the nonlinear FEM with respect to the shear span and depth (*a/d*) ratio is shown in **Table 6-8.** The depth of the beam is thirty-six inches. Although the difference between the utilization ratios is significantly higher between the nonlinear FEM and the STM-AASHTO, the trend with *a/d* ratios is similar. For a region with *a/d*=0.10, utilization ratio is maximum for both the STM-AASHTO and the nonlinear FEM.

Table 6-8: Comparison of utilization ratios with *a/d* ratio for Bridge 2

Shear span (a) (in.)	a/d	Utilization ratios		Remarks
		STM-AASHTO	Nonlinear FEM	
40.00	1.00	0.22	0.31	R1
30.00	0.76	0.18	0.31	R2
77.00	1.91	0.32	0.33	R3
4.00	0.10	0.35	0.36	R4

6.3.4 Global Response Results

The cracking pattern of the cap beam at the failure load and at the point of rebar yielding is shown in **Figure 6-34** and at the factored load is shown in **Figure 6-35.** Failure occurs in the interface of the column and at the mid-span of the pier cap due to shear. At failure load, flexural cracks are also seen at the mid-span of the beam, as reinforcement yields. The cracking pattern when rebar first yields is shown in **Figure 6-35.** This is an efficiently designed pier cap, where the loads are distributed along the pier cap, which can be seen from the cracking pattern. Although shear cracks cause the final failure of the beam, rebars also yield and the full beam is fully utilized. At the factored load level, very few cracks are predicted, which gives small utilization ratios for tension ties.

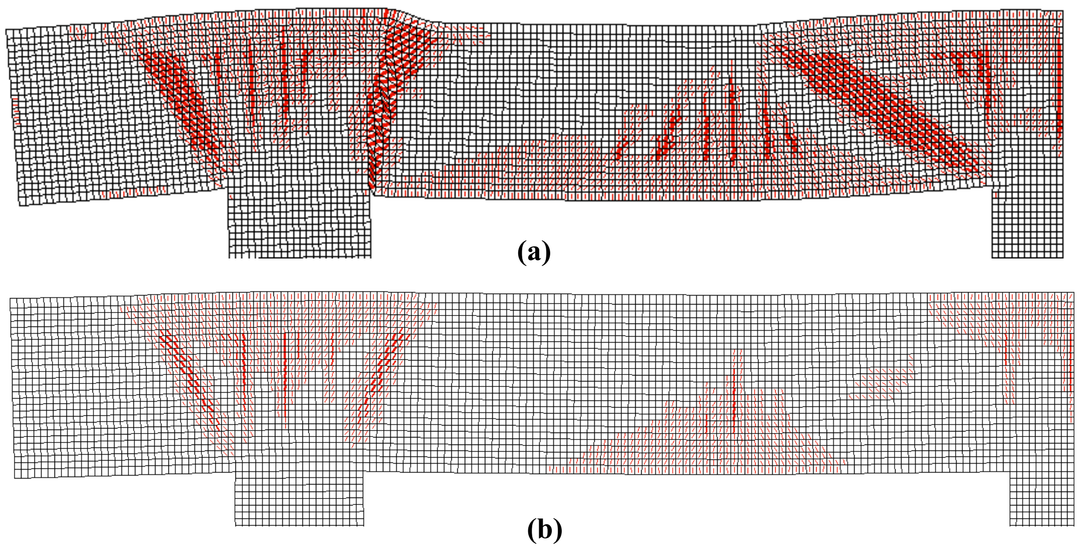


Figure 6-34: Cracking pattern (a) at failure load (b) when rebar yields first for Bridge 2

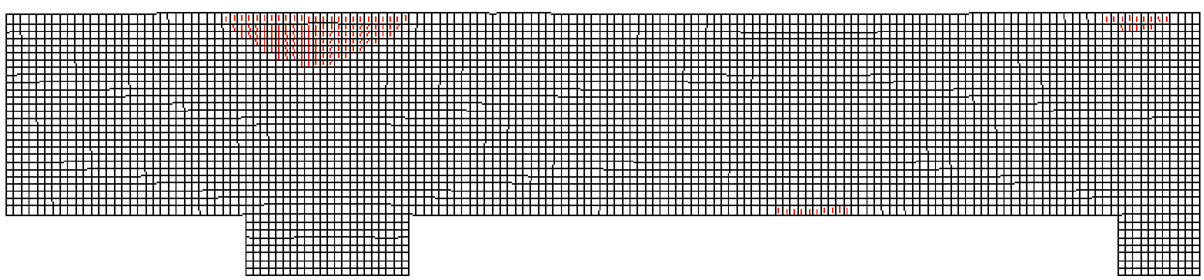


Figure 6-35: Cracking pattern at factored load for Bridge 2

The load-displacement curve is shown in **Figure 6-36.** Load corresponding to 783 kips is factored load acting on the pier cap. Failure occurs due to crushing of concrete at Load Stage 39 with convergence factor of 1.094, corresponding to a load of 2976 kips. The reinforcement yields at Load Stage 22, corresponding to a load of 1723 kips. After the first yielding of the reinforcement, two times higher load is resisted by the pier cap. Failure occurs due to crushing of concrete at Load

Stage 39 with convergence factor of 1.094, corresponding to a load of 2976 kips. The predicted capacity of pier cap from the STM-AASHTO is 784 kips.

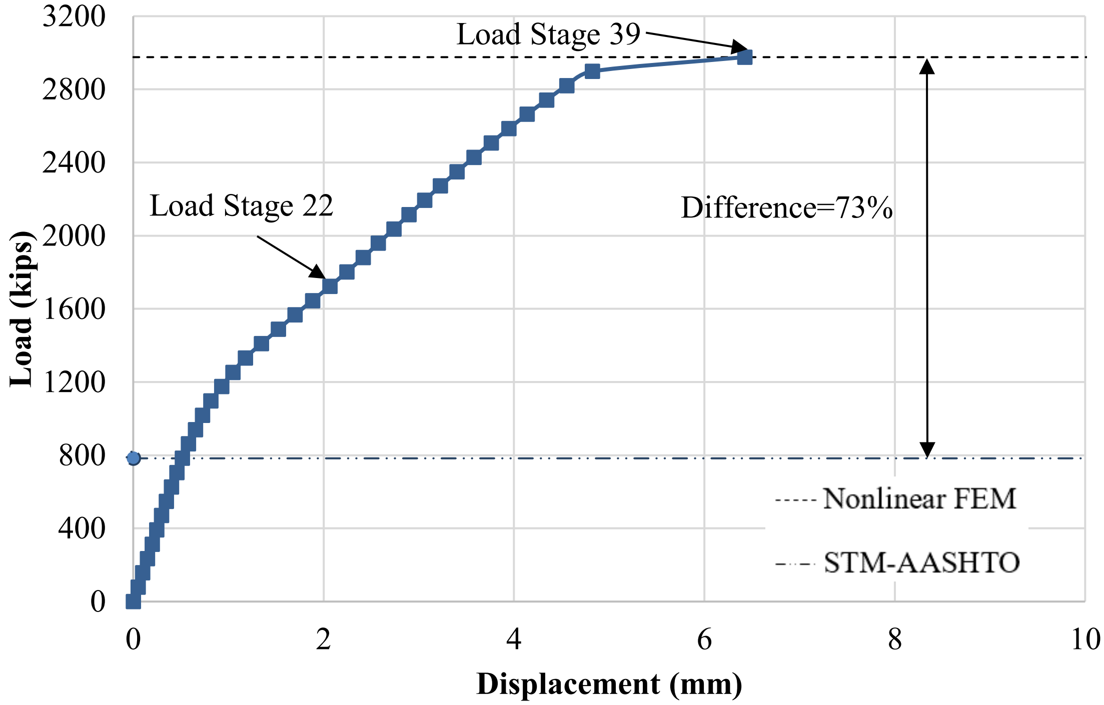


Figure 6-36: Load vs displacement response of cap beam for Bridge 2

6.3.5 Discussions

In this pier cap, the top reinforcing bar yields first but the pier cap doesn't fail. The failure occurs due to the crushing of the concrete at the left span, which is captured by the nonlinear FEM analysis. The STM-AASHTO, on the other hand, which is based on the lower-bound theorem, considers the top reinforcement yielding as failure and terminates the analysis conservatively. The governing member from the nonlinear FEM is the concrete member at the left span of the pier cap, whereas the top rebar at the mid-span yields in the STM-AASHTO and governs the failure.

The utilization ratios from the nonlinear FEM are calculated to be 42% on average of those from the STM-AASHTO. There is a significant difference between the utilization ratios determined for the reinforcement bars. The load applied is well distributed along the beam length. The ultimate failure occurs from yielding of reinforcement and crushing of the concrete along the beam length. After the top reinforcing bar yields, two times higher load is required to fail the pier cap due to the crushing of the concrete. This discrepancy shows significant difference in the utilization ratio of the governing member with the STM-AASHTO. In-spite of the differences, the utilization ratios of shear critical members from the nonlinear FEM and the STM-AASHTO show a similar trend with *a/d* ratios.

6.4 Bridge 3

This pier cap is symmetrical with cantilever ends, four supporting columns, and a total of seven girder loads as shown in **Figure 6-37.** The reinforcement detail is shown in **Figure 6-38**.

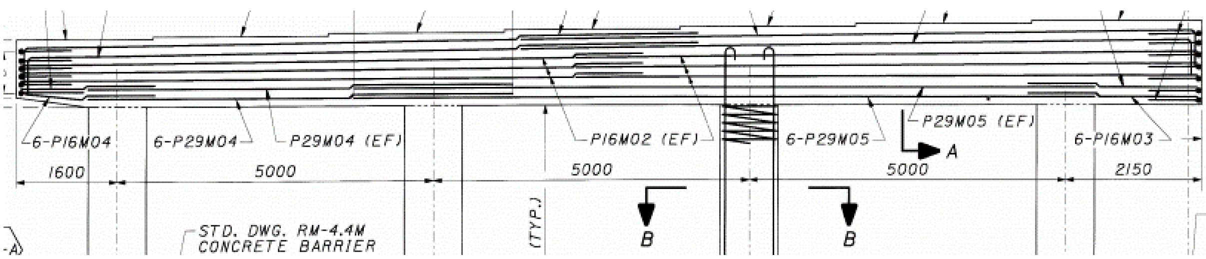


Figure 6-37: Pier cap details for Bridge 3

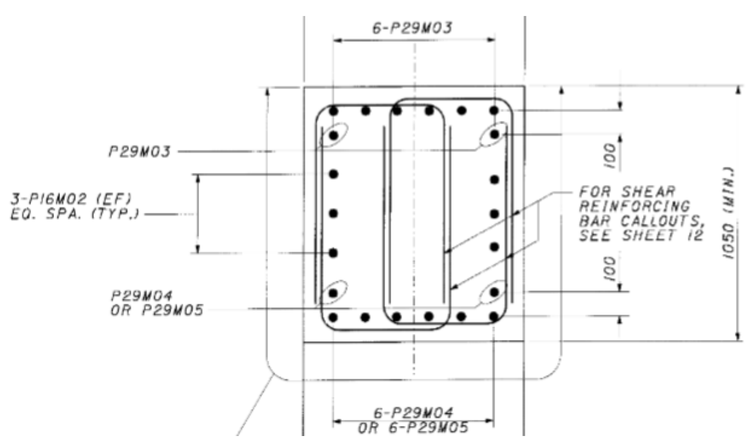


Figure 6-38: Section A-A of the beam for Bridge 3

6.4.1 Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (420 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are used. Five different regions are created to represent different smeared reinforcement conditions as shown in **Table 6-9.** Beam-I, Beam-II, and Beam-III are the regions of Section A-A (**Figure 6-38**) with a stirrup spacing of 165 mm, 305 mm, and 395 mm.

Table 6-9: Continuum region properties for Bridge 3

Region	Description	Color	f'c (MPa)	Reinforcement ratio
1	Beam-I		19.30	0.53%
2	Beam-II		19.30	0.29%
3	Beam-III		19.30	0.22%
4	Column		19.30	1.18% and 0.27%

The truss bar properties of reinforcement in cap beam is shown in **Table 6-10.**

Table 6-10: Truss bar properties for Bridge 3

Truss	Description	Color	Area(mm^2)	fy (MPa)	Diameter(mm)
1	Top Bar-I (6-P29)		3870	378	28.7
1	Top Bar-II (2-P29)		1290	378	28.7
3	Side Bar (2-P16)		398	378	15.9
4	Bottom Bar-I (2-P29)		1290	378	28.7
5	Bottom Bar-II (6-P29)		3870	378	28.7

The symmetry of the structure allows for modeling one-half of the beam. The finite element model of the cap beam developed in VecTor2 is shown in **Figure 6-39**. Pin supports are defined at the lowermost ends of the columns while rollers are defined at the axis of symmetry. The finite element mesh size of 75 x 75 mm is used.

Factored dead and live loads of 841 kN and 414 kN, respectively, are applied as four-point loads on the beam, the same as in previous pier caps. The load is applied monotonically at an increment of 10% up to the failure.

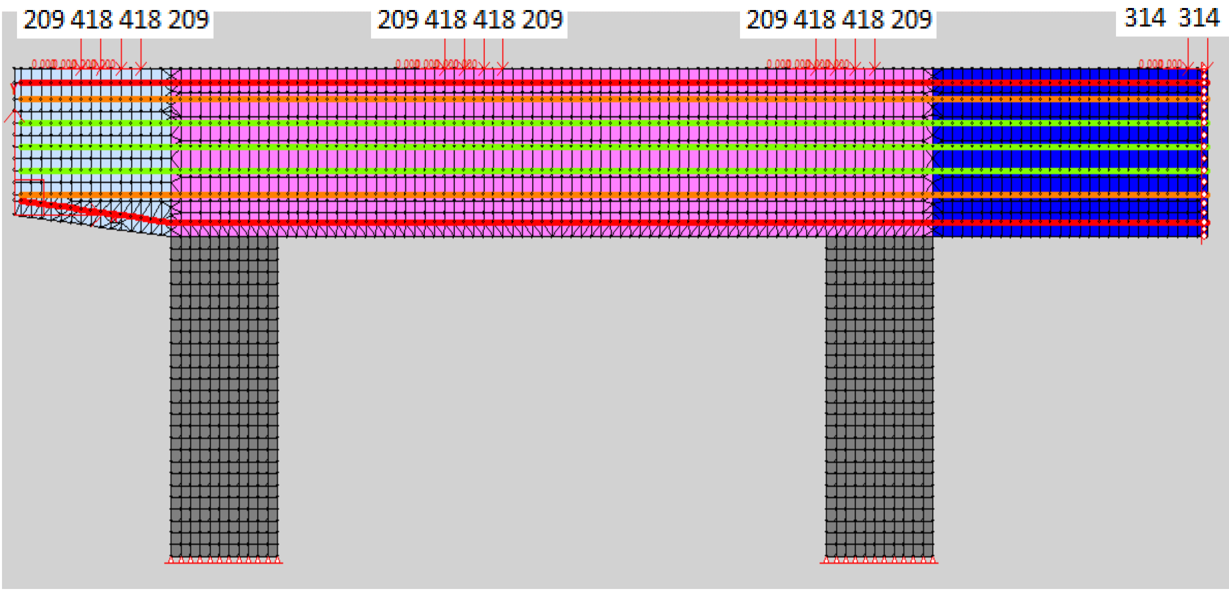


Figure 6-39: Cap beam model in VecTor2 for Bridge 3

6.4.2 Determination of Utilization Ratios

The same procedure as in the previous bridge is followed to calculate utilization ratio for concrete and reinforcement. The stresses and utilization ratio at the factored load for concrete and the main reinforcement bars is shown in **Figure 6-40** and **Figure 6-41**. The failure is governed by the region with the highest utilization ratio.

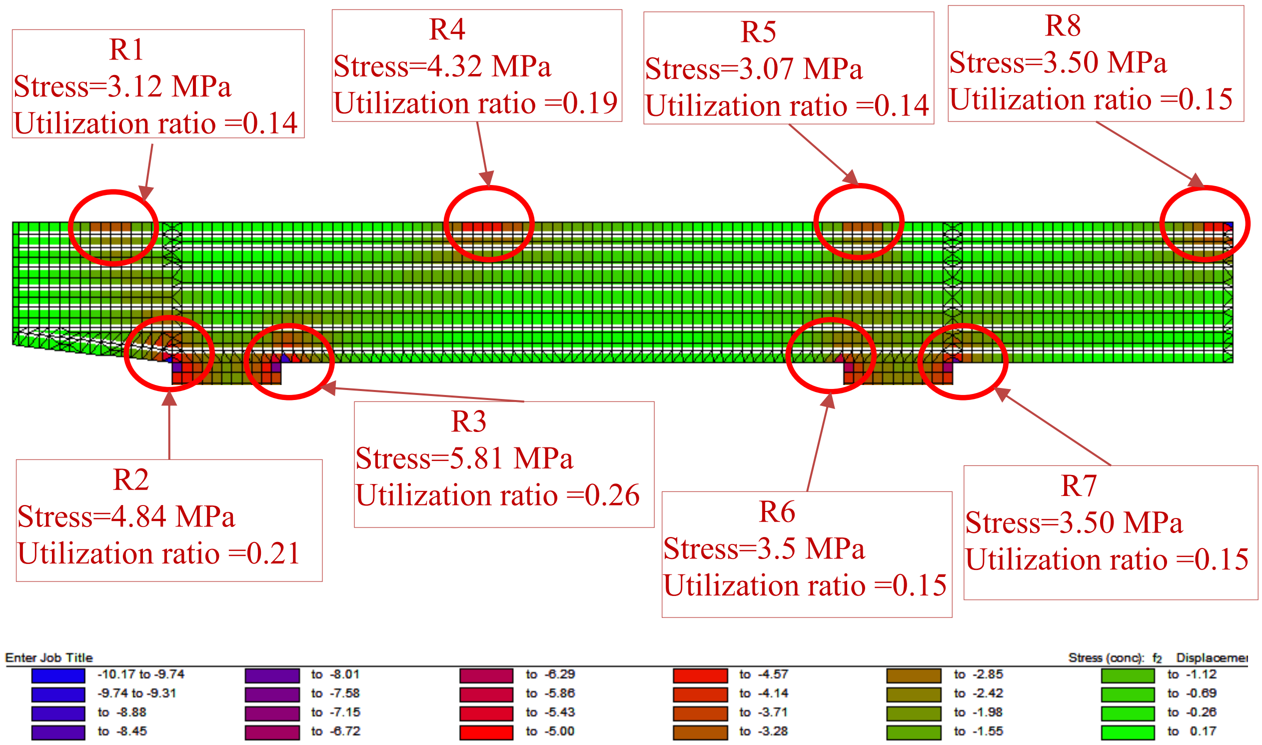


Figure 6-40: Concrete stresses and utilization ratio at factored load for Bridge 3

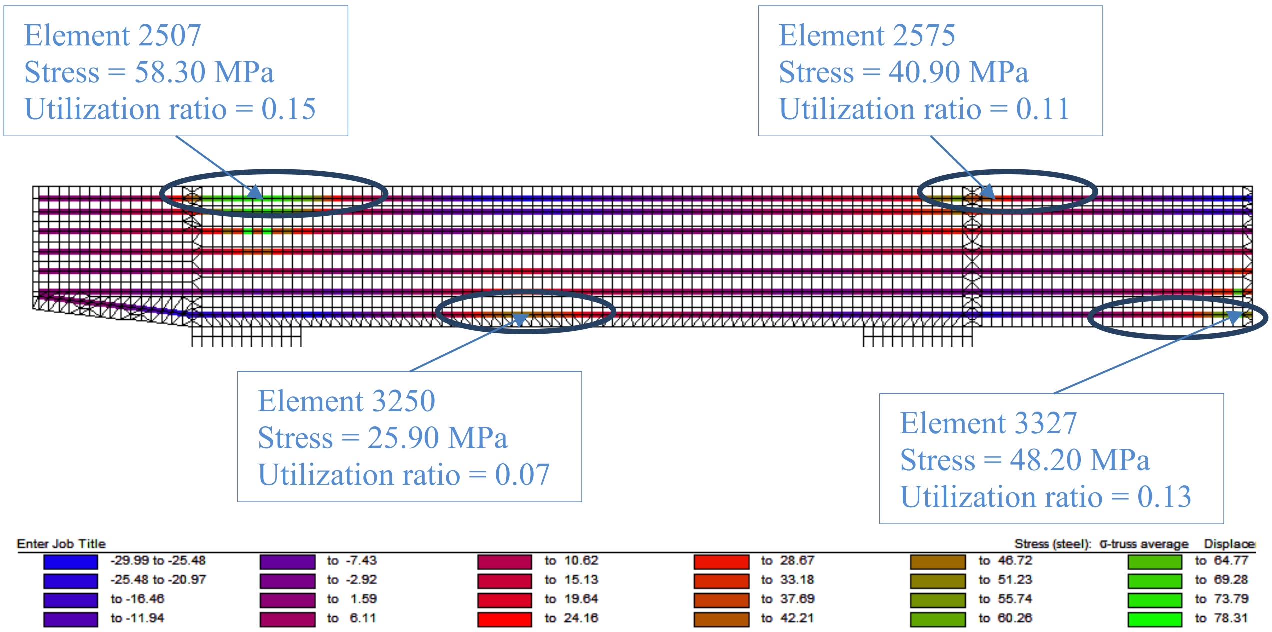


Figure 6-41: Average reinforcement stresses and utilization ratio at factored load, Bridge 3

6.4.3 Comparison of Utilization Ratios with the STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses in contrast to the ratios obtained from the STM-AASHTO in **Figure 6-42**.

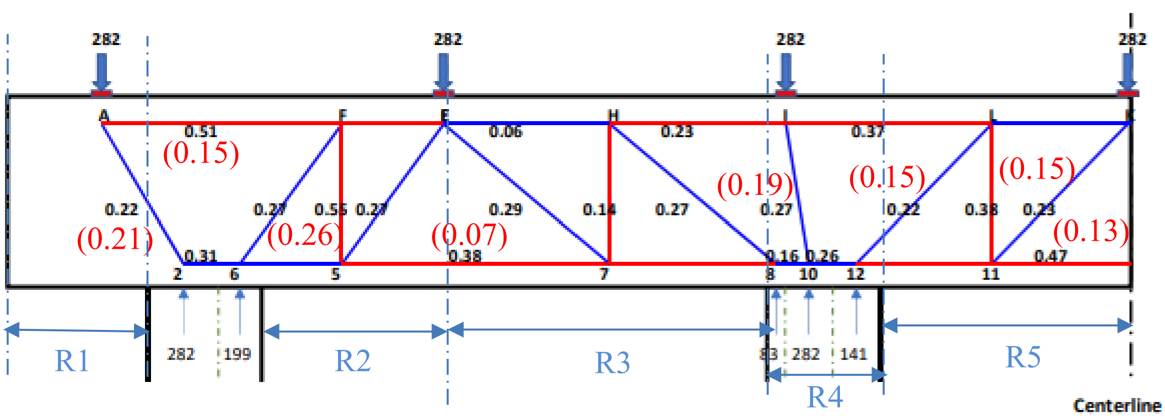


Figure 6-42: Utilization ratio from the STM-AASHTO and nonlinear FEM for Bridge 3

The member with highest utilization ratio governing the failure is vertical tie at the left span of the cap beam from the STM-AASHTO. The nonlinear FEM calculates the maximum utilization ratios for the concrete element in the same region. The utilization of the top rebar is significantly less than the STM-AASHTO.

The shear span to depth (*a/d*) ratios for each region in the pier caps are calculated and the corresponding utilization ratios of the most critical concrete member in the region are compared. The depth of beam (*d*) is thirty-eight inches. The comparison is shown in **Table 6-11**. The utilization ratios from the nonlinear FEM and the STM-AASHTO show a similar trend with *a/d* ratios. The utilization ratio of shear critical member, in the region with high *a/d* ratio, is higher from both the STM-AASHTO and the nonlinear FEM. The utilization ratio for the cantilever region (R1) is also calculated high in the nonlinear FEM.

Table 6-11: Comparison of utilization ratios with *a/d* ratio for Bridge 3

Shear span (*a*) (in.)	***a/d***	**Utilization ratios**		**Remarks**
		STM-AASHTO	**Nonlinear FEM**	
26.00	0.69	0.22	0.21	R1
65.00	1.72	0.55	0.26	R2
106.00	2.80	0.29	0.19	R3
7.00	0.19	0.27	0.15	R4
87.00	2.29	0.22	0.15	R5

6.4.4 Global Response Results

The cracking pattern of the cap beam at the failure load is shown in **Figure 6-43** and at the factored load is shown in **Figure 6-44.** Shear failure occurs, which is indicated by inclined red lines, and matches with the region having high utilization ratios in the STM-AASHTO.

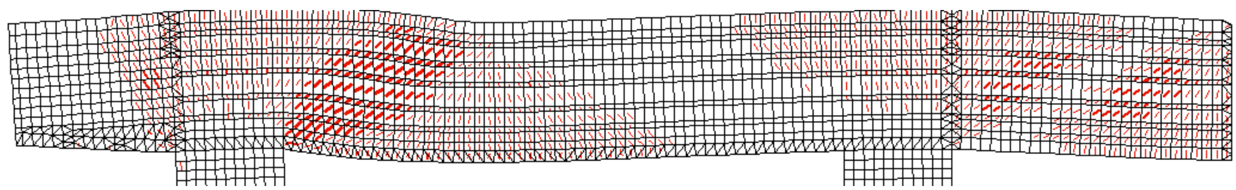


Figure 6-43: Cracking pattern at failure load for Bridge 3

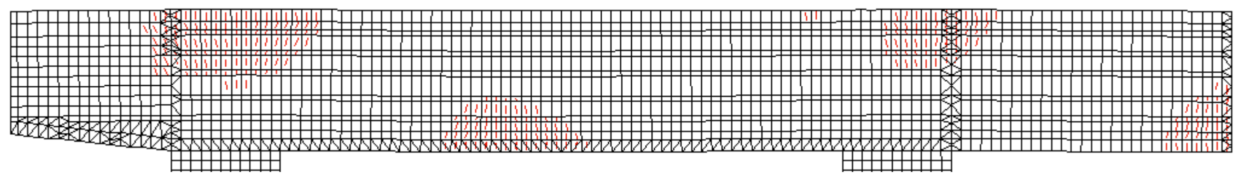


Figure 6-44: Cracking pattern at factored load for Bridge 3

The load-displacement curve is shown in **Figure 6-45.** Load corresponding to 987 kips is the factored load acting on the pier cap. Failure occurs at Load Stage 27 with the convergence factor of 1.016, corresponding to the load of 2565 kips. The predicted capacity of pier cap from the STM-AASHTO is 1795 kips. The nonlinear FEM is 1.4 times higher capacity prediction than the STM-AASHTO.

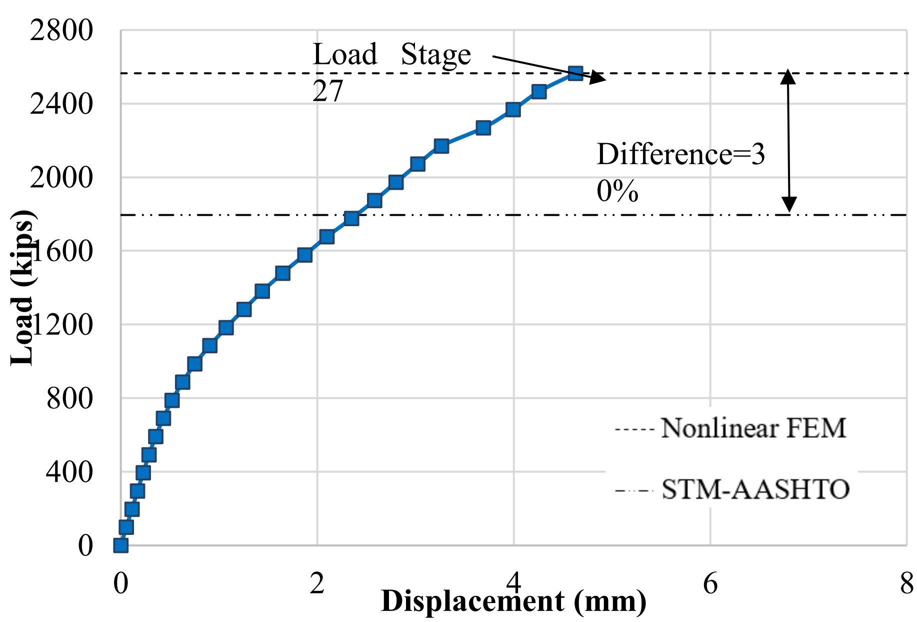


Figure 6-45: Load vs displacement response of cap beam for Bridge 3

6.4.5 Discussions

The governing mode for the failure matches with the STM-AASHTO result. The utilization ratios from the nonlinear FEM are calculated to be 49% on average of those from the STM-AASHTO. The nonlinear FEM is 1.4 times higher capacity prediction than the STM-AASHTO.

6.5 Bridge 4

Pier-line 1 of the bridge is selected for analysis. This pier cap is symmetrical with cantilever ends and four supporting columns as shown in **Figure 6-46.** The reinforcement details for the beam at Section A-A and B-B is shown in **Figure 6-47.**

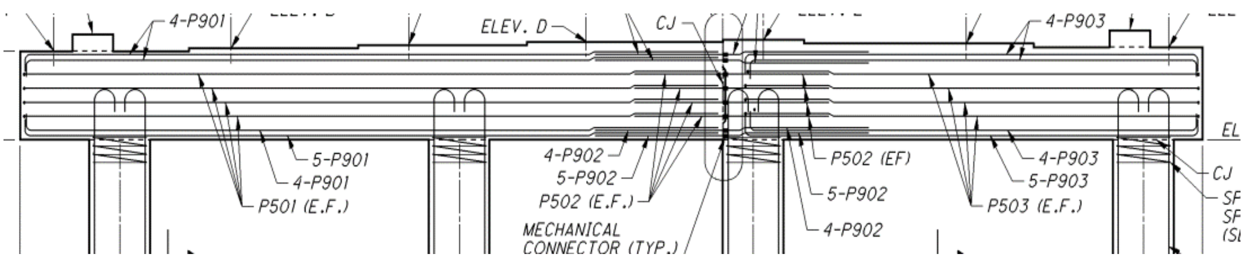


Figure 6-46: Pier cap details for Bridge 4

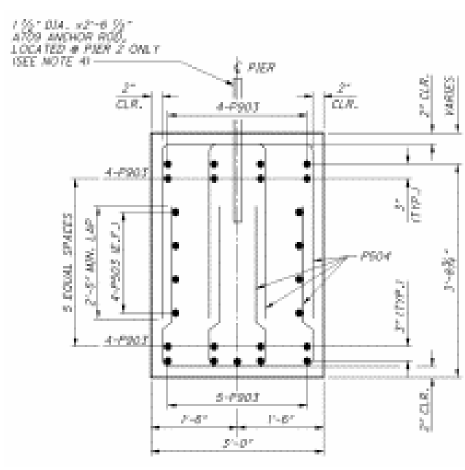


Figure 6-47: Section of the beam for Bridge 4

6.5.1 Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (415 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are used. Five different regions are created to represent different smeared reinforcement conditions as shown in **Table 6-12.** Beam-I, Beam-II, and Beam-III are the regions of sectional details (**Figure 6-47**) with a stirrup spacing of six inches (150 mm), twelve inches (300 mm), and eighteen inches (450 mm). The size of the concrete cover is given as two inches (50 mm).

Table 6-12: Continuum region properties for Bridge 4

Region	Description	Color	$f'c$ (MPa)	Reinforcement ratio
1	Beam-I		19.30	0.57%
2	Beam-II		19.30	0.28%
3	Beam-III		19.30	0.19%
4	Column		19.30	1.13% and 0.13%
5	Concrete cover		19.30	0.00%

The truss bar properties of reinforcement in cap beam is shown in **Table 6-13**.

Table 6-13: Truss bar properties for Bridge 4

Truss	Description	Color	Area(mm^2)	fy (MPa)	Diameter(mm)
1	Top Bar-I (4-P9)		2580	378	28.7
2	Side Bar (2-P16)		398	378	15.9
3	Bottom Bar (4-P9)		2580	378	28.7

The symmetry of the structure allowed for modeling one-half of the beam. The finite element model of the cap beam developed in VecTor2 is shown in **Figure 6-48**. Pin supports are defined at the lowermost ends of the columns while rollers are defined at the axis of symmetry. The finite element mesh size of 50 x 50 mm is used.

Factored dead and live loads of 151 kips and 105 kips, are applied as four-point loads on the beam, the same as in previous pier caps. The load is applied monotonically at an increment of 10% up to the failure.

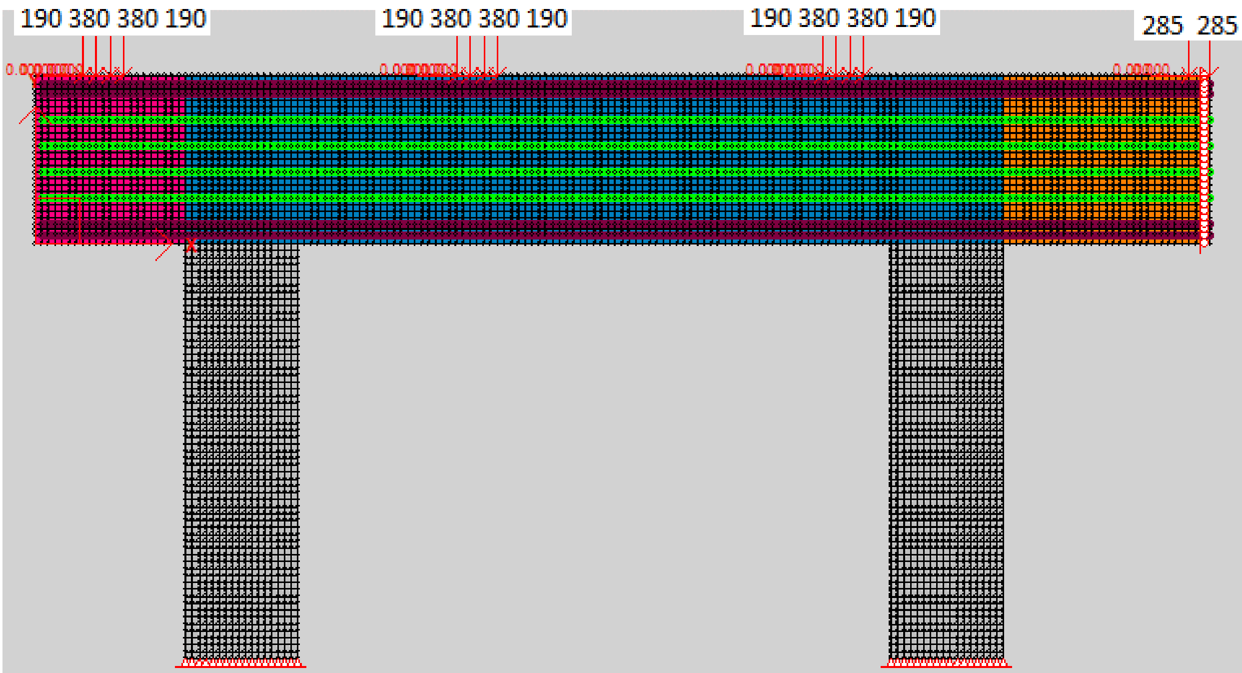


Figure 6-48: Cap beam model in VecTor2 for Bridge 4

6.5.2 Determination of Utilization Ratios

The same procedure as in previous bridge is followed to calculate utilization ratio for concrete and reinforcement. The stresses and utilization ratio at the factored load for concrete and main bars reinforcement is shown in **Figure 6-49** and **Figure 6-50**. The failure is governed by the region with the highest utilization ratio.

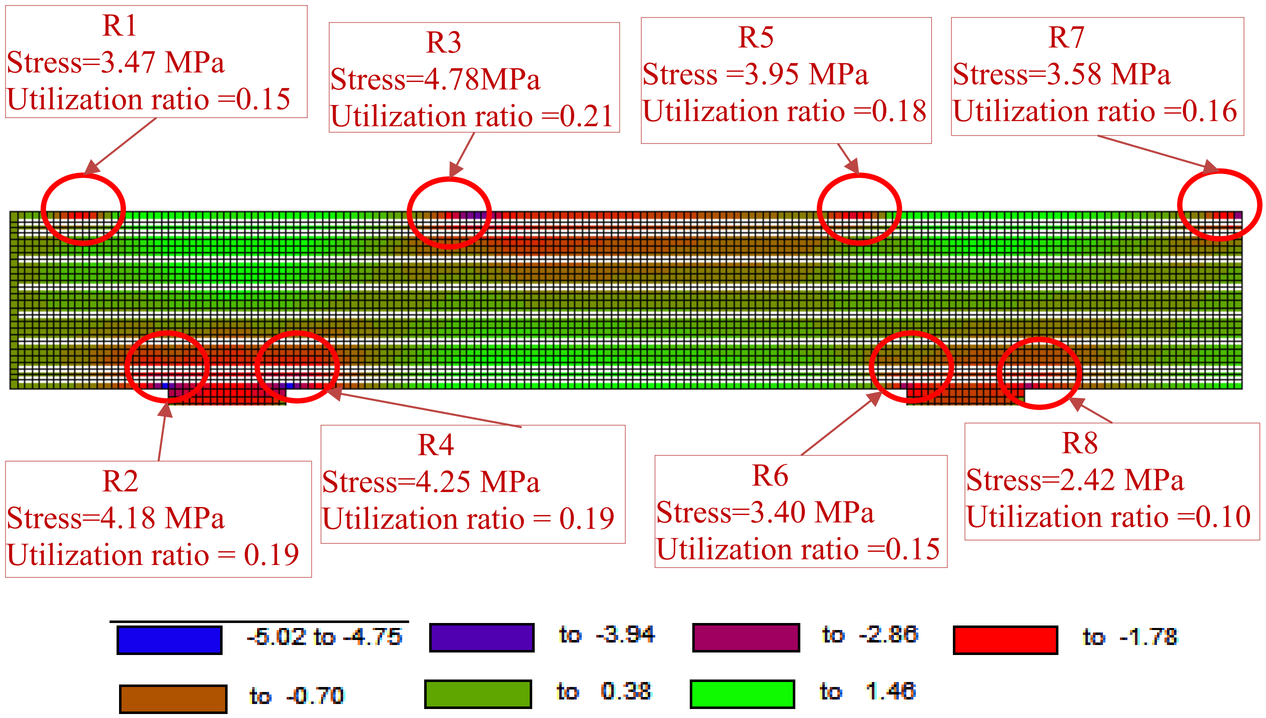


Figure 6-49: Concrete stress and utilization ratio at factored load for Bridge 4

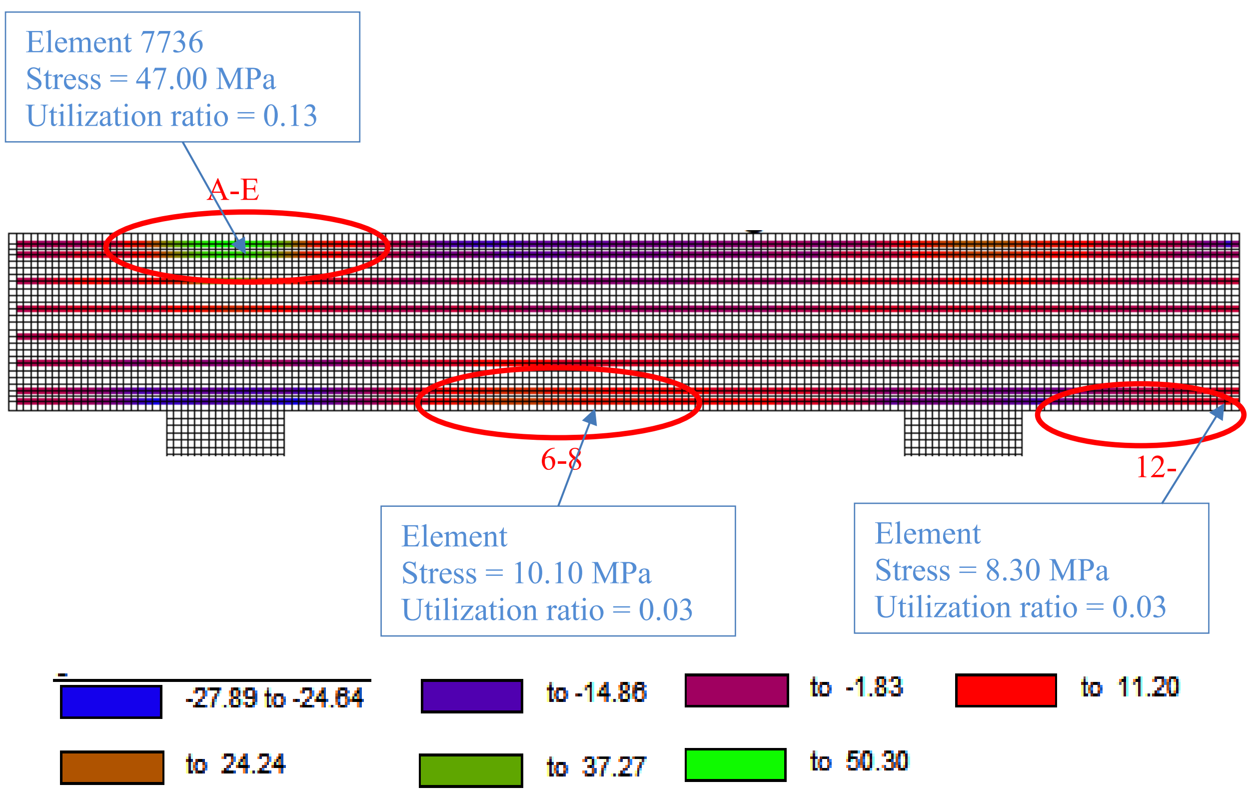


Figure 6-50: Average reinforcement stresses and utilization ratio at factored load, Bridge 4

6.5.3 Comparison of Utilization Ratios with the STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses in contrast to the ratios obtained from the STM-AASHTO in **Figure 6-51**.

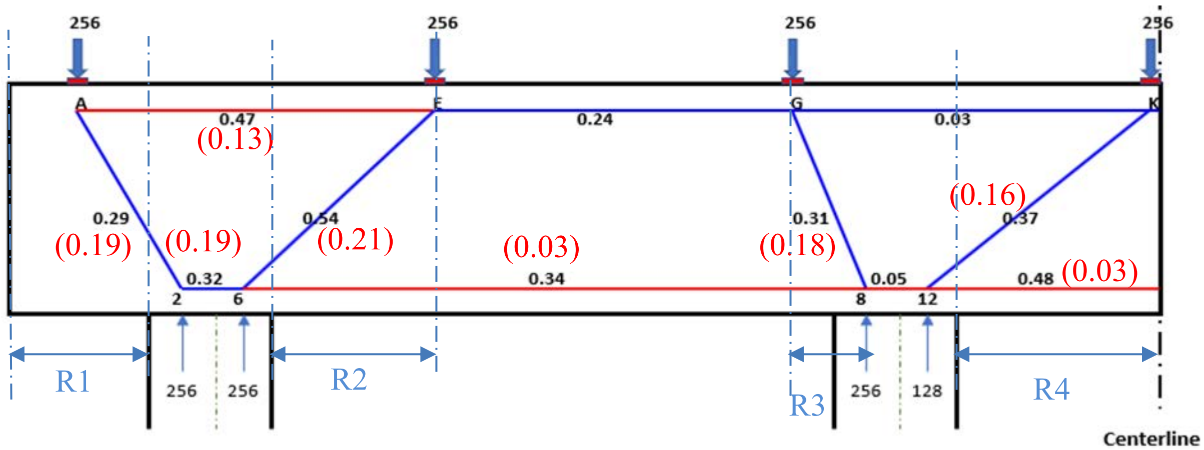


Figure 6-51: Utilization ratio from the STM-AASHTO and nonlinear FEM for Bridge 4

The maximum utilization is calculated for a concrete member in the left span of the pier cap, which matches Member E-6 from the STM-AASHTO. The failure is governed by Member E-5 for both analyses. Concrete in the cantilever span governs, whereas top rebar in cantilever span governs in the STM-AASHTO.

The utilization ratios were consistently less than those from the STM-AASHTO for concrete members by 34.48% and 61.11% at the lowest and highest extremes with 48.57% on average. The stresses in reinforcement are less than concrete. The utilization ratios are consistently less for reinforcement with 72.34% and 93.75% at the lowest and highest extremes and 85.73% on average than the STM-AASHTO.

The effect of the shear span and depth ratio with the utilization ratios is compared as shown in **Table 6-14.** The depth of beam (*d*) is forty-three inches. The utilization ratios from the nonlinear

FEM and the STM-AASHTO show a similar trend with *a/d* ratios except for the cantilever span where VecTor2 calculates high utilization ratio. The utilization ratio for cantilever region is calculated high from the nonlinear FEM.

Table 6-14: Comparison of utilization ratios with *a/d* ratio for Bridge 4

Shear span (a) (in.)	a/d	Utilization ratios		Remarks
		STM-AASHTO	Nonlinear FEM	
31.00	0.71	0.29	0.19	R1
56.00	1.30	0.54	0.21	R2
22.00	0.50	0.31	0.18	R3
65.00	1.51	0.37	0.16	R4

6.5.4 Global Response Results

The cracking pattern of the cap beam at the failure load is shown in **Figure 6-52** and at the factored load in **Figure 6-53.** Shear failure occurs at the interface of the column and left span of the beam. Flexural cracks occur above the column regions.

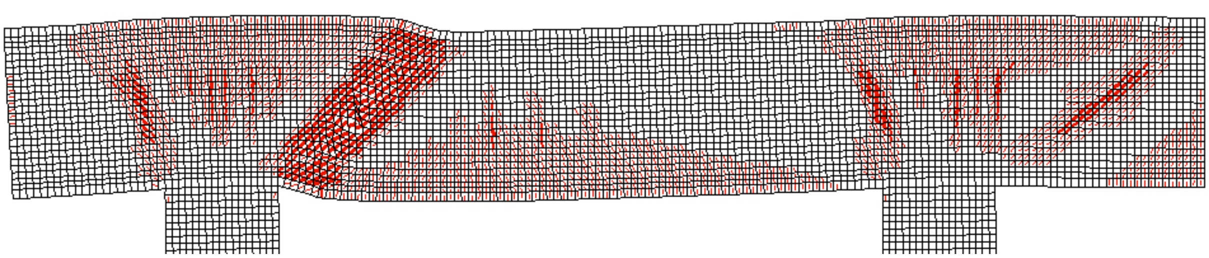


Figure 6-52: Cracking pattern at failure load for Bridge 4

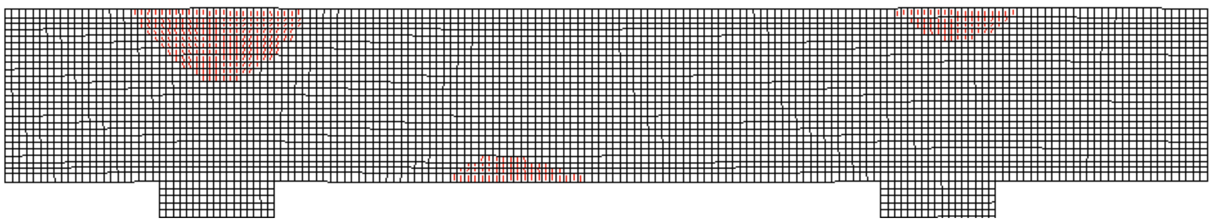


Figure 6-53: Cracking pattern at factored load for Bridge 4

The load-displacement curve is shown in **Figure 6-54**. Failure occurs at Load Stage 27 with a convergence factor of 1.016, corresponding to the load of 2340 kips. The calculated capacity of the pier cap from the STM-AASHTO is 1660 kips, which is 30% less than from the nonlinear FEM. The predicted capacity of this pier cap from sectional analysis is 1378 kips (From Chapter 7), which is 41% less capacity prediction than the nonlinear FEM.

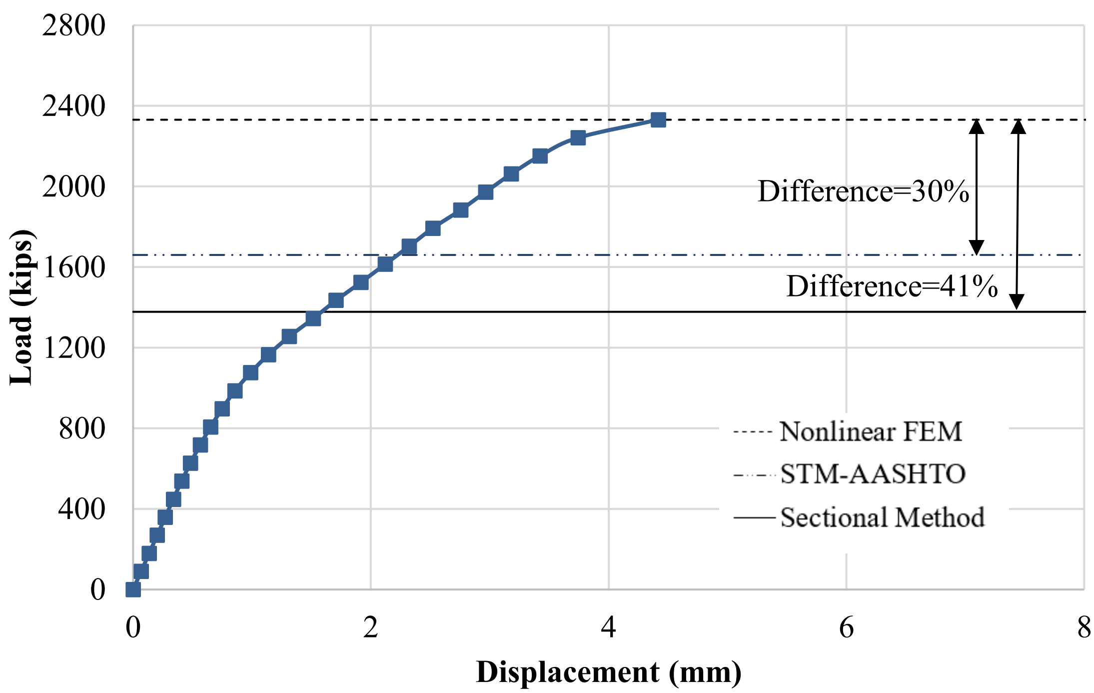


Figure 6-54: Load vs displacement response of cap beam for Bridge 4

6.5.5 Discussions

The highest utilization ratio is found for the concrete member at the interface of the column and left span, which matches with Member E-6 from the STM-AASHTO and governs the behavior. The utilization ratios from the nonlinear FEM are calculated to be 38% on average of those from the STM-AASHTO. The nonlinear FEM calculates 1.4 times higher capacity prediction than the STM-AASHTO.

6.6 Bridge 5

Pier 1 was modeled for the bridge whose elevation is shown in **Figure 6-55.** The reinforcement details for the beam at sectional detail is shown in **Figure 6-56.**

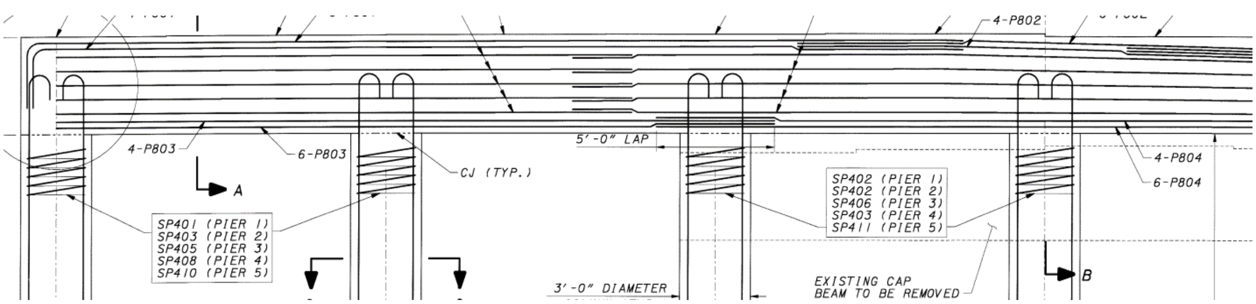


Figure 6-55: Pier cap details for Bridge 5

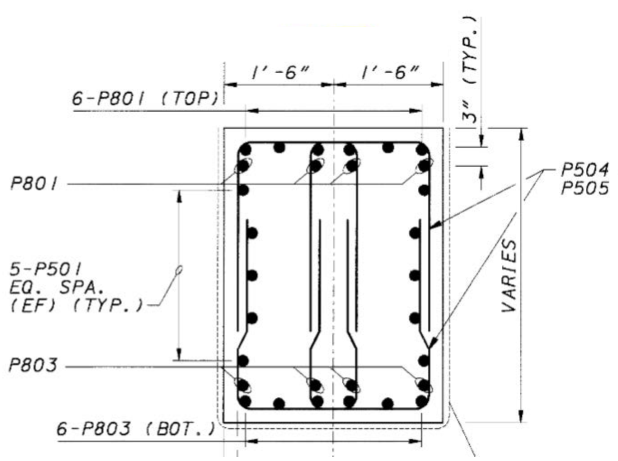


Figure 6-56: Section of the beam for Bridge 5

6.6.1 Modeling

The characteristic compressive strength of concrete (f_{ck}) is 4 ksi (27.6 MPa) and yield strength of reinforcement (f_y) is 60 ksi (420 MPa). For analysis, factored concrete strength (f_c) and factored reinforcement yield strength (f_s) are used.

Four different regions are created to represent different smeared reinforcement conditions as shown in **Table 6-15.** Beam-I and Beam-II are the regions of Section A-A with a stirrup spacing of eighteen inches and twenty inches **(Figure 6-56).** The size of the concrete cover is taken as three inches (75 mm).

Table 6-15: Continuum region properties for Bridge 5

Region	Description	Color	f'c (MPa)	Reinforcement ratio
1	Beam-I		19.30	0.19%
2	Beam-II		19.30	0.17%
3	Column		19.30	2.34% and 0.27%
4	Concrete cover		19.30	0.00%

The truss bar properties of reinforcement in cap beam is shown in **Table 6-16.**

Table 6-16: Truss bar properties for Bridge 5

Truss	Description	Color	Area(mm^2)	fy (MPa)	Diameter(mm)
1	Top Bar-I (6-P8)		3060	373	25.4
1	Top Bar-II (4-P8)		2040	373	25.4
3	Side Bar (2-P5)		398	373	15.9
3	Bottom Bar-I (4-P8)		2040	373	25.4
3	Bottom Bar-II (6-P8)		3060	373	25.4

The symmetry of the structure allows for modeling one-half of the beam. The finite element model of the cap beam developed in VecTor2 is shown in **Figure 6-57**. Only 2000 mm of the column is developed for the model, as columns have no significant effect on our analysis. Pin supports are defined at the lowermost ends of the columns while rollers are defined at the axis of symmetry. The finite element mesh size of 75 x 75 mm is used.

Factored dead and live loads of 142 kips and 80 kips are applied as four-point loads on the beam the same as in previous pier caps. The load is applied monotonically at an increment of 10% up to the failure.

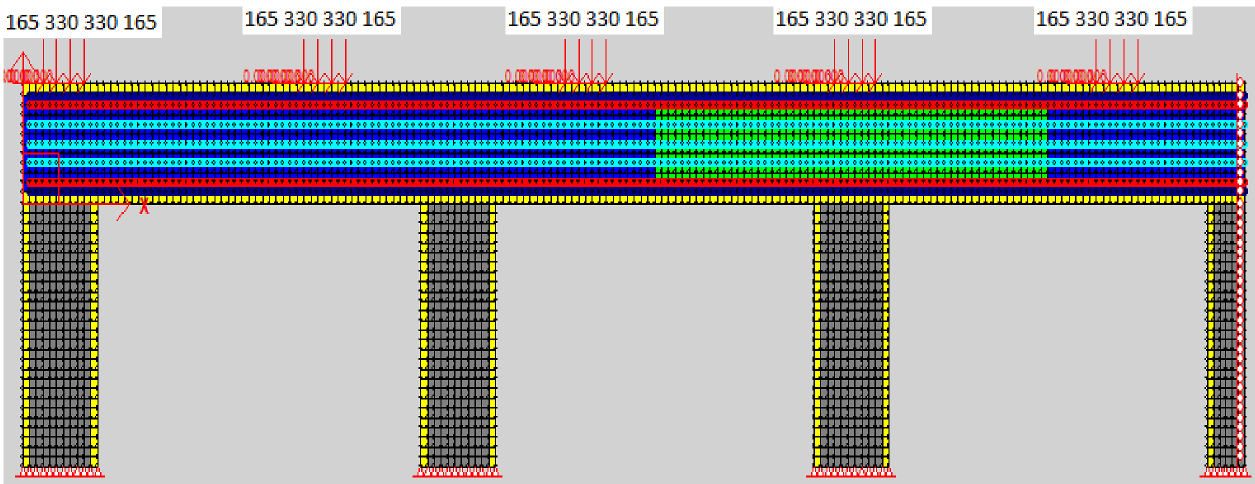


Figure 6-57: Cap beam model in VecTor2 for Bridge 5

6.6.2 Determination of Utilization Ratios

The stresses and utilization ratio at the factored load for concrete and main bars reinforcement is shown in **Figure 6-58** and **Figure 6-59**. The failure is governed by the region with the highest utilization ratio.

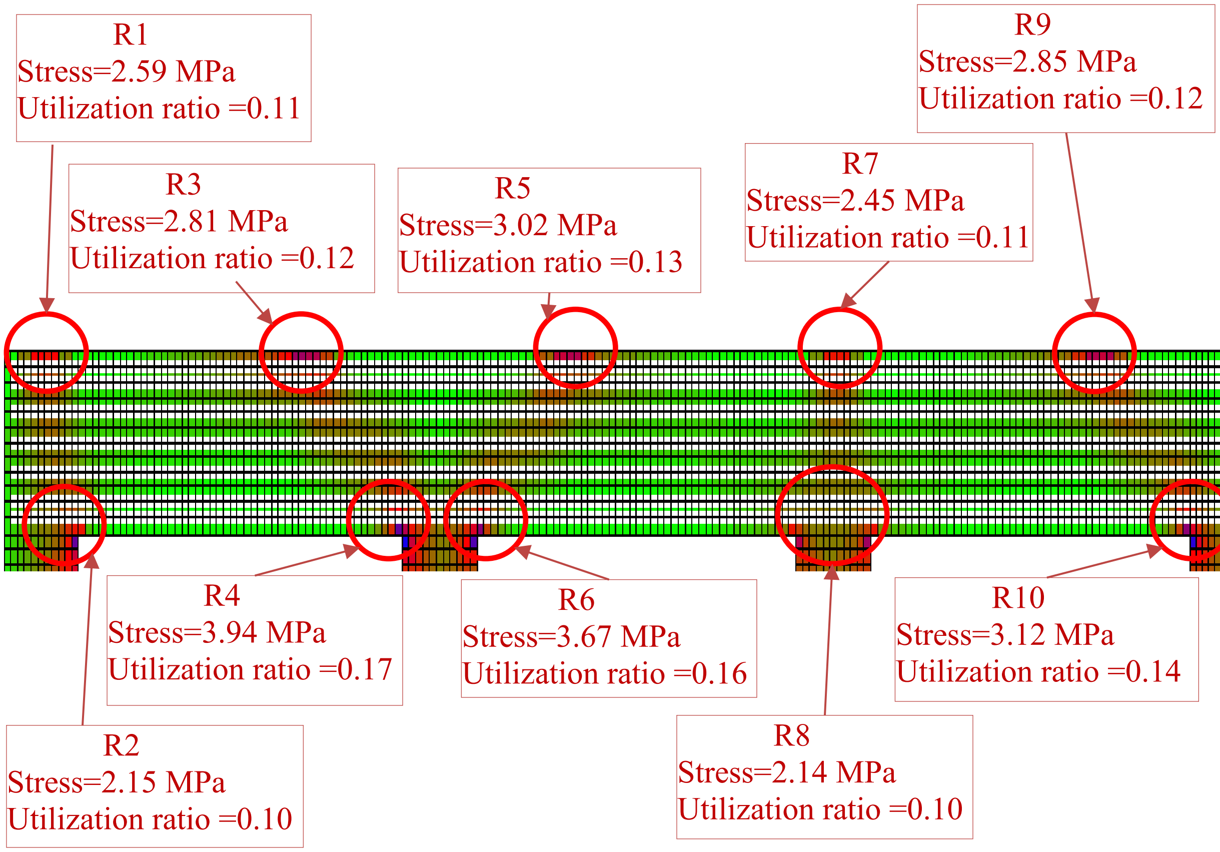


Figure 6-58: Concrete stress and utilization ratio at factored load for Bridge 5

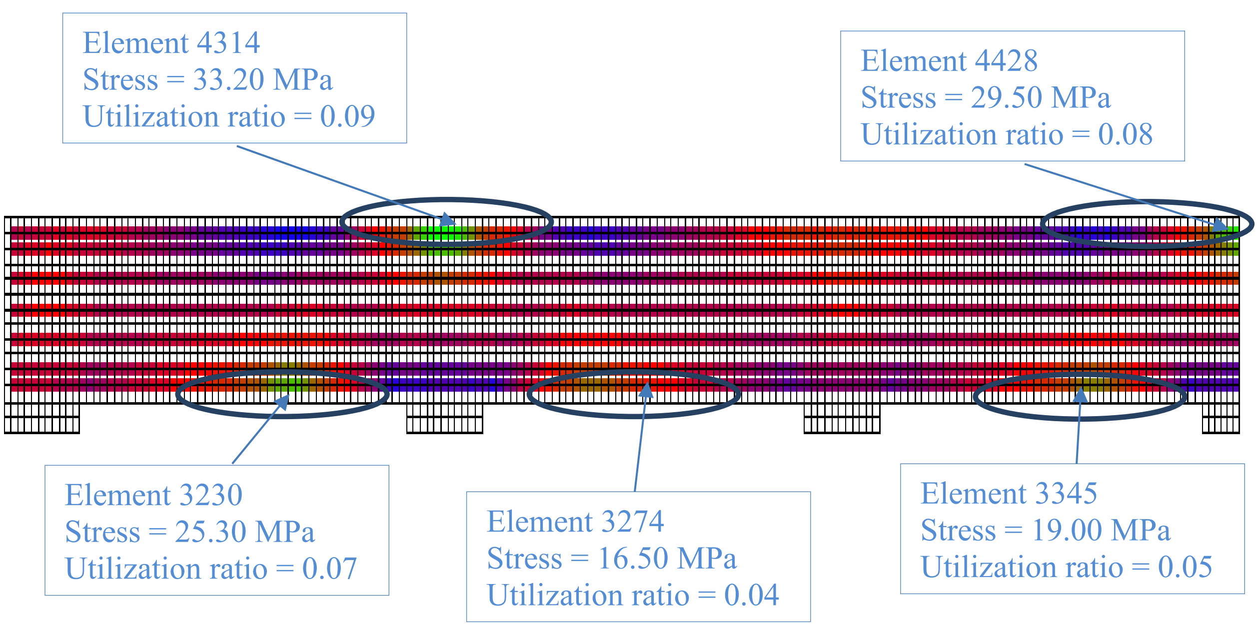


Figure 6-59: Average reinforcement stresses and utilization ratio at factored load, Bridge 5

6.6.3 Comparison of Utilization Ratios with the STM-AASHTO

The utilization ratios obtained from the nonlinear FEM is shown within parentheses in contrast to the ratios obtained from the STM-AASHTO in **Figure 6-60**. The maximum stress is found in concrete members in mid-region of the pier cap.

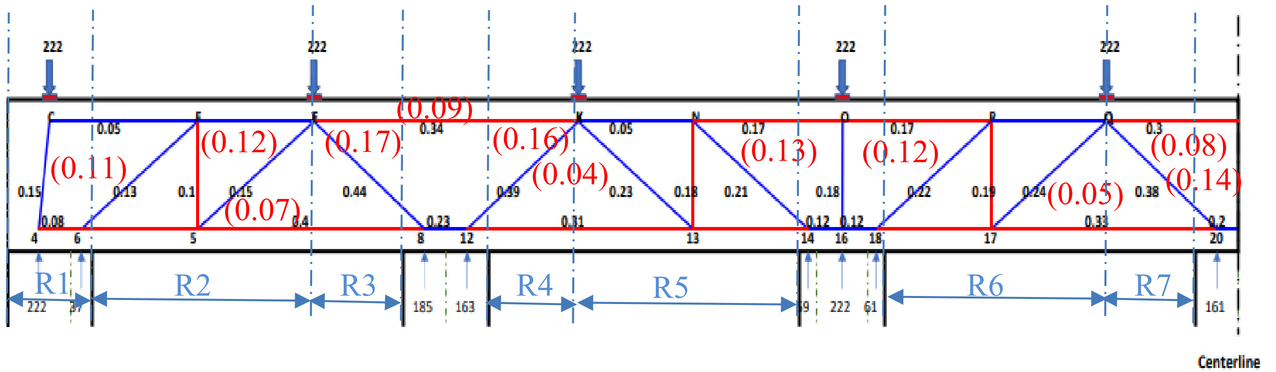


Figure 6-60: Utilization ratio from the STM-AASHTO and nonlinear FEM for Bridge 5

The shear span to depth (*a/d*) ratios for each region shown above in the pier caps are calculated and the corresponding utilization ratios of the most critical concrete member in the region are compared. The depth of beam (*d*) is thirty-two inches. The comparison is shown in **Table 6-17**. The utilization ratios from the nonlinear FEM show a similar trend with *a/d* ratios as from STM-AASHTO. For *a/d*=1.44, the utilization ratio is maximum for both the STM-AASHTO and the nonlinear FEM. However, utilization ratio is consistent for the deep and slender region.

Table 6-17: Comparison of utilization ratios with *a/d* ratio for Bridge 5

Shear span (a) (in.)	a/d	Utilization ratios		Remarks
		STM-AASHTO	Nonlinear FEM	
4.00	0.14	0.15	0.11	R1
98.00	3.03	0.15	0.12	R2
46.00	1.44	0.44	0.17	R3
47.00	1.45	0.39	0.16	R4
97.00	3.00	0.23	0.13	R5
97.00	3.01	0.24	0.12	R6
47.00	1.44	0.45	0.14	R7

6.6.4 Global Response Results

The cracking pattern of the cap beam at the failure load is shown in **Figure 6-61** and at the factored load is shown in **Figure 6-62.** Failure occurs due to shear compression in concrete.

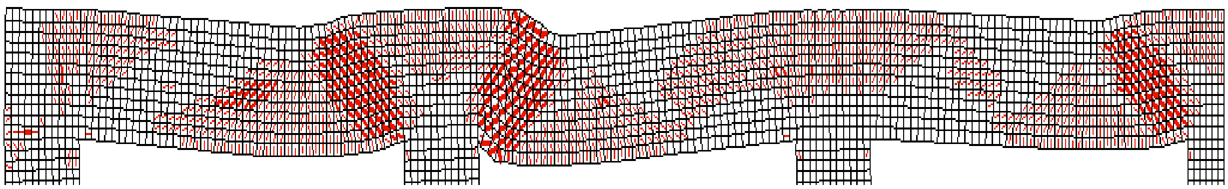


Figure 6-61: Cracking pattern at failure load for Bridge 5

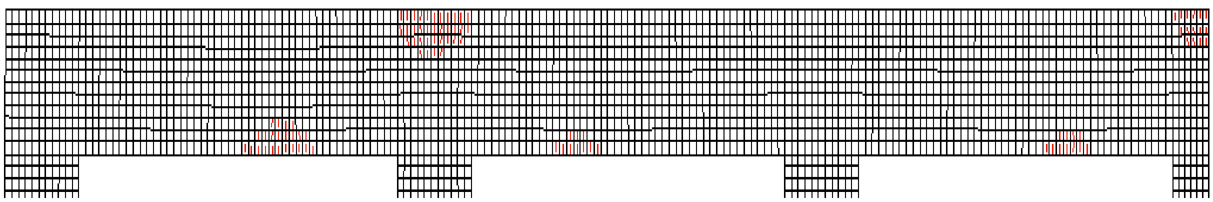


Figure 6-62: Cracking pattern at factored load for Bridge 5

The load-displacement curve is shown in **Figure 6-63.** Load corresponding to 1110 kips is the factored load acting on the pier cap. Failure occurs at Load Stage 36 with a convergence factor of 1.006, corresponding to the load of 3885 kips. The predicted capacity of pier cap from the STM-AASHTO is 2523 kips, which is 35% less predicted capacity than the nonlinear FEM. The capacity of pier cap from sectional analysis is 1338 kips (from Chapter 7), which is 65% less predicted capacity than the nonlinear FEM.

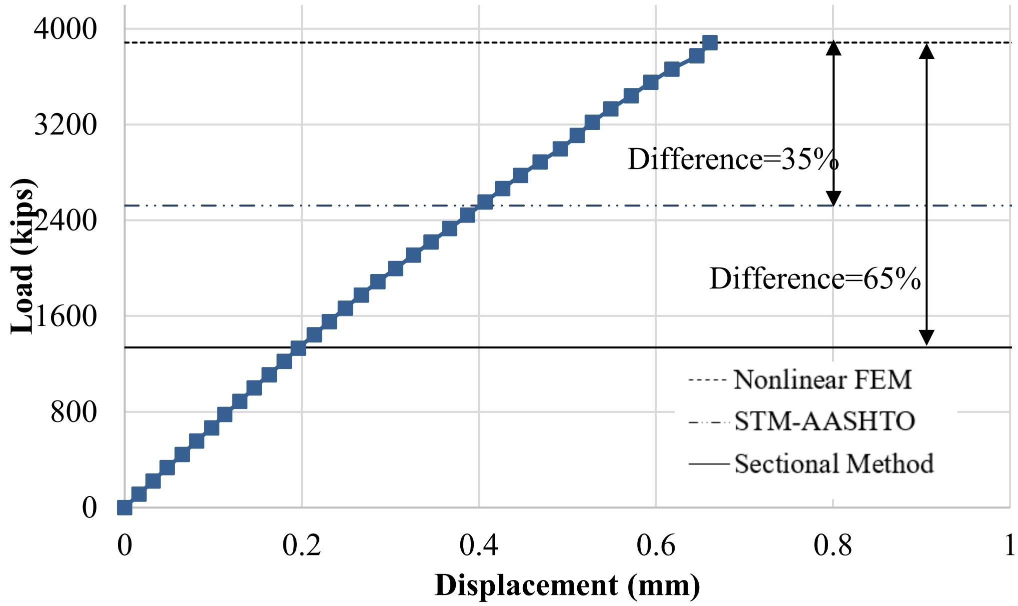


Figure 6-63: Load vs displacement response of cap beam for Bridge 5

6.6.5 Discussions

The mode of failure is a shear failure of the concrete member in the mid-region of the second span from left, which matches the STM-AASHTO. The utilization ratios from the nonlinear FEM are calculated to be 36% on average as those from the STM-AASHTO. The nonlinear FEM determines 1.5 times higher predicted capacity than the STM-AASHTO.

6.7 Discussions

The maximum utilization ratio of tension ties, horizontal struts, and inclined struts from the above bridges are summarized in **Table 6-18** from the three methods: the STM-AASHTO, CAST software, and nonlinear Finite Element Method (FEM). Finite element method calculates the maximum capacities for the pier caps. The optimized results from the STM-AASHTO truss model are used for the comparison. The maximum utilization ratio from all three methods, which govern the failure, is found in the same member for most cases. The utilization ratios from the nonlinear FEM using VecTor2 are calculated to be 40% on average of those from the STM-AASHTO. In

Bridge 3, inclined struts have the maximum utilization ratios and thus governs the failure. The governing behavior and the mode of failure matches for the pier caps. VecTor2 represents the strut behavior due to the deep beam action more accurately than the STM-AASHTO. In Bridge 2* the nonlinear FEM determines the failure mode to be the crushing of the concrete caused by shear, which occurs after yielding of the tensile reinforcement. The STM-AASHTO, on the other hand, is based on a lower-bound theorem and thus terminates the analysis at the first yielding of the reinforcement. However, after the first yielding of the reinforcement, the nonlinear FEM predicted two times higher load capacity before the failure by crushing of the concrete due to significant redistribution of forces.

Table 6-18: Bridge pier cap max utilization ratios summary table for Bridge 5

Bridge Name	**Pier Cap**	**Model**	**Utilization ratios**		**Nonlinear FEM/ STM-AASHTO**
			STM-AASHTO	**Nonlinear FEM**	
Bridge 1	Pier 2-Left	Tension Ties	0.71	0.37	0.52
		Horizontal Struts	0.69	0.39	0.57
		Inclined Struts	0.49	0.39	0.80
Bridge 2*	Pier 2-Left	Governing Member	1.02	0.15	0.15
Bridge 3	North pier cap	Tension Ties	0.51	0.15	0.29
		Horizontal Struts	0.31	0.15	0.48
		Inclined Struts	0.55	0.26	0.47
Bridge 4	Any	Tension Ties	0.48	0.13	0.27
		Horizontal Struts	0.32	0.19	0.59
		Inclined Struts	0.54	0.21	0.39
Bridge 5	Any	Tension Ties	0.34	0.09	0.26
		Horizontal Struts	0.05	0.02	0.20
		Inclined Struts	0.44	0.17	0.39

The shear span to depth ratios are compared with the utilization ratios of critical concrete members in **Table 6-19**. For all regions, the nonlinear FEM calculates lower utilization ratios and higher predicted capacities than STM-AASHTO. The comparison of utilization ratios from the STM-AASHTO and the nonlinear FEM is shown in **Figure 6-64**.

Table 6-19: Bridge pier caps *a/d* ratios with utilization ratios for Bridge 5

Bridge Name	a/d	Utilization ratios		Nonlinear FEM/ STM-AASHTO
		STM-AASHTO	Nonlinear FEM	
Bridge 1	1.40	0.49	0.39	0.80
	1.64	0.41	0.11	0.27
	1.89	0.42	0.28	0.67
Bridge 2	0.10	0.35	0.31	0.88
	0.76	0.18	0.31	1.72
	1.00	0.22	0.31	1.41
	1.91	0.43	0.36	0.83
Bridge 3	0.19	0.27	0.15	0.56
	0.69	0.22	0.21	0.95
	1.72	0.55	0.26	0.47
	2.29	0.22	0.15	0.68
	2.80	0.29	0.19	0.66
Bridge 4	0.50	0.31	0.18	0.58
	0.71	0.29	0.19	0.66
	1.30	0.54	0.21	0.39
	1.51	0.37	0.16	0.43
Bridge 5	1.44	0.44	0.14	0.32
	1.44	0.45	0.17	0.38
	1.45	0.39	0.16	0.41
	3.00	0.23	0.13	0.57
	3.01	0.24	0.12	0.50
	3.03	0.15	0.12	0.80

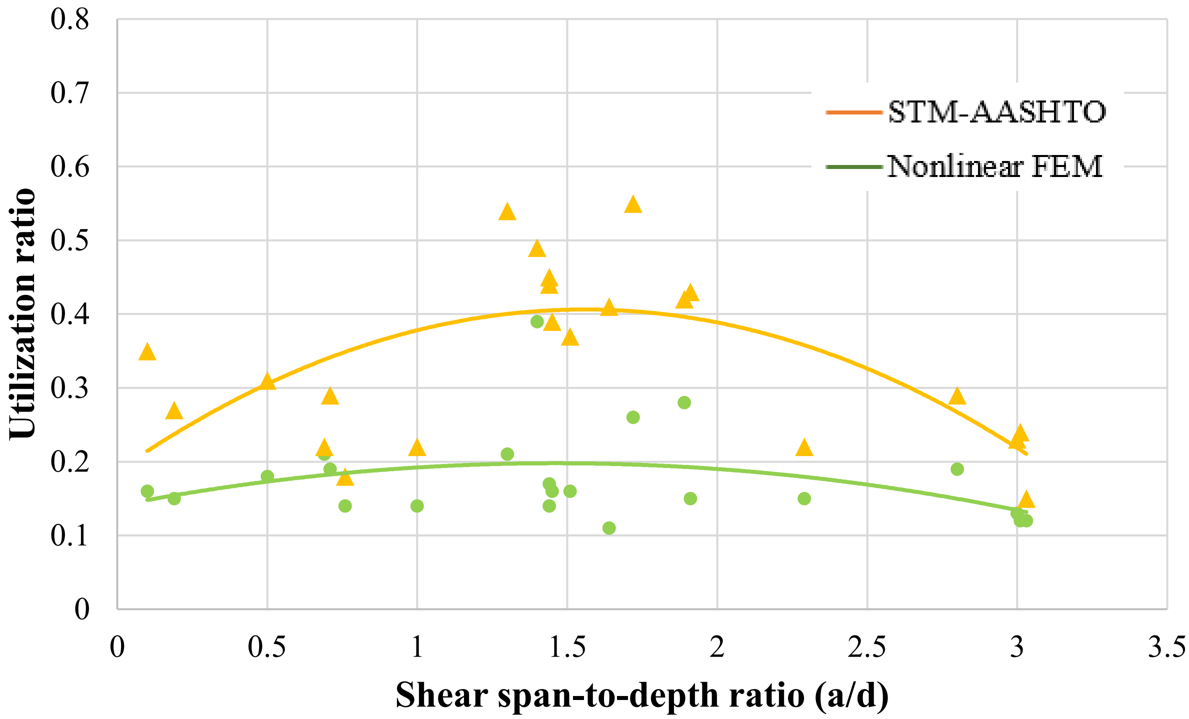


Figure 6-64: Comparison of utilization ratios from analysis methods with *a/d* ratio

The above graph shows the utilization ratios from the STM-AASHTO and the nonlinear FEM for regions with the calculated shear span-to-depth ratio. A single pier cap typically has different *a/d* ratios within each span or region with different utilization ratios, which are compared in **Figure 6-64**. The parabolic trendline drawn shows that the utilization ratios from the nonlinear FEM are less (i.e., higher predicted capacity) than the STM-AASHTO for the deep as well as slender regions. Three outliers between *a/d* ratio of 1.4 and 2.0, which have a higher utilization ratio in the nonlinear FEM, are the results from the cantilever span of the beam. For *a/d* ratios between 1.5 and 2.0, nonlinear FEM calculates lower utilization ratio and up to two times higher shear capacity prediction than the STM-AASHTO. With the decrease in the *a/d* ratio, the discrepancy between the nonlinear FEM and the STM-AASHTO decreases; the two curves converge at *a/d* ratios less than 0.2.

6.8 Conclusions

This chapter presents the simulation of the behavior of pier caps using nonlinear finite element analysis by the VecTor2 program. Nonlinear finite element analysis methods provide complete response simulation with highly accurate results but require significant knowledge and experience to obtain correct results. The STM-AASHTO provides the similar failure patterns for deep cap beams in less time. The behavior of pier caps from the nonlinear FEM were found to match the STM-AASHTO. The critical members are the same, and the failure patterns match well. The members with high utilization ratios from the STM-AASHTO match the highly stressed members in the nonlinear FEM analysis.

The utilization ratios from the nonlinear FEM are calculated to be 40% on average of those from STM-AASHTO. The utilization ratios for concrete and main rebar components are calculated and compare with an exception for vertical ties. The reasons for this are: 1) VecTor2 inherently represents the strut behavior due to the deep beam action more accurately, and 2) a single concentrated vertical tie is considered in the STM-AASHTO, which gives high-stress ratios, whereas VecTor2 uses a uniform spacing of stirrups which calculates more distributed and lower stress ratios.

The utilization ratios from the nonlinear FEM and the STM-AASHTO for different *a/d* ratios in pier cap are compared. Nonlinear FEM calculate higher capacity predictions, as expected for the deep as well as slender regions, than the STM-AASHTO. The STM-AASHTO is based on a lower-bound theorem and thus terminates the analysis at the first yielding of the reinforcement whereas nonlinear FEM continues the analysis till failure of the structure, consisting of nonlinear deformation and redistribution of stresses. The utilization ratios from nonlinear FEM are almost consistent for every region. For *a/d* ratios between 1.5 and 2.0, the nonlinear FEM give up to two times higher shear capacity predictions than the STM-AASHTO. With the decrease in *a/d* ratio, the discrepancy between the nonlinear FEM and the STM-AASHTO decreases and the response curves converge at *a/d* ratios less than 0.2.

The nonlinear FEM calculates on average 1.45 times higher global capacity prediction for pier caps than the STM-AASHTO. It should be noted that it takes approximately fifteen to twenty hours for each cap beam to model, run the simulation, and obtain/understand the analysis results in the nonlinear FEM.

Chapter 7 Sectional Method Vs Strut-and-Tie Method

7.1 Introduction

The sectional method is a structural analysis method valid for slender beams (i.e., shear span-to-depth ratios (*a/d*) >2.0). The sectional method assumes a linear strain distribution through a member's depth as per the Bernoulli hypothesis (Guner, 2008). The sectional method is simple but not appropriate for deep beams. The Strut-and-Tie Method (STM), which is based on the deep beam theory, does not assume a linear strain distribution which is more accurate for deep pier caps. Nonlinear finite element analysis methods (e.g., VecTor2) provide complete response simulation with highly accurate results but require significant knowledge and experience to obtain correct results. The strut-and-tie method and the STM-CAP provide a good compromise between complexity and accuracy. While it is as simple as the sectional method, it provides an accuracy closer to the finite element method. STM is based on the lower-bound theorem, which is still conservative when compared with nonlinear analysis or experimental tests. Although not recommended, five bridge pier caps are analyzed using the sectional method for comparison purposes. The flexural and shear utilization ratios at critical sections are determined and compared to the sectional method and STM. For the sectional method, the utilization ratios are determined from hand calculations. For STM, the optimized model from STM-CAP is used to obtain the maximum capacity from the pier cap. SAP2000 is used to determine the shear force diagram (SFD) and bending moment diagrams (BMD). The flexure utilization ratios are determined as the ratio of rebar stressed at factored loads to the yield stress of the rebar. For the sectional method, the tensile stress in the rebar is calculated based on the moment-curvature response. The shear utilization ratio is calculated as ratio of the shear force to shear capacity at each critical section. The factored sectional shear capacities are calculated based on empirical formulations from

AASHTO. The developed spreadsheet program STM-CAP is used to determine the utilization ratios for each STM member. The utilization ratios for flexure by sectional method is compared with utilization ratio of horizontal tension ties of the STM member at the critical sections. The utilization ratios of shear by the sectional method is compared with that of the inclined and vertical STM members. The utilization ratios obtained from the sectional method and STM-CAP are compared in **Table 7-1**.

Table 7-1: Comparison of shear utilization ratios (URs) for sectional method and STM

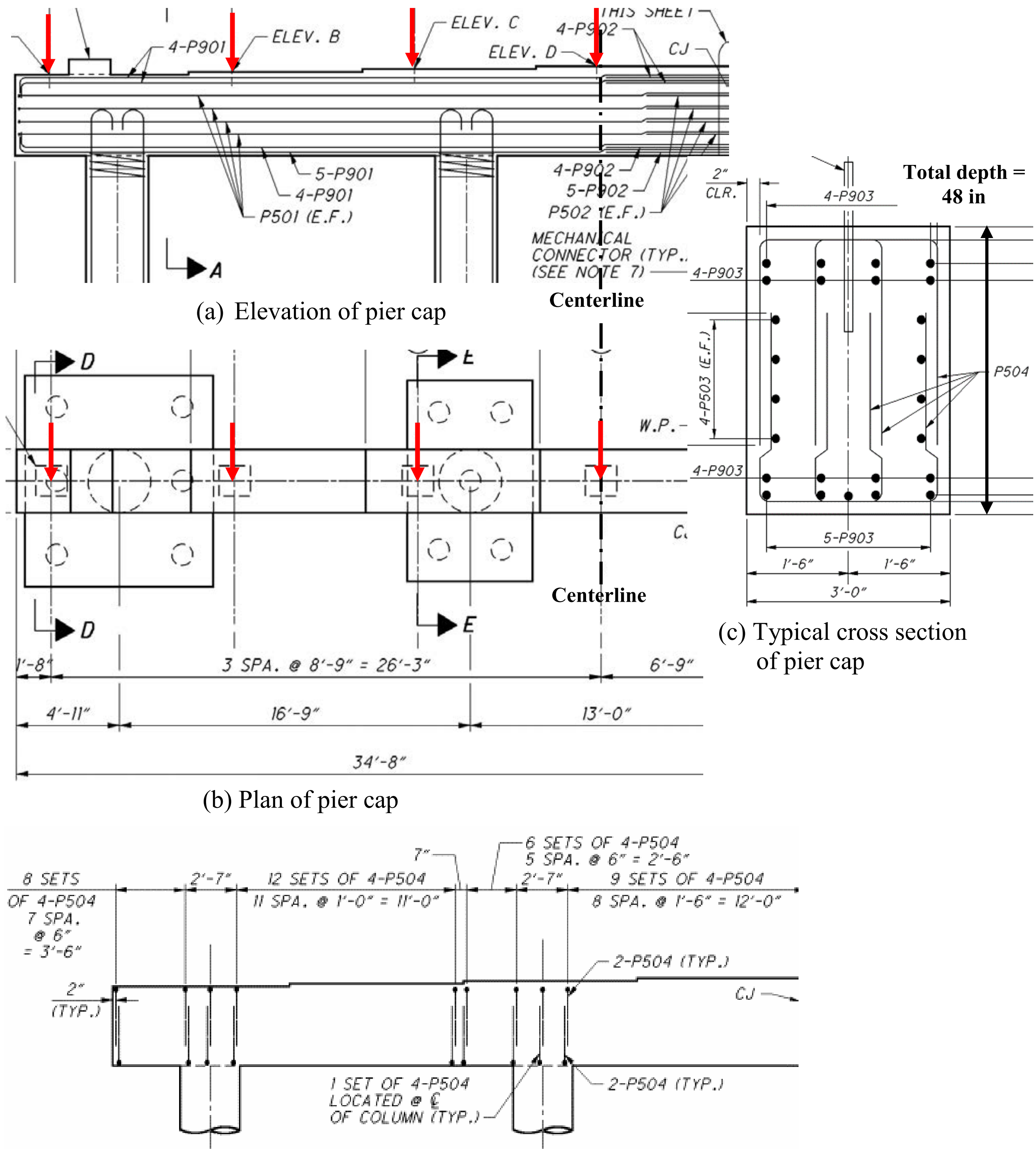
Shear Utilization Ratios (optimized) Comparison			
Bridge	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)
Bridge 1	0.71	0.43	0.29
	1.30	0.60	0.54
	0.50	0.48	0.31
Bridge 2	1.00	0.54	0.22
	0.76	0.54	0.18
	1.91	0.54	0.43
Bridge 3	0.69	0.54	0.22
	1.72	0.60	0.55
	2.80	0.25	0.30
	2.29	0.53	0.38
Bridge 4	3.03	0.17	0.15
	1.44	0.83	0.44
	1.41	0.73	0.39
	3.00	0.28	0.23
	3.01	0.29	0.24
	1.44	0.72	0.38
Bridge 5	0.46	1.09	0.26
	2.82	0.31	0.27
	1.23	0.78	0.39
	2.15	0.55	0.48

Table 7-1 shows that the sectional method gives higher utilization ratios than STM-CAP for all cases. A higher utilization ratio means lower capacity predictions for the same load. The deeper (lower *a/d* ratios) the beam or the region is, the more conservative the sectional method predicted

capacity is. The comparisons show that the sectional method systematically underestimates the shear capacity prediction of deep pier caps analyzed. The calculation details for the sectional method are shown below.

7.2 Bridge 1

The engineering drawing for Bridge 1 is shown in **Figure 7-1**. Bridge 1 corresponds to the 'Bridge Pier Cap 4' from Appendix A. The pier cap has four columns and is symmetric. It has four girder loads and two columns in the half symmetric section. The reinforcement cross section is shown in **Figure 7-1 (c)**. It has the same longitudinal reinforcement throughout the section. The shear reinforcement is four-legged, #5 bars with the spacing as shown in **Figure 7-1 (d)**.



(a) Elevation of pier cap

(b) Plan of pier cap

(c) Typical cross section of pier cap

(d) Shear reinforcement details

Material properties→ Concrete compressive strength (f'_c) = 4 ksi
Reinforcement yield strength (f_y) = 60 ksi

Figure 7-1: Bridge 1 pier cap details.

Shear Force Diagram (SFD) and Bending Moment Diagram (BMD) using SAP2000

The full pier cap is modeled as an indeterminate beam in SAP2000 to determine the SFD and BMD, as shown in **Figure 7-2**, to identify the critical sections. The critical section for BMD is the point of maximum bending moment value as indicated in Section 1-1, etc., as shown in **Figure 7-2**. Since the shear span is less than the depth of the beam, the critical section for the shear is assumed to be the face of the support based on Section 5.5.3.2 of AASHTO LRFD 2014. Therefore, the critical section for shear is the point at the faces of the columns (half pier width, i.e. eighteen inches away from the center of support), as shown in Section A-A, etc. The bending moment values are interpolated at Sections A-A, B-B, and C-C.

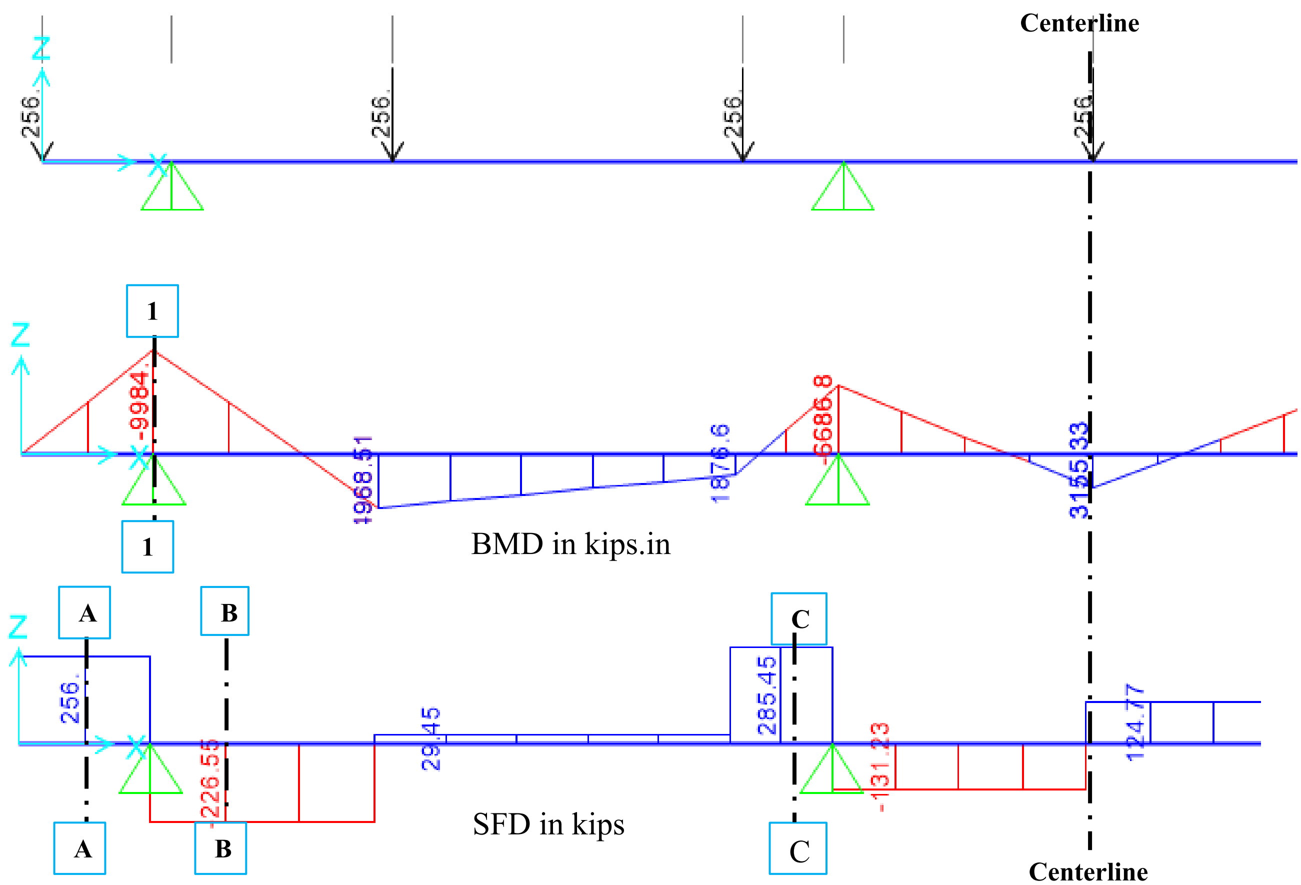


Figure 7-2: Bridge 1 pier cap BMD and SFD using SAP2000

7.2.1 Sectional Method

The pier cap is a doubly reinforced section. For flexure, the moment-curvature response is determined at each section. The moment-curvature is required to determine the rebar tensile stress. AASHTO formulations in Section 5.7.3 are used to calculate the shear capacity at each critical section.

Moment Capacity

Section 1-1

The section properties based on **Figure 7-3**, **Figure 7-4**, and **Figure 7-5** are used to calculate the moment curvature response of the pier cap.

$f_c^{'} = 4\text{ksi}$

$f_y = 60\text{ksi}$

Effective depth (d) = Total depth – centroid = 48 in – 4.5 in = 43.5 in

$E_c = 57\sqrt{f_c^{'}(\text{psi})} = 57\sqrt{4000}\text{ksi} = 3,600\text{ksi}$

Es=29,000ksi

Modular Ratio (n) = E_s/E_c = 8

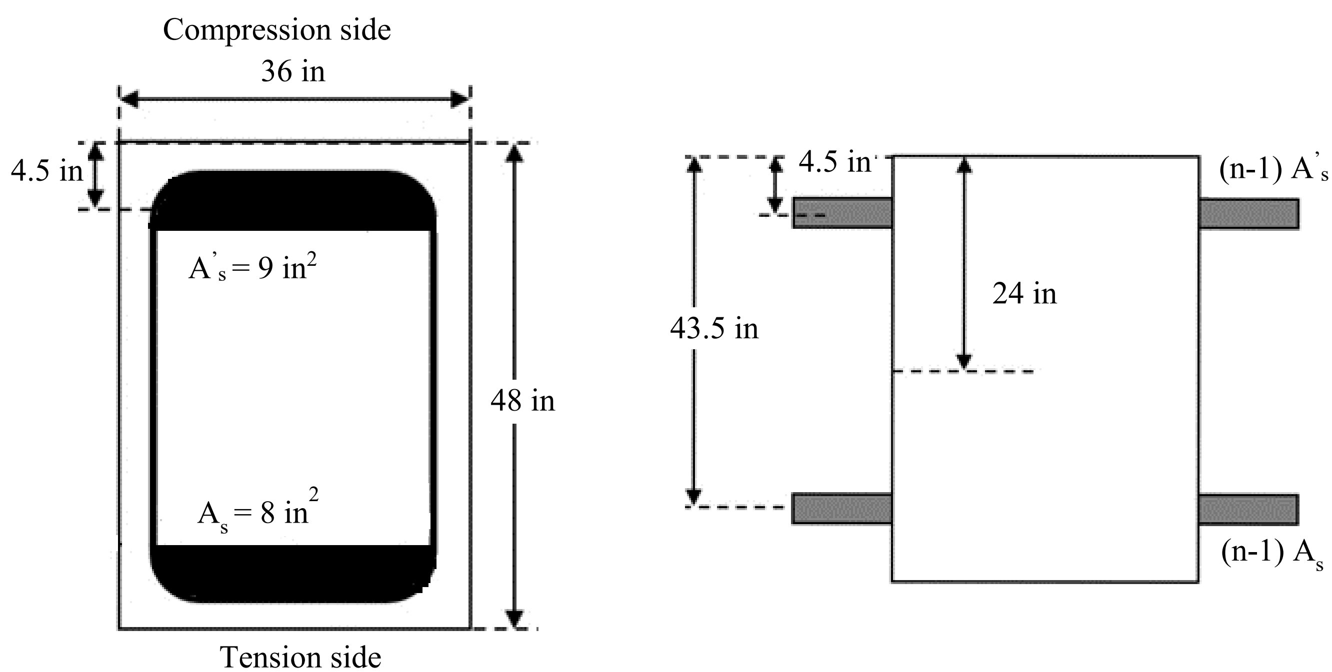


Figure 7-3: Section details and un-cracked transformed section properties

Tension at top

$A_{st,\,top} = A_s = 8$ #9 $= 8*1\ in^2 = 8\ in^2$

$A_{st,\,bottom} = A_s^{'} = 9$ #9 $= 9*1\ in^2 = 9\ in^2$

(1) Uncracked, transformed section

The position of centroid from the top is

$$\overline{Y} = \frac{\sum A_i Y_i}{\sum A_i}$$

$$\overline{Y} = \frac{36*48*24+(8-1)*9*4.5+(8-1)*8*43.5}{36*48*(8-1)*9+(8-1)*8} = 23.93in \approx 24in$$

The uncracked-transformed moment of inertia about neutral axis is

$$I_{un,tr} = \sum(I_i + A_i d_i^{\,2})$$

$$I_{un,tr} = \frac{b*d^3}{12} + (n-1)A_s^{'}*(24-4.5)^2 + (n-1)A_s*(43.5-24)^2 = 377,025in^4$$

The concrete tensile strength, $f_t = 0.20\sqrt{f_c^{'}} = 0.40\,ksi$

The cracking moment required, $M_{cr} = \dfrac{f_t * I_{un,tr}}{Y_b} = \dfrac{0.4ksi*377,025in^4}{(48-24)in} = 6,283kip.in$

The curvature at cracking, $\Phi_{cr} = \dfrac{M_{cr}}{E_c * I_{un,tr}} = \dfrac{6,283kip.in}{3,600ksi*377,025\,in^4} = 4.63*10^{-6}\,/\,in$

(2) Cracked, transformed section

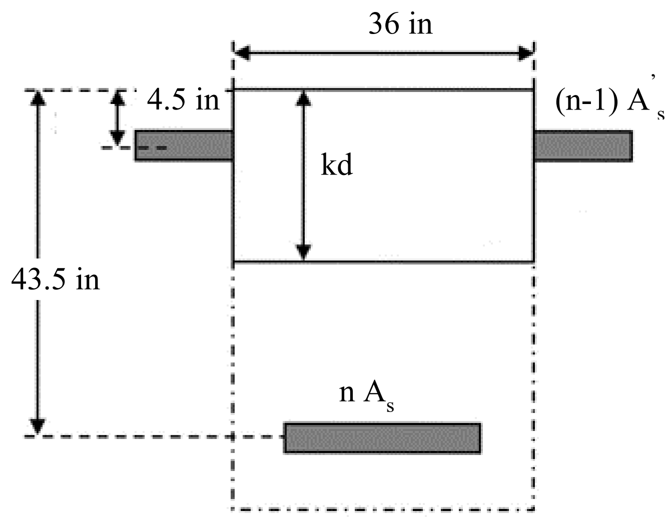


Figure 7-4: Cracked, transformed section properties

Let the neutral axis be kd,

$$\sum M_{area} = 0$$

$$b*kd*\frac{kd}{2}+(n-1)A_s^{'}*(kd-d^{'}) = nA_s*(d-kd)$$

$$18*(kd)^2+127(kd)-3067.5=0$$

Solving for kd, kd=10 in

The cracked-transformed moment of inertia about the neutral axis.

$$I_{cr} = \frac{36*10^3}{12}+36*10*(5)^2+(n-1)*A_s^{'}*(kd-4.5)+nA_s*(d-kd)^2$$
$$I_{cr} = 36000/12+36*10*25+7*9*(10-4.5)^2+8*8*(43.5-10)^2$$
$$I_{cr} = 85730 in^4$$

The curvature at cracking, $\Phi_{cr} = \frac{M_{cr}}{E_c * I_{un,tr}} = \frac{6,283 kip.in}{3,600 ksi * 85729 in^4} = 2.03*10^{-5} / in$

(3) Linear until yield of steel or concrete non-linear

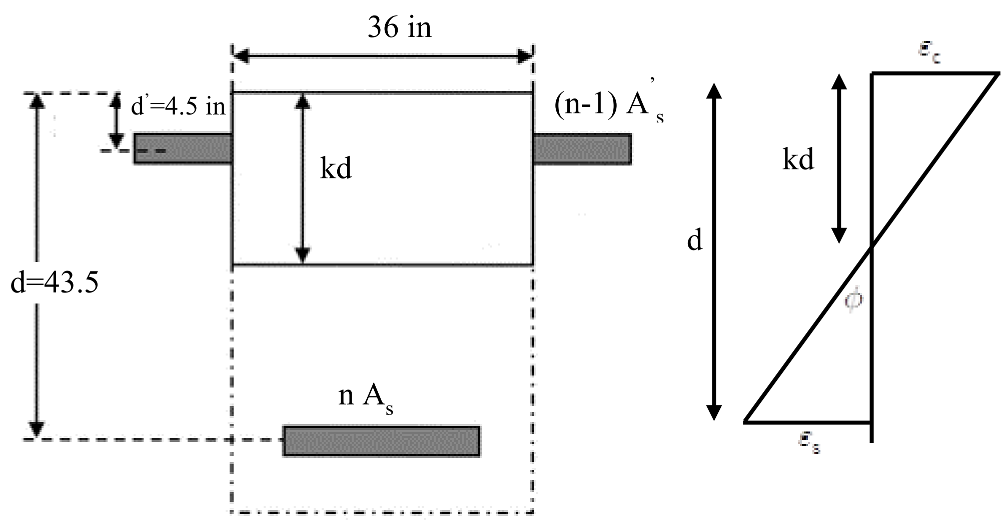


Figure 7-5: Strain distribution

Steel yields when, $\varepsilon_s = \varepsilon_y = \frac{f_y}{E_s} = \frac{60\text{ksi}}{29000\text{ksi}} = 2.1*10-3$

Concrete becomes non-linear when, $f_c = 0.7f_c^{'}$

First case: Let's consider steel yields

$$\phi_y = \frac{\varepsilon_s}{d-kd} = \frac{2.1*10^{-3}}{43.5-10} = 6.27*10^{-5}\ /\ \text{in}$$

$$M_y = \phi_y * E_c * I_{cr,tr} = 6.27*10^{-5}\ /\ \text{in}*3600\text{ksi}*85730\text{in}^4 = 19350\text{kip.in}$$

Second case: Let's consider concrete becomes non-linear

$$\phi_{0.7f_c^{'}} = \frac{0.7f_c^{'}}{E_c*kd} = \frac{0.7*5}{3600*10} = 7.77*10^{-5}\ /\ \text{in}$$

$$M_{0.7f_c^{'}} = \phi_{0.7f_c^{'}} * E_c * I_{cr,tr} = 7.77*10^{-5}\ /\ \text{in}*3600\text{ksi}*85730\text{in}^4 = 24,000\text{kip.in}$$

Since, $M_y < M_{0.7f_c^{'}}$, the steel (rebar) yields first.

Therefore, M_y=19,350 kip.in and $\phi_y = 6.27*10^{-5}\ /\ \text{in}$

(4) Nominal (ultimate) strength

Let's assume the stress in rebar on compressive side is 1 ksi

$$\text{Guess}, f_s^{'} = 1\text{ksi}$$
$$\varepsilon_s^{'} = f_s^{'} / E_s = 1\text{ksi} / 29{,}000\text{ksi} = 3.45*10^{-5}$$

$\beta_1 = 0.85$ for f'c = 4 ksi concrete

Depth of neutral axis, $c = \dfrac{A_s f_s - A_s^{'} f_s^{'}}{0.85 f_c^{'} * b * \beta_1} = \dfrac{8*60 - 9*1}{0.85*4*36*0.85} = 4.53\text{in}$

$$f_s^{'} = 0.003*E_s*\left(\frac{c-d^{'}}{c}\right) = 0.003*29{,}000\text{ksi}*\frac{4.53\text{in} - 4.5\text{in}}{4.53} = 0.6\text{ksi} \quad \text{(Not Matched)}$$

Upon iteration the value of f'$_s$ is found to be 0.8 ksi.

$$f_s^{'} = 0.8\text{ksi}$$
$$\varepsilon_s^{'} = f_s^{'} / E_s = 0.8\text{ksi} / 29{,}000\text{ksi} = 2.75*10^{-5}$$

Depth of neutral axis, $c = \dfrac{A_s f_s - A_s^{'} f_s^{'}}{0.85 f_c^{'} * b * \beta_1} = \dfrac{8*60 - 9*0.8}{0.85*4*36*0.85} = 4.54\text{in}$

$$f_s^{'} = 0.003*E_s*\left(\frac{c-d^{'}}{c}\right) = 0.003*29{,}000\text{ksi}*\frac{4.54\text{in} - 4.5\text{in}}{4.54} = 0.8\text{ksi} \quad \text{(Matched)}$$

The neutral axis (c) is found to be 4.54 in and f'$_s$ = 0.8 ksi

The curvature $\phi = \varepsilon_{cu} / c = 0.003 / 4.54 = 6.6*10^{-4} / \text{in}$

The ultimate moment capacity,

$$M_n = 0.85*f_c^{'}*b*\beta_1 c + A_s^{'} f_s^{'}*(d-d^{'}) = 0.84*4*36*0.85*4.54 + 9*0.8*(43.5-4.5) = 19916\text{kip.in}$$

<u>Summary</u>

The moment-curvature response captured is

ϕ (1/in)	M_n (kip. in)	M_u (kip. in)	M_u (kip.ft)	Comment
$4.63*10^{-6}$	6283	5655	471	before cracking
$2.03*10^{-5}$	6283	5655	471	after cracking

$6.27*10^{-5}$	19350	17415	1451	steel yields
$6.6*10^{-4}$	19916	17925	1494	ultimate condition

The rebar tensile stress is determined for ultimate moment of 9984 kip.in. Upon interpolation, the corresponding curvature at moment of 9984 kip.in is found to be $3.59*10^{-5}$.

Therefore, $\phi = 3.59*10^{-5}$ rad/ in

and strain in rebar, $\varepsilon_s = \phi*(d - kd) = 3.59*10^{-5}*(43.5-10) = 1.20*10^{-3}$

stress in tension rebar, $f_s = \varepsilon_s * E_s = 1.20*10^{-3}*29000 = 34.87\text{ksi}$

Utilization ratio for flexure at Section 1-1 $= \dfrac{\text{stress at loading}}{\text{yield stress}} = \dfrac{34.87\text{ksi}}{0.9*60\text{ksi}} = 0.65$

<u>Shear capacity</u>

Section A-A

At Section A-A, the critical section is assumed to be at the outer face of pier 1.

The shear capacity at any section cannot exceed $V_n = 0.25f_c^{'}b_vd_v$

Therefore, V_n is calculated as min of ($0.25f_c^{'}b_vd_v$, $V_c + V_s$)

where,

b_v = effective web width (in) = 36 in

d_v = effective shear depth (in) = max (0.9d, 0.72h) = max (0.9*43.5in, 0.72*48in) =39 in

The shear strength due to concrete (V_c) is calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}}*b_v*d_v$$

The shear strength due to stirrup (V_s) is calculated as:

$$V_s = \frac{A_vf_yd_v\cot\theta}{s}$$

where,

A_v = area of shear reinforcement within a distance s = 4 legged #5 = (4*0.31) in^2 = 1.24 in^2

s = spacing of transverse reinforced measured in a direction parallel to the longitudinal rebar (in)

s = 6 in

β = factor indicating the ability of reinforcement measured in a direction parallel to the rebar

θ = angle of inclination of diagonal compressive stresses

Section 5.8.3.4.1 of AASHTO LRFD 2014 specifies that "for other non-prestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement in Section 5.8.2.5", the value for β and θ can be taken as 2.0 and 45º respectively.

There is no axial force in the pier cap which satisfies the first condition. Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\frac{36*6}{60}$$

$$1.24 \geq 0.227 in^2 \qquad \text{satisfied.}$$

The value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}}*b_v*d_v = 0.0316*2*\sqrt{4}*36*39 = 177 kips$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*39*\cot(45)}{6} = 484 kips$$

And V_n = min of ($0.25f_c^{'}b_v d_v$, $V_c + V_s$)

= min of (0.25*4*36*39, 661)

= min of (1400, 661)

= 661 kips

The factored shear capacity, $V_u = \Phi * V_n = 0.9*661$ kips = 595 kips

Utilization ratio at Section A-A $= \dfrac{\text{shear force at A-A } (V_{A\text{-}A})}{\text{shear capacity at A-A } (V_u)} = \dfrac{256\text{kips}}{595\text{kips}} = 0.43$

Section B-B

At Section B-B, the shear is checked at the inner face of pier 1. The section properties of Section B-B are like those of Section A-A. The difference in reinforcement properties at Section B-B is the transverse reinforcement spacing of twelve inches.

The shear capacities are determined as:

For concrete, $V_c = 0.0316\beta\sqrt{f_c^{'}} * b_v * d_v$

For stirrup, $V_s = \dfrac{A_v f_y d_v \cot\theta}{s}$

Here,

b_v = 36 in

d_v = max (0.9d, 0.72h) = max (0.9*43.5in, 0.72*48in) =39 in

A_v = 4 legged #5 = (4*0.31) in^2 = 1.24 in^2

s = 12 in

The check for minimum transverse reinforcement is performed to determine the value of β and θ according to Section 5.7.2.5.

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\,\frac{36*12}{60}$$

$1.24 \geq 0.454\text{in}^2$ satisfied, OK.

Hence β = 2.0 and θ = 45°

Therefore, the shear capacities,

$$V_c = 0.0316\beta\sqrt{f_c^{'}} * b_v * d_v = 0.0316*2*\sqrt{4}*36*39 = 177\text{kips}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*39*\cot(45)}{12} = 242\text{kips}$$

$V_n = \min \text{ of } (0.25f_c^{'}b_v d_v, V_c + V_s)$

= min of (0.25*4*36*39, 177+242)

= min of (1400, 419)

= 419 kips

The factored shear capacity, $V_u = \Phi * V_n$ = 0.9*419 kips = 377 kips

Utilization ratio at Section B-B $= \dfrac{\text{shear force at B-B } (V_{B-B})}{\text{shear capacity at B-B } (V_u)} = \dfrac{226.5\text{kips}}{377\text{kips}} = 0.60$

Section C-C

At Section C-C, the shear is checked at the inner face of pier 2. The section properties of Section C-C are like those of Section A-A. Both sections have four-legged, #5 bars, spaced @ six inches center-to-center. Therefore, the shear capacities at Section C-C is the same as shear capacities determined at Section A-A.

The factored shear capacity, $V_u = \Phi * V_n$ = 0.9*661 kips = 595 kips

$$\text{Utilization ratio at Section C-C} = \frac{\text{shear force at C-C } (V_{C-C})}{\text{shear capacity at C-C } (V_u)} = \frac{285.5\text{kips}}{595\text{kips}} = 0.48$$

Verification of capacity calculation by Response-2000

The moment capacity and shear capacity at Section 1-1 and Section A-A are verified with Response-2000 software. Response-2000 is a sectional analysis program that calculates the moment and shear capacity of a beam subject to axial, shear, and moment loads. All loads are considered simultaneously to find the full load-deformation response using modified compression field theory (MCFT). The results from Response-2000 are shown in **Figure 7-6.**

Figure 7-6(a) shows the moment-shear interaction diagram. **Figure 7-6(b)** shows the moment-curvature response captured for the pier cap at Section 1-1, which is found to be 1431.5 kip.ft and an approximate value of 1494 kip.ft obtained from the hand calculation. Response-2000 verifies the hand calculation. **Figure 7-6(b)** shows the shear prediction made by Response-2000 at Section A-A, using the sectional method, is found to be 597.7 kips. This is the same value obtained at Section A-A of 595 kips from the hand calculation. Thus, the hand calculation results are verified with Response-2000.

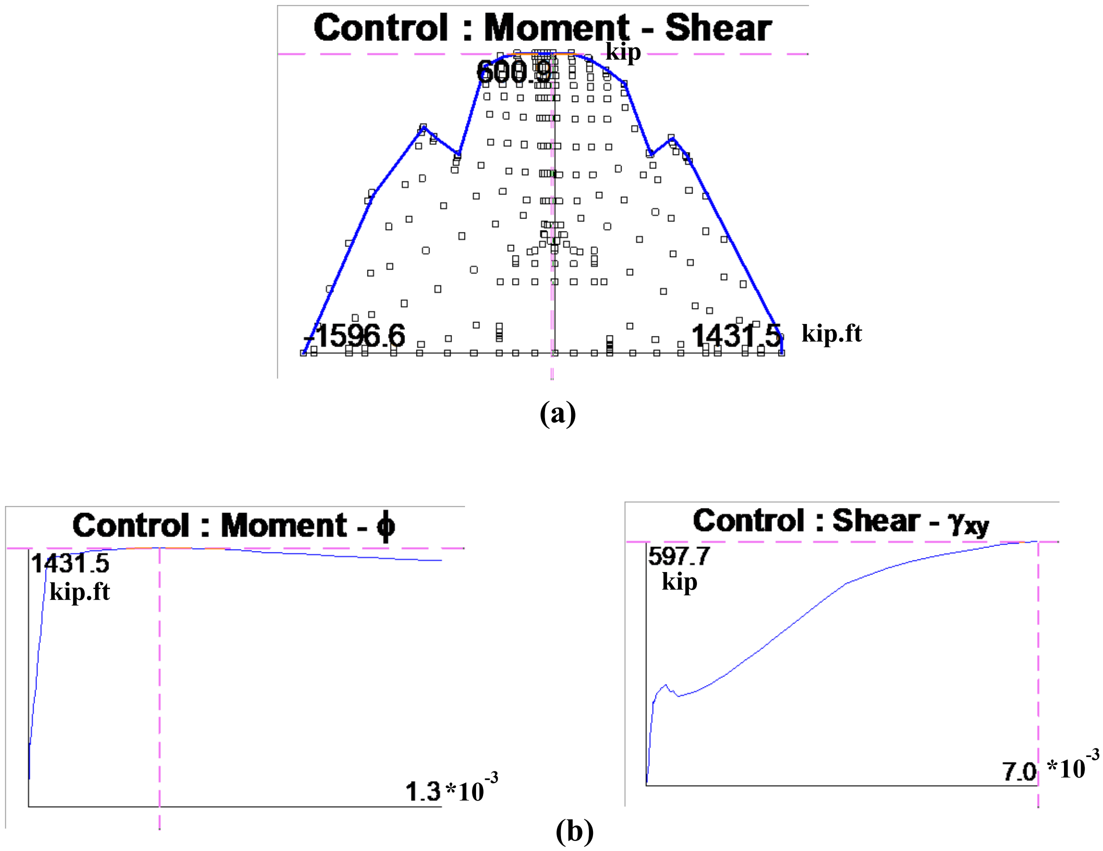


Figure 7-6: (a) Moment-shear interaction diagram **(b)** moment-curvature and shear response

7.2.2 Strut-and-Tie Method

The developed spreadsheet program STM-CAP is based on the strut-and-tie method. The pier cap is modeled in STM-CAP to determine the optimized utilization ratios. The full input and output fields using STM-CAP can be seen at Bridge 4 from Appendix A. **Figure 7-7** shows the screenshot of STM-CAP utilization ratios output screen.

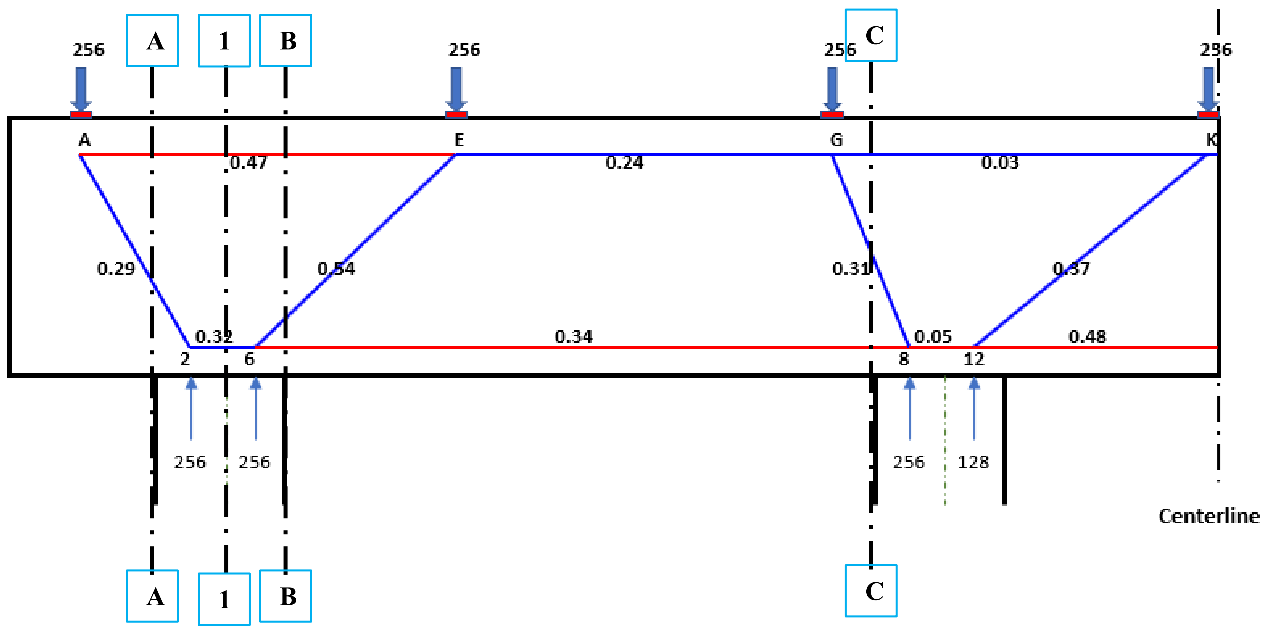


Figure 7-7: Utilization ratios calculated from STM-CAP for Bridge 1

7.2.3 Comparisons

The utilization ratio from the sectional method and STM-CAP are compared. For flexure at Section 1-1, the utilization ratio obtained from flexure using the sectional method is compared with the utilization ratio of the horizontal tension tie (Members A-E) of STM-CAP. Since both utilization ratios are calculated from the tensile stress in the rebar, they match the comparison concept. The shear utilization ratios from the sectional method is compared with the utilization ratios of inclined members of STM-CAP. To match the comparison for shear action, the load and capacity in the inclined strut can be resolved in the vertical direction to determine the shear utilization ratio. Since the angle is the same for both load and capacity, the resolving factor will be the same and thus cancel out in numerator and denominator to give the same utilization ratio as the inclined member. Hence, the inclined STM members represent the shear action. The utilization ratios from the sectional method and STM are compared in **Table 7-2** and **Table 7-3**.

Table 7-2: Comparison of flexure URs for the STM and the sectional method for Bridge 1

Flexural Utilization Ratios Comparison at Section 1-1				
Parameter	*a/d* ratio	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
Utilization Ratio	0.71	0.65	0.47	1.38

Table 7-3: Comparison of shear URs for the STM and the sectional method for Bridge 1

Shear Utilization Ratios Comparison at Different Sections				
Sections	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
A-A	0.71	0.43	0.29	1.48
B-B	1.30	0.60	0.54	1.11
C-C	0.50	0.48	0.31	1.55

7.2.4 Discussions

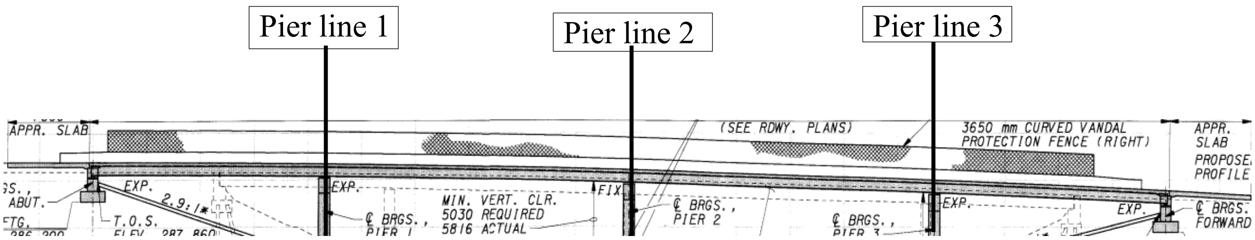
Table 7-2 and **Table 7-3** shows the utilization ratios calculated from the sectional method and STM-CAP. The higher the utilization ratio, the lower the capacity prediction for the same load. In all of the above cases, the sectional method calculates higher utilization ratios and hence lower capacity at each section. The reason for the highly conservative capacity prediction for all of the above case is that each of these regions is deep.

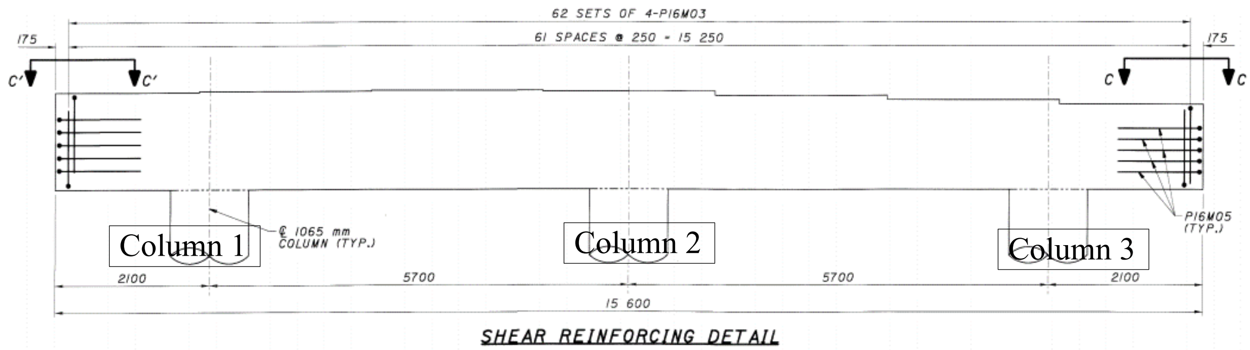
For flexure, the shear span-to-depth ratio is 0.71 and the utilization ratio using the sectional method is found to be 38% higher than STM-CAP. Hence the flexure capacity predicted by STM-CAP is 38% higher than the sectional method. For shear capacity predictions, the deeper the beam (lower *a/d* ratio), the higher conservative results are predicted from the sectional method as compared to the STM-CAP method. For shear span-to-depth ratio of 0.5 to 1.3, STM-CAP provides 55% to 11% higher capacity than the sectional method. STM is based on the lower-bound theorem that is still conservative as compared to nonlinear analysis methodology and experimental cases.

Therefore, the sectional method highly underestimates the capacity prediciton of deep pier caps and is not a recommended method for deep pier cap beams. AASHTO LRFD 2014 requires either the use of strut-and-tie or nonlinear analysis for deep structures.

Using similar procedures, the shear capacities of the other bridges are hand calculated using the sectional method and the utilization ratios obtained from the sectional method are compared with the utilization ratios from STM-CAP.

7.3 Bridge 2





Typical pier cap

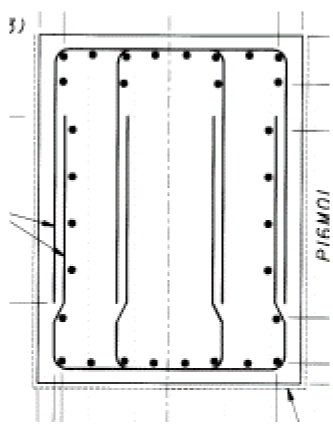
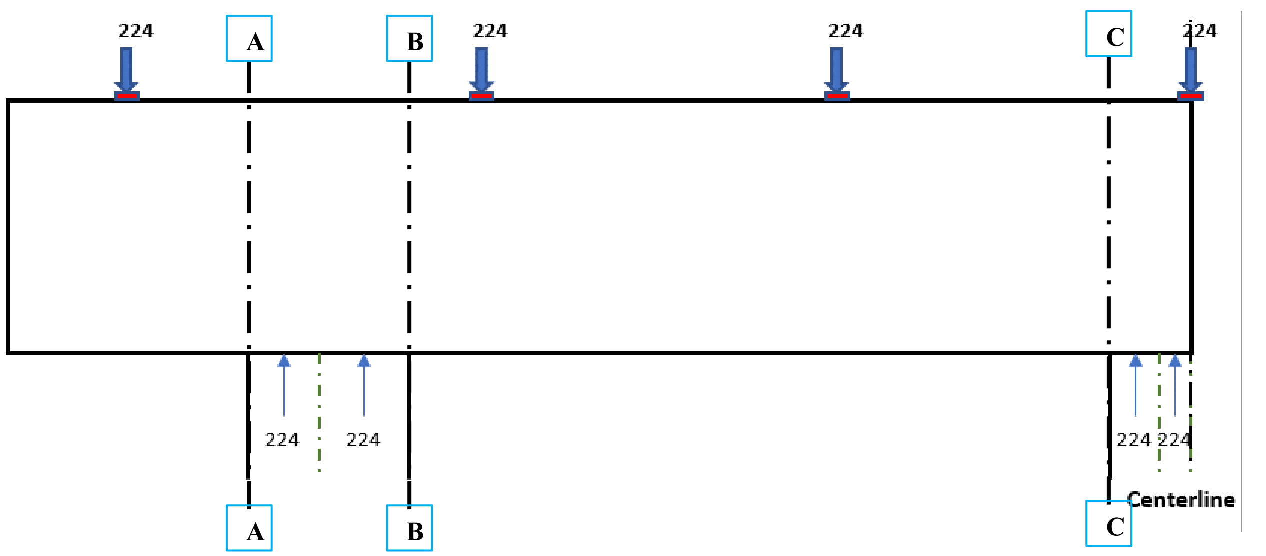


Figure 7-8: Bridge 2 details

The engineering drawing for **Bridge 2** is shown in **Figure 7-8**. Bridge 2 corresponds to the 'Bridge Pier Cap 2' from Appendix A. This pier cap has three columns and is symmetric. In the half symmetric section, it has four girder loads. The shear reinforcement details are shown in **Figure 7-8**. The stirrups are equally spaced at 250 mm or ten inches c/c throughout the pier cap section. Each stirrup is four-legged, #5 bars as shown in **Figure 7-8**.

7.3.1 Sectional Method

The utilization ratio for shear using the sectional method is determined by dividing the shear force with shear capacity. The shear force at each span can be determined using any software. STM-CAP calculates the reaction at each support and will be used to determine the shear force in the pier cap. The shear check is done at each face of the column. Therefore, there are three critical sections (e.g., A-A, B-B, etc.), as shown in **Figure 7-9.**



Shear Force at

A-A= 224 kip   B-B= 224 kip   C-C= 224 kip

Figure 7-9: Shear forces at critical sections for Bridge 2

The shear force at Section A-A is determined from the panel shear using the load. The reaction is found to be 224 kips. The shear force at each section is determined at each critical section as shown in **Figure 7-9.**

Using the AASHTO LRFD, the shear capacity at each critical section is determined as follows:

<u>Shear capacity</u>

Section A-A

At Section A-A, the critical section is assumed to be at the outer face of pier 1.

The shear capacity at any section cannot exceed $V_n = 0.25f_c^{'}b_v d_v$

Therefore, V_n is calculated as minimum of ($0.25f_c^{'}b_v d_v$, $V_c + V_s$)

where,

$b_v = 42$ in

d_v = max (0.9d, 0.72h) = max (0.9*40.5in, 0.72*45in) =36.5 in

A_v = 4 legged #5 = (4*0.31) in^2 = 1.24 in^2

s = 10 in

β = factor indicating the ability of reinforcement measured in a direction parallel to the rebar

θ = angle of inclination of diagonal compressive stresses

There is no axial force in the pier cap which satisfies the first condition. Checking for the minimum transverse according to Section 5.8.2.5 of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24 ? 0.0316 * \sqrt{4}\frac{42*10}{60}$$

$1.24 \geq 0.44 in^2$ satisfied, OK.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}} * b_v * d_v = 0.0316 * 2 * \sqrt{4} * 42 * 36.5 = 194 kips$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*36.5*\cot(45)}{10} = 271\text{kips}$$

And, $V_n = \text{min of} (0.25f_c^{'}b_v d_v$, $V_c + V_s$)

= min of (0.25*4*42*36.5, 465)

= min of (1533, 465)

= 465 kips

Hence, the factored shear capacity, $V_u = \Phi*V_n$ = 0.9*465 kips = 419 kips

$$\text{Utilization ratio at Section A-A } = \frac{\text{shear force at A-A } (V_{A\text{-}A})}{\text{shear capacity at A-A } (V_u)} = \frac{224\text{kips}}{419\text{kips}} = 0.54$$

Section B-B

At Section B-B, the shear is checked at the inner face of pier 1. The section properties of Section B-B are the same as Section A-A. Section B-B has the same stirrup spacing of ten inches.

The factored shear capacity, $V_u = \Phi*V_n$ = 0.9*465 kips = 419 kips

$$\text{Utilization ratio at Section B-B } = \frac{\text{shear force at B-B } (V_{B-B})}{\text{shear capacity at B-B } (V_u)} = \frac{224\text{kips}}{419\text{kips}} = 0.54$$

Section C-C

At Section C-C, the shear is checked at the inner face of pier 2. The section properties of Section C-C are like those of Section A-A. Both sections have four-legged, #5 bars, spaced @ ten inches center-to-center. Therefore, the shear capacities at Section C-C are the same as shear capacities determined at Section A-A.

The factored shear capacity, $V_u = \Phi*V_n$ = 0.9*465 kips = 419 kips

$$\text{Utilization ratio at Section C-C} = \frac{\text{shear force at C-C } (V_{C-C})}{\text{shear capacity at C-C } (V_u)} = \frac{224\text{kips}}{419\text{kips}} = 0.54$$

7.3.2 Strut-and-Tie Method

STM-CAP is based on the strut-and-tie method. The pier cap is modeled in STM-CAP to determine the optimized utilization ratios. The full input and output fields using STM-CAP can be seen at Bridge 2 from Appendix A. **Figure 7-10** shows the screenshot of STM-CAP utilization ratios output screen.

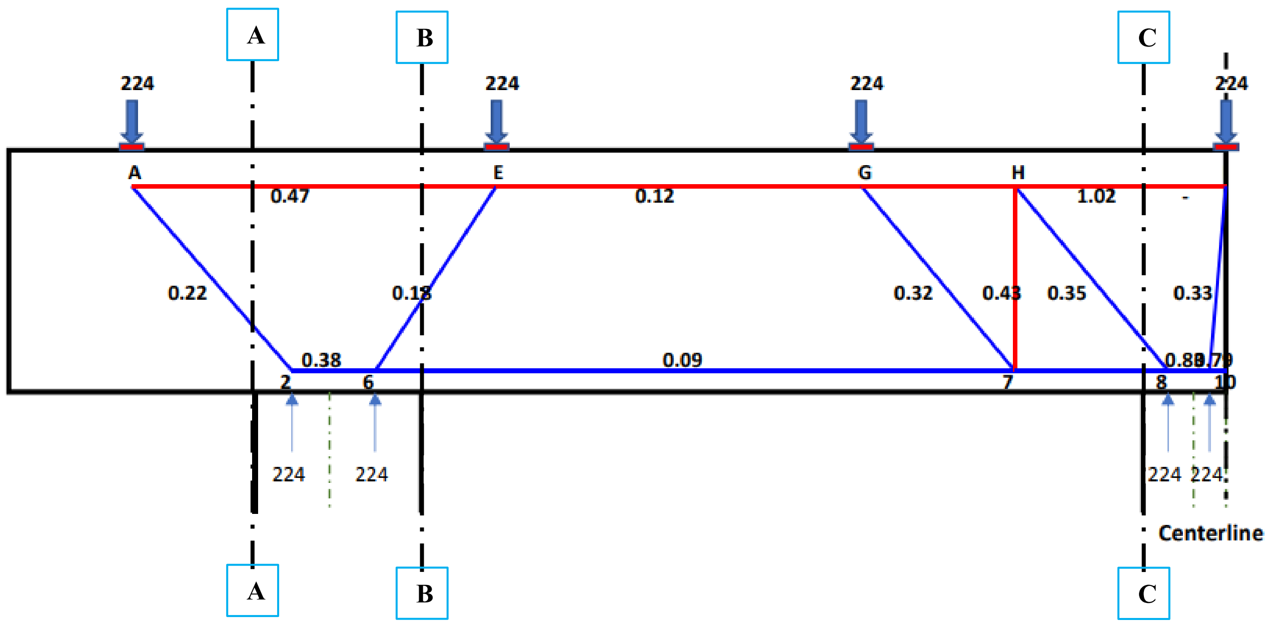


Figure 7-10: Utilization ratios calculated from STM-CAP for Bridge 2

7.3.3 Comparisons

The utilization ratio from the sectional method and STM-CAP are compared in **Table 7-4**. The shear utilization ratios from the sectional method is compared with the utilization ratios of critical inclined or vertical members of STM-CAP.

Table 7-4: Comparison of shear URs for the STM and the sectional method for Bridge 2

Shear Utilization Ratios Comparison at Different Sections				
Sections	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
A-A	1.0	0.54	0.22	2.45
B-B	0.76	0.54	0.18	3.0
C-C	1.91	0.54	0.43	1.26

7.3.4 Discussions

Table 7-4 shows the utilization ratios determined from the sectional method and STM-CAP. The sectional method predicts a constant utilization ratio of 0.54 for every region, both deep and slender. It does not consider effect due to load position but only upon the shear force and section properties for shear capacity. STM considers the effect due to the load positioning and many other factors; for deeper regions where load is near to a support, the concrete is strong in shear and vice versa.

The sectional method predicts 1.26 to 3.0 times lower capacity for sections with *a/d* ratios ranges of 1.91 to 0.76. The deeper the region the more highly conservative the sectional method. STM-CAP predicts higher and more accurate shear capacities for beams with shear span-to-depth ratios less than 3.0.

7.4 Bridge 3

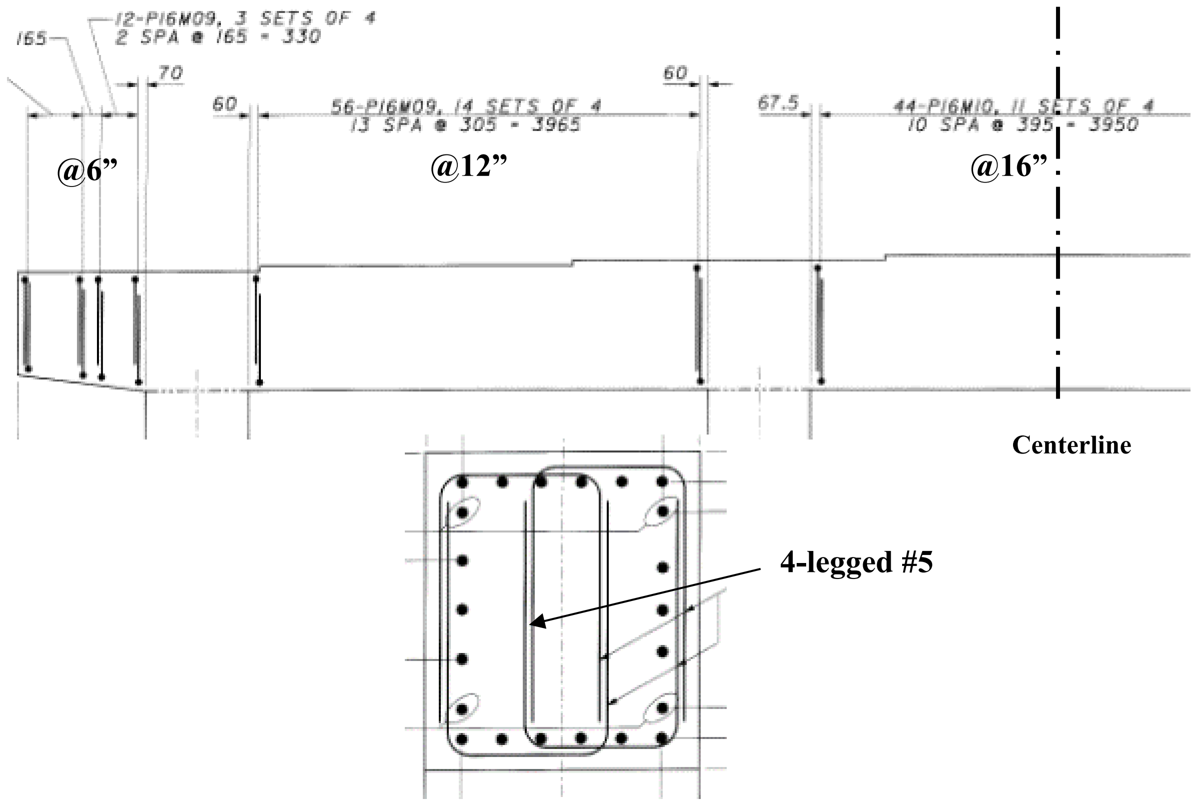
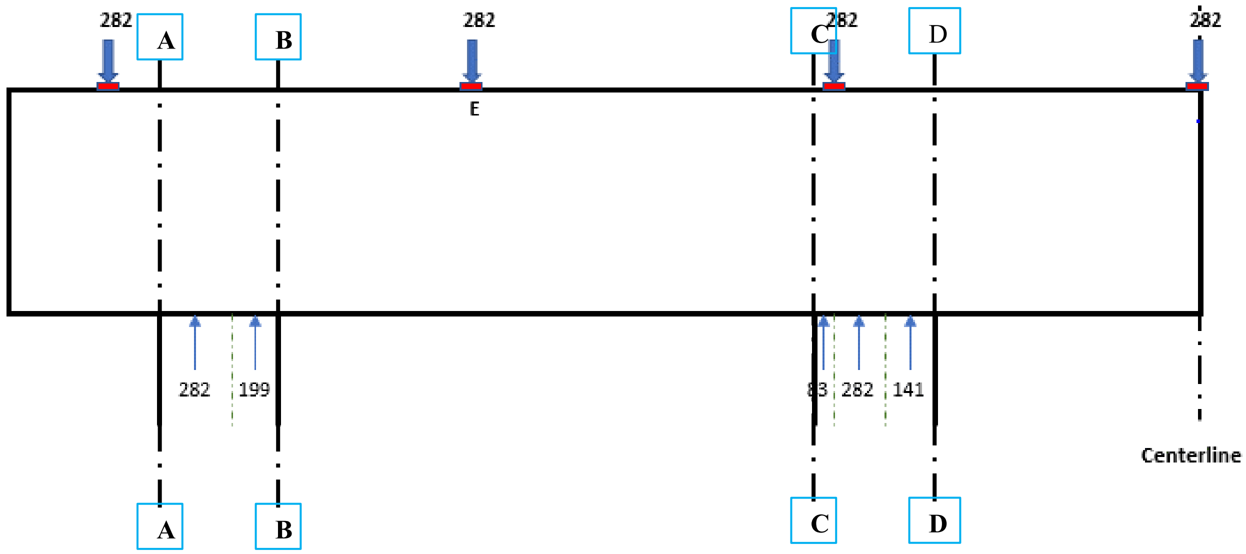


Figure 7-11: Bridge 3 details

The engineering drawing for Bridge 3 is shown in **Figure 7-11**. Bridge 3 corresponds to the 'Bridge Pier Cap 3' from Appendix A. The shear reinforcement details are shown in **Figure 7-11**. Each stirrup is four-legged, #5 bars and spaced as shown in **Figure 7-11**.

7.4.1 Sectional Method

The utilization ratio for shear using the sectional method is determined by dividing the shear force with shear capacity. The shear force at each span can be determined using any software. STM-CAP calculates the reaction at each support and will be used to determine the shear force in the pier cap. The shear check is done at each face of the column. There are four critical sections (e.g., A-A, B-B, etc.), as shown in **Figure 7-12.**



Shear Force at

A-A= 282 kip B-B= 199 kip C-C= 83 kip D-D= 141 kip

Figure 7-12: Shear forces at critical sections for Bridge 3

The shear force at Section A-A is determined from the panel shear using the load and the reaction and is found to be 282 kips. The shear force at each section is determined at each critical section as shown in **Figure 7-12.**

Using the AASHTO LRFD, the shear capacity at each critical section is determined as follows:

<u>Shear capacity</u>

Section A-A

At Section A-A, the critical section is assumed to be at the outer face of the pier 1.

The shear capacity at any section cannot exceed $V_n = 0.25f'_c b_v d_v$

Therefore, V_n is calculated as minimum of $(0.25f'_c b_v d_v, V_c + V_s)$

where,

$b_v = 36$ in

d_v = max (0.9d, 0.72h) = max (0.9*37.9in, 0.72*42in) =34.1 in

A_v = 4 legged #5 = (4*0.31) in^2 = 1.24 in^2

spacing (s) = 6 in

β = factor indicating the ability of reinforcement measured in a direction parallel to the rebar

θ = angle of inclination of diagonal compressive stresses

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 to determine the value as

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\frac{36*6}{60}$$

$1.24 \geq 0.227 in^2$ satisfied.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}} * b_v * d_v = 0.0316*2*\sqrt{4}*36*34.1 = 155\text{kips}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*34.1*\cot(45)}{6} = 423\text{kips}$$

And, V_n = min of ($0.25 f_c^{'} b_v d_v$, $V_c + V_s$)

= min of (0.25*4*36*34.1, 578)

= min of (1228, 578)

= 578 kips

Hence, the factored shear capacity, $V_u = \Phi * V_n$ = 0.9*578 kips = 520 kips

$$\text{Utilization ratio at Section A-A} = \frac{\text{shear force at A-A } (V_{A\text{-}A})}{\text{shear capacity at A-A } (V_u)} = \frac{282\text{kips}}{520\text{kips}} = 0.54$$

Section B-B

At Section B-B, the shear is checked at the right face of pier 1. The section properties of Section B-B are like those of Section A-A. The only difference is the spacing of stirrups in Section B-B is @ twelve-inch center-to-center.

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24 ? 0.0316 * \sqrt{4}\,\frac{36*12}{60}$$

$1.24 \geq 0.45\text{in}^2$ satisfied, OK.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}} * b_v * d_v = 0.0316 * 2 * \sqrt{4} * 36 * 34.1 = 155\text{kips}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*34.1*\cot(45)}{12} = 211\text{kips}$$

And, $V_n = \text{min of} \left(0.25 f_c^{'} b_v d_v ,\ V_c + V_s \right)$

= min of (0.25*4*36*34.1, 366)

= min of (1228, 366)

= 366 kips

Hence, the factored shear capacity, $V_u = \Phi * V_n$ = 0.9*366 kips = 329 kips

Utilization ratio at Section B-B $= \dfrac{\text{shear force at B-B } (V_{B-B})}{\text{shear capacity at B-B } (V_u)} = \dfrac{199\text{kips}}{329\text{kips}} = 0.60$

Section C-C

At Section C-C, the shear is checked at the left face of pier 2. The section properties of Section C-C are the same as Section B-B. Both have four-legged, #5 bars stirrups spaced @ twelve-inch center-to-center. The shear capacity is the same as Section B-B.

The factored shear capacity, $V_u = \Phi * V_n = 0.9*366$ kips = 329 kips

Utilization ratio at Section C-C $= \dfrac{\text{shear force at C-C } (V_{C-C})}{\text{shear capacity at C-C } (V_u)} = \dfrac{83\text{kips}}{329\text{kips}} = 0.25$

Section D-D

At Section D-D, the shear is checked at the right face of pier 2. The section properties of Section D-D are like those of Section A-A. The only difference is the spacing of stirrups in Section D-D @ eighteen-inch center-to-center.

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\frac{36*18}{60}$$

$1.24 \geq 0.68\text{in}^2$ satisfied, OK.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c'} * b_v * d_v = 0.0316*2*\sqrt{4}*36*34.1 = 155\text{kips}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*34.1*\cot(45)}{18} = 141\text{kips}$$

And, $V_n = \text{min of} (0.25 f_c' b_v d_v , V_c + V_s)$

= min of (0.25*4*36*34.1, 296)

= min of (1228, 296)

= 296 kips

Hence, the factored shear capacity, $V_u = \Phi * V_n = 0.9*296$ kips = 266 kips

Utilization ratio at Section D-D $= \dfrac{\text{shear force at D-D } (V_{D-D})}{\text{shear capacity at D-D } (V_u)} = \dfrac{141\text{kips}}{266\text{kips}} = 0.53$

7.4.2 Strut-and-Tie Method

STM-CAP is based on the strut-and-tie method. The pier cap is modeled in STM-CAP to determine the optimized utilization ratios. The full input and output fields using STM-CAP can be seen at Bridge 3 from Appendix A. **Figure 7-13** shows the screenshot of STM-CAP utilization ratios output screen.

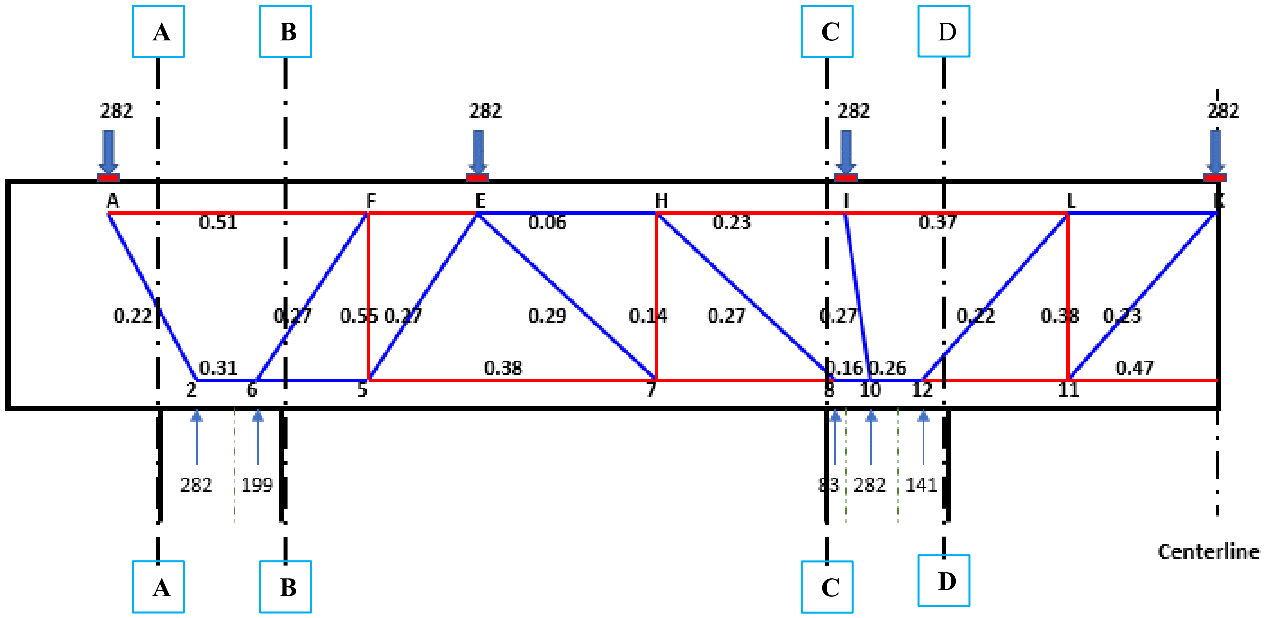


Figure 7-13: Utilization ratios calculated from STM-CAP for Bridge 3

7.4.3 Comparisons

The utilization ratio from the sectional method and STM-CAP are compared in **Table 7-5**. The shear utilization ratios from the sectional method is compared with the utilization ratios of critical inclined or vertical members of STM-CAP.

Table 7-5: Comparison of shear URs for the STM and the sectional method for Bridge 3

Shear Utilization Ratios Comparison at Different Sections				
Sections	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
A-A	0.69	0.54	0.22	2.45
B-B	1.72	0.60	0.55	1.09
C-C	2.80	0.25	0.3	0.83
D-D	2.29	0.53	0.38	1.39

7.4.4 Discussions

The utilization ratios determined from the sectional method and STM are compared in **Table 7-5**. Most of the regions are deep. The sectional method is underestimating the shear capacity of deep regions as expected. At Section C-C, which is a slender region (a/d ratio = 2.80 > 2.0), the STM method is slightly underestimating the capacity prediction (0.83 times capacity from the sectional method) as expected. At Section D-D, which is slender (a/d ratio = 2.29 > 2.0), the sectional method is conservative, predicting lower capacity by a factor of 1.39. Therefore, it can be concluded that STM-CAP predicts higher and more accurate shear capacities for beams with shear span-to-depth ratios less than 2.3.

7.5 Bridge 4

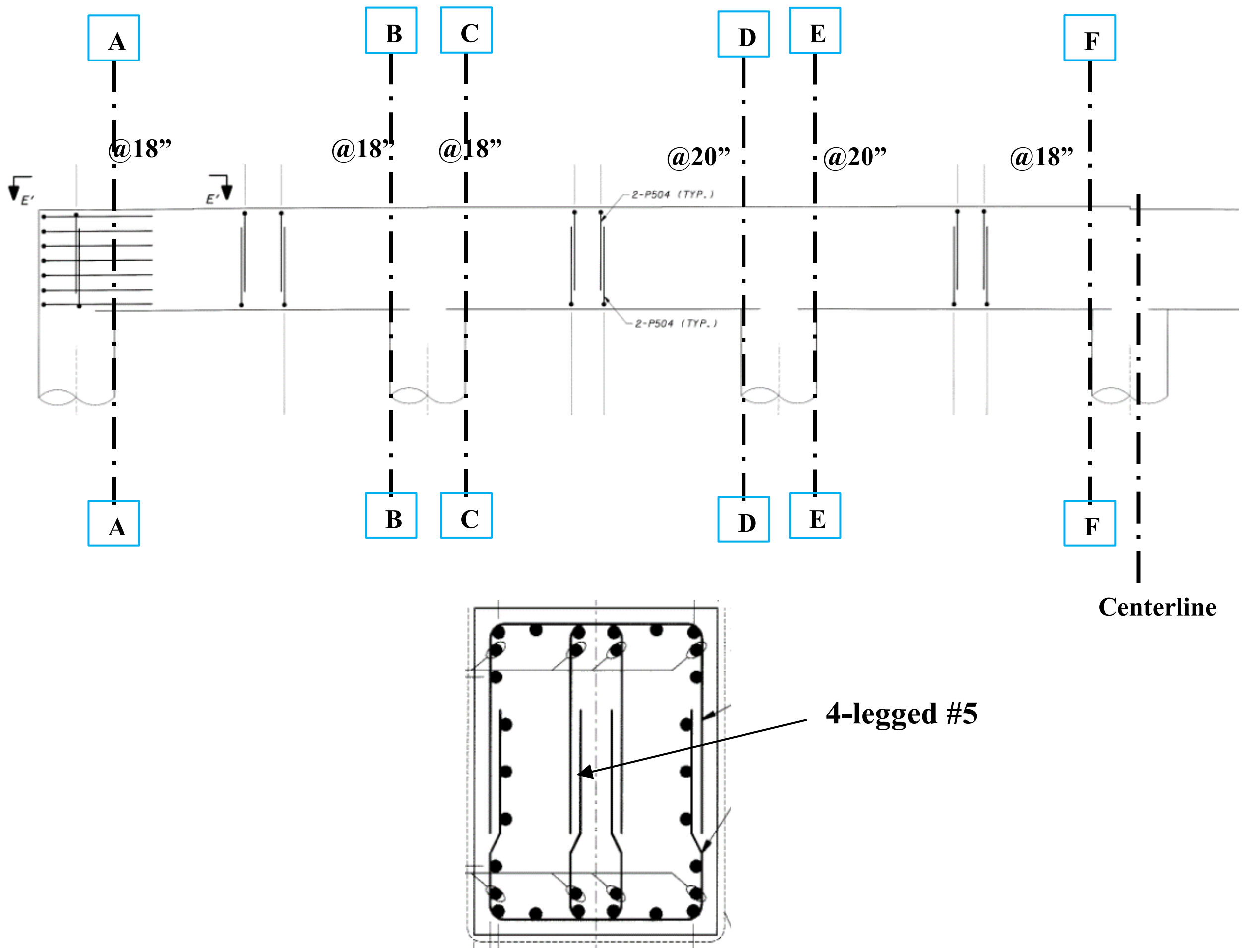
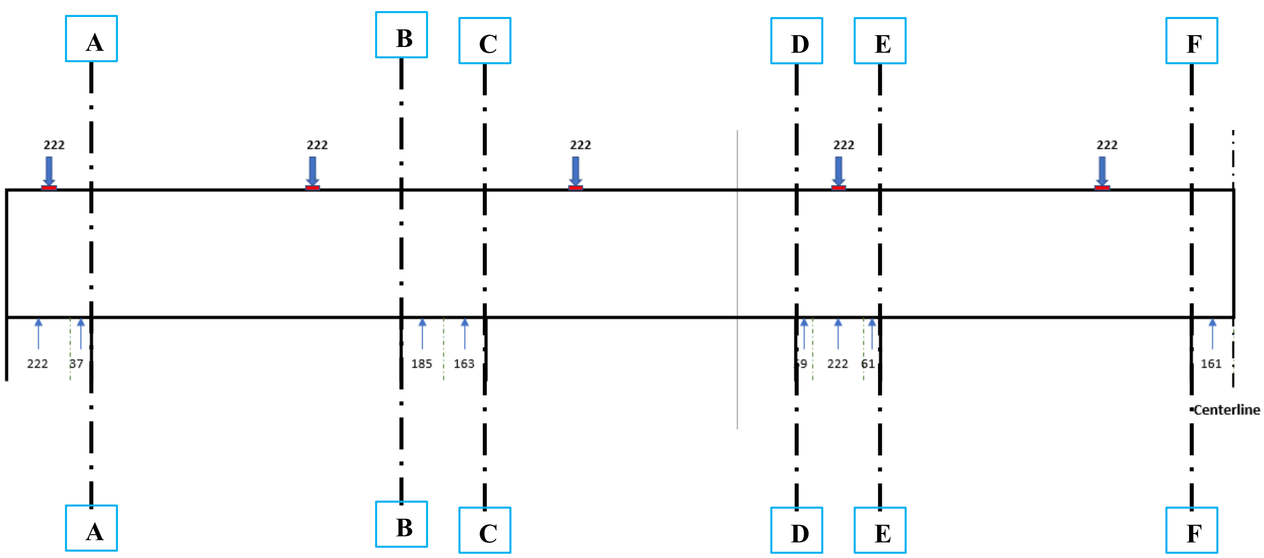


Figure 7-14: Bridge 4 details

The engineering drawing for Bridge 4 is shown in **Figure 7-14**. Bridge 4 corresponds to the 'Bridge Pier Cap 5' from Appendix A. The shear reinforcement details are shown in **Figure 7-14**. Each stirrup is four-legged, #5 bars and are as spaced as shown in **Figure 7-14**.

7.5.1 Sectional Method

The utilization ratio for shear using the sectional method is determined by dividing the shear force with shear capacity. The shear force at each span can be determined using any software. STM-CAP calculates the reaction at each support and will be used to determine the shear force in the pier cap. The shear check is done at each face of the column. There are six critical sections (e.g., A-A, B-B, etc.), as shown in **Figure 7-14** and **Figure 7-15.**



Shear Force at

A-A= 37 kip B-B= 185 kip C-C= 163 kip D-D= 59 kip E-E= 61 kip F-F= 161 kip

Figure 7-15: Shear forces at critical sections for Bridge 4

The shear force at Section A-A is determined from the panel shear using the load and the reaction and is 37 kips. The shear force at each section is determined at each critical section as shown in **Figure 7-15.**

Using the AASHTO LRFD, the shear capacity at each critical section is determined as follows:

Shear capacity

Section A-A

At Section A-A, the critical section is assumed to be at the right face of pier 1.

The shear capacity at any section cannot exceed $V_n = 0.25f'_c b_v d_v$

Therefore, V_n is calculated as min of ($0.25f'_c b_v d_v$, $V_c + V_s$)

where,

b_v = 36 in

d_v = max (0.9d, 0.72h) = max (0.9*31.8, 0.72*36in) =28.6 in

A_v = legged #5 = (4*0.31) in^2 = 1.24 in^2

s = 18 in

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f'_c}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\frac{36*18}{60}$$

$1.24 \geq 0.68in^2$ satisfied, OK.

Hence, the value of β and θ can be taken as 2.0 and 45° respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f'_c}*b_v*d_v = 0.0316*2*\sqrt{4}*36*28.6 = 130kips$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*28.6*\cot(45)}{18} = 118kips$$

And, $V_n = \text{min of}\,(0.25f'_c b_v d_v, V_c + V_s)$

$= \text{min of}\,(0.25*4*36*28.6, 248)$

$= \text{min of}\,(1030, 248)$

$= 248$ kips

Hence, the factored shear capacity, $V_u = \Phi*V_n = 0.9*248\text{kips} = 223$ kips

$$\text{Utilization ratio at Section A-A} = \frac{\text{shear force at A-A } (V_{A\text{-}A})}{\text{shear capacity at A-A } (V_u)} = \frac{37\text{kips}}{223\text{kips}} = 0.17$$

Section B-B

At Section B-B, the shear is checked at the left face of pier 2. The section properties of Section B-B are the same as Section A-A. Section B-B has the same stirrup spacing of eighteen inches.

Hence, the factored shear capacity, $V_u = \Phi*V_n = 0.9*248\text{kips} = 223$ kips

$$\text{Utilization ratio at Section B-B} = \frac{\text{shear force at B-B } (V_{B-B})}{\text{shear capacity at B-B } (V_u)} = \frac{185\text{kips}}{223\text{kips}} = 0.83$$

Section C-C

At Section C-C, the shear is checked at the right face of pier 2. The section properties of Section C-C are the same as Section A-A. Section C-C has the same stirrup spacing of eighteen inches.

Hence, the factored shear capacity, $V_u = \Phi*V_n = 0.9*248\text{kips} = 223$ kips

$$\text{Utilization ratio at Section C-C} = \frac{\text{shear force at C-C } (V_{C-C})}{\text{shear capacity at C-C } (V_u)} = \frac{163\text{kips}}{223\text{kips}} = 0.73$$

Section D-D

At Section D-D, the shear is checked at the left face of pier 3. The section properties of Section D-D are like those of Section A-A. The only difference is the spacing of stirrups in Section D-D @ twenty-inches center-to-center.

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24?0.0316*\sqrt{4}\frac{36*20}{60}$$

$1.24 \geq 0.75in^2$ satisfied, OK.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as;

$$V_c = 0.0316\beta\sqrt{f_c^{'}}*b_v*d_v = 0.0316*2*\sqrt{4}*36*28.6 = 130kips$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*28.6*\cot(45)}{20} = 106kips$$

And, $V_n = \text{min of}$ ($0.25f_c^{'}b_v d_v$, $V_c + V_s$)

= min of (0.25*4*36*28.6, 236)

= min of (1030, 236)

= 236 kips

Hence, the factored shear capacity, $V_u = \Phi * V_n$ = 0.9*236kips = 212 kips

$$\text{Utilization ratio at Section D-D} = \frac{\text{shear force at D-D } (V_{D-D})}{\text{shear capacity at D-D } (V_u)} = \frac{59kips}{212kips} = 0.28$$

Section E-E

At Section E-E, the shear is checked at the right face of pier 3. The section properties of Section E-E are the same as of Section D-D. Section E-E has the same stirrup spacing of twenty inches.

Hence, the factored shear capacity, $V_u = \Phi^* V_n = 0.9^*236\text{kips} = 212$ kips

Utilization ratio at Section E-E $= \dfrac{\text{shear force at E} - \text{E } (V_{E-E})}{\text{shear capacity at E} - \text{E } (V_u)} = \dfrac{61\text{kips}}{212\text{kips}} = 0.29$

Section F-F

At Section F-F, the shear is checked at the left face of pier 4. The section properties of Section F-F are the same as of Section A-A. Section F-F has the same stirrup spacing of eighteen inches.

Hence, the factored shear capacity, $V_u = \Phi^* V_n = 0.9^*248\text{kips} = 223$ kips

Utilization ratio at Section F-F $= \dfrac{\text{shear force at F} - \text{F } (V_{B-B})}{\text{shear capacity at F} - \text{F } (V_u)} = \dfrac{161\text{kips}}{223\text{kips}} = 0.72$

7.5.2 Strut-and-Tie Method

STM-CAP is based on the strut-and-tie method. The pier cap is modeled in STM-CAP to determine the optimized utilization ratios. The full input and output fields using STM-CAP can be seen at Bridge 5 from Appendix A. **Figure 7-16** shows the screenshot of STM-CAP utilization ratios output screen.

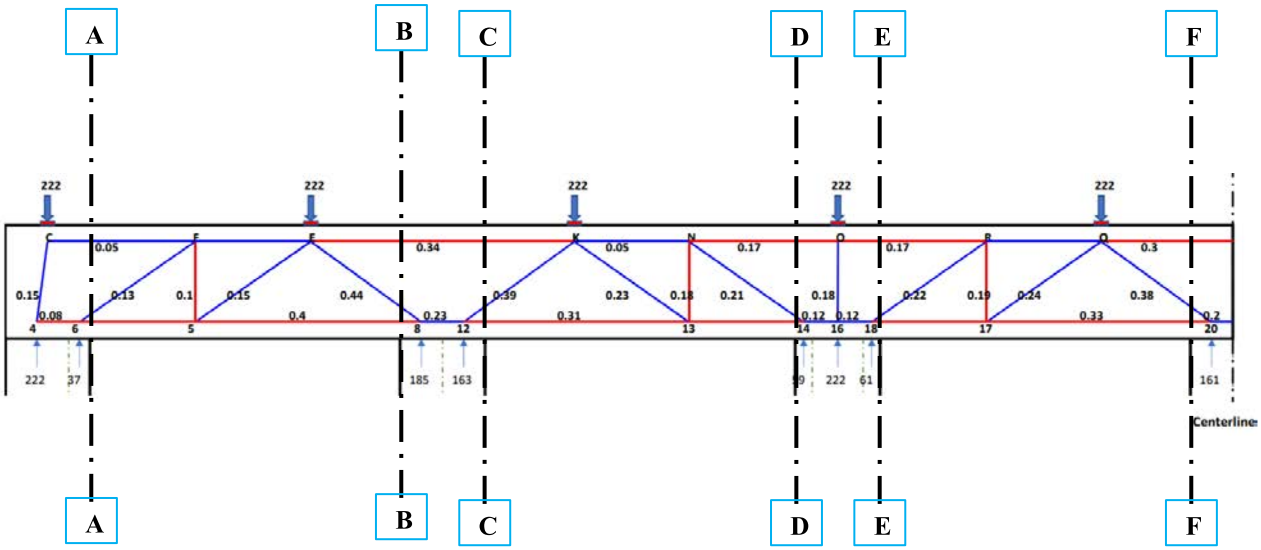


Figure 7-16: Utilization ratios calculated from STM-CAP for Bridge 4

7.5.3 Comparisons

The utilization ratio from the sectional method and the STM-CAP are compared in **Table 7-6**. The shear utilization ratios from the sectional method is compared with the utilization ratios of critical inclined or vertical members of STM-CAP.

Table 7-6: Comparison of shear URs for the STM and the sectional method for Bridge 4

Shear Utilization Ratios Comparison at Different Sections				
Sections	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
A-A	3.03	0.17	0.15	1.13
B-B	1.44	0.83	0.44	1.89
C-C	1.41	0.73	0.39	1.87
D-D	3.00	0.28	0.23	1.21
E-E	3.01	0.29	0.24	1.21
F-F	1.44	0.72	0.38	1.89

7.5.4 Discussions

The utilization ratios determined from the sectional method and the STM are compared in **Table 7-6**.

For every region considered, the sectional method predicted conservative results. A conservative result using the sectional method was expected for B-B, C-C, and F-F regions (*a/d* ratio < 2.0). The sectional method predicted nearly half of the capacity for B-B, C-C, and F-F regions with approximately *a/d* ratio of 1.4 than the STM-CAP. For slender regions, higher capacity is expected from the sectional method. However, slender regions with *a/d* ratios of 3.0, the sectional method is more conservative than the STM. STM-CAP predicts higher capacity for regions with *a/d* ratio of 3.0. STM-CAP predicts higher and more accurate shear capacities for beam with shear span-to-depth ratios less than 3.0.

7.6 Bridge 5

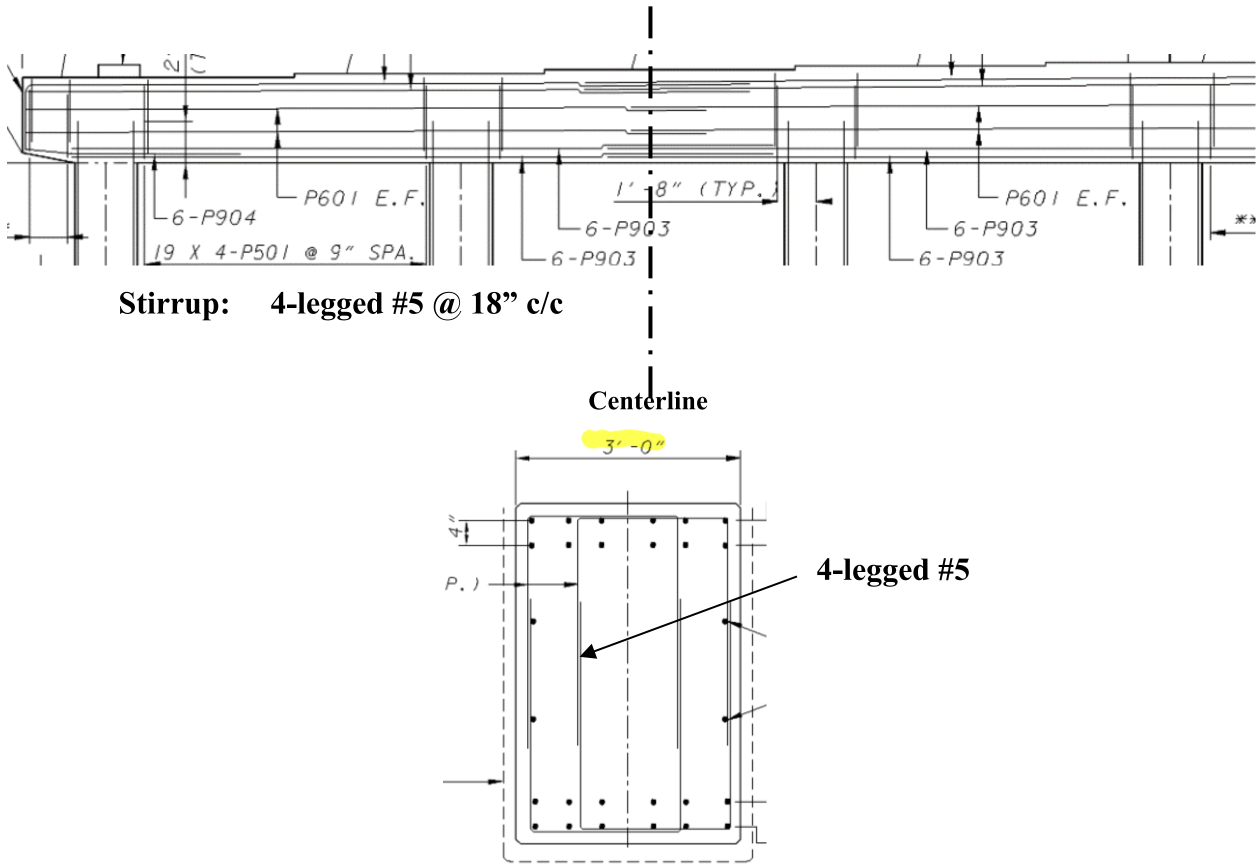
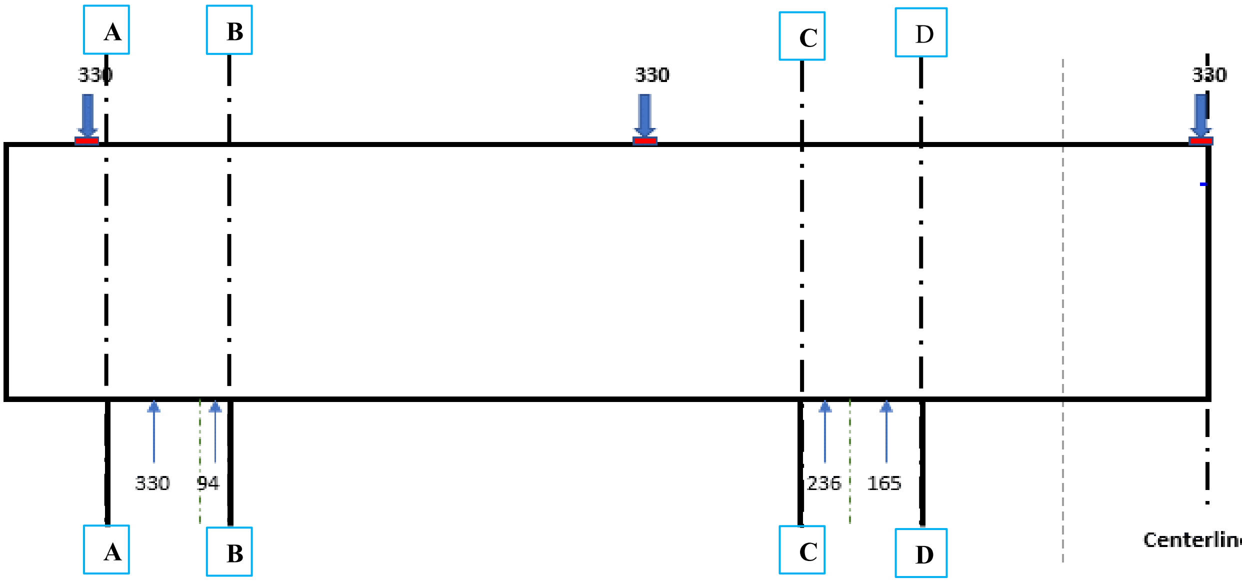


Figure 7-17: Bridge 5 details

The engineering drawing for Bridge 5 is shown in **Figure 7-17**. Bridge 5 corresponds to the 'Bridge Pier Cap 7' from Appendix A. The shear reinforcement details are shown in **Figure 7-17**. Each stirrup is four-legged, #5 bars and are spaced eighteen-inches c/c throughout the pier cap section as shown in **Figure 7-17**.

7.6.1 Sectional Method

The utilization ratio for shear using the sectional method is determined by dividing the shear force with shear capacity. The shear force at each span can be determined using any software. STM-CAP calculates the reaction at each support and will be used to determine the shear force in the pier cap. The shear check is done at each face of the column. There are four critical sections (e.g., A-A, B-B, etc.), as shown in **Figure 7-18.**



Shear Force at

A-A= 330 kip B-B= 94 kip C-C= 236 kip D-D= 165 kip

Figure 7-18: Shear forces at critical sections for Bridge 5

The shear force at Section A-A is determined from the panel shear using the load and the reaction and is 330 kips. The shear force at each section is determined at each critical section as shown in **Figure 7-18.**

Using the AASHTO LRFD, the shear capacity at each critical section is determined as follows:

<u>Shear capacity</u>

Section A-A

At Section A-A, the critical section is assumed to be at the outer face of pier 1.

The shear capacity at any section cannot exceed $V_n = 0.25f'_c b_v d_v$

Therefore, V_n is calculated as minimum of $(0.25f'_c b_v d_v, V_c + V_s)$

where,

b_v = 36 in

d_v = max (0.9d, 0.72h) = max (0.9*43in, 0.72*48in) =38.7 in

A_v = 4 legged #5 = (4*0.31) in^2 = 1.24 in^2

stirrup spacing (s) = 18 in

β = factor indicating the ability of reinforcement measured in a direction parallel to the rebar

θ = angle of inclination of diagonal compressive stresses

Checking for the minimum transverse according to Section 5.8.2.5of AASHTO LRFD 2014 as,

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y}$$

$$1.24 ? 0.0316*\sqrt{4}\frac{36*18}{60}$$

$1.24 \geq 0.68 in^2$ satisfied.

Hence, the value of β and θ can be taken as 2.0 and 45º respectively.

For non-prestressed beams the shear capacities are calculated as:

$$V_c = 0.0316\beta\sqrt{f_c^{'}}*b_v*d_v = 0.0316*2*\sqrt{4}*36*38.7 = 176kips$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{(4*0.31)*60*38.7*\cot(45)}{18} = 160kips$$

And, $V_n = \text{min of} (0.25f_c^{'}b_v d_v, V_c + V_s)$

= min of (0.25*4*36*38.7, 336)

= min of (1393, 336)

= 336 kips

Hence, the factored shear capacity, $V_u = \Phi * V_n = 0.9*336$ kips = 302 kips

$$\text{Utilization ratio at Section A-A} = \frac{\text{shear force at A-A } (V_{A\text{-}A})}{\text{shear capacity at A-A } (V_u)} = \frac{330\text{kips}}{302\text{kips}} = 1.09$$

Section B-B

At Section B-B, the shear is checked at the right face of pier 1. The section properties of Section B-B are the same as Section A-A. Section B-B has the same stirrup spacing of eighteen inches as A-A with a factored shear capacity of 302 kips.

$$\text{Utilization ratio at Section B-B} = \frac{\text{shear force at B-B } (V_{B-B})}{\text{shear capacity at B-B } (V_u)} = \frac{94\text{kips}}{302\text{kips}} = 0.31$$

Section C-C

At Section C-C, the shear is checked at the left face of pier 2. The section properties of Section C-C are almost like those of Section A-A. Both sections have four-legged, #5 bars, spaced @ eighteen-inches center-to-center. The shear capacities at Section C-C are the same as the shear capacities determined at Section A-A with a factored shear capacity of 302 kips.

$$\text{Utilization ratio at Section C-C} = \frac{\text{shear force at C-C } (V_{C-C})}{\text{shear capacity at C-C } (V_u)} = \frac{236\text{kips}}{302\text{kips}} = 0.78$$

Section D-D

At Section D-D, the shear is checked at the right face of pier 2. The section properties of Section D-D are the same as Section A-A. The Section D-D has the same stirrup spacing of eighteen inches.

Hence, the factored shear capacity, $V_u = \Phi * V_n = 0.9*336$ kips = 302 kips

$$\text{Utilization ratio at Section D-D} = \frac{\text{shear force at D} - \text{D } (V_{D-D})}{\text{shear capacity at D} - \text{D } (V_u)} = \frac{165\text{kips}}{302\text{kips}} = 0.55$$

7.6.2 Strut-and-Tie Method

STM-CAP is based on the strut-and-tie method. The pier cap is modeled in STM-CAP to determine the optimized utilization ratios. The full input and output fields using STM-CAP can be seen at Bridge 7 from Appendix A. **Figure 7-19** shows the screenshot of STM-CAP utilization ratios output screen.

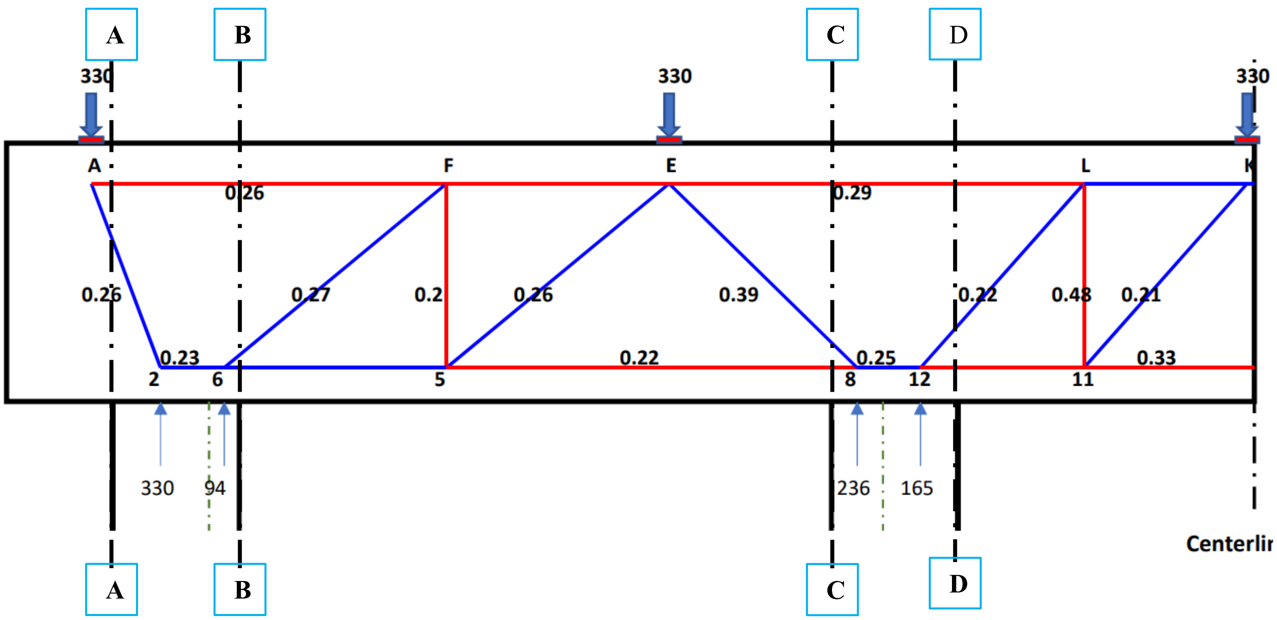


Figure 7-19: Utilization ratios calculated from STM-CAP for Bridge 5

7.6.3 Comparisons

The utilization ratio from the sectional method and the STM-CAP are compared in **Table 7-7**. The shear utilization ratios from the sectional method is compared with the utilization ratios of critical inclined or vertical members of STM-CAP.

Table 7-7: Comparison of shear URs for the STM and the sectional method for Bridge 5

Shear Utilization Ratios Comparison at Different Sections				
Sections	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM}-\text{CAP (UR)}}$
A-A	0.46	1.09	0.26	4.19
B-B	2.92	0.31	0.27	1.15
C-C	1.23	0.78	0.39	2.0
D-D	2.15	0.55	0.48	1.15

7.6.4 Discussions

The utilization ratios determined from the sectional method and STM are compared in **Table 7-7**. For every region considered, the sectional method predicts more conservative results. A conservative result using the sectional method is expected for A-A and C-C regions (*a/d* ratio < 2.0). The sectional method predicts nearly a quarter of the capacity for Section A-A and half the capacity for Section C-C than the STM-CAP results. For slender regions, higher capacities are expected from the sectional method. However, for slender beam with *a/d* ratio of 2.92 (B-B), the sectional method is more conservative that the STM. STM-CAP predicts higher and more accurate shear capacities for beams with shear span-to-depth ratios less than 3.0.

7.7 Conclusions

The optimized utilization ratios determined from the sectional method and STM-CAP for all sections from the bridges analyzed are summarized and tabulated in **Table 7-8**. The tabulated data is plotted along with the utilization ratios for nonlinear FEM obtained from Chapter 6 in **Figure 7-20.**

Table 7-8: Sectional method vs STM-CAP optimized utilization ratios (UR)

Shear Utilization Ratio Comparison				
Bridge	*a/d* ratios	Sectional Method (UR)	STM-CAP (UR)	$\frac{\text{Sectional Method (UR)}}{\text{STM} - \text{CAP (UR)}}$
Bridge 1	0.71	0.43	0.29	1.48
	1.30	0.60	0.54	1.11
	0.50	0.48	0.31	1.55
Bridge 2	1.00	0.54	0.22	2.45
	0.76	0.54	0.18	3.00
	1.91	0.54	0.43	1.26
Bridge 3	0.69	0.54	0.22	2.45
	1.72	0.60	0.55	1.09
	2.80	0.25	0.30	0.83
	2.29	0.53	0.38	1.39
Bridge 4	3.03	0.17	0.15	1.13
	1.44	0.83	0.44	1.89
	1.41	0.73	0.39	1.87
	3.00	0.28	0.23	1.22
	3.01	0.29	0.24	1.21
	1.44	0.72	0.38	1.89
Bridge 5	0.46	1.09	0.26	4.19
	2.82	0.31	0.27	1.15
	1.23	0.78	0.39	2.00
	2.15	0.55	0.48	1.15

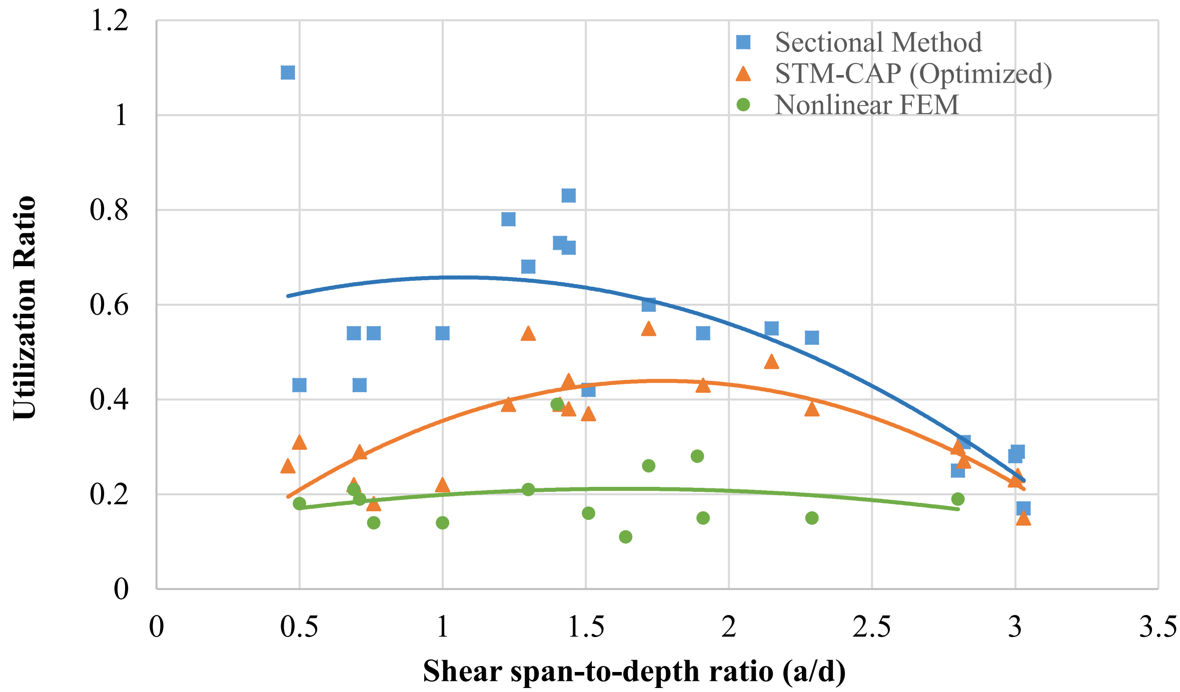


Figure 7-20: Comparison of results from analysis methods

Figure 7-20 shows the utilization ratios predicted using the sectional method and STM-CAP for twenty regions with various shear span-to-depth ratios ranging from 0.5 to 3.0. Most regions in all five pier caps fall within *a/d* ratio of 2.0 or lower. For almost all regions of all the pier caps, STM-CAP predicted lower utilization ratios than the sectional method and thus higher shear capacities for the same loads. For lower *a/d* ratios (*a/d*) of 0.50, STM-CAP predicted two to three times higher shear capacities. As the *a/d* ratio increased, the prediction by STM-CAP and the sectional method converged. The shear capacity prediction by STM-CAP was still higher than the sectional method as the *a/d* ratio reached a value 3.0. These STM capacity predictions are still conservative when compared with nonlinear FEM (from Chapter 6) because the STM is based on lower-bound theorem. The STM-CAP provided a good compromise between complexity and accuracy as compared to the sectional method and nonlinear FEM. While it is as simple as the sectional method, it provides an accuracy closer to the finite element method. Thus, in all cases, higher and more accurate shear capacity predictions were obtained with the simplified approach from STM-CAP.

Chapter 8 Updated AASHTO Provisions

The 8th edition of the AASHTO LRFD code was released during this study. While the results presented in this document are based on the 7th edition of the code, the STM-CAP calculation procedures are fully updated with the provisions contained in the 8th edition. The bridge database discussed in this study was re-analyzed using the latest code and the results are provided in Appendix B. While it is not the scope of this study, the results from both versions of the code were compared.

It was found that the new horizontal strut formulations result in minor capacity changes. In the 7^{th} edition, the capacity of horizontal struts is taken as the minimum capacity of either reinforced struts or the nodal zones, while in the 8^{th} edition, the horizontal strut capacities are equal to the sum of these two capacities. Thus, higher capacities are obtained from the horizontal struts where the node capacities were governing in the 7th edition. The new vertical tie formulations (i.e., Section 5.8.2.2 or Figure C5.8.2.2-2), on the other hand, results in a decrease in the tie capacities due to the new provision requiring 25° reduction from both ends of the shear spans (thus intersecting a smaller number of ties; compare Appendix A and B). The new inclined strut formulations result in higher capacities in most of the cases (compare Appendix A and B) under the same model conditions (same strut angles with no vertical ties). In addition, the new formulations (i.e., Section 5.8.2.5.3a) significantly reduce the strut capacities if the beam does not contain the minimum crack control reinforcement (compare Appendix A and B). It was found that the new horizontal tie capacities are the same as those from the 7th edition. The final version of the STM-CAP incorporates the updated formulations and will account for these influences.

8.1 Updated Sections

Section 5.8.2.1: "The STM should be considered for the design of deep footing and pile caps or other situations in which the distance between the centers of applied load and the supporting reactions is less than about twice the member depth."

From Section 5.8.2.3, the factored resistance, P_r of ties, struts and nodes are

$P_r = \phi P_n$ ………….. AASHTO LRFD 2017, Equation 5.8.2.3-1

where:

P_n = nominal resistance of strut or tie

ϕ = resistance factor for tension or compression based on Section 5.5.4.2 of AASHTO 2017

=0.9 for tension-controlled concrete sections

=0.7 for compression-controlled concrete sections

8.2 Strength of the Node Face

The nominal resistance of a node face as of Section 5.8.2.5.1 shall be taken as:

$P_n = f_{cu} A_{cn}$ ………….. AASHTO LRFD 2017, Equation 5.8.2.5.1-1

where:

P_n = nominal resistance of a node face (kips)

f_{cu} = limiting compressive stress at the node face (ksi) as explained below

A_{cs} = effective cross-sectional area of strut (in.^{2}) as shown in **Figure 8-1**.

The effective cross-sectional area of node face depends upon the type of node, anchorage condition, and size of the bearing. It can be calculated as the width of the node face times the thickness of the pier cap depending upon the type of node.

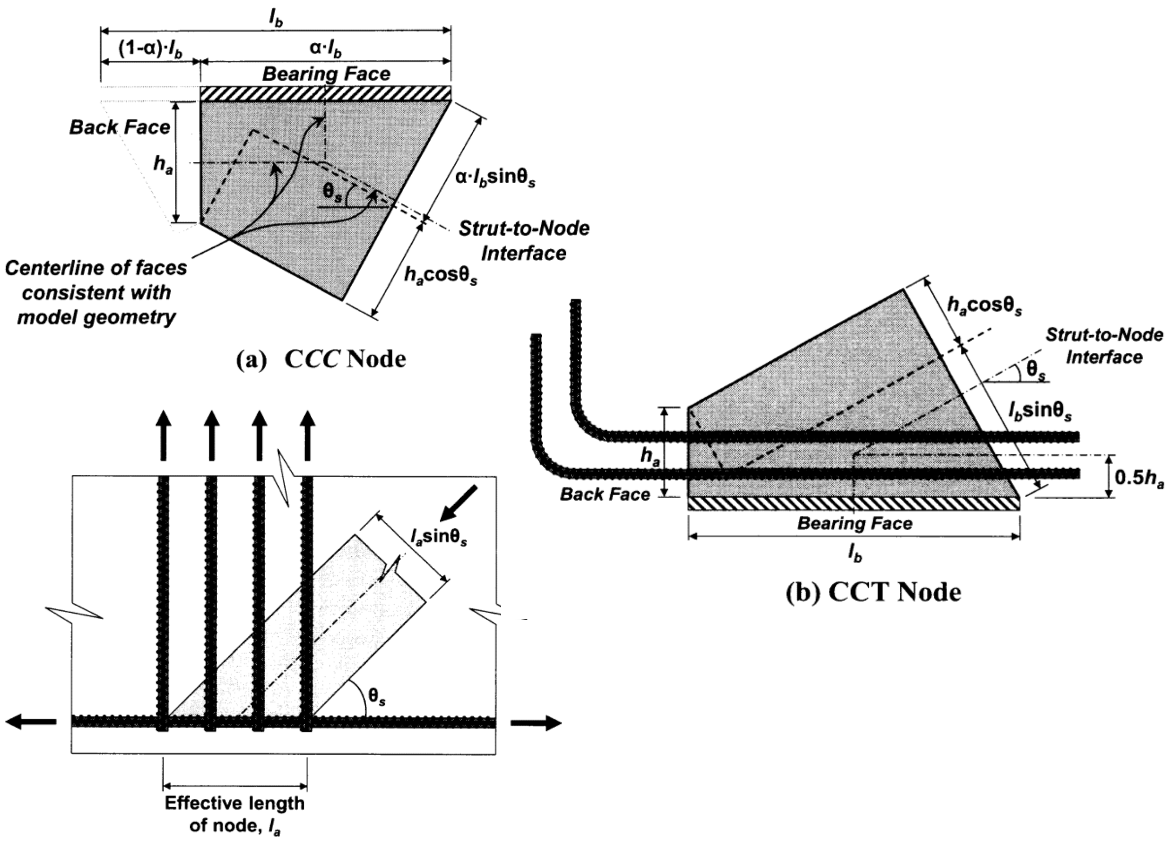


Figure 8-1: Nodal geometry and width of the node faces (AASHTO Figure 5.8.2.2-1)

Limiting compressive stress (Section 5.8.2.5.3a), f_{cu} is calculated as:

$f_{cu} = mvf_c^{'}$ ………….. AASHTO LRFD 2017, Equation 5.8.2.5.3a-1

where:

f'c = compressive strength of concrete for use in design (ksi)

m = confinement modification factor taken as $\sqrt{A_2 / A_1}$ but not more than 2.0

v = concrete efficiency factor

= 0.45, for structure that do not contain crack control reinforcement as in Section 2.5.4

= as explained in **Figure 8-2.**

	Node Type		
Face	CCC	CCT	CTT
Bearing Face	0.85	0.70	
Back Face	0.85	0.70	
Strut-to-Node Interface	$0.85 - \frac{f'_c}{20 \text{ ksi}}$ $0.45 \le v \le 0.65$	$0.85 - \frac{f'_c}{20 \text{ ksi}}$ $0.45 \le v \le 0.65$	$0.85 - \frac{f'_c}{20 \text{ ksi}}$ $0.45 \le v \le 0.65$

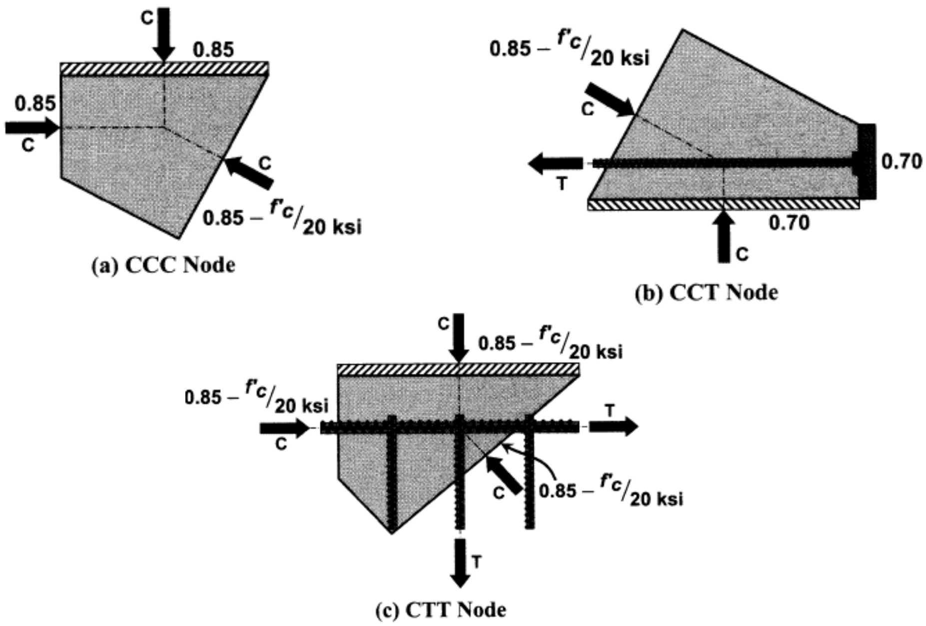


Figure 8-2: Concrete efficiency factor (AASHTO Figure and Table C5.8.2.5.3a-1)

If the node face contains non-prestressed compressive reinforcement, the nominal resistance shall be calculated as:

$$P_n = f_{cu} A_{cn} + f_y A_{ss}$$

where:

f_y = yield strength of longitudinal rebar (ksi)

A_{ss} = area of rebar in (in.^{2})

8.3 Strength of the Ties

Proper anchorage should be provided from the inner face of the nodal zone. The nominal strength of the tension tie (Section 5.8.2.4.1) is calculated as:

$P_n = f_y A_{st} + A_{ps}\left[f_{pe} + f_y\right]$………….. AASHTO LRFD 2017, Equation 5.8.2.4.1-1

where:

A_{st} = total area of longitudinal mild steel reinforcement in the tie (in.^{2})

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

f_y = yield strength of mild steel (ksi)

f_{pe} =stress in prestressing steel after losses (ksi)

8.4 Development Length Requirements

The end tie in a strut-and-tie model should be anchored properly in order to develop the tensile stress/force in the tie. The main longitudinal rebar must be developed/anchored a specific length beyond the nodal point. The development length is measured from the inner junction of strut and tie width. It can also be measured at the centroid of reinforcement where the tie leaves the intersection of effective strut width and the effective tie width. The development length is calculated as per the AASHTO LRFD 2017. The development length calculation for straight bar follows Section 5.10.8.2.1 while the development length for hook is based on Section 5.10.8.2.4. The development length is equal to basic development length times the modification factors.

In cases where the anchorage is not properly provided, the strength of tie reduces by a factor of deficient to full anchorage to the required length of anchorage.

In the analysis of the pier cap it is assumed that adequate development length is provided while lapping of rebar in the midsections. The only point to be checked is the ties at the end of the beam. The development length is discussed in detail in next chapter.

8.5 Crack Control Reinforcement

In the AASHTO LRFD Bridge Design Specifications 2017, crack control reinforcement is required if a strut-and-tie method is used as of Section 5.8.2.6. The reinforcement ratio in both the longitudinal and transverse direction must be at least 0.003. The amount of crack control reinforcement is required for the calculation of the concrete efficiency factor. These details are meant to improve the strength and serviceability of members designed using a strut-and-tie analysis, to limit the crack width, and to ensure a minimum ductility for the member.

$\frac{A_v}{b_w s_v} \geq 0.003$ ………….. AASHTO LRFD 2017, Equation 5.8.2.6-1

$\frac{A_h}{b_w s_h} \geq 0.003$ ………….. AASHTO LRFD 2017, Equation 5.8.2.6-2

where:

A_h and A_v = Total area of horizontal and vertical crack control reinforcement, respectively (in.^{2}) as shown in **Figure 8-3.**

b_w = width of member's web (in.) as in **Figure 8-3.**

s_h and s_v = spacing of horizontal and vertical crack control reinforcement, respectively (in.) as shown in **Figure 8-3.**

The maximum spacing of the bars in these girds (horizontal and vertical) should not exceed the smaller of d/4 and 12.0 in.

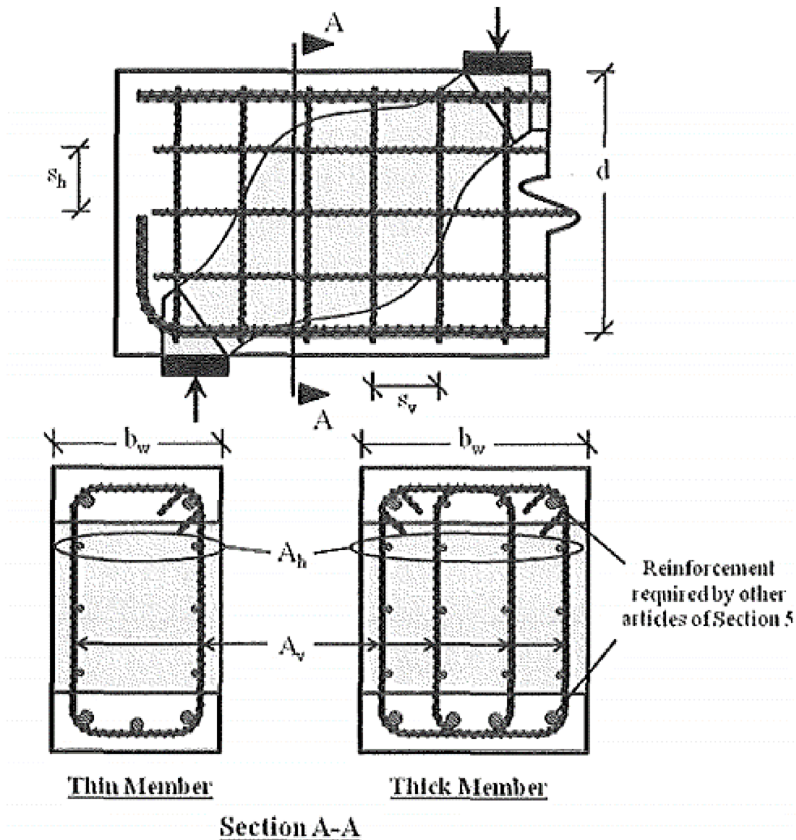


Figure 8-3: Distribution of crack control reinforcement (AASHTO Figure C5.8.2.6-1)

Chapter 9 Comparison of STM-CAP with Commercial STM Method

9.1 Introduction

This chapter compares a commercial Strut-and-Tie analysis method with STM-CAP. The advantage and disadvantage of each method are discussed in terms of total analysis time, utilization ratios, and governing mechanisms. Two bridge pier caps are modeled for this purpose.

9.1.1 Commercial STM Method

The commercial method starts with creating an appropriate STM model for the pier cap being analyzed. STAAD Pro (Bentley, 2019) is used to determine the STM member forces. Member capacities are manually determined using AASHTO LRFD design code and AASHTO Manual for Bridge Evaluation (MBE). A utilization ratio for each member is calculated and represented graphically in manually-drawn figures. If the STM model is not efficient or if any further alternative models are needed, the process is repeated with some re-useable data from the previous process. This process typically requires twenty to forty hours for a bridge pier analysis depending on the complexity of the pier caps and the experience of the engineer. This value is determined as a result of an interview with the senior engineers using this process.

i. Determining STM Member Forces

The STM model for sample Pier Cap 1 is shown in Figure 9-1, where the top nodes represent the girder positions while the bottom nodes represent the centroid of the columns. Truss elements with uniform stiffnesses (i.e., the same moment of inertias, section properties, and moduli of elasticity) are used. The loads from the girders are applied as concentrated loads at the centerline of the bearings on the pier caps. The support conditions are modelled as pinned. The member forces are determined as shown in Appendix D.

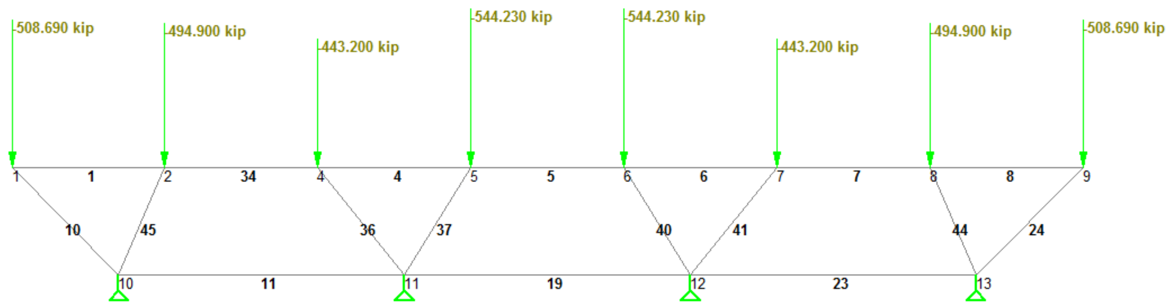


Figure 9-1: STAAD Pro STM model for Pier Cap I

ii. Determining the STM Capacities

Based on AASHTO LRFD design codes and AASHTO MBE, manual calculations are performed to determine the capacities of the ties, struts and nodes. The cross-sectional details, reinforcement details, and bearing details are obtained from the design drawings. Anchorage (i.e., development length) and serviceability checks (i.e., minimum skin reinforcement) are performed. Sample calculations are presented in Appendix D.

iii. Result Presentation

The rating factors (RFs) are calculated as the ratio of the capacity to the member force. These RFs are converted to equivalent utilization ratios (URs), as shown in **Figure 9-2,** to more consistently compare them with the STM-CAP results as shown below in. The governing member and governing mechanism are determined using the highest URs.

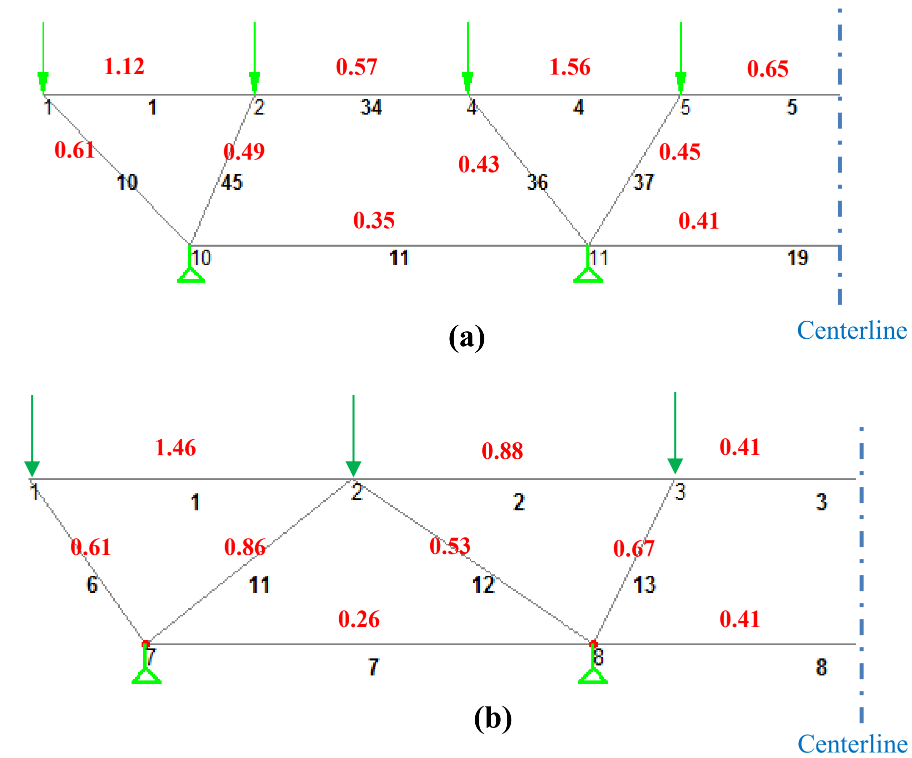


Figure 9-2: Utilization ratios for sample **(a)** Pier Cap 1; **(b)** Pier Cap 2

9.1.2 STM-CAP

The sample pier caps are also modeled in STM-CAP to obtain the URs for all members following the methodology, as shown in **Figure 9-3**.

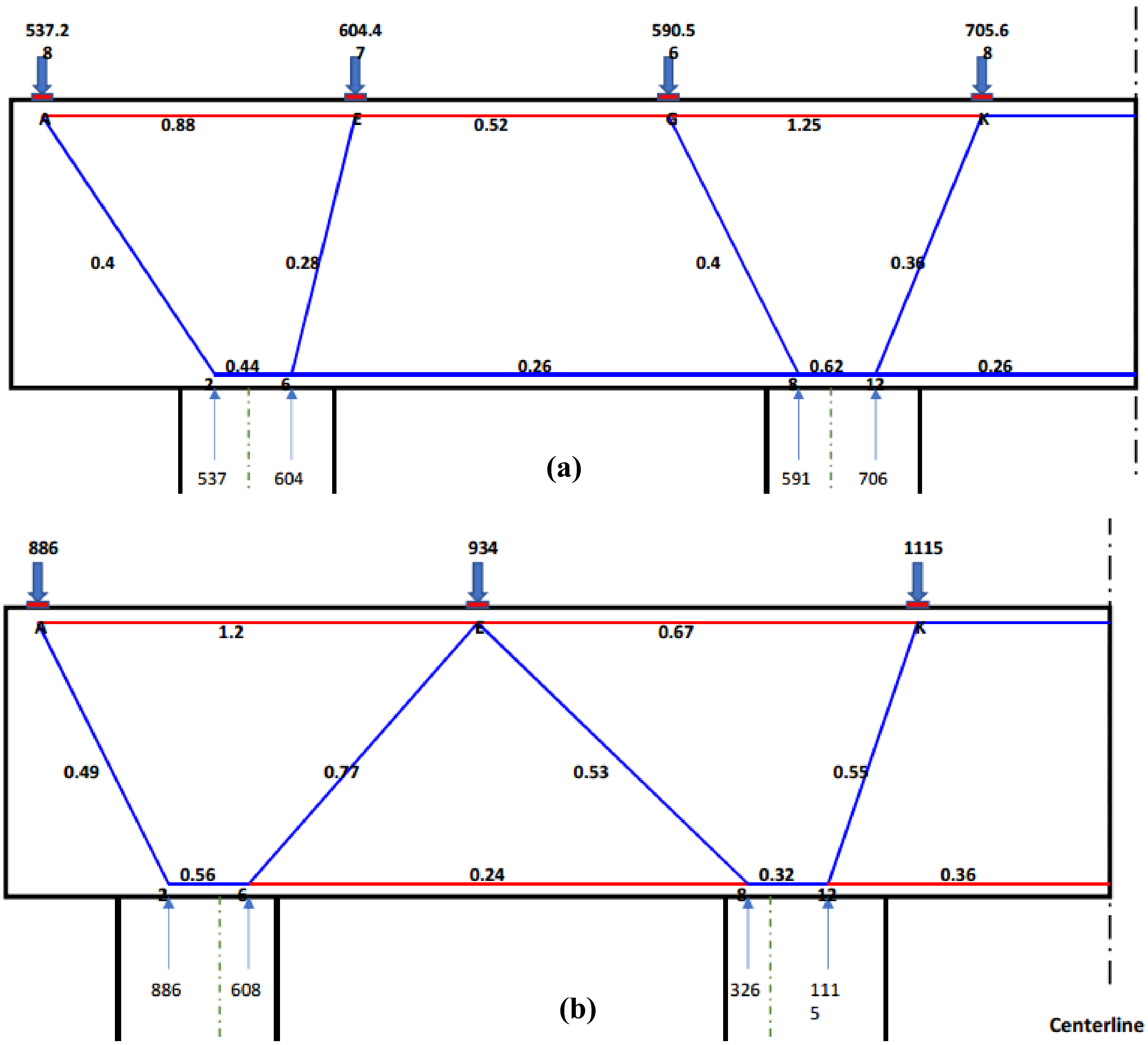


Figure 9-3: STM-CAP utilization ratio for **(a)** Pier Cap 1; **(b)** Pier Cap 2

9.2 Comparison

The commercial method and STM-CAP employ the same STM principles. The following sections compares the results obtained.

9.2.1 STM member force and Utilization Ratio (UR)

The member forces, capacities and URs are compared for both pier caps in **Tables 9-1** and **9-2** and **Figures 9-4** and **9-5**.

Table 9-1: Comparison of member forces, capacities and URs for sample Pier Cap 1

STM Members	STM-CAP			Commercial Method		
	STM Force (kip)	STM Capacity (kip)	UR	STM Force (kip)	STM Capacity (kip)	UR
A-E	420	476	0.88	531	476	1.12
E-G	246	476	0.52	271	476	0.57
G-K	594	476	1.25	745	476	1.56
K-Q	----	----	----	310	476	0.65
2-6	-420	-962	0.44	-----	-----	-----
6-8	-246	-962	0.26	-271	-764	0.35
8-12	-594	-962	0.62	-----	-----	-----
12-14	-255	-962	0.26	-310	-764	0.41
A-2	-682	-1713	0.40	-755	-1233	0.61
E-6	-629	-2224	0.28	-658	-1349	0.49
G-8	-686	-1726	0.40	-757	-1747	0.43
K-12	-783	-2169	0.36	-829	-1827	0.45

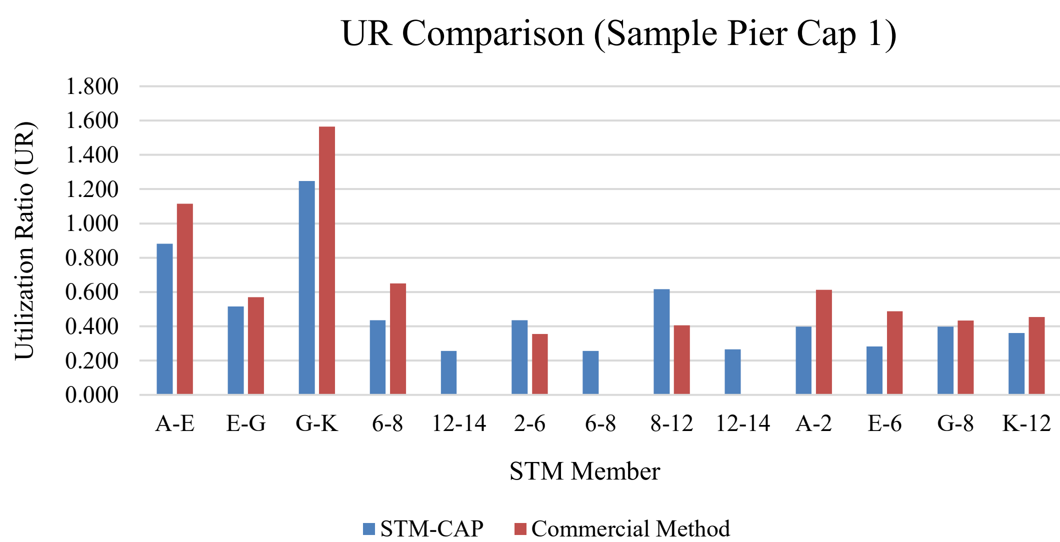


Figure 9-4: STM-CAP and commercial method UR comparison Pier Cap 1

Table 9-2: Comparison of member forces, capacities and URs for sample Pier Cap 2

STM Members	STM-CAP			Commercial Method		
	STM Force	STM Capacity	UR	STM Force	STM Capacity	UR
A-E	521	435	1.20	635	435	1.46
E-K	291	435	0.67	381	435	0.88
K-Q	158	435	0.36	179	435	0.41
2-6	-521	-921	0.56	-----	-----	-----
6-8	104	435	0.24	114	435	0.26
8-12	-291	-921	0.32	-----	-----	-----
12-14	158	435	0.36	179	435	0.41
A-2	-1028	-2098	0.49	-1090	-1782	0.61
E-6	-872	-1131	0.77	-959	-1113	0.86
E-8	-512	-974	0.53	-599	-1126	0.53
K-12	-1202	-2182	0.55	-1248	-1867	0.67

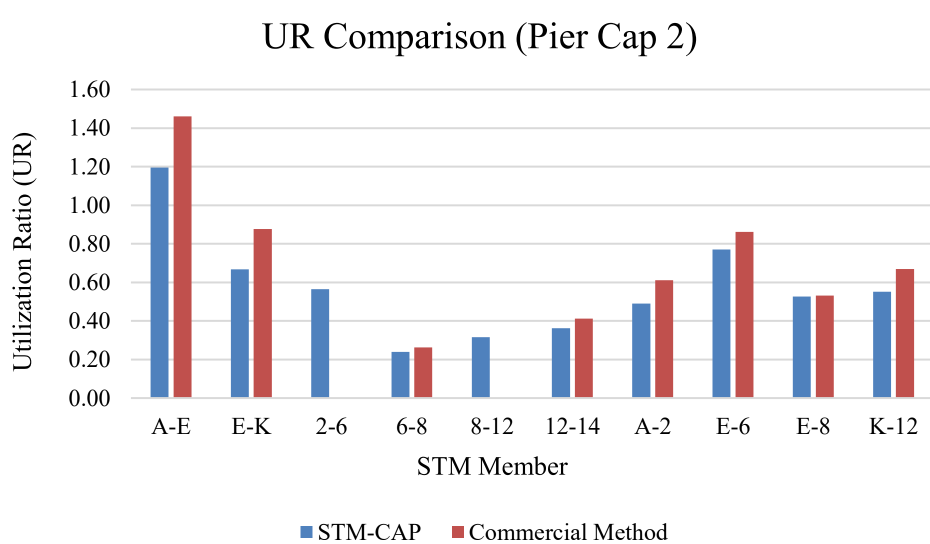


Figure 9-5: STM-CAP and commercial method UR comparison Pier Cap 2

The utilization ratios from both methods follow a similar trend with STM-CAP providing consistently lower utilization ratios. This discrepancy is due to the fact that the commercial method does not consider the column proportioning (i.e., division of column) as STM-CAP does. This results in less-efficient and smaller strut angles which results in higher member forces and lower capacities for the governing struts. Consequently, the commercial method has one less member per column in the pier cap due to not providing column proportioning. For the governing mechanism, both methods provide the same predictions.

9.2.2 Total Analysis Time

The breakdown of the time required is shown in **Figure 9-6**, which demonstrates that STM-CAP can reduce the effort and budget to one tenth of that required by the commercial method (CM).

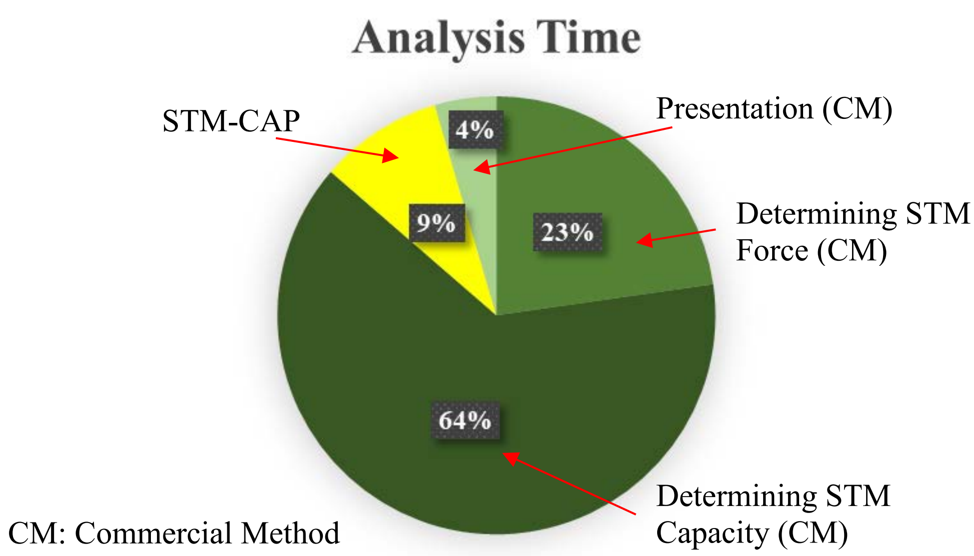


Figure 9-6: Total analysis time for STM-CAP and commercial method

9.3 Advantage of Commercial Method:

9.3.1 Flexibility

The commercial method provides flexibility to model any type of structure for an STM analysis. With a similar effort, it can analyze hammer head pier caps, variable depth pier caps, pier caps with any number of columns and nonsymmetrical conditions. However, the user should have the expert level knowledge on the STM modeling and create the truss model from scratch. No template or suggestion is given by the program.

9.3.2 User Control

The user has full control over the STM model and the capacity calculations according to any desired code. Any additional information such as section loss of rebar due to deterioration, special load case, etc. can be accounted. However, once again, the user should have the expert knowledge to do the required calculations on his own.

9.4 Advantages of STM-CAP

9.4.1 Simple Modeling Process

The model is automatically generated, and the user does not need expert knowledge. The capacity calculations are also done automatically. STM-CAP educates a beginner user. It guides the user with step by step input, etc.

9.4.2 Graphical Interface

STM-CAP provides dynamically adjusted graphical input and output sketches. Input sketches are drawn to confirm the accuracy to minimize the input mistakes whereas the output sketches provide a comprehensive overview of the analysis results in a graphical manner. Informatory sketches are also provided to clarify and explain the input parameters.

9.4.3 Faster Modeling and Analysis Process

A major advantage of STM-CAP is that it can complete an entire pier cap analysis ten times faster than the commercial method. The analysis can be performed almost immediately. It has the potential of saving large amounts of project budget for pier cap analyses.

Chapter 10 Summary and Conclusions

10.1 Summary

‘Pier caps’ or ‘bent caps’ transfer the load from bridge girders to piers. Due to short shear span, most pier caps act as ‘deep beams,’ which possess additional shear strength due to the formation of the strut action. Several theories are available to predict the failure modes and capacities of pier cap, with the Strut-and-Tie Method (STM) being one of the most commonly used methods. STM has been included in AASHTO LRFD Bridge Specification since 1994 for the analysis of deep beams.

STM is an analysis methodology where the internal stress distribution is idealized by a truss model which is termed the strut-and-tie model. Struts and ties are the elements of STM which represent the uniaxial compressive stress and tensile stress. The forces in the truss member act as loads and the AASHTO LRFD code provide specifications for the calculation of the capacity of the members. Thus, the loads can be compared with the capacities to determine the utilization ratio of a pier cap.

STM-CAP, which stands for Strut-and-Tie Method for pier CAPs, is a spreadsheet program that has been developed in this study for the analysis of deep pier caps subjected to static girder loads. STM-CAP uses Visual Basic coding and provides graphical solutions to aid the analyst to understand the system and identify potential input errors.

STM-CAP analyzes both symmetrical and asymmetrical pier caps with or without cantilevered end spans. It can analyze a symmetrical pier cap up to eight columns and an asymmetrical pier cap up to four columns. The generated truss model can be adjusted by the user for the optimization of the truss model. Several combinations are possible by activating or deactivating the option for vertical ties. STM-CAP considers horizontal ties, vertical ties, reinforced horizontal struts, inclined struts, and nodal regions for calculating the loads and capacities. STM-CAP determines the utilization ratios, a ratio of the load to the capacity, for each member. From the utilization ratio, STM-CAP predicts the possible failure modes and corresponding locations of the failure.

A total of eight pier caps, the drawings of which were received from ODOT, are modeled using STM-CAP. They consist of cantilevered, non-cantilevered, symmetrical, and asymmetrical pier caps with different numbers of columns and girder loads. The same pier caps are also modeled with CAST (Computer Aided Strut-and-Tie) and VecTor2 software. The utilization ratios, governing behaviors, and failure modes are compared to validate the accuracy of STM-CAP and provide sample applications to the users.

Five out of eight pier caps modeled by CAST are also modeled using VecTor2, a nonlinear finite element analysis software, to compare the results. The comparison of the STM-AASHTO results with the stress distribution from VecTor2 was performed based on the concept of utilization ratio, which is the ratio of stresses at the factored loads divided by the strength of the material. In addition, the nonlinear load-displacement responses obtained from VecTor2 is used to obtain the global capacity of the pier caps.

The sectional analysis method is compared with the STM analysis method to compare the results. The tensile rebar stresses at critical sections (i.e., sections with maximum moment) is calculated using the sectional method and is compared with the stress in tension ties using STM-CAP. The shear capacities are determined at critical sections (i.e., sections with maximum shear) using the sectional analysis method and compared with the utilization ratios of inclined struts or vertical ties obtained from STM-CAP at the same sections.

10.2 Conclusions

The literature review, both analytical and experimental, consistently indicated that STM estimates the load capacities for deep beams more accurately and less conservatively than the sectional method. Most pier caps qualify as deep beams and will be found overloaded using the sectional method. STM will give higher and more accurate capacity prediction but will still be conservative as compared to a nonlinear finite element analysis. The AASHTO LRFD 2017 consistently requires the use of either a strut-and-tie or nonlinear finite element model for the analysis and design of deep members. Both methods are more sophisticated and require more effort than the sectional method. Thus, a solution algorithm (through a computer program) based on STM was developed that can be used in practice for the analysis of the pier caps.

The developed spreadsheet program, STM-CAP, followed the AASHTO LRFD 2017. The factored load and factored material resistances are used to perform an LRFD analysis. STM-CAP defines the geometry configuration and detailing of STM elements based on the AASHTO provisions. Tie tensile capacities, strut compressive strengths and limiting nodal compressive strengths are calculated. It performs the reinforcement development checks, bearing checks, and crack control reinforcement checks as required by the AASHTO LRFD 2017.

STM-CAP determines the load, capacity, and utilization ratio for each element of STM, which reflects the condition (i.e., overloaded or reserve capacity) of the pier cap. Using the utilization ratios, overloaded bridges can be categorized/ranked and limited rehabilitation and strengthening funds can be directed to the caps with the largest utilization ratios. STM-CAP also indicates the governing failure mode and the location of the failure which will facilitate strengthening cap beams at the correct locations. The research results have potential to result in significant cost savings by rehabilitating a smaller number of pier caps and reducing the associated construction work and traffic disruption. STM-CAP can also be used when load rating concrete pier caps. It can also be used when determining a safe load limit for certain bridges and when assessing the feasibility of increasing the loads and extending lanes.

Eight bridge pier caps were modeled using STM-CAP; the results were validated using CAST. Based on the numerical modeling of the pier caps, STM-CAP provided identical results to CAST in most cases because both programs work with the same principles of the strut-and-tie conceptualization. In other cases, the STM-CAP provided higher utilization ratios than CAST and remains on the conservative side. In such cases of discrepancy, the difference in the utilization ratios between the two methods was under 5%. One reason for these discrepancies was related to the geometrical simplifications made in CAST which uses a grid with constant spacing. STM-CAP permitted more accurate input of the bridge geometry (e.g., a girder spacing of 13' and 11.5"). The other reason may involve round off errors. Verification with the hand calculations indicated that STM-CAP is more accurate in such cases of discrepancies.

The simulation of the behavior of five pier caps was undertaken using a nonlinear finite element method (FEM) and the analysis method VecTor2. The behavior of pier caps was found to match STM-AASHTO. The critical members were the same and the failure patterns matched reasonably well. The members with high utilization ratios from the STM-AASHTO matched the highly stressed members in the nonlinear FEM analysis. The nonlinear FEM predicted higher capacities, as expected, for the deep as well as slender regions than the STM-AASHTO. The STM-AASHTO is based on a lower-bound theorem and thus terminates the analysis at the first yielding of the reinforcement whereas nonlinear FEM continues the analysis until ultimate failure of the structure due to significant re-distribution of forces. The utilization ratios from the nonlinear FEM and STM-AASHTO showed a similar trend with *a/d* ratios. For *a/d* ratios between 1.5 and 2.0, nonlinear

FEM calculated up to two times larger shear load capacities. As the *a/d* ratio decreased, the results from the nonlinear FEM and STM-AASHTO converged. The utilization ratios from the nonlinear FEM were calculated to be 40% on average of those from STM-AASHTO. The nonlinear FEM provided complete response simulation with highly accurate results but require significant knowledge, analysis time, and experience to obtain correct results. It took approximately fifteen to twenty hours for each cap beam to create the analysis model, run the simulation, and obtain/understand the analysis results.

The results from the sectional method and the STM-CAP for the same pier caps were compared. These comparisons showed that the sectional method systematically underestimates the shear capacity prediction of deep pier caps. The deeper the pier cap, the higher the discrepancy between calculated shear capacities. For lower *a/d* ratios (*a/d* = 0.50), STM-CAP predicted two to three times higher shear capacities. As the *a/d* ratio increased, the prediction by STM-CAP and sectional method converged. The shear capacity prediction by STM-CAP was still higher than the sectional method as the *a/d* ratio reached a value 3.0. These STM predicted capacities were still conservative when compared with nonlinear FEM (from Chapter 6) because STM is based on lower-bound theorem. The STM and STM-CAP program provided a good compromise between complexity and accuracy as compared to the sectional method and nonlinear FEM. While it is as simple as the sectional method, it provides an accuracy closer to the finite element method. Thus, the STM-CAP predicts more accurate and higher capacity for deep pier cap.

Two sample bridge pier caps were modeled using the commercial method and STM-CAP. Both methods predict similar trend of utilization ratio with same controlling member and controlling mechanism. The analysis time with the commercial method is approximately ten times more time than the STM-CAP analysis time. Using STM-CAP for analysis of bridge pier cap can save huge amount of funds and effort. STM-CAP provides easy modeling approach to users with dynamic generated informative and output sketches. However, the commercial method used is a general-case solution to any bridge pier cap. The commercial method is a good technique where the application of STM-CAP is limited (more than 8 column pier cap, hammerhead pier cap etc.).

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Appendix A: STM-CAP Solved Examples (AASHTO LRFD 2014)

BRIDGE PIER CAP 1

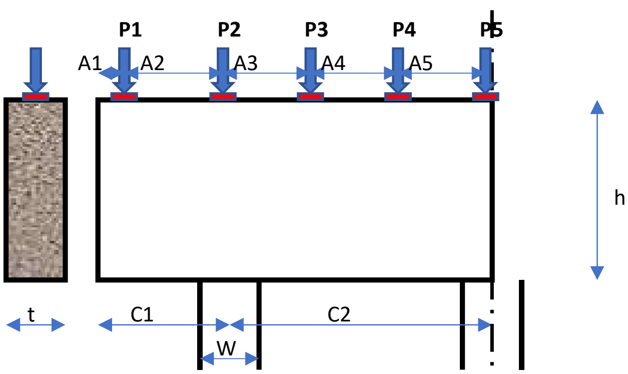
Analysis Input

Bridge Details:

Bridge Name:	Bridge 1	Pier Number:	Pier 2-Left
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	3

☐ Unsymmetrical



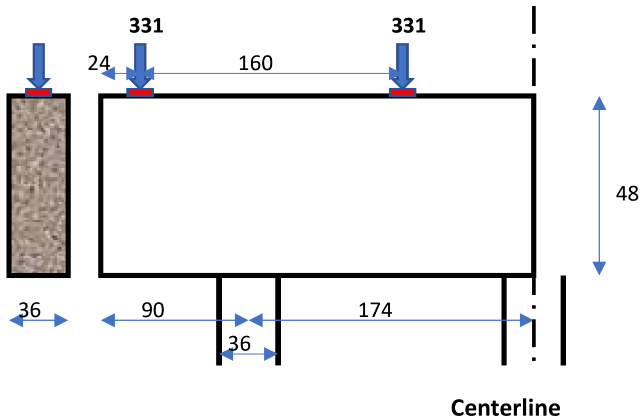
Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	7 ft	6 in	**90 in**
Distance from center of first column to center of second column (C2)	14 ft	6 in	**174 in**
Column width (W)	36 in	Square	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	0 in	**24 in**
Spacing between the girders	13 ft	4 in	**160 in**
Factored Load	331 k		

Factored Load		***Distance***			
P1	331 k	2 ft	0.0 in	**24.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	331 k	13 ft	4.0 in	**160.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5

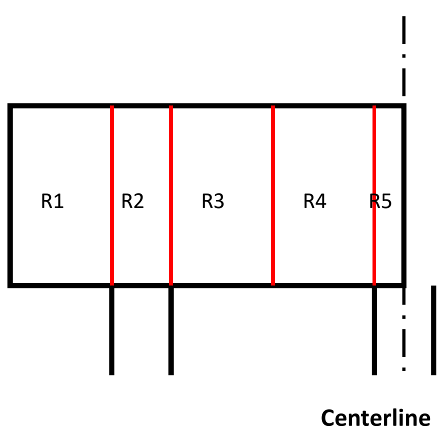


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	60.33 in	1.40	Deep Region
R2	0.00 in	0.00	Zero Region
R3	81.67 in	1.89	Deep Region
R4	71.00 in	1.64	Deep Region
R5	0.00 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

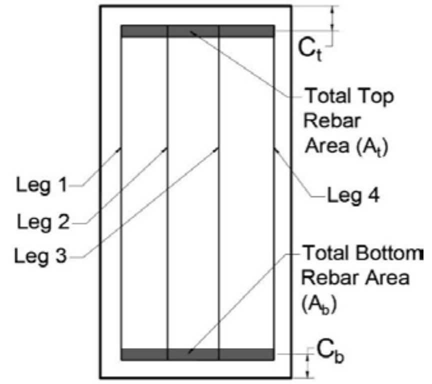
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



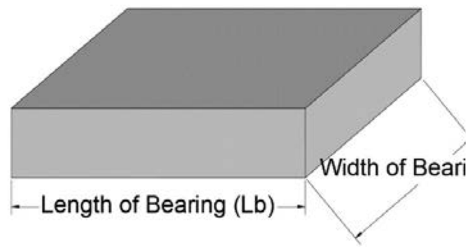
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	13.97	6	7	4.5
R2	13.97	6	7	4.5
R3	13.97	6	7	4.5
R4	13.97	6	7	4.5
R5	13.97	6	7	4.5



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	5 in
R2	0	0 in
R3	4	10 in
R4	2	12 in
R5	0	0 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	13.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in



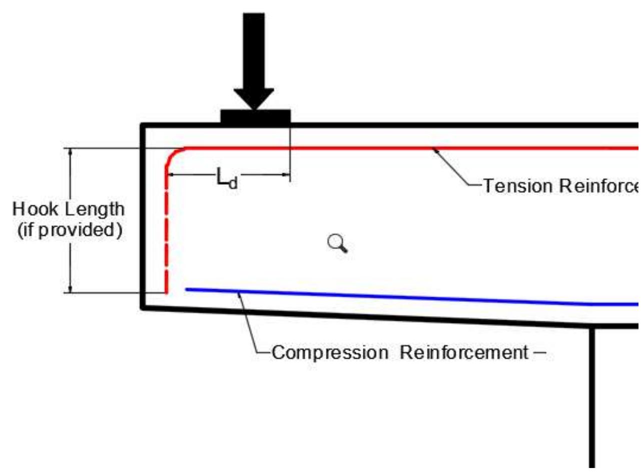
11. Reinforcement Development

Horizontal length available (L_d)	**33 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	**24 in**

It qualifies for 90° hook.

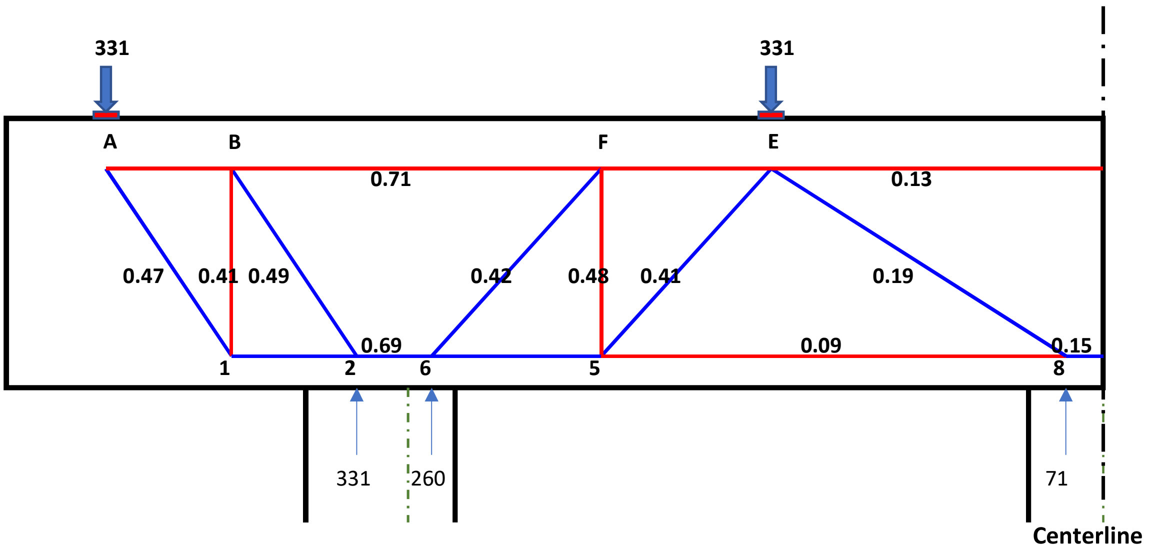
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**



Required development length	**29 in**
Available development length (L_d)	**33 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

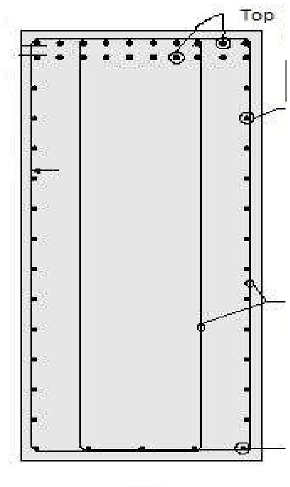
Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here

1
1
0

Bearing Areas Nodes at ⇉

Summary				
Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***
B-F	533	754	0.71	PASS
E-K	101	754	0.13	PASS
2-6	-533	-771	0.69	PASS
5-8	34	378	0.09	PASS
8-12	-101	-680	0.15	PASS
B-1	331	808	0.41	PASS
F-5	260	547	0.48	PASS
H-7	-	-	0.00	-
A-1	-425	-896	0.47	PASS
B-2	-425	-868	0.49	PASS
F-6	-384	-923	0.42	PASS
E-5	-384	-937	0.41	PASS
E-8	-152	-780	0.19	PASS
A	331	573	0.58	PASS
E	331	497	0.67	PASS
2	331	1727	0.19	PASS
6	260	1357	0.19	PASS
8	71	1361	0.05	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement



Region	Area of the Crack Control Rebar (in^2)	Spacing of Crack Control Rebar (in)	No of layers of Crack Control Rebars	Spacing between skin bars	Crack Control Reinforcement
				Code Required Min skin reinforcement	0.30%
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%

BRIDGE PIER CAP 2

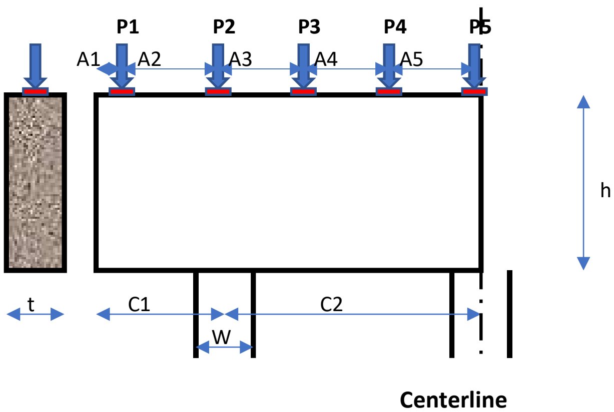
Analysis Input

Bridge Details:

Bridge Name:	Bridge 2	Pier Number:	Pier 2
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	3

☐ Unsymmetrical

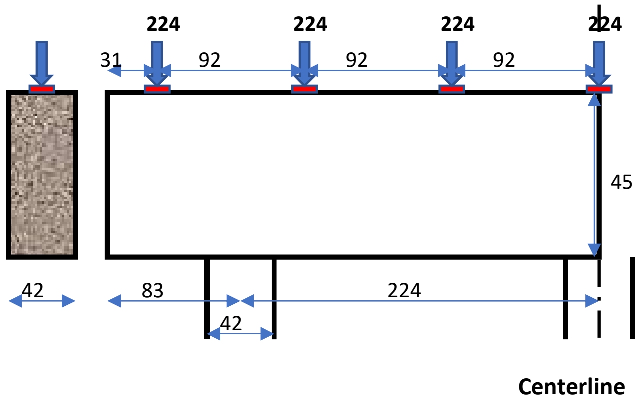


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	6 ft	11 in	**83 in**
Distance from center of first column to center of second column (C2)	18 ft	8 in	**224 in**
Column width (W)	42 in	**Circular**	
Depth of pier cap (h)	45 in		
Thickness of pier cap (t)	42 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	7 in	**31 in**
Spacing between the girders	7 ft	8 in	**92 in**
Factored Load	224 k		

Factored Load		***Distance***			
P1	224 k	2 ft	7.0 in	**31.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	224 k	7 ft	8.0 in	**92.0 in**	A3
P4	224 k	7 ft	8.0 in	**92.0 in**	A4
P5	224 k	7 ft	8.0 in	**92.0 in**	A5

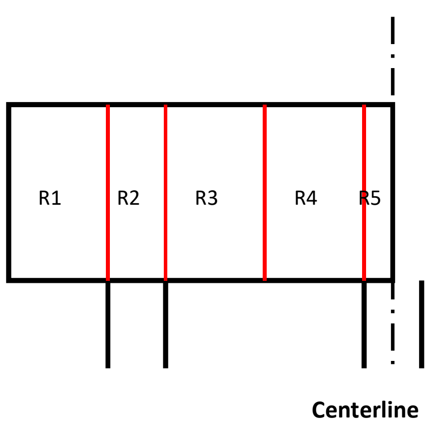


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	**40.41 in**	**1.00**	**Deep Region**
R2	**0.00 in**	**0.00**	**Zero Region**
R3	**30.59 in**	**0.76**	**Deep Region**
R4	**77.36 in**	**1.91**	**Deep Region**
R5	**4.14 in**	**0.10**	**Deep Region**

This pier cap is deep.
Please continue with Section 7.

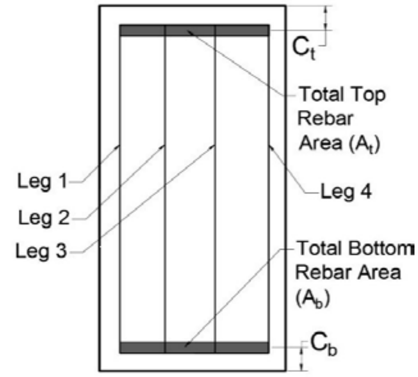
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



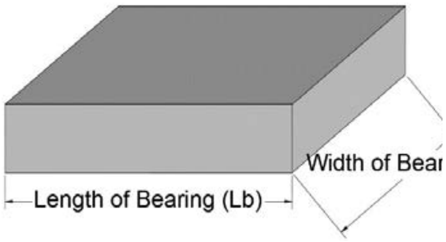
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	**Top Steel (in^2, in)**		**Bottom Steel (in^2, in)**	
	Total Area (A_t)	**Centroid (C_t)**	**Total Area (A_b)**	**Centroid (C_b)**
R1	9.46	4.45	8	3.15
R2	9.46	4.45	8	3.15
R3	9.46	4.45	8	3.15
R4	9.46	4.45	8	3.15
R5	9.46	4.45	8	3.15



9B. Transverse Reinforcement		
Region	**No. of Legs**	**Stirrup Spacing**
R1	4	10 in
R2	4	10 in
R3	4	10 in
R4	4	10 in
R5	4	10 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	20.0 in
Base plate width perpendicular to the pier cap (W_b)	13.0 in



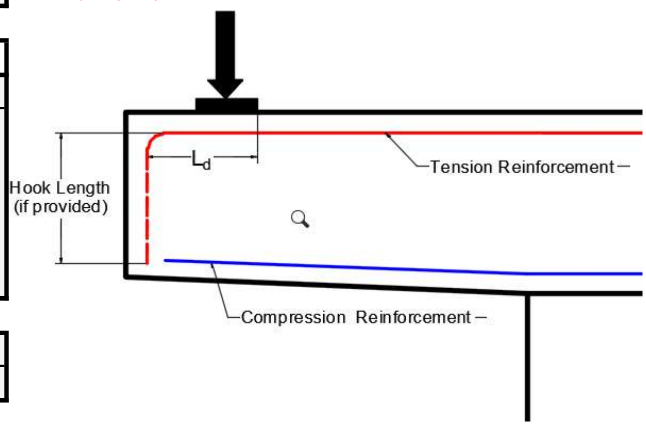
11. Reinforcement Development

Horizontal length available (L_d)	**41 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	**24 in**

It qualifies for 90° hook.

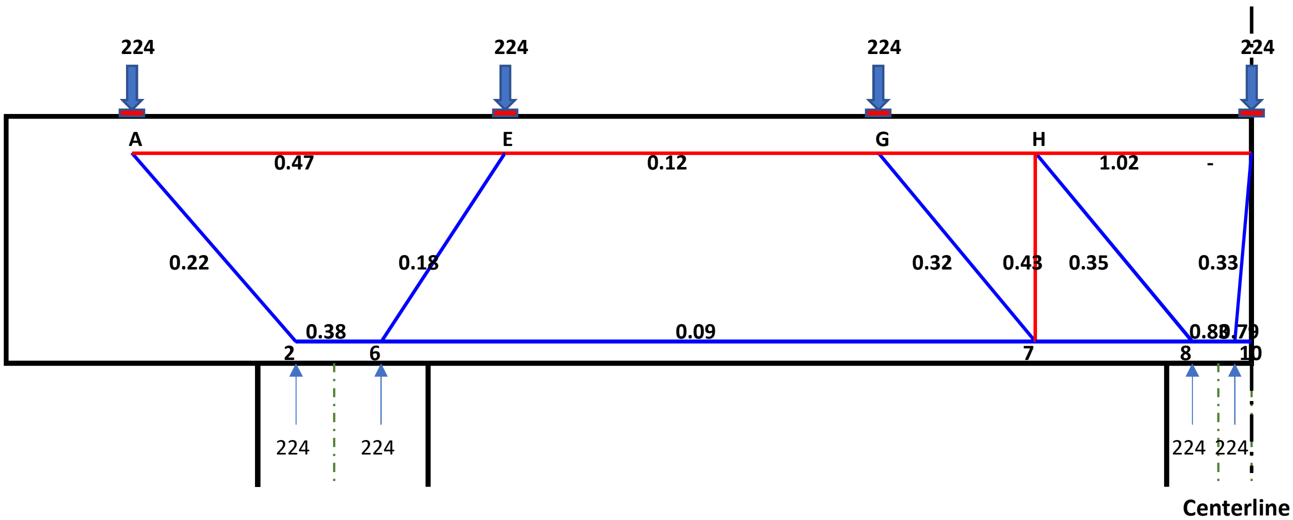
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**



Required development length	**29 in**
Available development length (L_d)	**41 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

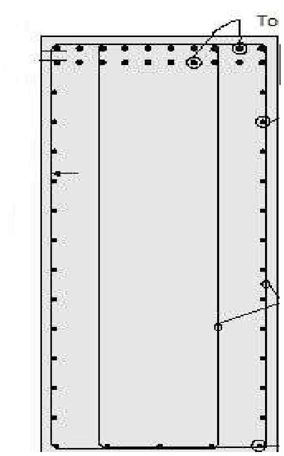
STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary				
	Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***
	A-E	242	511	0.47	PASS
	E-G	59	511	0.12	PASS
	H-I	522	511	1.02	Flexure Overloaded
	2-6	-242	-630	0.38	PASS
	6-7	-59	-630	0.09	PASS
	8-10	-522	-630	0.83	PASS
	10-12	-497	-630	0.79	PASS
	B-1	-	-	0.00	-
	F-5	-	-	0.00	-
	H-7	224	518	0.43	PASS
	J-9	-	-	0.00	-
0	A-2	-330	-1506	0.22	PASS
0	E-6	-289	-1614	0.18	PASS
1	G-7	-322	-1022	0.32	PASS
	H-8	-322	-933	0.35	PASS
0	I-10	-225	-682	0.33	PASS
Bearing Areas Nodes at ⇉	A	224	546	0.41	PASS
	E	224	473	0.47	PASS
	G	224	473	0.47	PASS
	I	224	473	0.47	PASS
	2	224	1649	0.14	PASS
	6	224	1649	0.14	PASS
	8	224	824	0.27	PASS
	10	224	824	0.27	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

Code Required Min skin reinforcement					0.30%
Region	***Area of the Crack Control Rebar (in^2)***	***Spacing of Crack Control Rebar (in)***	***No of layers of Crack Control Rebars***	***Spacing between skin bars***	***Crack Control Reinforcement***
Region 1	0.31	8.0	2	Good	0.18%
Region 2	0.31	8.0	2	Good	0.18%
Region 3	0.31	8.0	2	Good	0.18%
Region 4	0.31	8.0	2	Good	0.18%
Region 5	0.31	8.0	2	Good	0.18%



BRIDGE PIER CAP 3

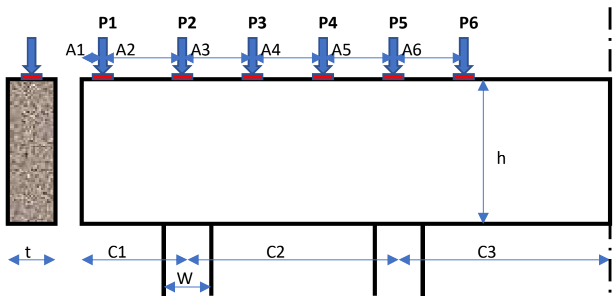
Analysis Input

Bridge Details:

Bridge Name:	Bridge 3	Pier Number:	North Pier
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Unsymmetrical

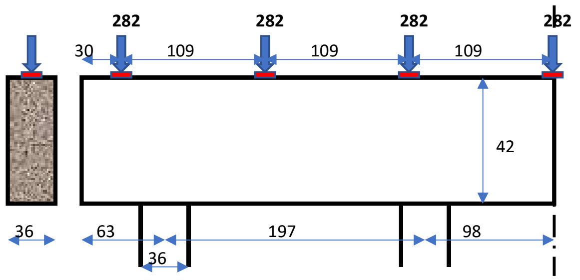


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	5 ft	3 in	**63 in**
Distance from center of first column to center of second column (C2)	16 ft	5 in	**197 in**
Distance from center of second column to centerline of pier cap (C3)	8 ft	2 in	**98 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	42 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	6 in	**30 in**
Spacing between the girders	9 ft	1 in	**109 in**
Factored Load	282 k		

Factored Load		***Distance***			
P1	282 k	2 ft	6.0 in	**30.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	282 k	9 ft	1.0 in	**109.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	282 k	9 ft	1.0 in	**109.0 in**	A5
P6	282 k	9 ft	1.0 in	**109.0 in**	A6

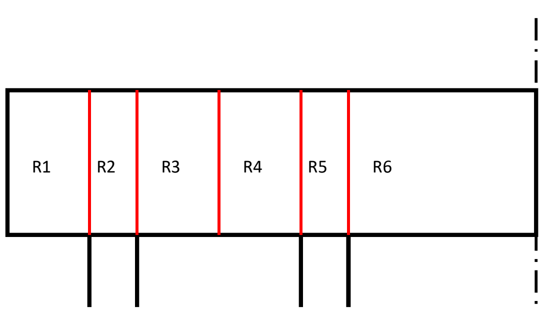


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	26.15 in	0.69	Deep Region
R2	0.00 in	0.00	Zero Region
R3	64.85 in	1.72	Deep Region
R4	105.91 in	2.80	Slender Region
R5	7.37 in	0.19	Deep Region
R6	87 in	2.29	Deep Region

This pier cap is deep.
Please continue with Section 7.

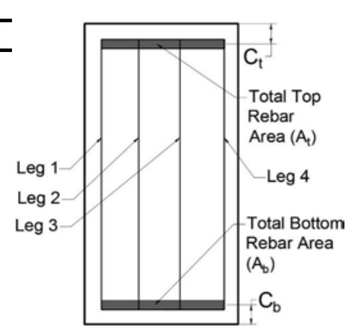
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



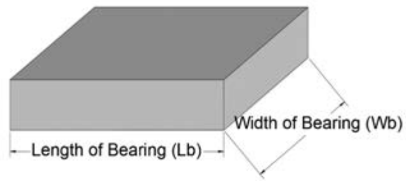
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	*Top Steel (in², in)*		*Bottom Steel (in², in)*	
	Total Area (A_t)	*Centroid (C_t)*	*Total Area (A_b)*	*Centroid (C_b)*
R1	8	4.1	8	4.1
R2	8	4.1	8	4.1
R3	8	4.1	8	4.1
R4	8	4.1	8	4.1
R5	8	4.1	8	4.1
R6	8	4.1	8	4.1



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	7 in
R2	0	0 in
R3	4	12 in
R4	4	12 in
R5	0	0 in
R6	4	16 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	21.0 in
Base plate width perpendicular to the pier cap (W_b)	13.0 in



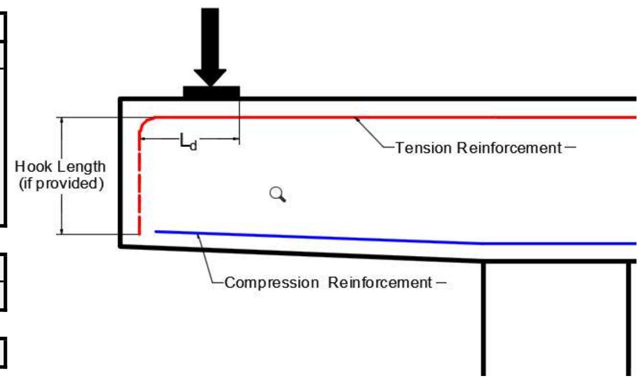
11. Reinforcement Development

Horizontal length available (L_d)	**40 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	**19 in**

It qualifies for 90° hook.

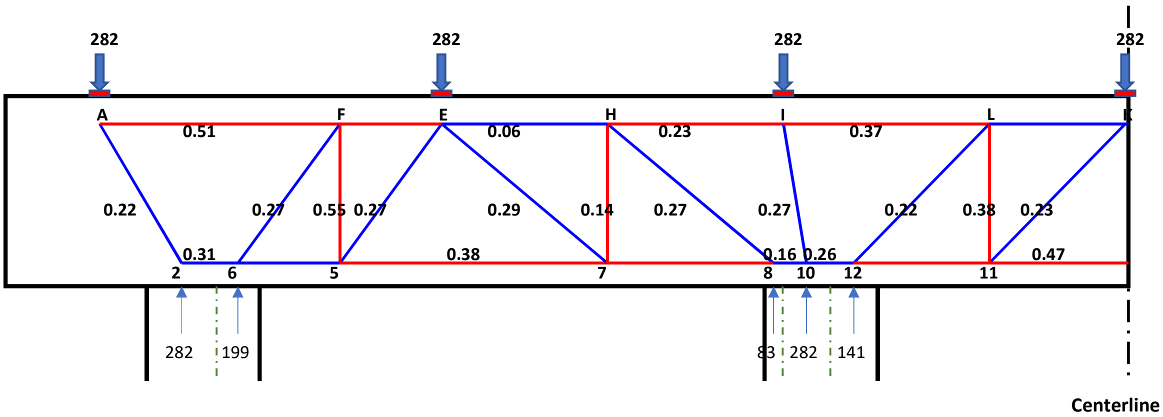
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**



Required development length	**23 in**
Available development length (L_d)	**40 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

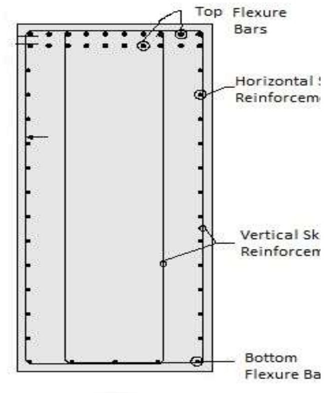
STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result	
	A-F	218	432	0.51	PASS	Top Members
	E-H	-32	-537	0.06	PASS	
	H-I	98	432	0.23	PASS	
	I-L	160	432	0.37	PASS	
	2-6	-218	-703	0.31	PASS	Bottom Members
	5-7	163	432	0.38	PASS	
	8-10	-98	-620	0.16	PASS	
	10-12	-160	-620	0.26	PASS	
	11-14	201	432	0.47	PASS	
	B-1	-	-	0.00	-	Vertical Members
	F-5	199	362	0.55	PASS	
	H-7	83	591	0.14	PASS	
	J-9	-	-	0.00	-	
	L-11	141	374	0.38	PASS	
0	A-2	-357	-1635	0.22	PASS	Inclined Members
1	F-6	-275	-1020	0.27	PASS	
	E-5	-275	-1020	0.27	PASS	
1	E-7	-155	-538	0.29	PASS	
	H-8	-155	-576	0.27	PASS	
0	I-10	-289	-1080	0.27	PASS	
1	L-12	-229	-1032	0.22	PASS	
	K-11	-229	-1010	0.23	PASS	
Bearing Areas Nodes at ⇉	A	282	573	0.49	PASS	1
	E	282	573	0.49	PASS	
	I	282	497	0.57	PASS	
	K	282	650	0.43	PASS	
	2	282	1422	0.20	PASS	2
	6	199	1001	0.20	PASS	
	8	83	352	0.24	PASS	
	10	282	1349	0.21	PASS	
	12	141	595	0.24	PASS	

14. Informational Check: Min Horizontal Crack Control Reinforcement

				Code Required Min skin reinforcement	0.30%
Region	***Area of the Crack Control Rebar (in ^{2})***	***Spacing of Crack Control Rebar (in)***	***No of layers of Crack Control Rebars***	***Spacing between skin bars***	***Crack Control Reinforcement***
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%
Region 6	0.31	8.0	2	Good	0.22%



BRIDGE PIER CAP 4

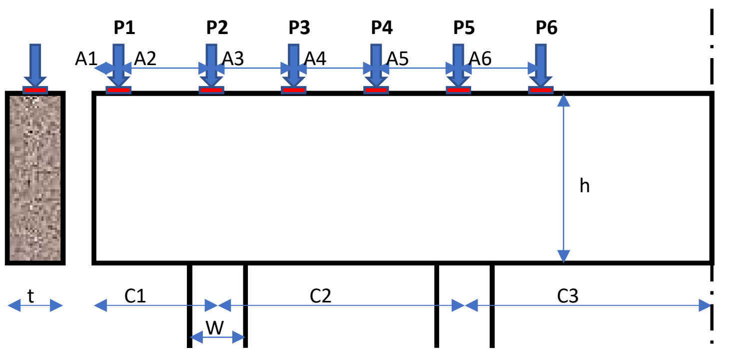
Analysis Input

Bridge Details:

Bridge Name:	Bridge 4	Pier Number:	Left-Unsymmetric
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Unsymmetrical

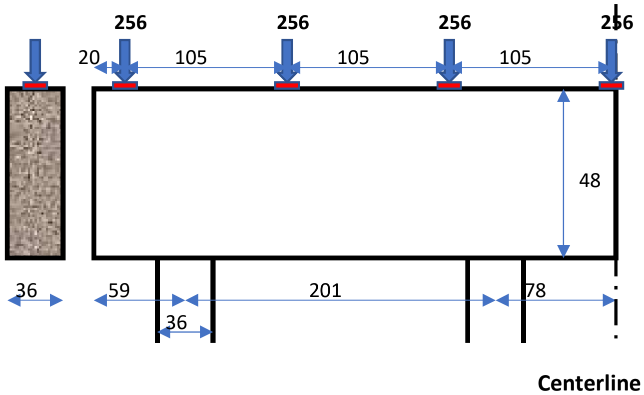


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	4 ft	11 in	**59 in**
Distance from center of first column to center of second column (C2)	16 ft	9 in	**201 in**
Distance from center of second column to centerline of pier cap (C3)	6 ft	6 in	**78 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft	8 in	**20 in**
Spacing between the girders	8 ft	9 in	**105 in**
Factored Load	256 k		

Factored Load		***Distance***			
P1	256 k	1 ft	8.0 in	**20.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	256 k	8 ft	9.0 in	**105.0 in**	A3
P4	256 k	8 ft	9.0 in	**105.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5
P6	256 k	8 ft	9.0 in	**105.0 in**	A6

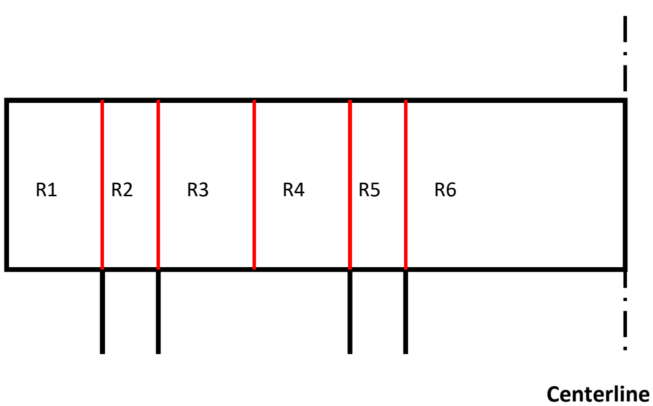


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	**30.89 in**	**0.71**	**Deep Region**
R2	**0.00 in**	**0.00**	**Zero Region**
R3	**56.11 in**	**1.30**	**Deep Region**
R4	**21.74 in**	**0.50**	**Deep Region**
R5	**0.00 in**	**0.00**	**Zero Region**
R6	**65 in**	**1.51**	**Deep Region**

This pier cap is deep.
Please continue with Section 7.

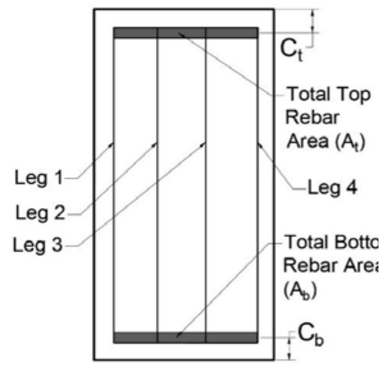
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	0.79 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



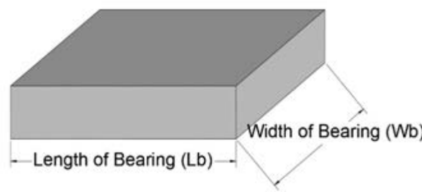
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in², in)***		***Bottom Steel (in², in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	8	4.5	9	4.2
R2	8	4.5	9	4.2
R3	8	4.5	9	4.2
R4	8	4.5	9	4.2
R5	8	4.5	9	4.2
R6	8	4.5	9	4.2



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	6 in
R2	0	0 in
R3	4	12 in
R4	4	6 in
R5	0	0 in
R6	4	18 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	11.5 in
Base plate width perpendicular to the pier cap (W_b)	19.0 in



11. Reinforcement Development

Horizontal length available (L_d)	**26 in**

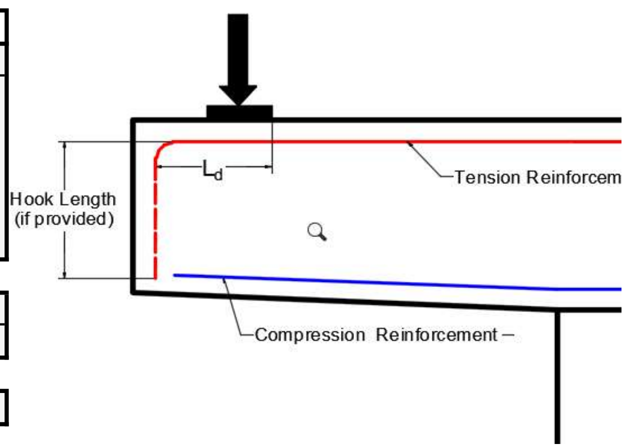
Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	**19 in**

It qualifies for 90° hook.

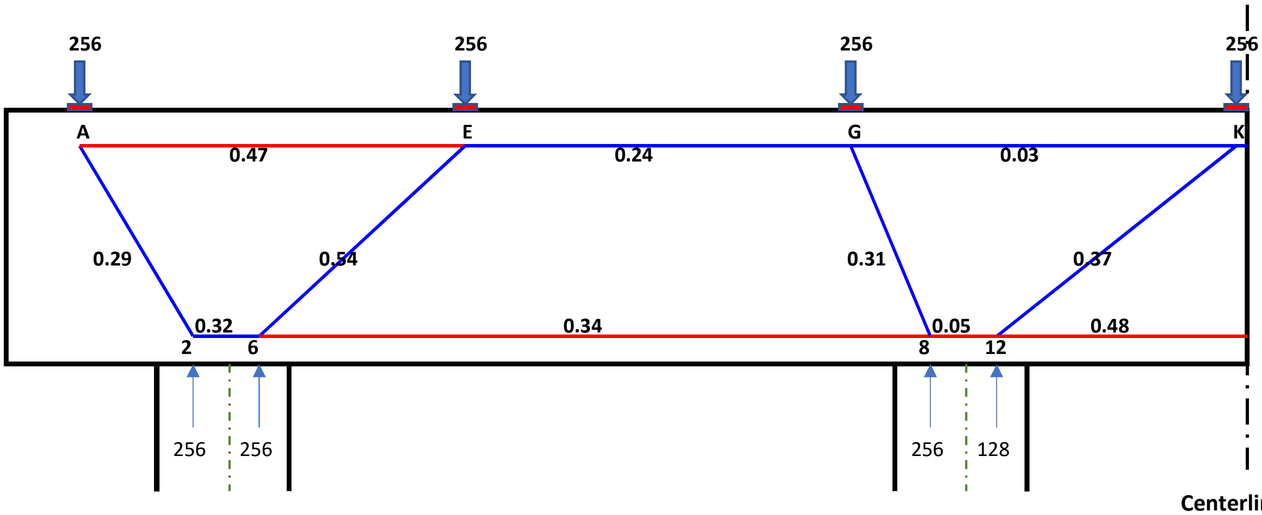
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**

Required development length	**23 in**
Available development length (L_d)	**26 in**

Reinforcement Capacity Multiplier:	**1.00**



Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

<u>**STM Members**</u>

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result
	Summary				
	A-E	201	432	0.47	PASS
	E-G	-164	-680	0.24	PASS
	G-K	-23	-771	0.03	PASS
	2-6	-201	-635	0.32	PASS
	6-8	164	486	0.34	PASS
	8-12	23	486	0.05	PASS
	12-14	235	486	0.48	PASS
	B-1	-	-	0.00	-
	F-5	-	-	0.00	-
	H-7	-	-	0.00	-
	L-11	-	-	0.00	-
0	A-2	-326	-1104	0.29	PASS
0	E-6	-446	-820	0.54	PASS
0	G-8	-293	-945	0.31	PASS
0	K-12	-248	-670	0.37	PASS
<u>**Bearing Areas**</u> Nodes at ⇉	A	256	459	0.56	PASS
	E	256	459	0.56	PASS
	G	256	520	0.49	PASS
	K	256	520	0.49	PASS
	2	256	1212	0.21	PASS
	6	256	1069	0.24	PASS
	8	256	1235	0.21	PASS
	12	128	618	0.21	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

Code Required Min skin reinforcement					0.30%
Region	***Area of the Crack Control Rebar (in ^{2})***	***Spacing of Crack Control Rebar (in)***	***No of layers of Crack Control Rebars***	***Spacing between skin bars***	***Crack Control Reinforcem ent***
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%
Region 6	0.31	8.0	2	Good	0.22%

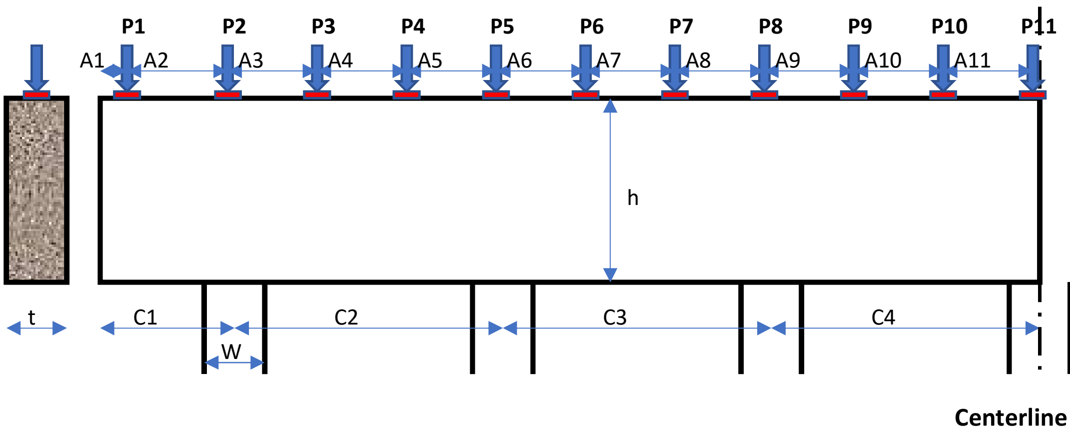
BRIDGE PIER CAP 5

Analysis Input

Bridge Details:

Bridge Name:	Bridge 5	Pier Number:	Pier 4
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	7

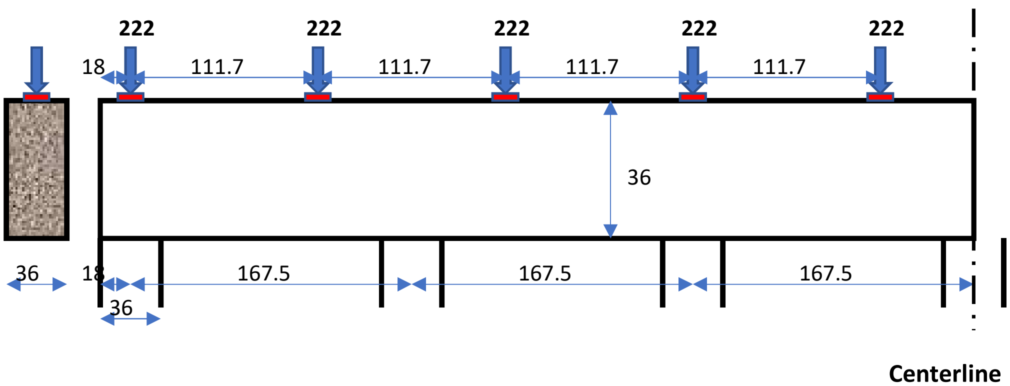


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	1 ft	6 in	**18 in**
Distance from center of first column to center of second column (C2)	13 ft	12 in	**168 in**
Distance from center of second column to center of third column (C3)	13 ft	12 in	**168 in**
Distance from center of third column to center of fourth column (C4)	13 ft	12 in	**168 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	36 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft	6 in	18 in
Spacing between the girders	9 ft	4 in	112 in
Factored Load	222 k		

Factored Load		*Distance*			
P1	0 k	0 ft	0.0 in	0.0 in	A1
P2	222 k	1 ft	6.0 in	18.0 in	A2
P3	222 k	9 ft	3.7 in	111.7 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	222 k	9 ft	3.7 in	111.7 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	222 k	9 ft	3.7 in	111.7 in	A8
P9	222 k	9 ft	3.7 in	111.7 in	A9
P10	0 k	0 ft	0.0 in	0.0 in	A10
P11	0 k	0 ft	0.0 in	0.0 in	A11

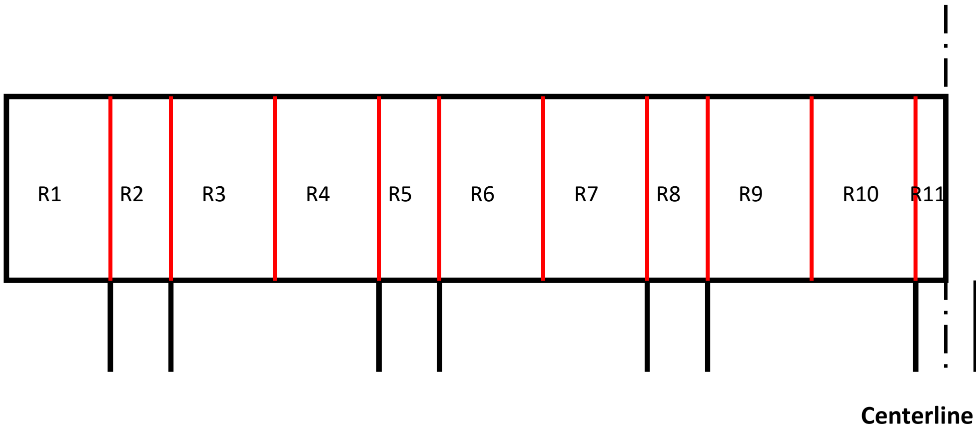


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	0.00 in	0.00	Zero Region
R2	4.50 in	0.14	Deep Region
R3	98.20 in	3.03	Slender Region
R4	46.80 in	1.44	Deep Region
R5	0.00 in	0.00	Zero Region
R6	47 in	1.45	Deep Region
R7	97 in	3.00	Slender Region
R8	0 in	0.00	Deep Region
R9	97 in	3.01	Slender Region
R10	47 in	1.44	Deep Region
R11	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

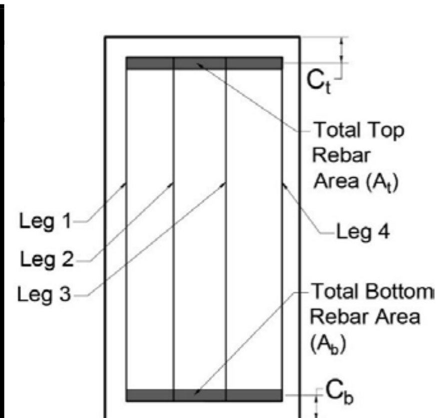
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	0.79 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



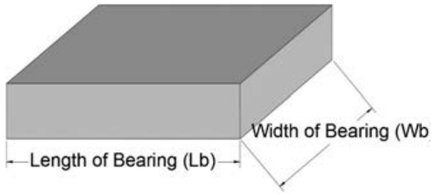
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	7.9	4.2	7.9	4.2
R2	7.9	4.2	7.9	4.2
R3	7.9	4.2	7.9	4.2
R4	7.9	4.2	7.9	4.2
R5	7.9	4.2	7.9	4.2
R6	7.9	4.2	7.9	4.2
R7	7.9	4.2	7.9	4.2
R8	7.9	4.2	7.9	4.2
R9	7.9	4.2	7.9	4.2
R10	7.9	4.2	7.9	4.2
R11	7.9	4.2	7.9	4.2



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	0	0 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	20 in
R8	4	20 in
R9	4	20 in
R10	4	18 in
R11	4	18 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	19.0 in
Base plate width perpendicular to the pier cap (W_b)	12.0 in



11. Reinforcement Development

Horizontal length available	**25 in**

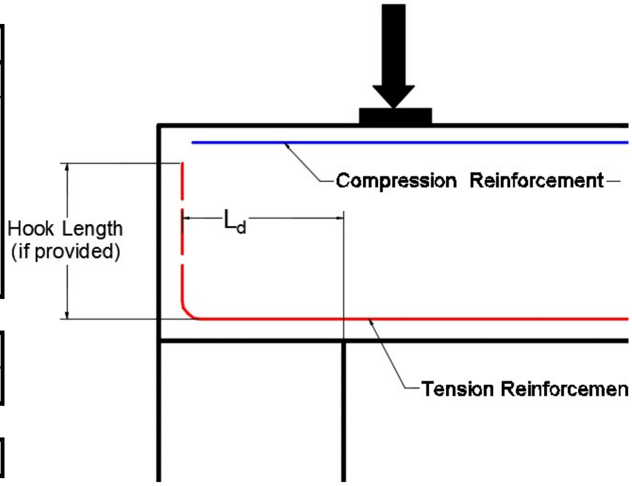
Bottom Tension Bars	
Enter the Diameter of the Bottom longitudinal bar:	0.79 in
Enter the Length of the hook Provided:	27 in
Basic Development Length	**15 in**

It qualifies for 90° hook.

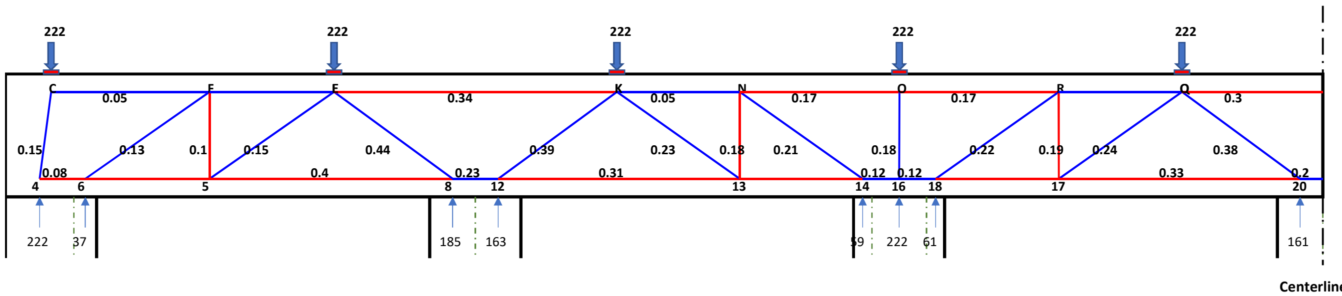
Modification Factor		
1. Are those bars epoxy coated?	No	**1**
2. Is the side cover for No. 11 Bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**

Required development length	**15 in**
Available development length (L_d)	**25 in**

Reinforcement Capacity Multiplier:	**1.00**



Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Tie Option	Member Code	Load (k)	Capacity (k)	Utilization Ratio	Result
		Summary			
	C-F	-36	-720	0.05	PASS
	E-K	144	427	0.34	PASS
	K-N	-29	-550	0.05	PASS
	N-O	74	427	0.17	PASS
	O-R	73	427	0.17	PASS
	Q-W	130	427	0.30	PASS
	4-6	36	427	0.08	PASS
	5-8	169	427	0.40	PASS
	8-12	-144	-635	0.23	PASS
	12-13	133	427	0.31	PASS
	14-16	-74	-635	0.12	PASS
	16-18	-73	-635	0.12	PASS
	17-20	142	427	0.33	PASS
	20-24	-130	-635	0.20	PASS
1	F-5	37	365	0.10	PASS
0	H-7	-	-	0.00	-
0	L-11	-	-	0.00	-
1	N-13	59	325	0.18	PASS
1	R-17	61	326	0.19	PASS
0	T-19	-	-	0.00	-
	C-4	-225	-1520	0.15	PASS
	F-6	-76	-566	0.13	PASS
	E-5	-76	-515	0.15	PASS
	E-8	-364	-817	0.44	PASS
	K-12	-322	-829	0.39	PASS
	K-13	-119	-521	0.23	PASS
	N-14	-119	-557	0.21	PASS
	O-16	-222	-1247	0.18	PASS
	R-18	-124	-555	0.22	PASS
	Q-17	-124	-514	0.24	PASS
	Q-20	-316	-835	0.38	PASS
	C	222	543	0.41	PASS
	E	222	543	0.41	PASS
	K	222	479	0.46	PASS
	O	222	415	0.53	PASS
	Q	222	479	0.46	PASS
	4	222	1830	0.12	PASS
	6	37	267	0.14	PASS
	8	185	1135	0.16	PASS
	12	163	1003	0.16	PASS
	14	59	368	0.16	PASS
	16	222	1573	0.14	PASS
	18	61	382	0.16	PASS
	20	161	1069	0.15	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

Region	Area of the Crack Control Rebar (in^2)	Spacing of Crack Control Rebar (in)	No of layers of Crack Control Rebars	Spacing between skin bars	Crack Control Reinforcement
				Code Required Min skin reinforcement	0.30%
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%
Region 6	0.31	8.0	2	Good	0.22%
Region 7	0.31	6.5	2	Good	0.26%
Region 8	0.31	6.5	2	Good	0.26%
Region 9	0.31	6.5	2	Good	0.26%
Region 10	0.31	6.5	2	Good	0.26%
Region 11	0.31	6.5	2	Good	0.26%

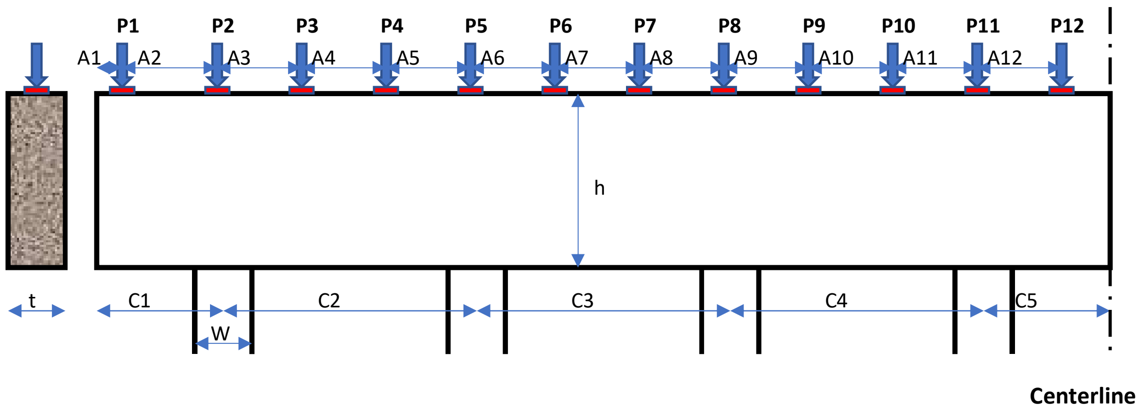
BRIDGE PIER CAP 6

Analysis Input

Bridge Details:

Bridge Name:	Bridge 6	Pier Number:	Pier-2 Left
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	8	☑ Unsymmetrical

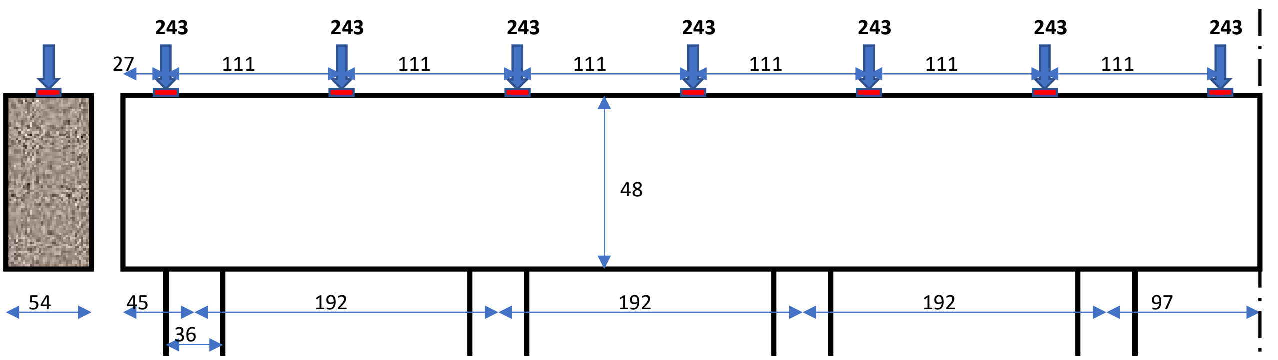


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	3 ft	9 in	**45 in**
Distance from center of first column to center of second column (C2)	16 ft	0 in	**192 in**
Distance from center of second column to center of third column (C3)	16 ft	0 in	**192 in**
Distance from center of third column to center of fourth column (C4)	16 ft	0 in	**192 in**
Distance from center of fourth column to centerline of pier cap (C5)	8 ft	1 in	**97 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	54 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	3 in	27 in
Spacing between the girders	9 ft	3 in	111 in
Factored Load	243 k		

Factored Load		*Distance*			
P1	243 k	2 ft	3.0 in	27.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	243 k	9 ft	3.0 in	111.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	243 k	9 ft	3.0 in	111.0 in	A5
P6	243 k	9 ft	3.0 in	111.0 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	0 k	0 ft	0.0 in	0.0 in	A8
P9	243 k	9 ft	3.0 in	111.0 in	A9
P10	243 k	9 ft	3.0 in	111.0 in	A10
P11	0 k	0 ft	0.0 in	0.0 in	A11
P12	243 k	9 ft	3.0 in	111.0 in	A12

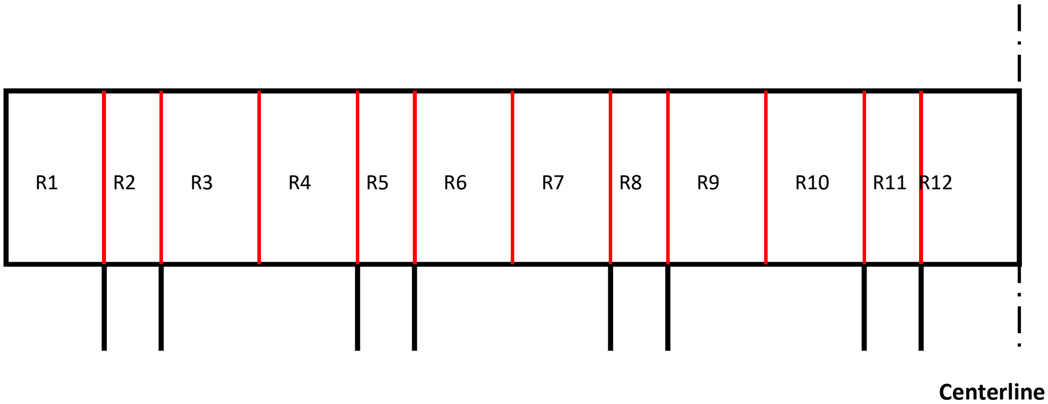


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	11.88 in	0.27	Deep Region
R2	0.00 in	0.00	Zero Region
R3	81.12 in	1.88	Deep Region
R4	85.73 in	1.98	Deep Region
R5	10.78 in	0.25	Deep Region
R6	109 in	2.51	Slender Region
R7	58 in	1.34	Deep Region
R8	0 in	0.00	Zero Region
R9	17 in	0.39	Deep Region
R10	30 in	0.70	Deep Region
R11	0 in	0.00	Zero Region
R12	45 in	1.04	Deep Region

This pier cap is deep.
Please continue with Section 7.

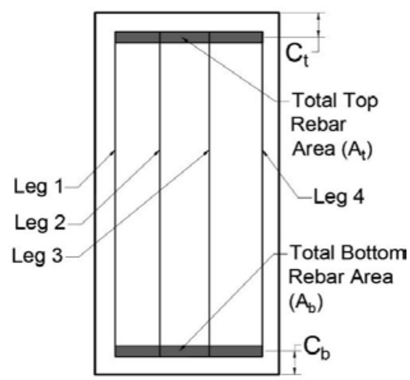
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.27 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



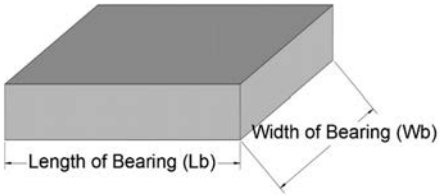
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	Top Steel (in^2, in)		Bottom Steel (in^2, in)	
	Total Area (A_t)	Centroid (C_t)	Total Area (A_b)	Centroid (C_b)
R1	22.86	5.5	11.43	3
R2	22.86	5.5	11.43	3
R3	22.86	5.5	11.43	3
R4	22.86	5.5	11.43	3
R5	22.86	5.5	11.43	3
R6	22.86	5.5	11.43	3
R7	22.86	5.5	11.43	3
R8	22.86	5.5	11.43	3
R9	22.86	5.5	11.43	3
R10	22.86	5.5	11.43	3
R11	22.86	5.5	11.43	3
R12	22.86	5.5	11.43	3



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	0	0 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	20 in
R8	4	20 in
R9	4	20 in
R10	4	18 in
R11	4	18 in
R12	0	0 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	13.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in



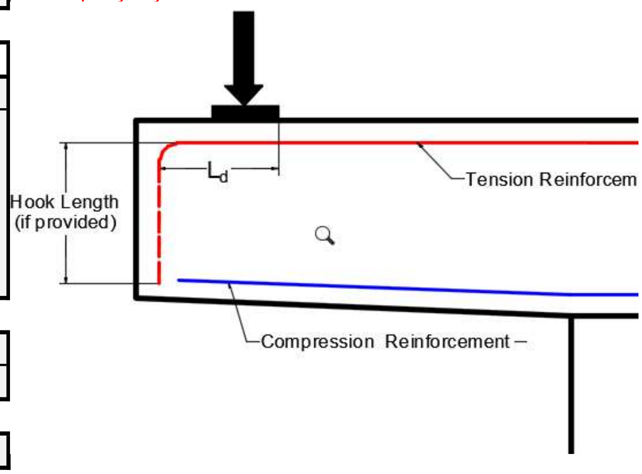
11. Reinforcement Development

Horizontal length available (L_d)	**32 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	**24 in**

It qualifies for 90° hook.

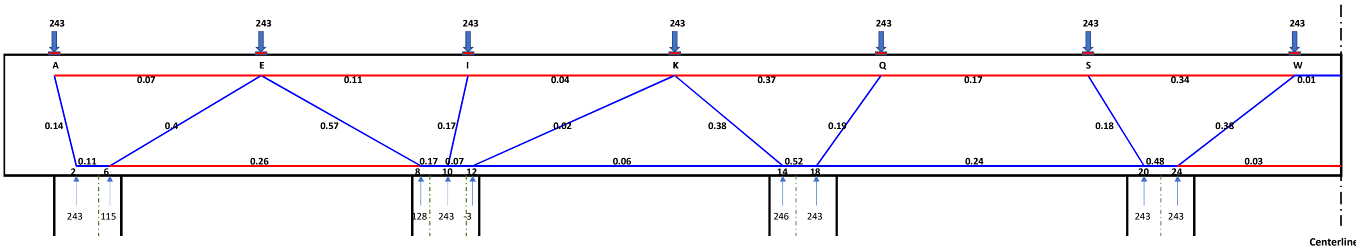
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**



Required development length	**29 in**
Available development length (L_d)	**32 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary				
	Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***
	A-E	73	1081	0.07	PASS
	E-I	114	1081	0.11	PASS
	I-K	48	1081	0.04	PASS
	K-Q	401	1081	0.37	PASS
	Q-S	187	1081	0.17	PASS
	S-W	372	1081	0.34	PASS
	W+	-16	-1247	0.01	PASS
	2-6	-73	-680	0.11	PASS
	6-8	164	617	0.26	PASS
	8-10	-114	-680	0.17	PASS
	10-12	-48	-680	0.07	PASS
	12-14	-40	-680	0.06	PASS
	14-18	-401	-771	0.52	PASS
	18-20	-187	-771	0.24	PASS
	20-24	-372	-771	0.48	PASS
	24+	16	617	0.03	PASS
	B-1	-	-	0.00	-
	F-5	-	-	0.00	-
	H-7	-	-	0.00	-
	J-9	-	-	0.00	-
	L-11	-	-	0.00	-
	N-13	-	-	0.00	-
	R-17	-	-	0.00	-
	T-19	-	-	0.00	-
	X-23	-	-	0.00	-
0	A-2	-254	-1771	0.14	PASS
0	E-6	-263	-663	0.40	PASS
0	E-8	-305	-539	0.57	PASS
0	I-10	-252	-1517	0.17	PASS
0	K-12	-8	-351	0.02	PASS
0	K-14	-437	-1141	0.38	PASS
0	Q-18	-324	-1673	0.19	PASS
0	S-20	-305	-1671	0.18	PASS
0	W-24	-457	-1203	0.38	PASS
Bearing Areas Nodes at ⇉	A	243	573	0.42	PASS
	E	243	497	0.49	PASS
	I	243	497	0.49	PASS
	K	243	497	0.49	PASS
	Q	243	497	0.49	PASS
	S	243	497	0.49	PASS
	W	243	573	0.42	PASS
	2	243	1644	0.15	PASS
	6	115	688	0.17	PASS
	8	128	743	0.17	PASS
	10	243	1600	0.15	PASS
	12	-3	-16	0.17	PASS
	14	246	1219	0.20	PASS
	18	243	1204	0.20	PASS
	20	243	1212	0.20	PASS
	24	243	1212	0.20	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

Region	Area of the Crack Control Rebar (in^2)	Spacing of Crack Control Rebar (in)	No of layers of Crack Control Rebars	Spacing between skin bars	Crack Control Reinforcement
				Code Required Min skin reinforcement	0.30%
Region 1	0.31	8.0	2	Good	0.14%
Region 2	0.31	8.0	2	Good	0.14%
Region 3	0.31	8.0	2	Good	0.14%
Region 4	0.31	8.0	2	Good	0.14%
Region 5	0.31	8.0	2	Good	0.14%
Region 6	0.31	8.0	2	Good	0.14%
Region 7	0.31	6.5	2	Good	0.18%
Region 8	0.31	6.5	2	Good	0.18%
Region 9	0.31	6.5	2	Good	0.18%
Region 10	0.31	6.5	2	Good	0.18%
Region 11	0.31	6.5	2	Good	0.18%
Region 12	0.31	6.5	4	Good	0.35%

BRIDGE PIER CAP 7

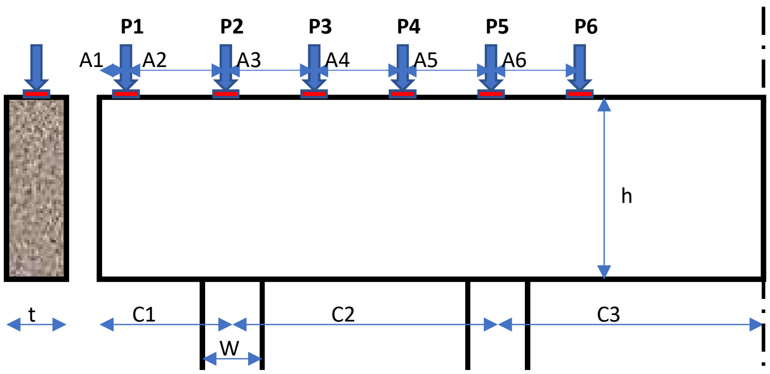
Analysis Input

Bridge Details:

Bridge Name:	Bridge 7	Pier Number:	Southbound (Left)
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Unsymmetrical



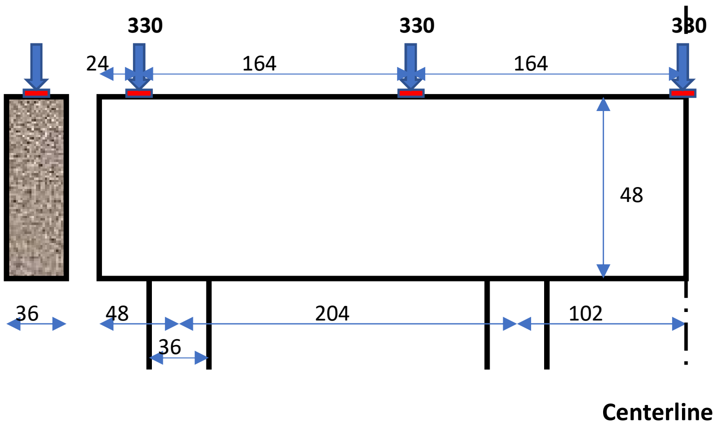
Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	4 ft	0 in	**48 in**
Distance from center of first column to center of second column (C2)	17 ft	0 in	**204 in**
Distance from center of second column to centerline of pier cap (C3)	8 ft	6 in	**102 in**
Column width (W)	36 in	**Square**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	0 in	**24 in**
Spacing between the girders	13 ft	8 in	**164 in**
Factored Load	330 k		

Factored Load		***Distance***			
P1	330 k	2 ft	0.0 in	**24.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	330 k	13 ft	8.0 in	**164.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5
P6	330 k	13 ft	8.0 in	**164.0 in**	A6

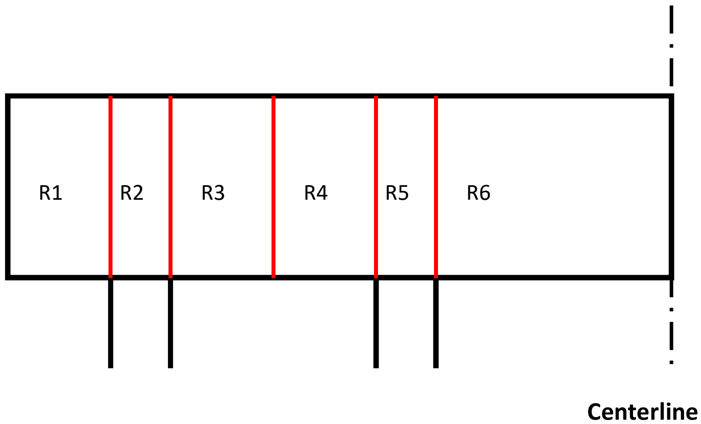


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	19.70 in	0.46	Deep Region
R2	0.00 in	0.00	Zero Region
R3	126.30 in	2.92	Slender Region
R4	53.33 in	1.23	Deep Region
R5	0.00 in	0.00	Zero Region
R6	93 in	2.15	Deep Region

This pier cap is deep.
Please continue with Section 7.

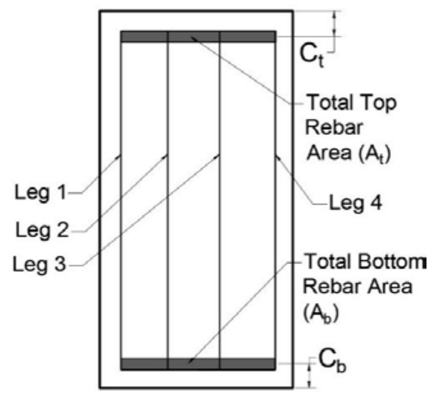
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



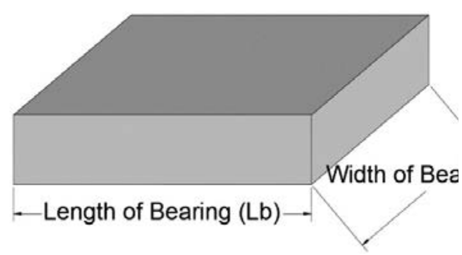
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in², in)***		***Bottom Steel (in², in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	12	5	12	5
R2	12	5	12	5
R3	12	5	12	5
R4	12	5	12	5
R5	12	5	12	5
R6	12	5	12	5



9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	18 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	16.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in

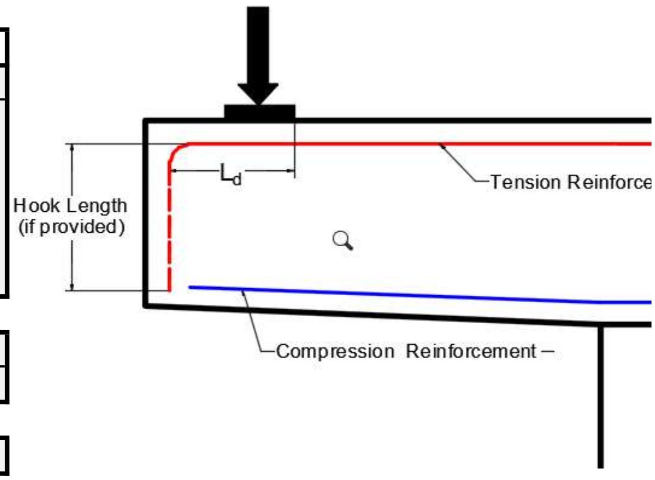


11. Reinforcement Development

Horizontal length available (L_d)	31 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	19 in

It qualifies for 90° hook.

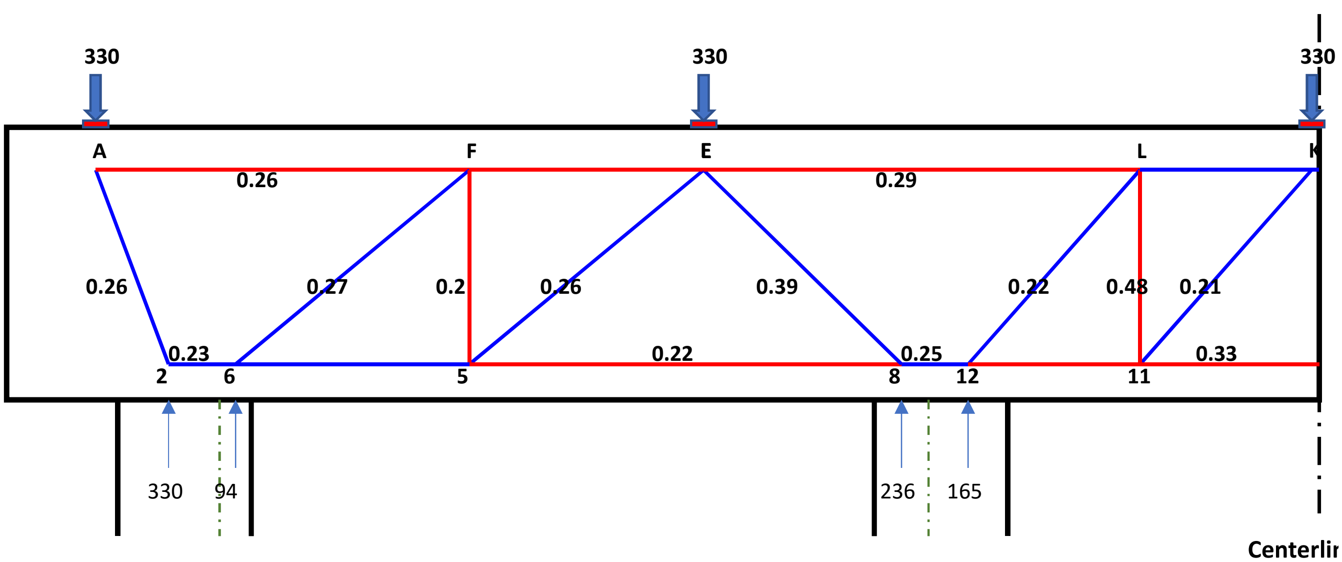


Modification Factor		
1. Are bars epoxy coated?	No	1
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	19 in
Available development length (L_d)	31 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary				
	Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***
	A-F	171	648	0.26	PASS
	E-L	189	648	0.29	PASS
	2-6	-171	-756	0.23	PASS
	5-8	142	648	0.22	PASS
	8-12	-189	-756	0.25	PASS
	11-14	214	648	0.33	PASS
	B-1	-	-	0.00	-
	F-5	94	470	0.20	PASS
	H-7	-	-	0.00	-
	L-11	165	345	0.48	PASS
0	A-2	-372	-1422	0.26	PASS
1	F-6	-183	-686	0.27	PASS
	E-5	-183	-701	0.26	PASS
0	E-8	-406	-1044	0.39	PASS
1	L-12	-260	-1171	0.22	PASS
	K-11	-260	-1246	0.21	PASS
Bearing Areas Nodes at ⇉	A	330	706	0.47	PASS
	E	330	706	0.47	PASS
	K	330	706	0.47	PASS
	2	330	2399	0.14	PASS
	6	94	604	0.16	PASS
	8	236	1601	0.15	PASS
	12	165	1120	0.15	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

Code Required Min skin reinforcement					0.30%
Region	***Area of the Crack Control Rebar (in^2)***	***Spacing of Crack Control Rebar (in)***	***No of layers of Crack Control Rebars***	***Spacing between skin bars***	***Crack Control Reinforcement***
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%
Region 6	0.31	8.0	2	Good	0.22%

BRIDGE PIER CAP 8

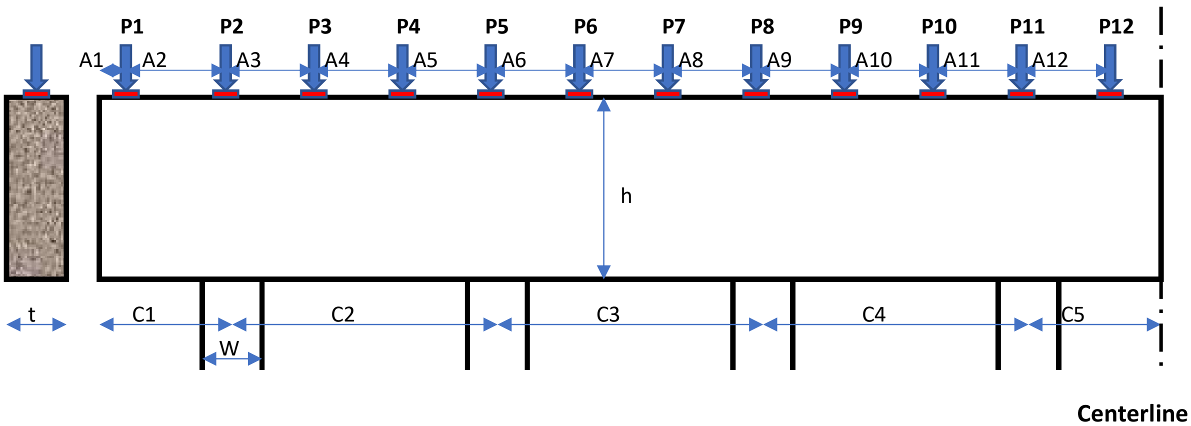
Analysis Input

Bridge Details:

Bridge Name:	Bridge 8	Pier Number:	Southbound (Left)
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	8

☑ Unsymmetrical

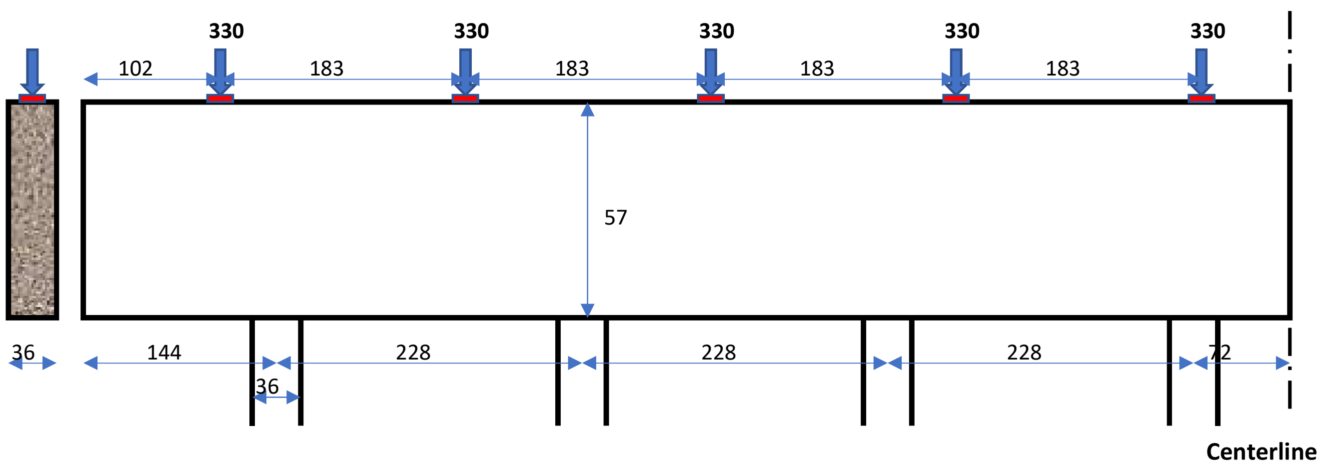


Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	12 ft	0 in	**144 in**
Distance from center of first column to center of second column (C2)	19 ft	0 in	**228 in**
Distance from center of second column to center of third column (C3)	19 ft	0 in	**228 in**
Distance from center of third column to center of fourth column (C4)	19 ft	0 in	**228 in**
Distance from center of fourth column to centerline of pier cap (C5)	6 ft	0 in	**72 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	57 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	8 ft	6 in	102 in
Spacing between the girders	15 ft	3 in	183 in
Factored Load	330 k		

Factored Load		Distance			
P1	330 k	8 ft	6.0 in	102.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	330 k	15 ft	3.0 in	183.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	330 k	15 ft	3.0 in	183.0 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	0 k	0 ft	0.0 in	0.0 in	A8
P9	330 k	15 ft	3.0 in	183.0 in	A9
P10	0 k	0 ft	0.0 in	0.0 in	A10
P11	330 k	15 ft	3.0 in	183.0 in	A11
P12	0 k	0 ft	0.0 in	0 in	A12

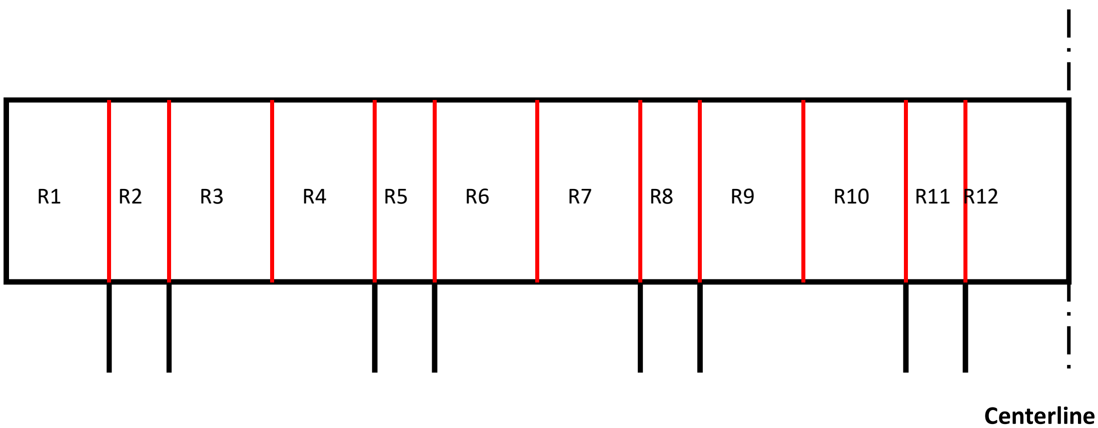


6. Check whether the Pier Cap is Deep			
Region	**Shear span (a)**	**a/d ratio:**	**Result**
R1	37.03 in	0.72	Deep Region
R2	0.00 in	0.00	Zero Region
R3	127.97 in	2.49	Deep Region
R4	78.30 in	1.53	Deep Region
R5	0.00 in	0.00	Zero Region
R6	87 in	1.69	Deep Region
R7	120 in	2.35	Deep Region
R8	0 in	0.00	Zero Region
R9	45 in	0.87	Deep Region
R10	162 in	3.16	Slender Region
R11	3 in	0.05	Deep Region
R12	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

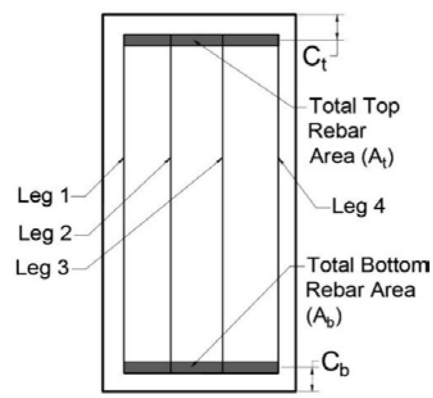
7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC Node multiplier	0.85
CCT Node multiplier	0.75
CTT Node multiplier	0.65



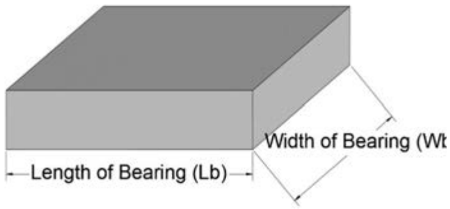
9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	12	5	12	5
R2	12	5	12	5
R3	12	5	12	5
R4	12	5	12	5
R5	12	5	12	5
R6	12	5	12	5
R7	12	5	12	5
R8	12	5	12	5
R9	12	5	12	5
R10	12	5	12	5
R11	12	5	12	5
R12	12	5	12	5



9B. Transverse Reinforcement		
Region	**No. of Legs**	**Stirrup Spacing**
R1	4	18 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	18 in
R8	4	18 in
R9	4	18 in
R10	4	18 in
R11	4	18 in
R12	4	18 in

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	16.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in



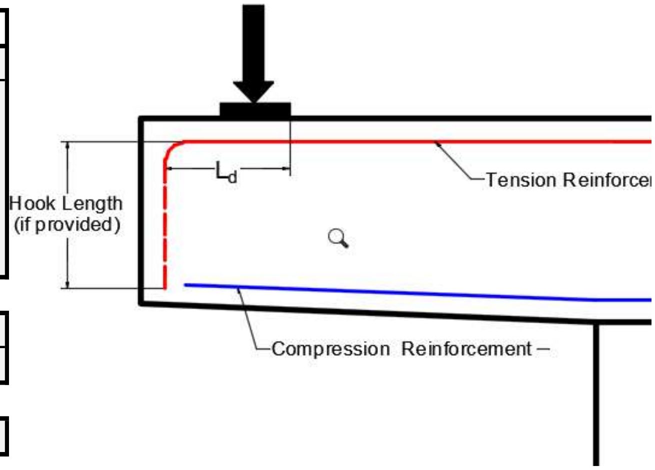
11. Reinforcement Development

Horizontal length available (L_d)	**110 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	**19 in**

It qualifies for 90° hook.

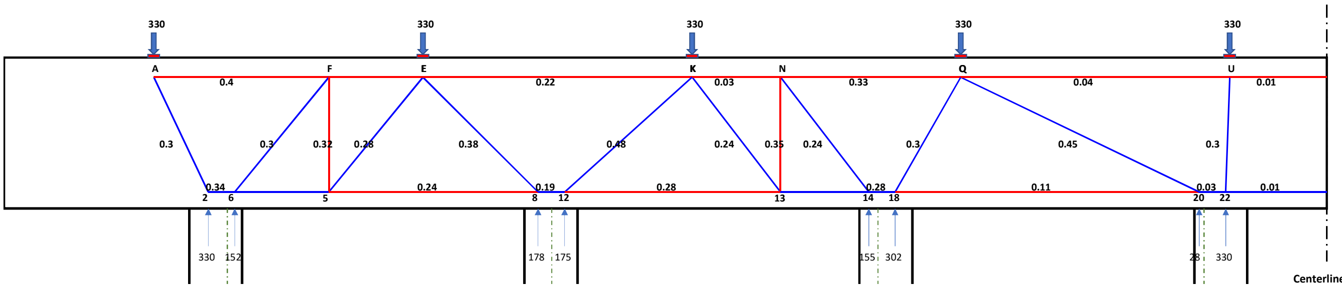
Modification Factor		
1. Are bars epoxy coated?	No	**1**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**



Required development length	**19 in**
Available development length (L_d)	**110 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary				
	Member Code	***Load (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***
	A-F	260	648	0.40	PASS
	E-K	142	648	0.22	PASS
	K-N	17	648	0.03	PASS
	N-Q	215	648	0.33	PASS
	Q-U	25	648	0.04	PASS
	U-W	6	648	0.01	PASS
	2-6	-260	-756	0.34	PASS
	5-8	154	648	0.24	PASS
	8-12	-142	-756	0.19	PASS
	12-13	181	648	0.28	PASS
	14-18	-215	-756	0.28	PASS
	18-20	72	648	0.11	PASS
	20-22	-25	-756	0.03	PASS
	22-24	-6	-857	0.01	PASS
	B-1	-	-	0.00	-
	F-5	152	476	0.32	PASS
	H-7	-	-	0.00	-
	L-11	-	-	0.00	-
	N-13	155	448	0.35	PASS
	R-17	-	-	0.00	-
	T-19	-	-	0.00	-
	V-21	-	-	0.00	-
0	A-2	-420	-1418	0.30	PASS
1	F-6	-257	-853	0.30	PASS
	E-5	-257	-914	0.28	PASS
0	E-8	-346	-899	0.38	PASS
0	K-12	-368	-769	0.48	PASS
1	K-13	-251	-1027	0.24	PASS
	N-14	-251	-1027	0.24	PASS
0	Q-18	-417	-1398	0.30	PASS
0	Q-20	-101	-227	0.45	PASS
0	U-22	-331	-1084	0.30	PASS
Bearing Areas Nodes at ⇉	A	330	706	0.47	PASS
	E	330	706	0.47	PASS
	K	330	706	0.47	PASS
	Q	330	706	0.47	PASS
	U	330	612	0.54	PASS
	2	330	1659	0.20	PASS
	6	152	674	0.23	PASS
	8	178	1077	0.17	PASS
	12	175	1061	0.17	PASS
	14	155	724	0.21	PASS
	18	302	1414	0.21	PASS
	20	28	168	0.17	PASS
	22	330	2233	0.15	PASS

14. Informational Check: Min Horizontal Crack Control Reinforcement

				Code Required Min skin reinforcement	0.30%
Region	***Area of the Crack Control Rebar (in ^{2})***	***Spacing of Crack Control Rebar (in)***	***No of layers of Crack Control Rebars***	***Spacing between skin bars***	***Crack Control Reinforcement***
Region 1	0.31	8.0	2	Good	0.22%
Region 2	0.31	8.0	2	Good	0.22%
Region 3	0.31	8.0	2	Good	0.22%
Region 4	0.31	8.0	2	Good	0.22%
Region 5	0.31	8.0	2	Good	0.22%
Region 6	0.31	8.0	2	Good	0.22%
Region 7	0.31	6.5	2	Good	0.26%
Region 8	0.31	6.5	2	Good	0.26%
Region 9	0.31	6.5	2	Good	0.26%
Region 10	0.31	6.5	2	Good	0.26%
Region 11	0.31	6.5	2	Good	0.26%
Region 12	0.31	6.5	4	Good	0.53%

Appendix B: STM-CAP Solved Examples (AASHTO LRFD 2017)

BRIDGE PIER CAP 1

Analysis Input

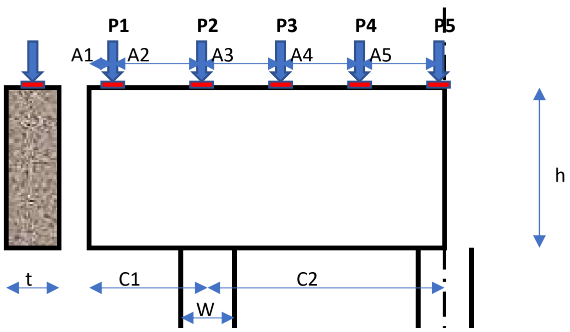
Bridge Details:

Bridge Name:	Bridge 1	Pier Number:	Pier 2-Left
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	3

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

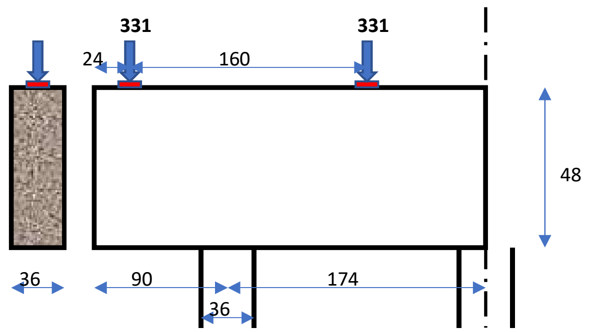
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	7 ft	6.0 in	**90.0 in**
Distance from center of first column to center of second column (C2)	14 ft	6.0 in	**174.0 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	0.0 in	**24.0 in**
Spacing Between the Girders	13 ft	4.0 in	**160.0 in**
Factored Load	331 k		

Generate Load Table

Factored Load		***Distance***			
P1	331 k	2 ft	0.0 in	**24.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	331 k	13 ft	4.0 in	**160.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5

5. Generate

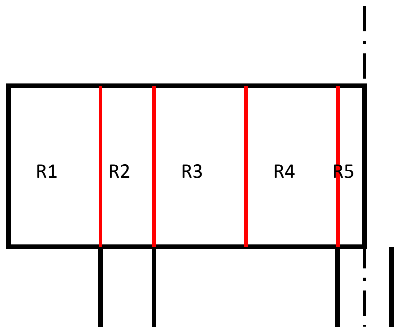


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	60.3 in	1.40	Deep Region
R2	0.0 in	0.00	Zero Region
R3	81.7 in	1.89	Deep Region
R4	71.0 in	1.64	Deep Region
R5	0.0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.27 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	0.65



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	Top Steel (in^2, in)		Bottom Steel (in^2, in)	
	Total Area (A_t)	Centroid (C_t)	Total Area (A_b)	Centroid (C_b)
R1	13.97	6	7	4.5
R2	13.97	6	7	4.5
R3	13.97	6	7	4.5
R4	13.97	6	7	4.5
R5	13.97	6	7	4.5

9B. Transverse Reinforcement		
Region	No. of Legs	Stirrup Spacing
R1	4	5 in
R2	0	0 in
R3	4	10 in
R4	2	12 in
R5	0	0 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.44
Spacing (in)	6.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	0.41%

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	13.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in

11. Reinforcement Development

Horizontal length available (L_d)	33 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	24 in

It qualifies for 90° hook.

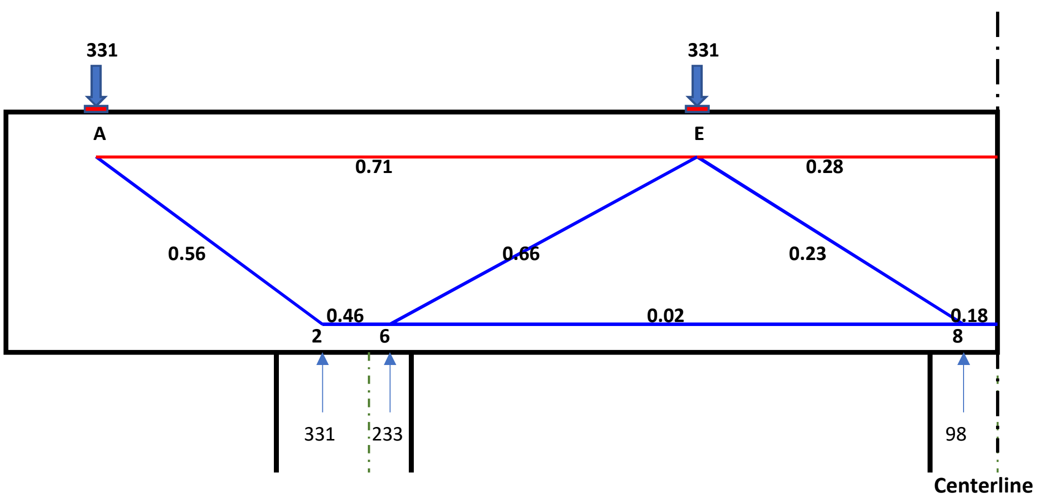
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	29 in
Available development length (L_d)	33 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

		Summary					
		Member Code	***Force (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***	
		A-E	533	754	0.71	PASS	**Top Members**
		E-K	210	754	0.28	PASS	
		2-6	-533	-1149	0.46	PASS	**Bottom Members**
		6-8	-25	-1149	0.02	PASS	
		8-12	-210	-1149	0.18	PASS	
Input 0 for "Do not use Tie"		B-1	0	-	0.00	-	**Vertical Members**
Input 1 for "Use Tie"		F-5	0	-	0.00	-	
Input Your Option Down Here		H-7	0	-	0.00	-	
	0	A-2	-627	-1117	0.56	PASS	**Inclined Members**
	0	E-6	-559	-846	0.66	PASS	
	0	E-8	-210	-910	0.23	PASS	
Bearing Areas	Nodes at ⇉	A	331	1028	0.32	**PASS**	**1**
		E	331	**955**	0.35	**PASS**	
		2	331	**1422**	0.23	**PASS**	**2**
		6	233	**1001**	0.23	**PASS**	
		8	98	**1212**	0.08	**PASS**	

Re-Generate Output Model

BRIDGE PIER CAP 2

Analysis Input

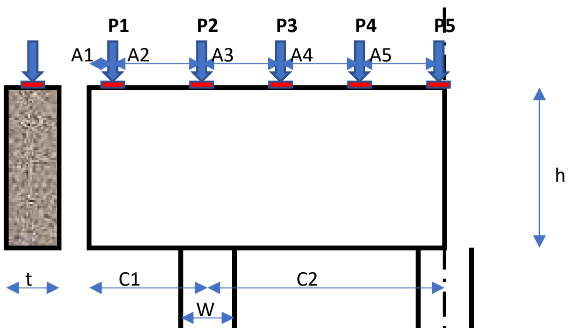
Bridge Details:

Bridge Name:	Bridge 2	Pier Number:	Pier 2
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	3

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

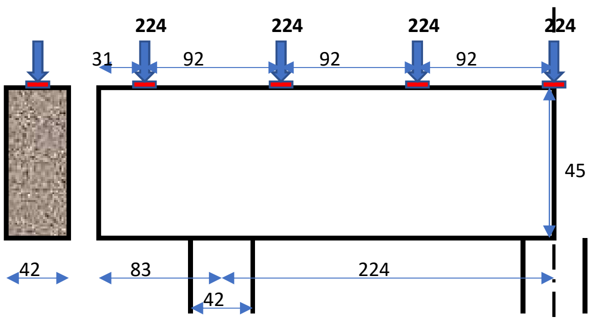
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	6 ft	11.0 in	**83.0 in**
Distance from center of first column to center of second column (C2)	18 ft	8.0 in	**224.0 in**
Column width (W)	42 in	**Circular**	
Depth of pier cap (h)	45 in		
Thickness of pier cap (t)	42 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	7.0 in	**31.0 in**
Spacing Between the Girders	7 ft	8.0 in	**92.0 in**
Factored Load	224 k		

Generate Load Table

Factored Load		***Distance***			
P1	224 k	2 ft	7.0 in	**31.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	224 k	7 ft	8.0 in	**92.0 in**	A3
P4	224 k	7 ft	8.0 in	**92.0 in**	A4
P5	224 k	7 ft	8.0 in	**92.0 in**	A5

5. Generate

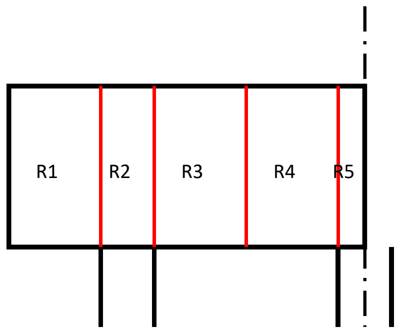


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	**40.4 in**	**1.00**	**Deep Region**
R2	**0.0 in**	**0.00**	**Zero Region**
R3	**30.6 in**	**0.76**	**Deep Region**
R4	**77.4 in**	**1.91**	**Deep Region**
R5	**4.1 in**	**0.10**	**Deep Region**

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	**0.65**



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	9.46	4.45	8	3.15
R2	9.46	4.45	8	3.15
R3	9.46	4.45	8	3.15
R4	9.46	4.45	8	3.15
R5	9.46	4.45	8	3.15

9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	10 in
R2	4	10 in
R3	4	10 in
R4	4	10 in
R5	4	10 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.31
Spacing (in)	6.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.25%**

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	20.0 in
Base plate width perpendicular to the pier cap (W_b)	13.0 in

11. Reinforcement Development

Horizontal length available (L_d)	41 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	19 in

It qualifies for 90° hook.

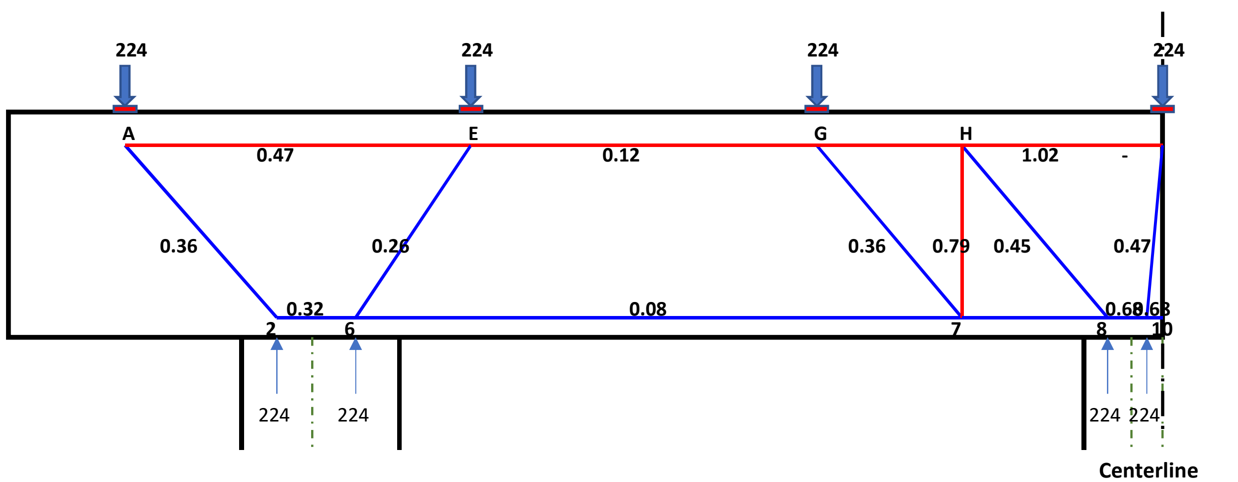
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	23 in
Available development length (L_d)	41 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	Group
	A-E	242	511	0.47	PASS	Top Members
	E-G	59	511	0.12	PASS	Top Members
	H-I	520	511	1.02	Flexure Overloaded	Top Members
	2-6	-242	-765	0.32	PASS	Bottom Members
	6-7	-59	-765	0.08	PASS	Bottom Members
	8-10	-520	-765	0.68	PASS	Bottom Members
	10-12	-520	-765	0.68	PASS	Bottom Members
Input 0 for "Do not use Tie"	B-1	0	-	0.00	-	Vertical Members
Input 1 for "Use Tie"	F-5	0	-	0.00	-	Vertical Members
Input Your Option Down Here	H-7	224	284	0.79	PASS	Vertical Members
↓↓↓↓↓↓↓	J-9	0	-	0.00	-	Vertical Members
0	A-2	-330	-921	0.36	PASS	Inclined Members
0	E-6	-289	-1117	0.26	PASS	Inclined Members
1	G-7	-324	-904	0.36	PASS	Inclined Members
	H-8	-319	-708	0.45	PASS	Inclined Members
0	I-10	-224	-472	0.47	PASS	Inclined Members
Bearing Areas — Nodes at ⇉	A	224	655	0.34	PASS	1
	E	224	655	0.34	PASS	1
	G	224	655	0.34	PASS	1
	I	224	655	0.34	PASS	1
	2	224	873	0.26	PASS	2
	6	224	873	0.26	PASS	2
	8	224	436	0.51	PASS	2
	10	224	436	0.51	PASS	2

Re-Generate Output Model

BRIDGE PIER CAP 3

Analysis Input

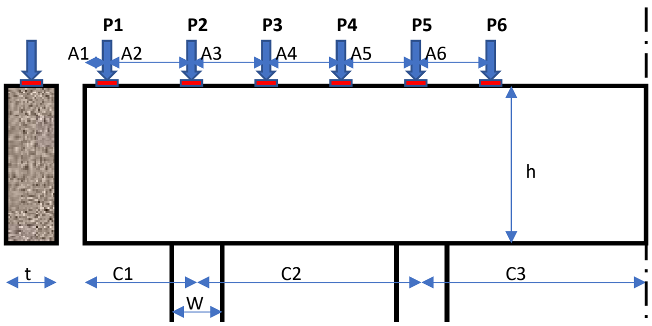
Bridge Details:

Bridge Name:	Bridge 3	Pier Number:	North Pier
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

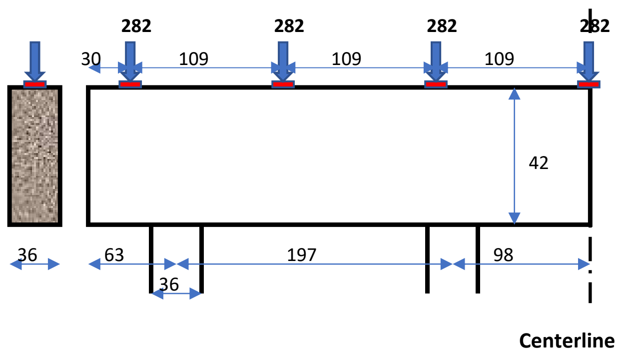
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	5 ft	3 in	**63.0 in**
Distance from center of first column to center of second column (C2)	16 ft	5 in	**197.0 in**
Distance from center of second column to centerline of pier cap (C3)	8 ft	2 in	**98.0 in**
Column width (W)	36 in	Circular	
Depth of pier cap (h)	42 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	6.0 in	**30.0 in**
Spacing Between the Girders	9 ft	1.0 in	**109.0 in**
Factored Load	282 k		

Generate Load Table

Factored Load		***Distance***			
P1	282 k	2 ft	6.0 in	**30.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	282 k	9 ft	1.0 in	**109.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	282 k	9 ft	1.0 in	**109.0 in**	A5
P6	282 k	9 ft	1.0 in	**109.0 in**	A6

5. Generate

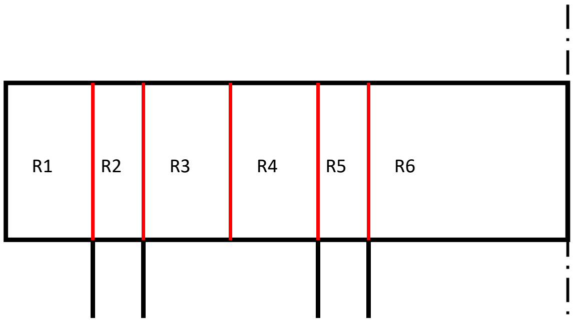


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	26.2 in	0.69	Deep Region
R2	0.0 in	0.00	Zero Region
R3	64.8 in	1.72	Deep Region
R4	105.9 in	2.80	Slender Region
R5	7.4 in	0.19	Deep Region
R6	87 in	2.29	Slender Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	0.65



9. Reinforcement Details

9A. Longitudinal Reinforcement

Region	Top Steel (in², in)		Bottom Steel (in², in)	
	Total Area (A_t)	Centroid (C_t)	Total Area (A_b)	Centroid (C_b)
R1	8	4.1	8	4.1
R2	8	4.1	8	4.1
R3	8	4.1	8	4.1
R4	8	4.1	8	4.1
R5	8	4.1	8	4.1
R6	8	4.1	8	4.1

9B. Transverse Reinforcement

Region	No. of Legs	Stirrup Spacing
R1	4	7 in
R2	0	0 in
R3	4	12 in
R4	4	12 in
R5	0	0 in
R6	4	16 in

9C. Min Horizontal Crack Control Reinforcement

Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in²)	0.31
Spacing (in)	8.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.22%**

10. Base Plate Dimensions

Base plate length parallel to the pier cap (L_b)	21.0 in
Base plate width perpendicular to the pier cap (W_b)	13.0 in

11. Reinforcement Development

Horizontal length available (L_d)	40 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	19 in

It qualifies for 90° hook.

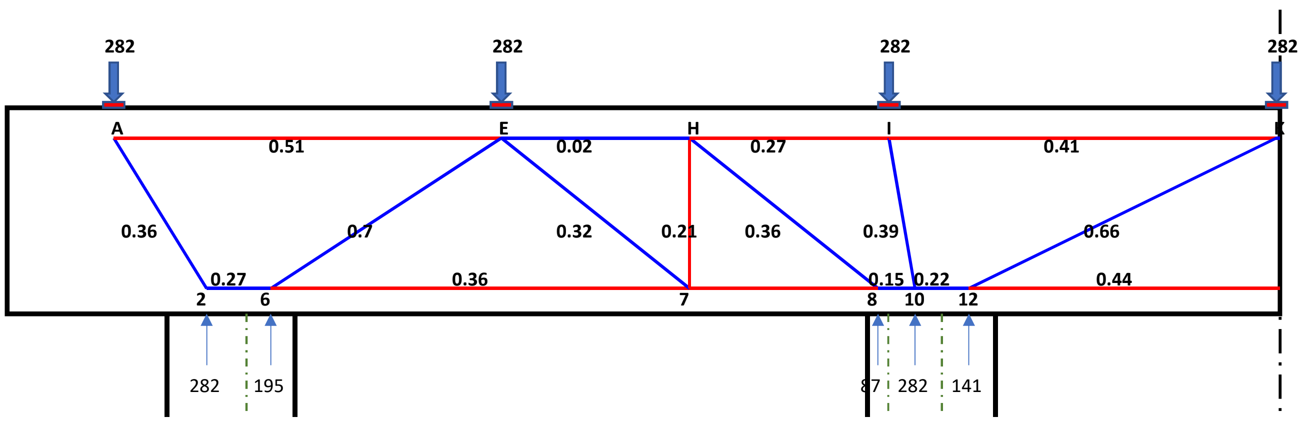
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	23 in
Available development length (L_d)	40 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-E	218	432	0.51	PASS	**Top Members**
	E-H	-19	-804	0.02	PASS	
	I-K	177	432	0.41	PASS	
	K-Q	0	432	0.00	-	
	2-6	-218	-804	0.27	PASS	**Bottom Members**
	6-7	155	432	0.36	PASS	
	8-10	-118	-804	0.15	PASS	
	10-12	-177	-804	0.22	PASS	
	12-14	189	432	0.44	PASS	
	B-1	0	-	0.00	-	**Vertical Members**
	F-5	0	-	0.00	-	
	H-7	87	415	0.21	PASS	
	J-9	0	-	0.00	-	
	L-11	0	-	0.00	-	
0	A-2	-357	-981	0.36	PASS	**Inclined Members**
0	E-6	-421	-600	0.70	PASS	
1	E-7	-162	-511	0.32	PASS	
	H-8	-162	-456	0.36	PASS	
0	I-10	-288	-748	0.39	PASS	
0	K-12	-391	-595	0.66	PASS	
Bearing Areas Nodes at ⇉	A	282	688	0.41	PASS	**1**
	E	282	688	0.41	PASS	
	I	282	688	0.41	PASS	
	K	282	688	0.41	PASS	
	2	282	759	0.37	PASS	**2**
	6	195	524	0.37	PASS	
	8	87	219	0.40	PASS	
	10	282	709	0.40	PASS	
	12	141	354	0.40	PASS	

Re-Generate Output Model

BRIDGE PIER CAP 4

Analysis Input

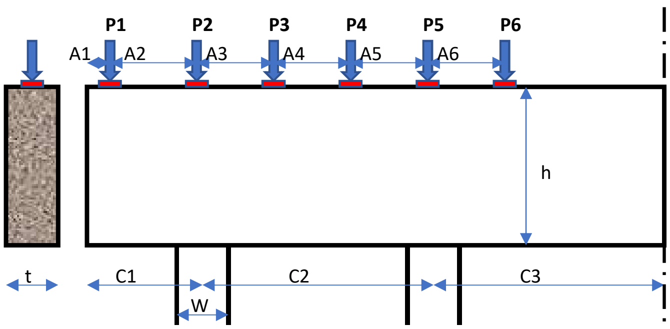
Bridge Details:

Bridge Name:	Bridge 4	Pier Number:	Left-Unsymmetric
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

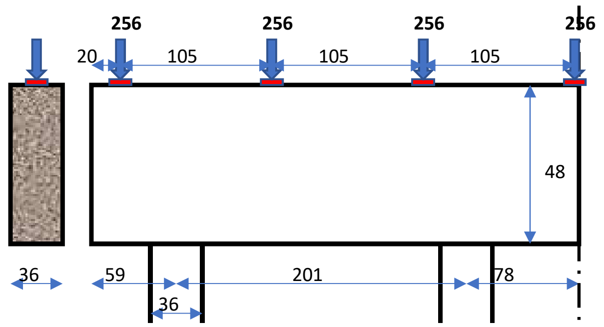
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	4 ft	11 in	**59.0 in**
Distance from center of first column to center of second column (C2)	16 ft	9 in	**201.0 in**
Distance from center of second column to centerline of pier cap (C3)	6 ft	6 in	**78.0 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft	8.0 in	**20.0 in**
Spacing Between the Girders	8 ft	9.0 in	**105.0 in**
Factored Load	256 k		

Generate Load Table

Factored Load		***Distance***			
P1	256 k	1 ft	8.0 in	**20.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	256 k	8 ft	9.0 in	**105.0 in**	A3
P4	256 k	8 ft	9.0 in	**105.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5
P6	256 k	8 ft	9.0 in	**105.0 in**	A6

5. Generate

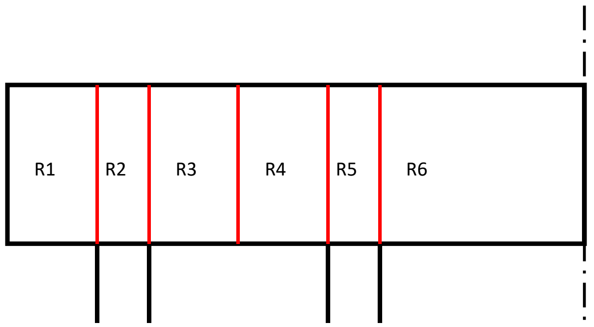


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	**30.9 in**	**0.71**	**Deep Region**
R2	**0.0 in**	**0.00**	**Zero Region**
R3	**56.1 in**	**1.30**	**Deep Region**
R4	**21.7 in**	**0.50**	**Deep Region**
R5	**0.0 in**	**0.00**	**Zero Region**
R6	**65 in**	**1.51**	**Deep Region**

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	**0.65**



Centerline

9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in², in)***		***Bottom Steel (in², in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	8	4.5	9	4.2
R2	8	4.5	9	4.2
R3	8	4.5	9	4.2
R4	8	4.5	9	4.2
R5	8	4.5	9	4.2
R6	8	4.5	9	4.2

9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	4	7 in
R2	0	0 in
R3	4	12 in
R4	4	12 in
R5	0	0 in
R6	4	16 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.31
Spacing (in)	5.5
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.31%**

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	11.5 in
Base plate width perpendicular to the pier cap (W_b)	19.0 in

11. Reinforcement Development

Horizontal length available (L_d)	26 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	19 in

It qualifies for 90° hook.

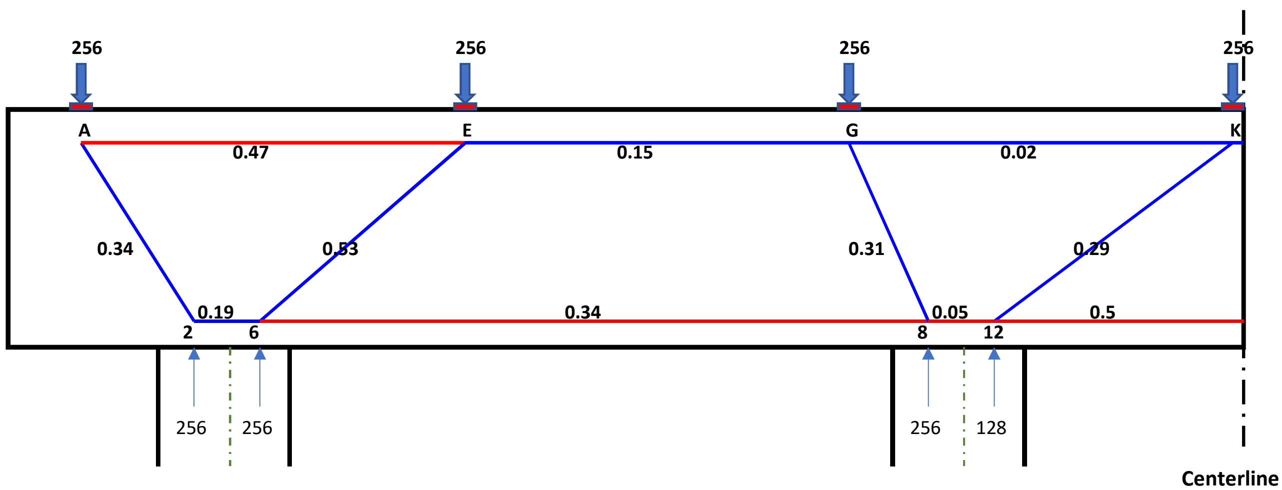
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	23 in
Available development length (L_d)	26 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Bearing Areas Nodes at ⇉

Input	Summary					
	Member Code	***Force (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***	
	A-E	201	432	0.47	PASS	**Top Members**
	E-G	-164	-1067	0.15	PASS	
	G-K	-23	-1203	0.02	PASS	
	K-Q	-23	-1203	0.02	PASS	
	2-6	-201	-1079	0.19	PASS	**Bottom Members**
	6-8	164	486	0.34	PASS	
	8-12	23	486	0.05	PASS	
	12-14	245	486	0.50	PASS	
	B-1	0	-	0.00	-	**Vertical Members**
	F-5	0	-	0.00	-	
	H-7	0	-	0.00	-	
	L-11	0	-	0.00	-	
0	A-2	-326	-957	0.34	PASS	**Inclined Members**
0	E-6	-446	-838	0.53	PASS	
0	G-8	-293	-945	0.31	PASS	
0	K-12	-257	-894	0.29	PASS	
	A	256	857	0.30	PASS	**1**
	E	256	857	0.30	PASS	
	G	256	1040	0.25	PASS	
	K	256	1040	0.25	PASS	
	2	256	1212	0.21	PASS	**2**
	6	256	998	0.26	PASS	
	8	256	1235	0.21	PASS	
	12	128	618	0.21	PASS	

Re-Generate Output Model

BRIDGE PIER CAP 5

Analysis Input

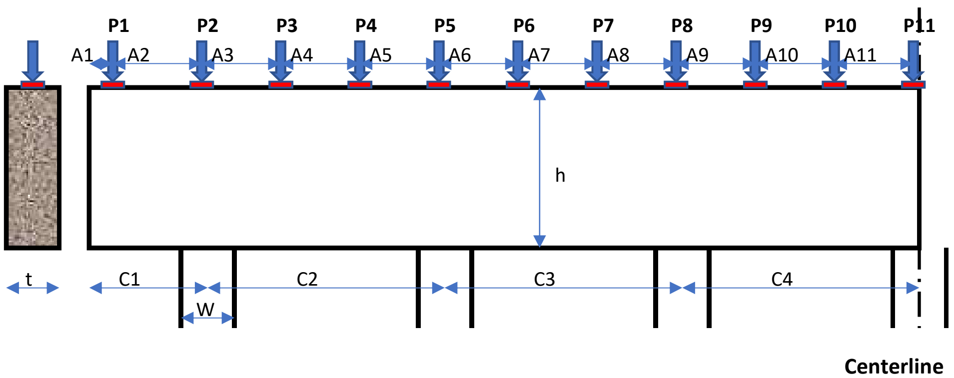
Bridge Details:

Bridge Name:	Bridge 5	Pier Number:	Pier 4
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	7

☐ Asymmetrical

2. Generate



Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

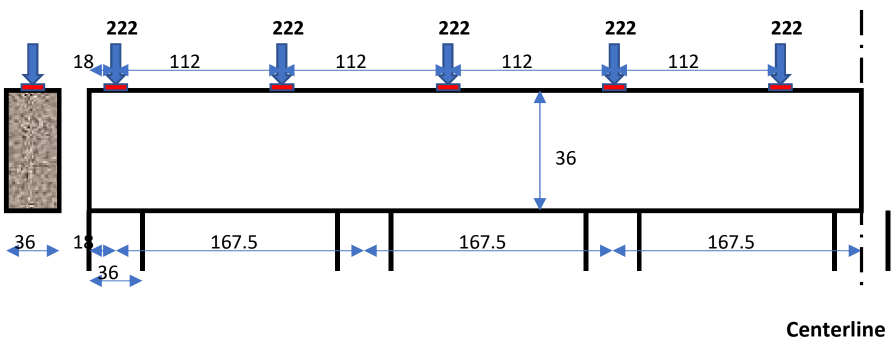
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	1 ft	6 in	**18.0 in**
Distance from center of first column to center of second column (C2)	13 ft	12 in	**167.5 in**
Distance from center of second column to center of third column (C3)	13 ft	12 in	**167.5 in**
Distance from center of third column to center of fourth column (C4)	13 ft	12 in	**167.5 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	36 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft	6.0 in	**18.0 in**
Spacing Between the Girders	9 ft	4.0 in	**112.0 in**
Factored Load	222 k		

Generate Load Table

Factored Load		*Distance*			
P1	0 k	0 ft	0.0 in	0.0 in	A1
P2	222 k	1 ft	6.0 in	18.0 in	A2
P3	222 k	9 ft	4.0 in	112.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	222 k	9 ft	4.0 in	112.0 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	222 k	9 ft	4.0 in	112.0 in	A8
P9	222 k	9 ft	4.0 in	112.0 in	A9
P10	0 k	0 ft	0.0 in	0.0 in	A10
P11	0 k	0 ft	0.0 in	0.0 in	A11

5. Generate

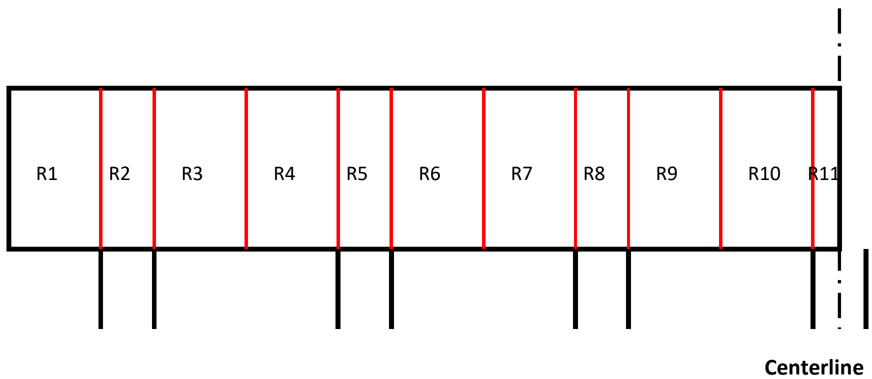


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	0.0 in	0.00	Zero Region
R2	4.5 in	0.14	Deep Region
R3	98.5 in	3.04	Slender Region
R4	46.5 in	1.44	Deep Region
R5	0.0 in	0.00	Zero Region
R6	47 in	1.46	Deep Region
R7	97 in	2.98	Slender Region
R8	1 in	0.03	Deep Region
R9	99 in	3.04	Slender Region
R10	46 in	1.40	Deep Region
R11	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	**0.65**



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	7.9	4.2	7.9	4.2
R2	7.9	4.2	7.9	4.2
R3	7.9	4.2	7.9	4.2
R4	7.9	4.2	7.9	4.2
R5	7.9	4.2	7.9	4.2
R6	7.9	4.2	7.9	4.2
R7	7.9	4.2	7.9	4.2
R8	7.9	4.2	7.9	4.2
R9	7.9	4.2	7.9	4.2
R10	7.9	4.2	7.9	4.2
R11	7.9	4.2	7.9	4.2

9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	0	0 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	20 in
R8	4	20 in
R9	4	20 in
R10	4	18 in
R11	4	18 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.31
Spacing (in)	7.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.25%**

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	19.0 in
Base plate width perpendicular to the pier cap (W_b)	12.0 in

11. Reinforcement Development

Horizontal length available	**25 in**

Bottom Tension Bars	
Enter the Diameter of the Bottom longitudinal bar:	1.00 in
Enter the Length of the hook Provided:	27 in
Basic Development Length	**19 in**

It qualifies for 90° hook.

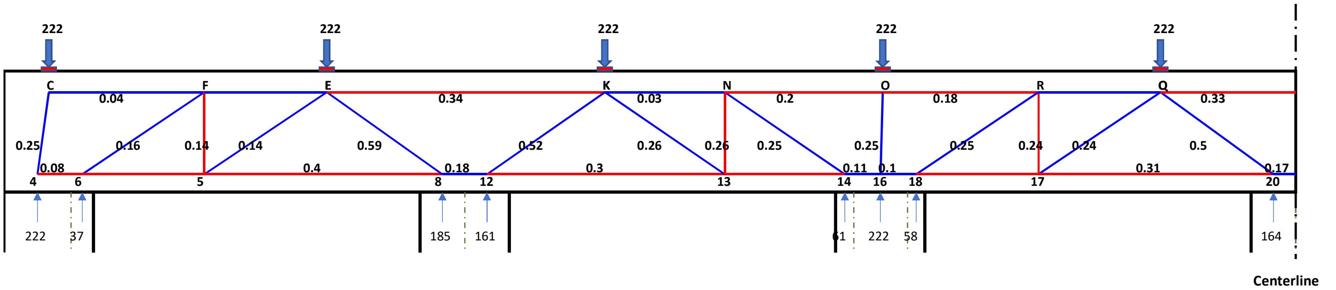
Modification Factor		
1. Are those bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 Bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**

Required development length	**23 in**
Available development length (L_d)	**25 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Tie Option	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	
	C-F	-36	-808	0.04	PASS	
	E-K	144	427	0.34	PASS	Top Members
	K-N	-23	-808	0.03	PASS	
	O-R	78	427	0.18	PASS	
	Q-W	139	427	0.33	PASS	
	4-6	36	427	0.08	PASS	
	5-8	170	427	0.40	PASS	
	8-12	-144	-808	0.18	PASS	
	12-13	129	427	0.30	PASS	Bottom Members
	14-16	-86	-808	0.11	PASS	
	16-18	-78	-808	0.10	PASS	
	17-20	131	427	0.31	PASS	
	20-24	-139	-808	0.17	PASS	
	D-3	0	-	0.00	-	
	F-5	37	271	0.14	PASS	
	H-7	0	-	0.00	-	
	L-11	0	-	0.00	-	Vertical Members
	N-13	61	237	0.26	PASS	
	P-15	0	-	0.00	-	
	R-17	58	244	0.24	PASS	
	T-19	0	-	0.00	-	
0	C-4	-225	-912	0.25	PASS	
1	F-6	-77	-472	0.16	PASS	
	E-5	-76	-531	0.14	PASS	
0	E-8	-364	-622	0.59	PASS	
0	K-12	-318	-616	0.52	PASS	
1	K-13	-123	-475	0.26	PASS	Inclined Members
	N-14	-123	-495	0.25	PASS	
0	O-16	-222	-873	0.25	PASS	
1	R-18	-119	-469	0.25	PASS	
	Q-17	-119	-489	0.24	PASS	
0	Q-20	-316	-627	0.50	PASS	
	C	222	575	0.39	PASS	
	E	222	575	0.39	PASS	
	K	222	575	0.39	PASS	1
	O	222	575	0.39	PASS	
	Q	222	575	0.39	PASS	
	4	222	1097	0.20	PASS	
	6	37	185	0.20	PASS	
	8	185	685	0.27	PASS	
	12	161	597	0.27	PASS	
	14	61	230	0.27	PASS	2
	16	222	834	0.27	PASS	
	18	58	218	0.27	PASS	
	20	164	641	0.26	PASS	

Re-Generate Output Model

BRIDGE PIER CAP 6

Analysis Input

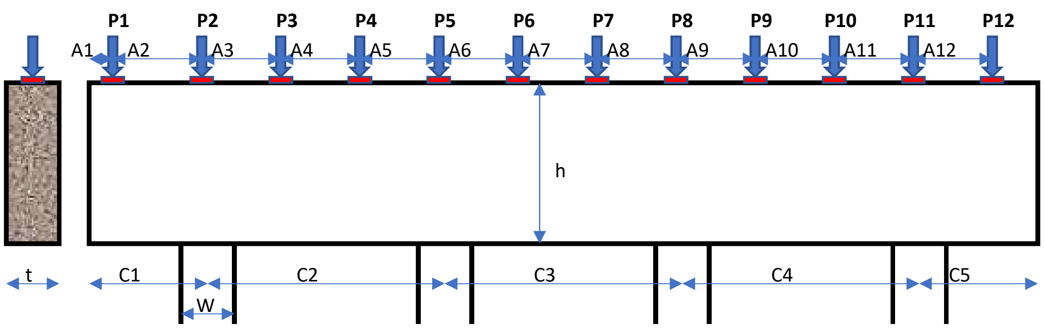
Bridge Details:

Bridge Name:	Bridge 6	Pier Number:	Pier 2-Left
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☑ Asymmetrical

2. Generate



Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

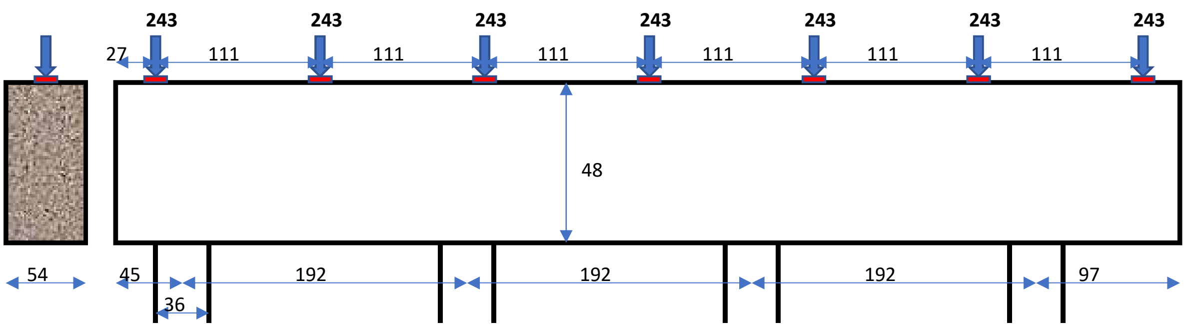
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	3 ft	9.0 in	**45.0 in**
Distance from center of first column to center of second column (C2)	16 ft	0.0 in	**192.0 in**
Distance from center of second column to center of third column (C3)	16 ft	0.0 in	**192.0 in**
Distance from center of third column to center of fourth pier cap (C4)	16 ft	0.0 in	**192.0 in**
Distance from center of fourth column to the end of the pier cap (C4)	8 ft	1.0 in	**97.0 in**
Column width (W)	36 in	**Circular**	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	54 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	3.0 in	**27.0 in**
Spacing Between the Girders	9 ft	3.0 in	**111.0 in**
Factored Load	243 k		

Generate Load Table

Factored Load		Distance			
P1	243 k	2 ft	3.0 in	27.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	243 k	9 ft	3.0 in	111.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	243 k	9 ft	3.0 in	111.0 in	A5
P6	243 k	9 ft	3.0 in	111.0 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	0 k	0 ft	0.0 in	0.0 in	A8
P9	243 k	9 ft	3.0 in	111.0 in	A9
P10	243 k	9 ft	3.0 in	111.0 in	A10
P11	0 k	0 ft	0.0 in	0.0 in	A11
P12	243 k	9 ft	3.0 in	111.0 in	A12

5. Generate

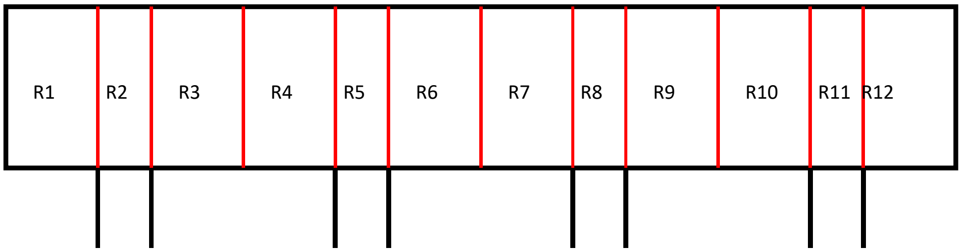


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	11.9 in	0.27	Deep Region
R2	0.0 in	0.00	Zero Region
R3	81.1 in	1.88	Deep Region
R4	85.7 in	1.98	Deep Region
R5	10.8 in	0.25	Deep Region
R6	109 in	2.51	Slender Region
R7	58 in	1.34	Deep Region
R8	0 in	0.00	Zero Region
R9	17 in	0.39	Deep Region
R10	30 in	0.70	Deep Region
R11	0 in	0.00	Zero Region
R12	45 in	1.04	Deep Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.27 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	**0.65**



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	***Top Steel (in^2, in)***		***Bottom Steel (in^2, in)***	
	Total Area (A_t)	***Centroid (C_t)***	***Total Area (A_b)***	***Centroid (C_b)***
R1	22.86	5.5	11.43	3
R2	22.86	5.5	11.43	3
R3	22.86	5.5	11.43	3
R4	22.86	5.5	11.43	3
R5	22.86	5.5	11.43	3
R6	22.86	5.5	11.43	3
R7	22.86	5.5	11.43	3
R8	22.86	5.5	11.43	3
R9	22.86	5.5	11.43	3
R10	22.86	5.5	11.43	3
R11	22.86	5.5	11.43	3
R12	22.86	5.5	11.43	3

9B. Transverse Reinforcement		
Region	***No. of Legs***	***Stirrup Spacing***
R1	0	0 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	20 in
R8	4	20 in
R9	4	20 in
R10	4	18 in
R11	4	18 in
R12	0	0 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.20
Spacing (in)	5.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.15%**

10. Base Plate Dimensions

Base plate length parallel to the pier cap (L_b)	13.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in

11. Reinforcement Development

Horizontal length available (L_d)	**32 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.27 in
Enter the length of the hook provided:	30 in
Basic development length	**24 in**

It qualifies for 90° hook.

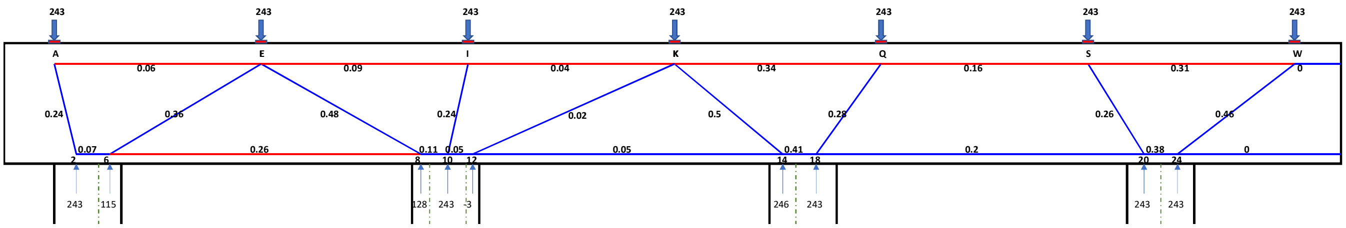
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**

Required development length	**29 in**
Available development length (L_d)	**32 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

	Summary					
	Member Code	***Force (k)***	***Capacity (k)***	***Utilization Ratio***	***Result***	
	A-E	73	1234	0.06	PASS	
	E-I	114	1234	0.09	PASS	
	I-K	48	1234	0.04	PASS	**Top Members**
	K-Q	417	1234	0.34	PASS	
	Q-S	202	1234	0.16	PASS	
	S-W	387	1234	0.31	PASS	
	W+	0	1234	0.00	-	
	2-6	-73	-1025	0.07	PASS	
	6-8	164	617	0.26	PASS	
	8-10	-114	-1025	0.11	PASS	
	10-12	-48	-1025	0.05	PASS	
	12-14	-55	-1025	0.05	PASS	**Bottom Members**
	14-18	-417	-1025	0.41	PASS	
	18-20	-202	-1025	0.20	PASS	
	20-24	-387	-1025	0.38	PASS	
	24+	0	617	0.00	-	
	B-1	0	-	0.00	-	
	F-5	0	-	0.00	-	
	H-7	0	-	0.00	-	
	J-9	0	-	0.00	-	
	L-11	0	-	0.00	-	**Vertical Members**
	N-13	0	-	0.00	-	
	R-17	0	-	0.00	-	
	T-19	0	-	0.00	-	
	X-23	0	-	0.00	-	
0	A-2	-254	-1063	0.24	PASS	
0	E-6	-263	-732	0.36	PASS	
0	E-8	-305	-640	0.48	PASS	
0	I-10	-252	-1050	0.24	PASS	
0	K-12	-8	-547	0.01	PASS	**Inclined Members**
0	K-14	-437	-881	0.50	PASS	
0	Q-18	-324	-1148	0.28	PASS	
0	S-20	-305	-1157	0.26	PASS	
0	W-24	-457	-992	0.46	PASS	
Bearing Areas Nodes at ⇉	A	243	688	0.35	PASS	
	E	243	688	0.35	PASS	
	I	243	688	0.35	PASS	
	K	243	688	0.35	PASS	**1**
	Q	243	688	0.35	PASS	
	S	243	688	0.35	PASS	
	W	243	688	0.35	PASS	
	2	243	870	0.28	PASS	
	6	115	413	0.28	PASS	
	8	128	446	0.29	PASS	
	10	243	847	0.29	PASS	
	12	-3	-10	0.29	PASS	**2**
	14	246	645	0.38	PASS	
	18	243	638	0.38	PASS	
	20	243	641	0.38	PASS	
	24	243	641	0.38	PASS	

Re-Generate Output Model

BRIDGE PIER CAP 7

Analysis Input

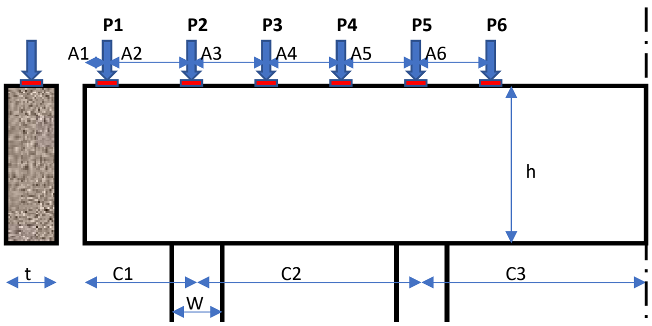
Bridge Details:

Bridge Name:	Bridge 7	Pier Number:	Southbound (Left)
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

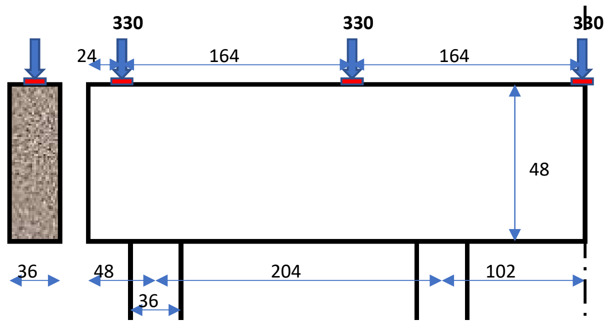
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	4 ft	0 in	**48.0 in**
Distance from center of first column to center of second column (C2)	17 ft	0 in	**204.0 in**
Distance from center of second column to centerline of pier cap (C3)	8 ft	6 in	**102.0 in**
Column width (W)	36 in	Circular	
Depth of pier cap (h)	48 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	2 ft	0.0 in	**24.0 in**
Spacing Between the Girders	13 ft	8.0 in	**164.0 in**
Factored Load	330 k		

Generate Load Table

Factored Load		***Distance***			
P1	330 k	2 ft	0.0 in	**24.0 in**	A1
P2	0 k	0 ft	0.0 in	**0.0 in**	A2
P3	330 k	13 ft	8.0 in	**164.0 in**	A3
P4	0 k	0 ft	0.0 in	**0.0 in**	A4
P5	0 k	0 ft	0.0 in	**0.0 in**	A5
P6	330 k	13 ft	8.0 in	**164.0 in**	A6

5. Generate

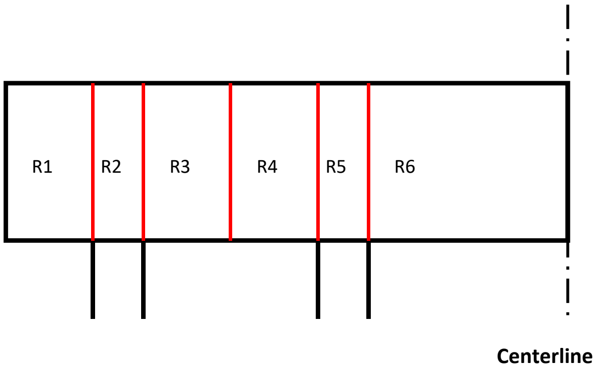


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	**19.7 in**	**0.46**	**Deep Region**
R2	**0.0 in**	**0.00**	**Zero Region**
R3	**126.3 in**	**2.92**	**Slender Region**
R4	**53.3 in**	**1.23**	**Deep Region**
R5	**0.0 in**	**0.00**	**Zero Region**
R6	**93 in**	**2.15**	**Slender Region**

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	**0.65**



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	**Top Steel (in^2, in)**		**Bottom Steel (in^2, in)**	
	Total Area (A_t)	**Centroid (C_t)**	**Total Area (A_b)**	**Centroid (C_b)**
R1	12	5	12	5
R2	12	5	12	5
R3	12	5	12	5
R4	12	5	12	5
R5	12	5	12	5
R6	12	5	12	5

9B. Transverse Reinforcement		
Region	**No. of Legs**	**Stirrup Spacing**
R1	4	18 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18.0 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.31
Spacing (in)	9.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.19%**

10. Base Plate Dimensions	
Base plate length parallel to the pier cap (L_b)	16.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in

11. Reinforcement Development

Horizontal length available (L_d)	31 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	19 in

It qualifies for 90° hook.

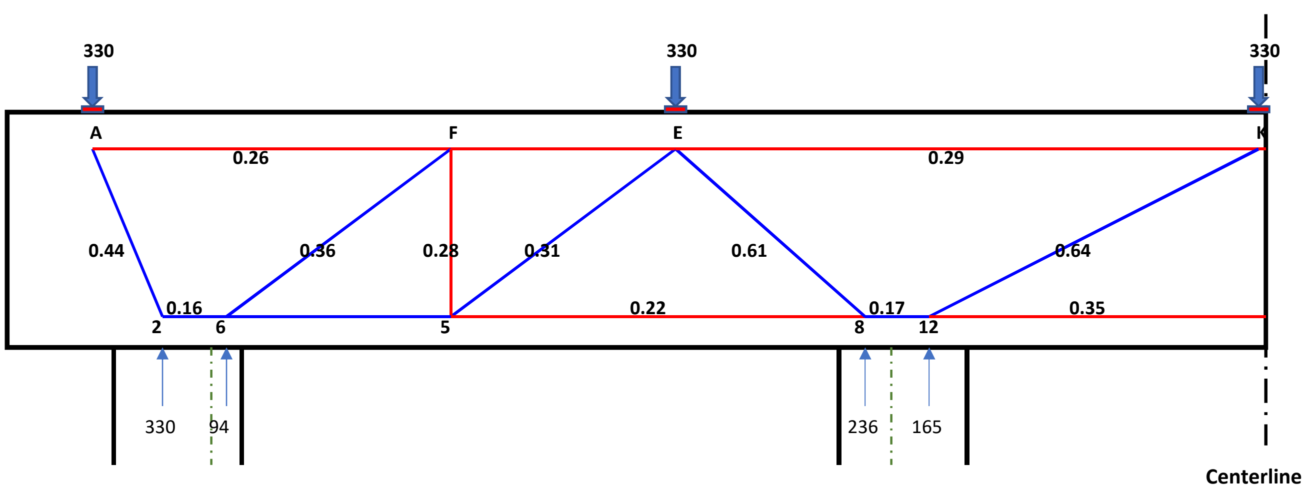
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	1

Required development length	23 in
Available development length (L_d)	31 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Input
0
1
0
0

Bearing Areas Nodes at ⇉

Summary					
Member Code	*Force (k)*	*Capacity (k)*	*Utilization Ratio*	*Result*	
A-F	171	648	0.26	PASS	
E-K	187	648	0.29	PASS	Top Members
K-Q	187	648	0.29	PASS	
2-6	-171	-1102	0.16	PASS	
5-8	142	648	0.22	PASS	Bottom Members
8-12	-187	-1102	0.17	PASS	
12-14	224	648	0.35	PASS	
B-1	0	-	0.00	-	
F-5	94	338	0.28	PASS	Vertical Members
H-7	0	-	0.00	-	
L-11	0	-	0.00	-	
A-2	-372	-853	0.44	PASS	
F-6	-183	-506	0.36	PASS	
E-5	-182	-590	0.31	PASS	Inclined Members
E-8	-405	-658	0.61	PASS	
K-12	-443	-695	0.64	PASS	
A	330	772	0.43	PASS	
E	330	772	0.43	PASS	1
K	330	772	0.43	PASS	
2	330	998	0.33	PASS	
6	94	285	0.33	PASS	2
8	236	755	0.31	PASS	
12	165	528	0.31	PASS	

Re-Generate Output Model

BRIDGE PIER CAP 8

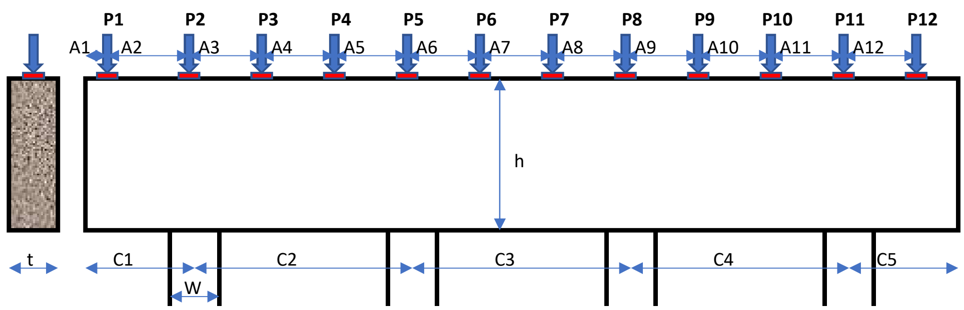
Analysis Input

Bridge Details:

Bridge Name:	Bridge 8	Pier Number:	Southbound (Left)
SFN Number:	570XXXX	Designer:	XXXX
PID No.:	77XXX	Date:	XXXX

1. Total Number of Columns (Piers)	4	☑ Asymmetrical

2. Generate



Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

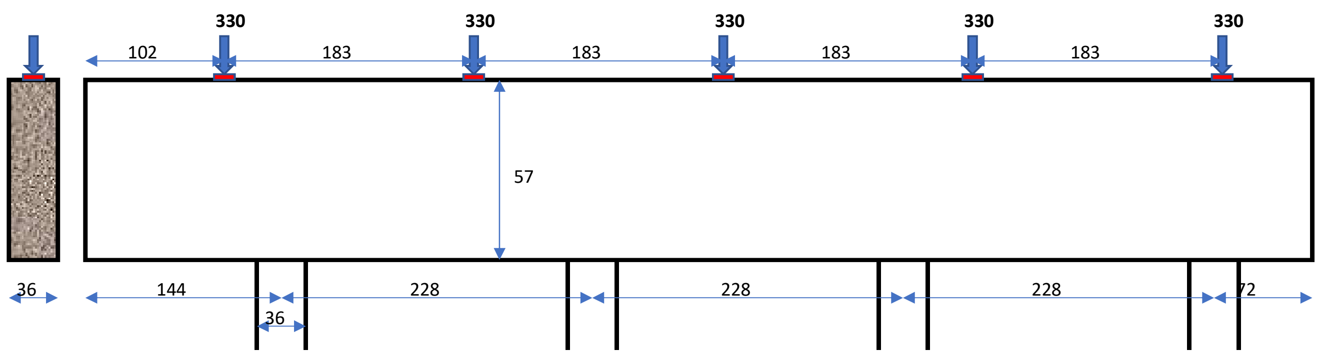
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	12 ft	0 in	144.0 in
Distance from center of first column to center of second column (C2)	19 ft	0 in	228.0 in
Distance from center of second column to center of third column (C3)	19 ft	0 in	228.0 in
Distance from center of third column to center of fourth pier cap (C4)	19 ft	0 in	228.0 in
Distance from center of fourth column to the end of the pier cap (C4)	6 ft	0 in	72.0 in
Column width (W)	36 in	Circular	
Depth of pier cap (h)	57 in		
Thickness of pier cap (t)	36 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	8 ft	6.0 in	102.0 in
Spacing Between the Girders	15 ft	3.0 in	183.0 in
Factored Load	330 k		

Generate Load Table

Factored Load		***Distance***			
P1	330 k	8 ft	6.0 in	102.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	330 k	15 ft	3.0 in	183.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	330 k	15 ft	3.0 in	183.0 in	A6
P7	0 k	0 ft	0.0 in	0.0 in	A7
P8	0 k	0 ft	0.0 in	0.0 in	A8
P9	330 k	15 ft	3.0 in	183.0 in	A9
P10	0 k	0 ft	0.0 in	0.0 in	A10
P11	330 k	15 ft	3.0 in	183.0 in	A11
P12	0 k	0 ft	0.0 in	0.0 in	A12

5. Generate

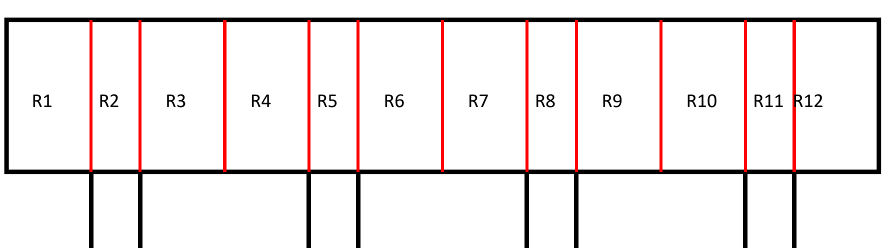


6. Check whether the Pier Cap is Deep			
Region	***Shear span (a)***	***a/d ratio:***	***Result***
R1	37.0 in	0.72	Deep Region
R2	0.0 in	0.00	Zero Region
R3	128.0 in	2.49	Slender Region
R4	78.3 in	1.53	Deep Region
R5	0.0 in	0.00	Zero Region
R6	87 in	1.69	Deep Region
R7	120 in	2.35	Slender Region
R8	0 in	0.00	Zero Region
R9	45 in	0.87	Deep Region
R10	162 in	3.16	Slender Region
R11	3 in	0.05	Deep Region
R12	0 in	0.00	Zero Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	4.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Diameter of biggest rebar (d_b)	1.00 in
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebars	0.9
For stirrup	0.9
CCC v-factor for bearing and back face	0.85
CCT v-factor for bearing and back face	0.7
CTT v-factor for bearing and back face	0.65



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	**Top Steel (in^2, in)**		**Bottom Steel (in^2, in)**	
	Total Area (A_t)	**Centroid (C_t)**	**Total Area (A_b)**	**Centroid (C_b)**
R1	12	5	12	5
R2	12	5	12	5
R3	12	5	12	5
R4	12	5	12	5
R5	12	5	12	5
R6	12	5	12	5
R7	12	5	12	5
R8	12	5	12	5
R9	12	5	12	5
R10	12	5	12	5
R11	12	5	12	5
R12	12	5	12	5

9B. Transverse Reinforcement		
Region	**No. of Legs**	**Stirrup Spacing**
R1	4	18 in
R2	4	18 in
R3	4	18 in
R4	4	18 in
R5	4	18 in
R6	4	18 in
R7	4	18 in
R8	4	18 in
R9	4	18 in
R10	4	18 in
R11	4	18 in
R12	4	18 in

9C. Min Horizontal Crack Control Reinforcement	
Code Required Crack Control Reinforcement	0.30%
Crack Control Rebar Area (in^2)	0.31
Spacing (in)	9.0
No of layers of Crack Control Rebars	2
Crack Control Reinforcement	**0.19%**

10. Base Plate Dimensions

Base plate length parallel to the pier cap (L_b)	16.0 in
Base plate width perpendicular to the pier cap (W_b)	21.0 in

11. Reinforcement Development

Horizontal length available (L_d)	**110 in**

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.00 in
Enter the length of the hook provided:	30 in
Basic development length	**19 in**

It qualifies for 90° hook.

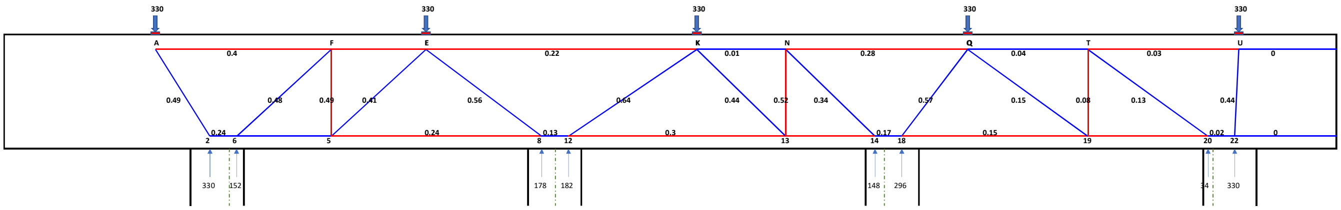
Modification Factor		
1. Are bars epoxy coated?	Yes	**1.2**
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	No	**1**

Required development length	**23 in**
Available development length (L_d)	**110 in**

Reinforcement Capacity Multiplier:	**1.00**

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Tie Input	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	
	Summary					
	A-F	260	648	0.40	PASS	
	E-K	142	648	0.22	PASS	
	K-N	-6	-1102	0.01	PASS	**Top Members**
	Q-T	-40	-1102	0.04	PASS	
	U-W	0	648	0.00	-	
	2-6	-260	-1102	0.24	PASS	
	5-8	153	648	0.24	PASS	
	8-12	-142	-1102	0.13	PASS	
	12-13	195	648	0.30	PASS	
	14-18	-183	-1102	0.17	PASS	**Bottom Members**
	18-19	100	648	0.15	PASS	
	20-22	-19	-1102	0.02	PASS	
	22-24	0	648	0.00	-	
	B-1	0	-	0.00	-	
	F-5	152	313	0.49	PASS	
	H-7	0	-	0.00	-	
	L-11	0	-	0.00	-	
	N-13	148	285	0.52	PASS	**Vertical Members**
	R-17	0	-	0.00	-	
	T-19	34	441	0.08	PASS	
	V-21	0	-	0.00	-	
0	A-2	-420	-851	0.49	PASS	
1	F-6	-256	-530	0.48	PASS	
	E-5	-257	-633	0.41	PASS	
0	E-8	-345	-620	0.56	PASS	
0	K-12	-383	-599	0.64	PASS	
1	K-13	-240	-546	0.44	PASS	**Inclined Members**
	N-14	-240	-711	0.34	PASS	
0	Q-18	-408	-721	0.57	PASS	
1	Q-19	-69	-474	0.15	PASS	
	T-20	-69	-542	0.13	PASS	
0	U-22	-331	-751	0.44	PASS	
Bearing Areas Nodes at ⇉	A	330	772	0.43	PASS	
	E	330	772	0.43	PASS	
	K	330	772	0.43	PASS	**1**
	Q	330	772	0.43	PASS	
	U	330	772	0.43	PASS	
	2	330	879	0.38	PASS	
	6	152	404	0.38	PASS	
	8	178	634	0.28	PASS	
	12	182	648	0.28	PASS	
	14	148	428	0.35	PASS	**2**
	18	296	855	0.35	PASS	
	20	34	121	0.28	PASS	
	22	330	1161	0.28	PASS	

Re-Generate Output Model

Appendix C: STM-CAP Error and Warning Message Displayed by STM-CAP

Please note that these are not intended as recommended values. The main objective is to check for accidental input of unrealistically large or small values.

Table C-1: Error and warning message

SN	Check	Message
1	Number of columns not numeric	Total number of columns should be a numerical value.
2	Number of columns not integer	Total number of columns should be an integer value.
3	Number of columns not between 2 to 8	STM-CAP is designed for 2 to 8 columns for symmetrical and 2 to 4 columns for asymmetrical pier caps.
4	Input in Section X not numeric (all sections)	The input should be a numerical value in Section X.
5	Column width < 1 or > 500 inches	Please check the column width in Section 3.
6	Pier cap depth < 1 or > 500 inches	Please check the pier cap depth in Section 3.
7	Pier cap thickness < 1 or > 300 inches	Please check the pier cap thickness in Section 3.
8	Distance from edge of pier cap to the center of first column (C1) < half-width of column	C1 cannot be less than half-width of the column. Please check the data and re-input C1 or W in Section 1.
9	Any clear span (C2, C3, C4…) > 10000 inches	Please check the geometry details of the pier cap in Section3.
10	Any clear span (C2, C3, C4…) < half-width of column	Please check the geometry details of the pier cap in Section3.
11	Sum of all girder distances > sum of pier cap span	Please check Section 3 and/or Section 4. The loads lie out of the pier cap boundary.
12	Any load < 1 or > 99999 kips	Please check the loads in Section 4.

13	Any girder spacing < 1 or > 1000 feet	Please check the girder spacing in Section 4.
14	Load value ≠0 but corresponding distance = 0	Please check the loads and respective distances in Section 4. Both should be either zero or non-zero.
15	Load value =0 but corresponding distance ≠ 0	Please check the loads and respective distances in Section 4. Both should be either zero or non-zero.
16	Concrete compressive strength > 15 ksi	Please check the concrete strength in Section 7. It should not be greater than 15 ksi for STM analysis as per AASHTO 2017.
17	Rebar yield strength > 75 ksi	Please check the rebar yield strength in Section 7. It should not be greater than 75 ksi for STM analysis as per AASHTO 2017.
18	Stirrup yield strength > 75 ksi	Please check the stirrup yield strength in Section 7. It should not be greater than 75 ksi for STM analysis as per AASHTO 2017.
19	Clear cover > 100 in	Please check the clear cover in Section 7.
20	Stirrup bar area > 4 in^2	Please check the stirrup bar area in Section 7.
21	Concrete resistance factor > 1	The concrete resistance factor should not be greater than 1 in Section 8.
22	Rebar resistance factor > 1	The rebar resistance factor should not be greater than 1 in Section 8.
23	Stirrup resistance factor > 1	The stirrup resistance factor should not be greater than 1 in Section 8.
24	CCC v-factor > 1	The CCC v-factor multiplier should not be greater than 1 in Section 8.
25	CCT v-factor > 1	The CCT v-factor multiplier should not be greater than 1 in Section 8.
26	CTT v-factor > 1	The CTT v-factor multiplier should not be greater than 1 in Section 8.
27	Input in Section 7 = 0	The input should not be zero in Section 7.

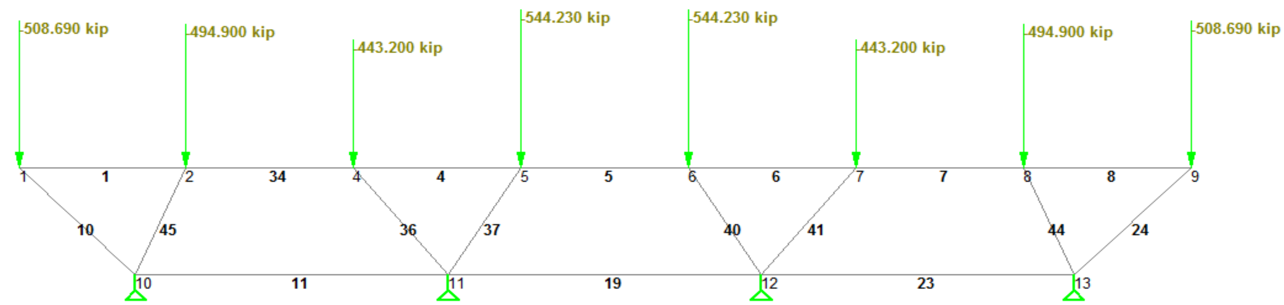
28	Input in Section 8 = 0	The input should not be zero in Section 8.
29	Input in Section 9C = 0	The input should not be zero in Section 9C.
30	Input in Section 10= 0	The input should not be zero in Section 10.
31	Crack control rebar area > 4 in^2	Please check the crack control rebar area in Section 9C.
32	Top/bottom horizontal rebar area > 5000 in^2	Please check the total top/bottom rebar area in Section 9A.
33	Top/bottom horizontal rebar centroid > 500 inches	Please check the top/bottom rebar centroid in Section 9A.
34	Number of legs > 30	Please check the number of stirrup legs in Section 9B.
35	Stirrup spacing > 300 inches	Please check the stirrup spacing in Section 9B.
36	Length/width of base plate > thickness of pier cap	Please check the length/width of bearing/base plate in Section 10.
37	Length of hook > depth of pier cap	Please check the length of hook in Section 11.
38	Vertical tie option ≠ 0 or 1	Please input either 0 or 1 for vertical tie option in Section 13.
39	Any change in vertical tie option	Some changes have been made for vertical tie. Please Re-Generate Output Model.

Appendix D: STM-CAP vs Commercial Method Solved Examples

Commercial Method

Sample Pier Cap 1

STM Model for STAAD



Coordinates Strut and Tie

Node	x (ft)	y (ft)	z (ft)
1	0.000	0.000	0
2	10.167	0.000	0
4	20.333	0.000	0
5	30.500	0.000	0
6	40.667	0.000	0
7	50.833	0.000	0
8	61.000	0.000	0
9	71.167	0.000	0
10	7.083	-7.167	0
11	26.083	-7.167	0
12	45.083	-7.167	0
13	64.083	-7.167	0

Geometry Strut and Tie

Beam	Node A	Node B	Theta (deg)
1	1	2	0.00
4	4	5	0.00
5	5	6	0.00
6	6	7	0.00
7	7	8	0.00
8	8	9	0.00
10	1	10	45.34
11	10	11	0.00
19	11	12	0.00
23	12	13	0.00
24	13	9	45.33
34	2	4	0.00
36	4	11	51.26
37	5	11	58.35
40	6	12	58.36
41	12	7	51.26
44	8	13	66.72
45	2	10	66.72

STM Member Forces and Support Reactions

Member Forces

Beam	Node	Axial Force (kip)	Beam	Node	Axial Force (kip)
1	1	-531.008	23	12	270.914
	2	531.008		13	-270.914
4	4	-744.753	24	13	755.426
	5	744.753		9	-755.426
5	5	-309.874	34	2	-270.929
	6	309.874		4	270.929
6	6	-744.713	36	4	757.146
	7	744.713		11	-757.146
7	7	-270.914	37	5	828.917
	8	270.914		11	-828.917
8	8	-531.036	40	6	828.897
	9	531.036		12	-828.897
10	1	755.406	41	12	757.131
	10	-755.406		7	-757.131
11	10	270.929	44	8	658.276
	11	-270.929		13	-658.276
19	11	309.874	45	2	658.046
	12	-309.874		10	-658.046

Reaction

Node	Reaction (kip)
10	1141.75
11	1296.24
12	1296.24
13	1141.982

Tie Capacity:

Proportion Longitudinal Ties

Location

Staad Model	Beam 1
Description	*between girders 1 and 2*

Purpose/Objective

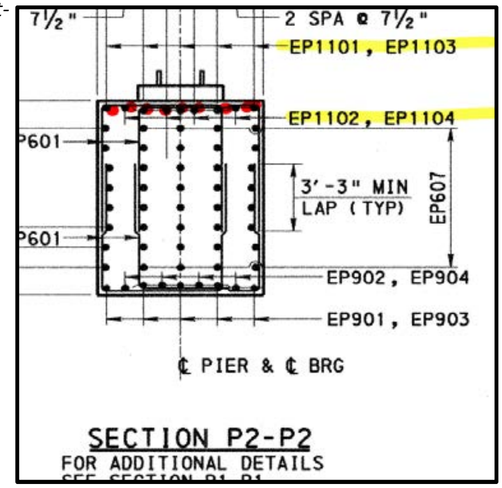
Verify the strength of the longitudinal tension reinforcement using STM

Critical Factored Load *per Staad output*

COMBINED	531.008	kip	

AASHTO LRFD 5.8.2.4 - Proportioning of Ties

Phi	0.9	*AASHTO LRFD 5.5.4.2, "for tension in strut-*	
f_y	60	ksi	*per plans*
Layer 1			
Size Rebar	11		*per plans*
Area Rebar	1.56	in^2	*area of one rebar*
No. Bar	9		*number of bars*
Layer 2			
Size Rebar	0		
Area Rebar	0	in^2	
No. Bar	0		
A_{st}	14.04	in^2	*area of steel per plans*
P_n	842.4	kip	*nominal resistance*
Nodal Strength	476	kip	
P_r	**476**	**kip**	



Design Ratio

condition factor	1
system factor	1

DR	**0.896**	NOT OK	*calculated design ratio*

Nodal and Strut Capacity:

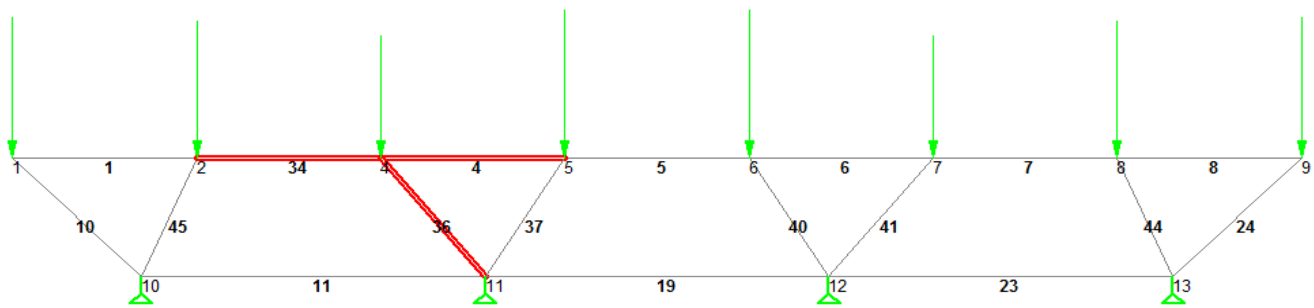
Proportion Nodal Regions

Location

Staad Model	Node 4
Description	*Girder 3*

Purpose/Objective

Verify the strength of the nodal zone using STM



Critical Factored Load

Girder Load	**G3**		
Combined	590.56	kip	*per BrR output*
Tension Tie	**4**		
Combined	744.753	kip	*per Staad output*
Compression Strut	**36**		
Combined	757	kip	*per Staad output*

AASHTO LRFD 5.8.2.5 - Proportioning Node Regions

AASHTO LRFD 5.8.2.5.1 - Strength of nodal face

Phi	0.7		*AASHTO LRFD 5.5.4.2, "for compression in strut-and-tie models"*
m	1.42		confinement modification factor
f'c	3.00	ksi	per plans
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (bearing face)*
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (back face)*
v	0.60		*Concrete efficiency factor, Table 5.8.2.5.3a-1: assume 0.60 (strut-node interface)*

Bearing Girder/Column Face

fcu	2.98	ksi	*limiting compressive stress of the node face*
bearing area	1025.58	in^2	*bearing area of masonry plate*
Pn	3061.25	kip	*nominal resistance of node face*
Pr	**2142.87**	**kip**	

Design Ratio

condition factor	1
system factor	1

DR	**3.63**	OK	*Design Ratio*

Strut-Node Interface

fcu	2.56	ksi	*limiting compressive stress of the node face*
bearing width	43.875	in	*bearing area of masonry plate, per plans*
theta	51.26	deg	*angle between compression strut and tension tie*
height back face	12	in	*at a "CCT" node, twice the height to the centroid of the tie reinforcement*
width strut	41.73	in	*calculated width of strut*
bearing depth	23.38	in	
bearing area	975.48	in^2	*calculated bearing area*
Pn	2495.75	kip	
Pr	**1747.03**	**kip**	

Design Ratio

condition factor	1
system factor	1

DR	**2.31**	OK	*Design Ratio*

Serviceability Check

Purpose/Objective

determine if adequate reinforment has been provided to prevent cracking (temperature and shrinkage)

AASHTO LRFD Eq. C5.8.2.6-1 (Vertical Reinforcement)

Stirrup Information

Size	5.00		*smallest size stirrup*
diameter	0.63	in	*diameter of rebar*
Area	0.31	in^2	*area of one rebar*
Spacing	11.38	in	*maximum spacing used*
No. Stirrup legs	4.00		*number of stirrup legs to be considered in the cross-section*
width member	6.00	ft	

A_h	1.23	in^2	
b_w	72.00	in	
s_h	11.38	in	
A_{sh}	0.001498394	*minimum crack control reinforcement not provided per AASHTO 5.8.2.6*	

AASHTO LRFD Eq. C5.8.2.6-2 (Horizontal Reinforcement)

Reinforcement Information

Size	7.00		*smallest size stirrup*
diameter	0.88	in	*diameter of rebar*
Area	0.60	in^2	*area of one rebar*
Spacing	9.50	in	*maximum spacing used*
No. Bar	2.00		*number of bars to be considered in the cross-section*
width member	6.00	ft	

A_v	1.20	in^2	
b_w	72.00	in	
s_v	9.50	in	
A_{sh}	0.001758247	*minimum crack control reinforcement not provided per AASHTO 5.8.2.6*	

BRIDGE PIER CAP 1

Analysis Input

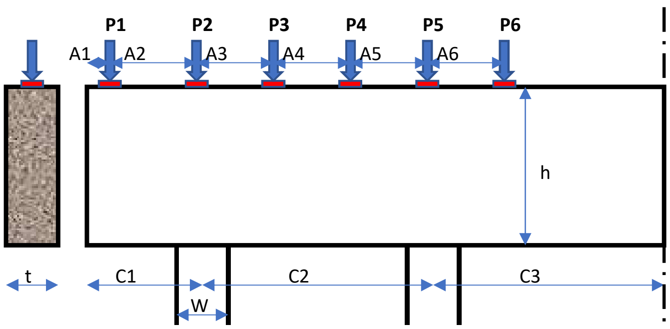
Bridge Details:

Bridge Name:	Bridge 1	Pier Number:	XXXXXXXX
SFN Number:	XXXXXXX	Designer:	XXXXXXXX
PID No.:	XXXXXXX	Date:	XXXXXXXX

1. Total Number of Columns (Piers)	4

☐ Asymmetrical

2. Generate



Centerline

Note: Input for Section 3 and Section 4 is based on the above-generated sketch. The loads shown in the above sketch are not the actual loads; these are shown for representation only.

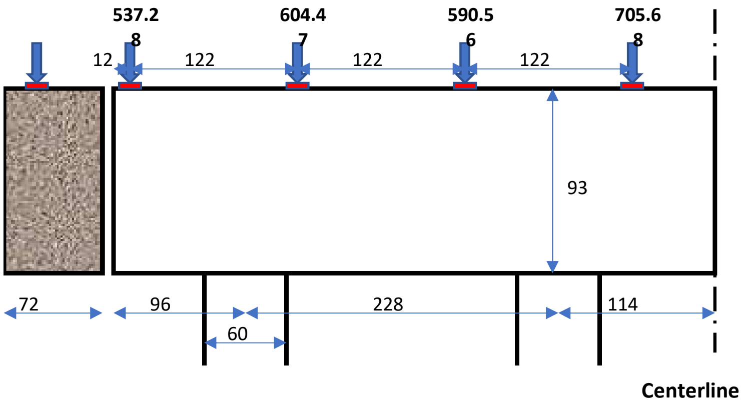
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	8 ft	0.0 in	**96.0 in**
Distance from center of first column to center of second column (C2)	19 ft	0.0 in	**228.0 in**
Distance from center of second column to centerline of pier cap (C3)	9 ft	6.0 in	**114.0 in**
Column width (W)	60 in	Circular	
Depth of pier cap (h)	93 in		
Thickness of pier cap (t)	72 in		

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft	0.0 in	**12.0 in**
Spacing Between the Girders	10 ft	2.0 in	**122.0 in**
Factored Load	400 k		

Generate Load Table

Factored Load		Distance			
P1	537 k	1 ft	0.0 in	12.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	604 k	10 ft	2.0 in	122.0 in	A3
P4	591 k	10 ft	2.0 in	122.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	706 k	10 ft	2.0 in	122.0 in	A6

5. Generate

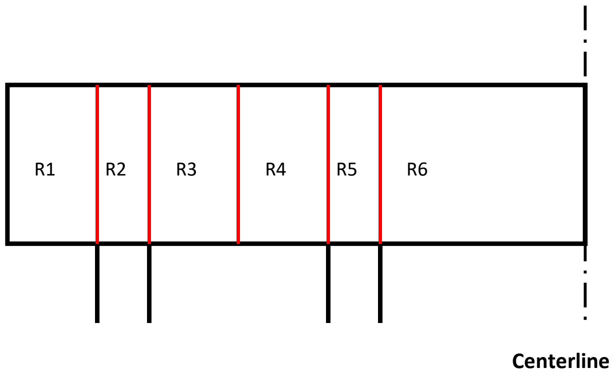


6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	67.2 in	0.80	Deep Region
R2	0.0 in	0.00	Zero Region
R3	24.8 in	0.30	Deep Region
R4	50.7 in	0.61	Deep Region
R5	0.0 in	0.00	Zero Region
R6	41 in	0.49	Deep Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	3.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Enter the clear cover	7.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebar	0.9
For stirrup	0.9
CCC v-factor for the bearing and back face	0.85
CCT v-factor for the bearing and back face	0.7
CTT v-factor for the bearing and back face	0.65



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	Top Steel (in^2, in)		Bottom Steel (in^2, in)	
	Total Area (A_t)	Centroid (C_t)	Total Area (A_b)	Centroid (C_b)
R1	14.04	3.5	9	3.5
R2	14.04	3.5	9	3.5
R3	14.04	3.5	9	3.5
R4	14.04	3.5	9	3.5
R5	14.04	3.5	9	3.5
R6	14.04	3.5	9	3.5

Rebar Area & Centroid'!A1

9B. Transverse Reinforcement		
Region	No. of Legs	Stirrup Spacing
R1	4	9.0 in
R2	4	9.0 in
R3	4	9.0 in
R4	4	9.0 in
R5	4	9.0 in
R6	4	9.0 in

9C. Min Horizontal Crack Control Reinforcement	
Crack control rebar area (in^2)	0.44
Spacing (in)	11.0
No of layers of crack control rebar	2
Provided crack control reinforcement	**0.11%**
Code required crack control reinforcement	0.30%

10. Base Plate Dimensions	
Base plate (bearing plate) length parallel to the pier cap (L_b)	32.0 in
Base plate (bearing plate) width perpendicular to the pier cap (W_b)	25.4 in

11. Reinforcement Development

Horizontal length available (L_d)	22 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.13 in
Enter the length of the hook provided:	32 in
Basic development length	25 in

It qualifies for 90° hook.

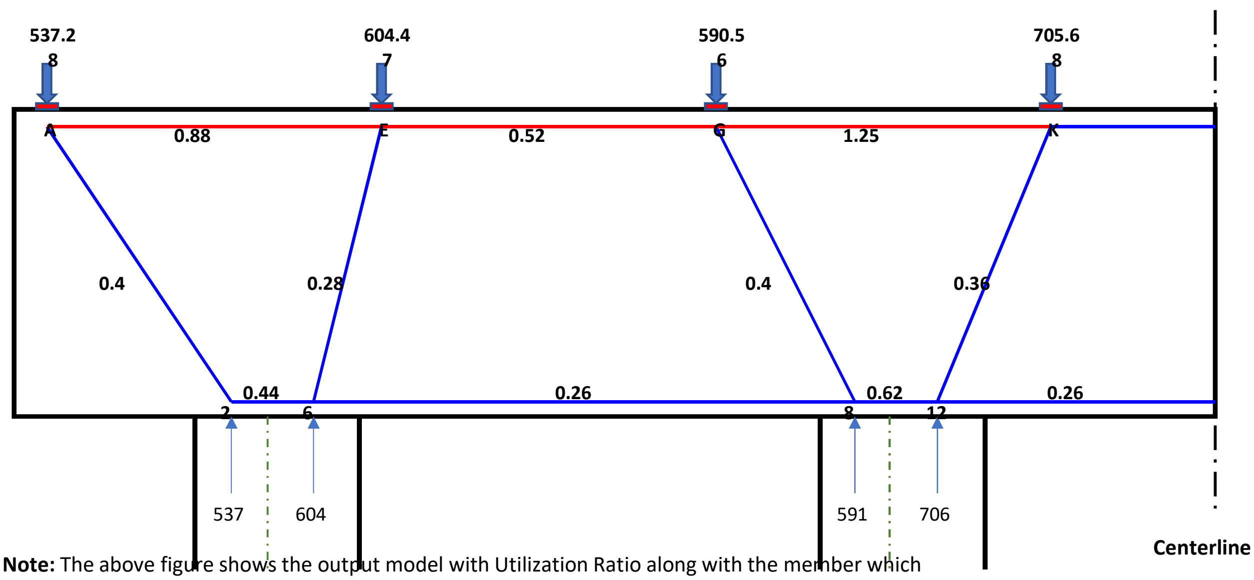
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	Yes	0.7

Required development length	21 in
Available development length (L_d)	22 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Bearing Areas Nodes at ⇉

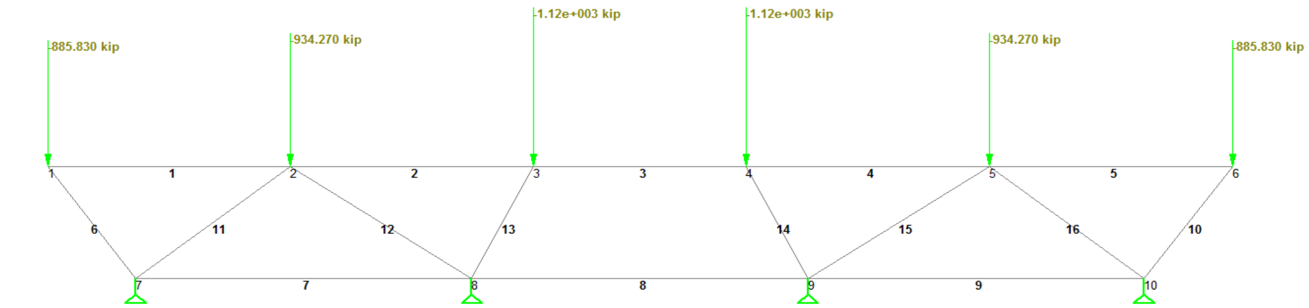
Tie Option	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	Group
	A-E	420	476	0.88	PASS	Top Members
	E-G	246	476	0.52	PASS	Top Members
	G-K	594	476	1.25	Flexure Overloaded	Top Members
	K-Q	0	476	0.00	-	Top Members
	2-6	-420	-962	0.44	PASS	Bottom Members
	6-8	-246	-962	0.26	PASS	Bottom Members
	8-12	-594	-962	0.62	PASS	Bottom Members
	12-14	-255	-962	0.26	PASS	Bottom Members
	B-1	0	-	0.00	-	Vertical Members
	F-5	0	-	0.00	-	Vertical Members
	H-7	0	-	0.00	-	Vertical Members
	L-11	0	-	0.00	-	Vertical Members
0	A-2	-682	-1713	0.40	PASS	Inclined Members
0	E-6	-629	-2224	0.28	PASS	Inclined Members
0	G-8	-686	-1726	0.40	PASS	Inclined Members
0	K-12	-783	-2169	0.36	PASS	Inclined Members
	A	537	1535	0.35	PASS	1
	E	604	1535	0.39	PASS	1
	G	591	1535	0.38	PASS	1
	K	706	1535	0.46	PASS	1
	2	537	1258	0.43	PASS	2
	6	604	1415	0.43	PASS	2
	8	591	1218	0.49	PASS	2
	12	706	1455	0.49	PASS	2

Re-Generate Output Model

Commercial Method

Sample Pier Cap 2

STM Model for STAAD



Coordinates Strut and Tie

Node	x (ft)	y (ft)	z (ft)
1	0	0	0
2	13.75	0	0
3	27.5	0	0
4	39.5	0	0
5	53.25	0	0
6	67	0	0
7	5	-6.97	0
8	24	-6.97	0
9	43	-6.97	0
10	62	-6.97	0

Geometry Strut and Tie

Beam	Node A	Node B	Theta (deg)
1	1	2	0.00
2	2	3	0.00
3	3	4	0.00
4	4	5	0.00
5	5	6	0.00
6	1	7	54.33
7	7	8	0.00
8	8	9	0.00
9	9	10	0.00
10	10	6	54.33
11	2	7	38.53
12	2	8	34.20
13	3	8	63.33
14	4	9	63.33
15	5	9	34.20
16	5	10	38.53

STM Member Forces and Support Reactions

Member Forces

Beam	Node	Axial Force (kip)	Beam	Node	Axial Force (kip)
1	1	-635.459	9	9	-114.364
	2	635.459		10	114.364
2	2	-381.199	10	10	1090.185
	3	381.199		6	-1090.185
3	3	178.892	11	2	958.638
	4	-178.892		7	-958.638
4	4	-381.199	12	2	599.282
	5	381.199		8	-599.282
5	5	-635.459	13	3	1248.108
	6	635.459		8	-1248.108
6	1	1090.185	14	4	1248.108
	7	-1090.185		9	-1248.108
7	7	-114.364	15	5	599.282
	8	114.364		9	-599.282
8	8	-178.892	16	5	958.638
	9	178.892		10	-958.638

Reaction

Node	Reaction (kip)
7	1483.118
8	1452.362
9	1452.362
10	1483.118

Tie Capacity:

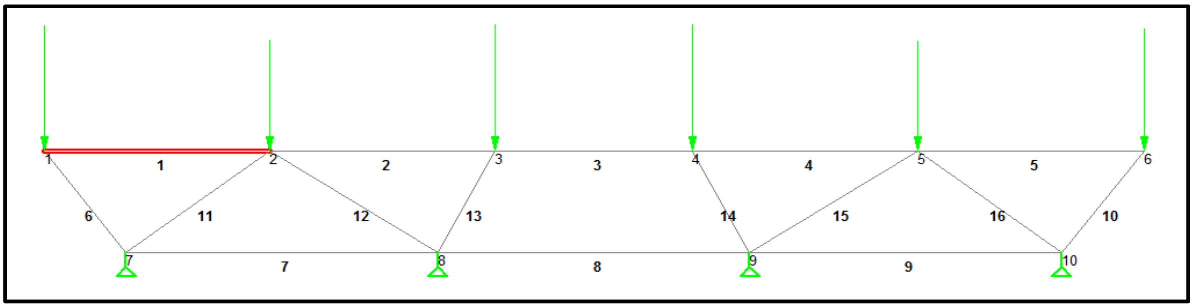
Proportion Longitudinal Ties

Location

Staad Model	Beam 1
Description	*between girders 9 and 10*

Purpose/Objective

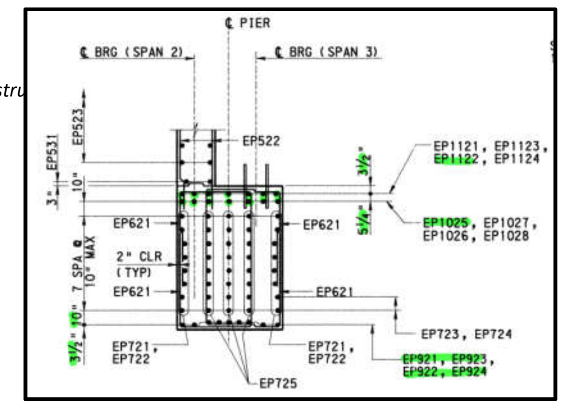
Verify the strength of the longitudinal tension reinforcement using STM



Critical Factored Load *per Staad output*

Dead Load	546.671	kip	
Live Load	88.795	kip	

AASHTO LRFD 5.8.2.4 - Proportioning of Ties



Phi	0.9	*AASHTO LRFD 5.5.4.2, "for tension in stru and-tie models"*	
f_y	60	ksi	*per plans*
Layer 1			
Size Rebar	11		*per plans*
Area Rebar	1.56	in^2	*area of one rebar*
No. Bar	7		*number of bars*
Layer 2			
Size Rebar	10		
Area Rebar	1.27	in^2	
No. Bar	7		
A_{st}	19.81	in^2	*area of steel per plans*
P_n	1188.6	kip	*nominal resistance*
Nodal Strength	435	kip	
P_r	**435**	**kip**	

Rating Factor

condition factor	1		
system factor	1		
RF1	0.796		*rating factor (C/DL)*
RF2	-1.258		*traditional rating factor (C-DL)/LL*
RF	**0.796**	NOT OK	*minimum of two calculated rating factors*

Design Ratio

DR	**0.685**	NOT OK	*calculated design ratio*

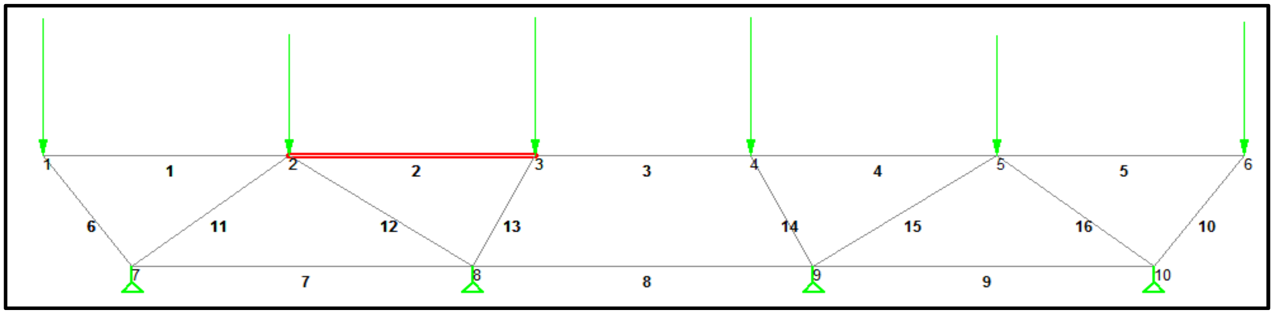
Proportion Longitudinal Ties

Location

Staad Model	Beam 2
Description	*between girders 10 and 11*

Purpose/Objective

Verify the strength of the longitudinal tension reinforcement using STM

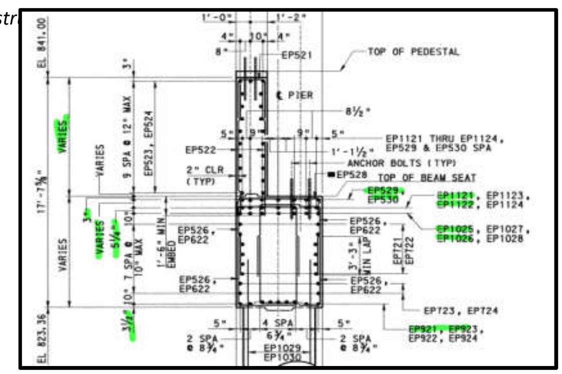


Critical Factored Load

per Staad output

Dead Load	277.878	kip	
Live Load	103.585	kip	

AASHTO LRFD 5.8.2.4 - Proportioning of Ties



Phi	0.9	*AASHTO LRFD 5.5.4.2, "for tension in str and-tie models"*	
f_y	60	ksi	*per plans*
Layer 1	**top layer #5 bar ignored*		
Size Rebar	11		*per plans*
Area Rebar	1.56	in^2	*area of one rebar*
No. Bar	7		*number of bars*
Layer 2			
Size Rebar	10		
Area Rebar	1.27	in^2	
No. Bar	7		
A_{st}	19.81	in^2	*area of steel per plans*
P_n	1188.6	kip	*nominal resistance*
Nodal Strength	435	kip	
P_r	**435**	**kip**	

Rating Factor

condition factor	1		
system factor	1		
RF1	1.565		*rating factor (C/DL)*
RF2	1.517		*traditional rating factor (C-DL)/LL*
RF	**1.517**	OK	*minimum of two calculated rating factors*

Design Ratio

DR	**1.140**	OK	*calculated design ratio*

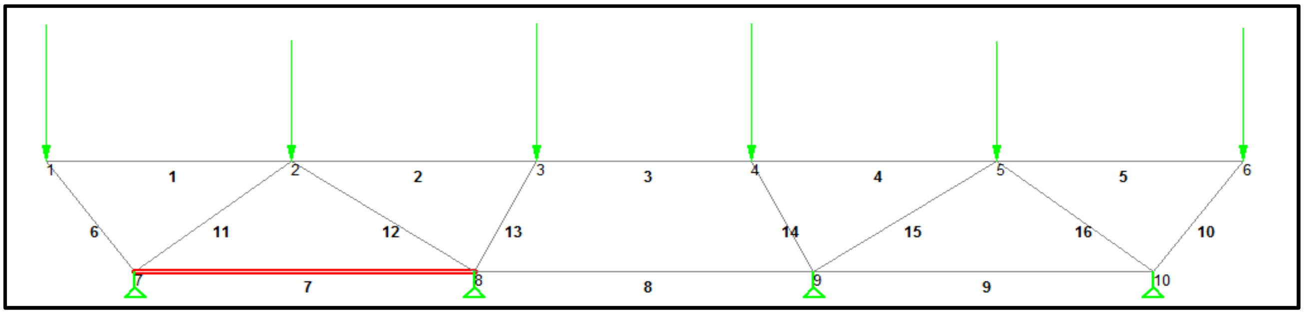
Proportion Longitudinal Ties

Location

Staad Model	Beam 7
Description	*between columns 1 and 2*

Purpose/Objective

Verify the strength of the longitudinal tension reinforcement using STM

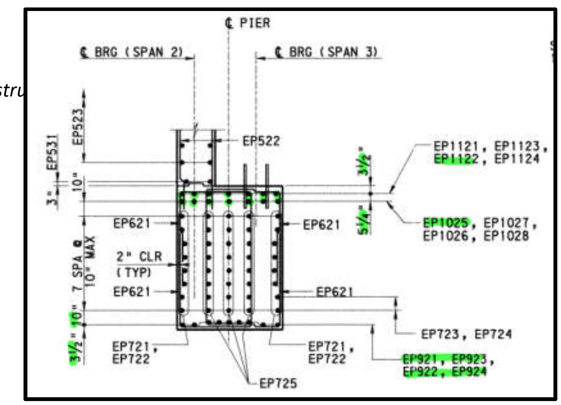


Critical Factored Load *per Staad output*

Dead Load	33.503	kip	
Live Load	80.735	kip	

AASHTO LRFD 5.8.2.4 - Proportioning of Ties

Phi	0.9	*AASHTO LRFD 5.5.4.2, "for tension in stru*	
		and-tie models"	
f_y	60	ksi	*per plans*
Layer 1			
Size Rebar	9		*per plans*
Area Rebar	1	in^2	*area of one rebar*
No. Bar	9		*number of bars*
Layer 2			
Size Rebar			
Area Rebar		in^2	
No. Bar			
A_{st}	9	in^2	*area of steel per plans*
P_n	540	kip	*nominal resistance*
Nodal Strength	435	kip	
P_r	**435**	**kip**	



Rating Factor

condition factor	1		
system factor	1		
RF1	12.984		*rating factor (C/DL)*
RF2	4.973		*traditional rating factor (C-DL)/LL*
RF	**4.973**	OK	*minimum of two calculated rating factors*

Design Ratio

DR	**3.808**	OK	*calculated design ratio*

Nodal and Strut Capacity:

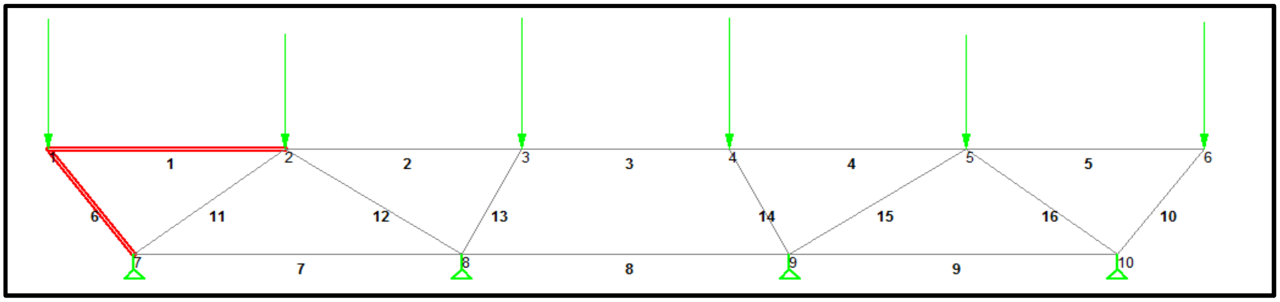
Proportion Nodal Regions

Location

Staad Model	Node 1
Description	*Girder 9*

Purpose/Objective

Verify the strength of the nodal zone using STM



Critical Factored Load

Girder Load	**G9**		
Dead Load	606.26	kip	*per BrR output*
Live Load	112.52	kip	*per BrR output*
Tension Tie	**1**		
Dead Load	546.671	kip	*per Staad output*
Live Load	88.795	kip	*per Staad output*
Compression Strut	**6**		
Dead Load	937.86	kip	*per Staad output*
Live Load	152.34	kip	*per Staad output*

AASHTO LRFD 5.8.2.5 - Proportioning Node Regions

AASHTO LRFD 5.8.2.5.1 - Strength of nodal face

Phi	0.7		*AASHTO LRFD 5.5.4.2, "for compression in strut-and-tie models"*
m	1.51		confinement modification factor
f'c	3.00	ksi	per plans
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (bearing face)*
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (back face)*
v	0.60		*Concrete efficiency factor, Table 5.8.2.5.3a-1: assume 0.60 (strut-node interface)*

Bearing Girder/Column Face of a CCT Node

fcu	3.17	ksi	*limiting compressive stress of the node face*
bearing area	970.59	in^2	*bearing area of masonry plate*
Pn	3074.24	kip	*nominal resistance of node face*
Pr	**2151.97**	**kip**	

Rating Factor

condition factor	1
system factor	1

RF1	3.55		*rating factor (C/DL)*
RF2	13.74		*traditional rating factor (C-DL)/LL*
RF	**3.55**	OK	*minimum*

Design Ratio

DR	**2.99**	OK	*Design Ratio*

Strut-Node Interface of a CCT Node

fcu	2.71	ksi	*limiting compressive stress of the node face*
bearing width	44.63	in	*bearing width of masonry plate, per plans "c"*
theta	54.33	deg	*angle between compression strut and tension tie*
height back face	11.78	in	*The height of the back face is taken as twice the distance from the bottom surface of the bent cap to the centroid of the longitudinal reinforcement (CCT nodes)*
width strut	43.12	in	*calculated width of strut*
bearing depth	21.75	in	*depth of bearing*
bearing area	937.92	in^2	*calculated bearing area*
Pn	2546.36	kip	
Pr	**1782.45**	**kip**	

Rating Factor

30.33

condition factor	1
system factor	1

RF1	1.90		*rating factor (C/DL)*
RF2	5.54		*traditional rating factor (C-DL)/LL*
RF	**1.90**	OK	*minimum*

Design Ratio

DR	**1.63**	OK	*Design Ratio*

Proportion Nodal Regions

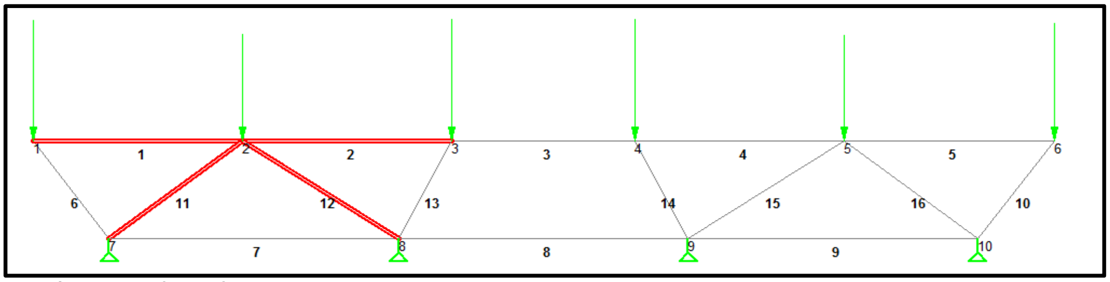
Location

Staad Model: Node 2

Description: *Girder 10*

Purpose/Objective

Verify the strength of the nodal zone using STM



Critical Factored Load

LEFT	**CCT**		**Right**	**CCT**		
Girder Load	**Girder 10**					
Dead Load	673.89	kip				*girder/column load before load consolidation*
Live Load	260.38	kip				*girder/column load before load consolidation*
Girder/Col. Load	**Girder 10**		**Girder/Col. Load**	**Girder 10**		
Dead Load	430.63	kip	Dead Load	243.26	kip	
Live Load	166.39	kip	Live Load	93.99	kip	
Tension Tie	**1**		**Tension Tie**	**2**		
Dead Load	546.67	kip	Dead Load	277.88	kip	
Live Load	88.80	kip	Live Load	103.59	kip	
Compression Strut	**11**	*resolved*	**Compression Strut**	**12**	*resolved*	
Dead Load	741.75	kip	Dead Load	376.55	kip	
Live Load	216.74	kip	Live Load	222.90	kip	
Back Face Force			**Back Face Force**			
Dead Load	-	kip	Dead Load	-	kip	*back face force need not be calculated for a CCT node, when the reinforcement is developed*
Live Load	-	kip	Live Load	-	kip	
Bearing Width	40.13	in	Bearing Width	40.13	in	*bearing width of girder/col*
height panel	6.97	ft	height panel	6.97	ft	*height of panel*
length panel	8.75	ft	length panel	10.25	ft*	*length of panel is back calculated from the known, resolved angle*
theta	38.53	deg	theta	34.20	deg	*angle between compression strut and tension tie*
bearing width	25.64	in	bearing width	14.47	in	*calculated bearing width at sub-divided node*
distance edge	12.82	in	distance edge	7.24	in	*edge distance at subdivided node*
shortened distance	7.24	in	shortened distance	12.83	in	*distance from centroid of bearing*
revised length	8.15	ft	revised length	9.18	ft	*length of panel at subdivided node*
revised theta	40.54	deg	revised theta	37.19	deg	*angle between compression strut and tension*

AASHTO LRFD 5.8.2.5 - Proportioning Node Regions

Phi	0.70		Phi	0.70		*AASHTO LRFD 5.5.4.2, "for compression in strut-and-tie models"*
m	1.60		m	1.60		*confinement modification factor*
f'c	3.00		f'c	3.00		*ksi, per plans*
v	0.70		v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (bearing face)*
v	0.70		v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (back face)*
v	0.60		v	0.60		*Concrete efficiency factor, Table 5.8.2.5.3a-1: assume 0.60 (strut-node interface)*

Bearing Column/Girder Face of a CCT Node

fcu	3.35	ksi	fcu	3.35	ksi	*limiting compressive stress of the node face*
bearing width	21.63	in	bearing width	21.63	in	*bearing depth of masonry plate, "b"*
bearing area	554.48	in^2	bearing area	312.96	in^2	*bearing area of the node face*
Pn	1857.46	kip	Pn	1048.38	kip	*nominal resistance of node face*
Pr	**1300.22**	**kip**	**Pr**	**733.87**	**kip**	

Rating Factor			**Rating Factor**			
condition factor	1		condition factor	1		
system factor	1		system factor	1		
RF1	3.02		RF1	3.02		*rating factor (C/DL)*
RF2	5.23		RF2	5.22		*traditional rating factor (C-DL)/LL*
RF	**3.02**	OK	**RF**	**3.02**	OK	*minimum*
Design Ratio			**Design Ratio**			
DR	**2.18**	OK	**DR**	**2.18**	OK	*design ratio*

Strut-Node Interface of a CCT Node

fcu	2.87	ksi	fcu	2.87	ksi	*limiting compressive stress of the node face*
bearing width	25.64	in	bearing width	14.47	in	*bearing width, previously calculated above*
angle strut	40.54	deg	angle strut	37.19	deg	
height	11.78	in	height	11.78	in	*The height of the back face is taken as twice the distance from the bottom surface of the bent cap to the centroid of the longitudinal reinforcement (CCT nodes)*
width strut	25.62	in	width strut	18.13	in	*calculated width of strut*
bearing depth	21.63	in	bearing depth	21.63	in	*depth of beairng*
bearing area	553.98	in^2	bearing area	392.11	in^2	*calculated bearing area*
Pn	1590.66	kip	Pn	1125.89	kip	*nominal resistance of node face*
Pr	**1113.46**	**kip**	**Pr**	**788.12**	**kip**	

Rating Factor			**Rating Factor**			
condition factor	1		condition factor	1		
system factor	1		system factor	1		
RF1	1.50		RF1	2.09		*rating factor (C/DL)*
RF2	1.72		RF2	1.85		*traditional rating factor (C-DL)/LL*
RF	**1.50**	OK	**RF**	**1.85**	OK	*minimum*
Design Ratio			**Design Ratio**			
DR	**1.16**	OK	**DR**	**1.31**	OK	*design ratio*

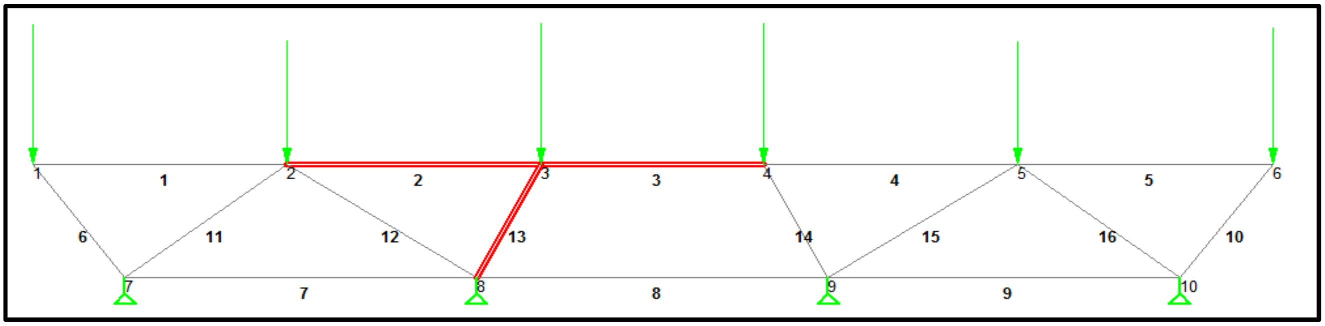
Proportion Nodal Regions

Location

Staad Model	Node 3
Description	*Girder 11*

Purpose/Objective

Verify the strength of the nodal zone using STM



Critical Factored Load

Girder Load	**G11**		
Dead Load	770.14	kip	*per BrR output*
Live Load	345.24	kip	*per BrR output*
Tension Tie	**2**		
Dead Load	277.878	kip	*per Staad output*
Live Load	103.585	kip	*per Staad output*
Compression Strut	**13**		
Dead Load	861.79	kip	*per Staad output*
Live Load	386.32	kip	*per Staad output*
Back Face Force	**3**		
Dead Load	108.85	kip	*per Staad output*
Live Load	69.78	kip	*per Staad output*

AASHTO LRFD 5.8.2.5 - Proportioning Node Regions

AASHTO LRFD 5.8.2.5.1 - Strength of nodal face

Phi	0.7		*AASHTO LRFD 5.5.4.2, "for compression in strut-and-tie models"*
m	1.51		confinement modification factor
f'c	3.00	ksi	per plans
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (bearing face)*
v	0.70		*Concrete efficiency factor, Table 5.8.2.5.3a-1 (back face)*
v	0.60		*Concrete efficiency factor, Table 5.8.2.5.3a-1: assume 0.60 (strut-node interface)*

Bearing Girder/Column Face of a CCT Node

fcu	3.17	ksi	*limiting compressive stress of the node face*
bearing area	970.59	in^2	*bearing area of masonry plate*
Pn	3074.24	kip	*nominal resistance of node face*
Pr	**2151.97**	**kip**	

Rating Factor

condition factor	1		
system factor	1		
RF1	2.79		*rating factor (C/DL)*
RF2	4.00		*traditional rating factor (C-DL)/LL*
RF	**2.79**	OK	*minimum*

Design Ratio

DR	**1.93**	OK	*Design Ratio*

Strut-Node Interface of a CCT Node

fcu	2.71	ksi	*limiting compressive stress of the node face*
bearing width	44.63	in	*bearing width of masonry plate, per plans "c"*
theta	63.33	deg	*angle between compression strut and tension tie*
height back face	11.78	in	*The height of the back face is taken as twice the distance from the bottom surface of the bent cap to the centroid of the longitudinal reinforcement (CCT nodes)*
width strut	45.16	in	*calculated width of strut*
bearing depth	21.75	in	*depth of bearing*
bearing area	982.31	in^2	*calculated bearing area*
Pn	2666.88	kip	
Pr	**1866.82**	**kip**	

Rating Factor

condition factor	1		
system factor	1		
RF1	2.17		*rating factor (C/DL)*
RF2	2.60		*traditional rating factor (C-DL)/LL*
RF	**2.17**	OK	*minimum*

Design Ratio

DR	**1.50**	OK	*Design Ratio*

Serviceability Check

Purpose/Objective

determine if adequate reinforment has been provided to prevent cracking (temperature and shrinkage)

AASHTO LRFD Eq. C5.8.2.6-1 (Vertical Reinforcement)

Stirrup Information

Size	6.00		*smallest size stirrup*
diameter	0.75	in	*diameter of rebar*
Area	0.44	in^2	*area of one rebar*
Spacing	18.00	in	*input largest spacing*
No. Stirrup legs	10.00		*number of stirrup legs to be considered in the cross-section*
width member	6.00	ft	
A_h	4.42	in^2	
b_w	72.00	in	
s_h	18.00	in	
A_{sh}	0.003408846	*OK, minimum horizontal crack control reinforcement provided*	

AASHTO LRFD Eq. C5.8.2.6-2 (Horizontal Reinforcement)

Reinforcement Information

Size	9.00		*smallest size rebar*
diameter	1.13	in	*diameter of rebar*
Area	0.99	in^2	*area of one rebar*
Spacing	5.25	in	*maximum spacing used*
No. Bar	7.00		*number of bars to be considered in the cross-section*
width member	6.00	ft	
A_v	6.96	in^2	
b_w	72.00	in	
s_v	5.25	in	
A_{sh}	0.018407769	*OK, minimum horizontal crack control reinforcement provided*	

BRIDGE PIER CAP 2

Analysis Input

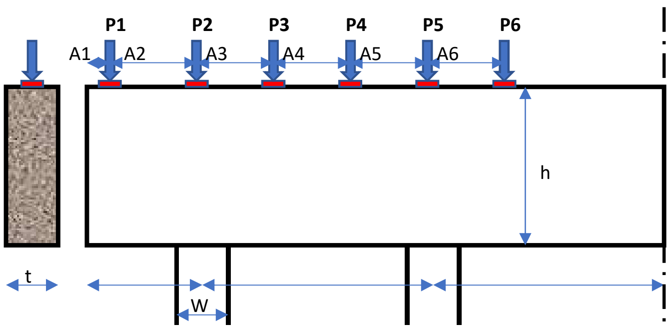
Bridge Details:

Bridge Name:		Pier Number:	
	XXXXXXX	Designer:	XXXXXXXX
	XXXXXXX	Date:	

1. Total Number of Columns (Piers)	4

☐ Asymmetrical

2. Generate



Centerline

Note:

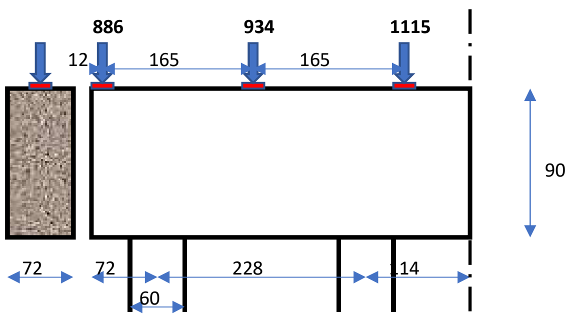
3. Geometry Details			
Distance from start of the pier cap to center of first column (C1)	6 ft		
Distance from center of first column to center of second column (C2)	19 ft		
Distance from center of second column to centerline of pier cap (C3)	9 ft		
Column width (W)	60 in	Circular	
Depth of pier cap (h)			
Thickness of pier cap (t)			

4. Factored Loads and their Position			
Distance of First Load from the Edge of Pier Cap	1 ft		
Spacing Between the Girders	14 ft		
Factored Load	400 k		

Generate Load Table

Factored Load		Distance			
P1	886 k	1 ft	0.0 in	12.0 in	A1
P2	0 k	0 ft	0.0 in	0.0 in	A2
P3	934 k	13 ft	9.0 in	165.0 in	A3
P4	0 k	0 ft	0.0 in	0.0 in	A4
P5	0 k	0 ft	0.0 in	0.0 in	A5
P6	1115 k	13 ft	9.0 in	165.0 in	A6

5. Generate



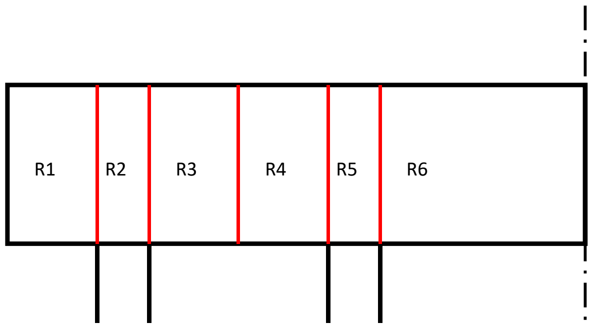
Centerline

6. Check whether the Pier Cap is Deep			
Region	*Shear span (a)*	*a/d ratio:*	*Result*
R1	49.1 in	0.61	Deep Region
R2	0.0 in	0.00	Zero Region
R3	85.9 in	1.06	Deep Region
R4	101.4 in	1.25	Deep Region
R5	0.0 in	0.00	Zero Region
R6	34 in	0.42	Deep Region

This pier cap is deep.
Please continue with Section 7.

7. Material Properties	
Concrete strength (f'_c)	3.00 ksi
Rebar yield strength (f_y)	60.0 ksi
Enter the clear cover	2.0 in
Stirrup yield strength(f_y)	60.0 ksi
Stirrup bar area	0.31 in^2

8. Resistance Factors Used	
For concrete	0.7
For longitudinal rebar	0.9
For stirrup	0.9
CCC v-factor for the bearing and back face	0.85
CCT v-factor for the bearing and back face	0.7
CTT v-factor for the bearing and back face	0.65



9. Reinforcement Details

9A. Longitudinal Reinforcement				
Region	Top Steel (in^2, in)		Bottom Steel (in^2, in)	
	Total Area (A_t)	Centroid (C_t)	Total Area (A_b)	Centroid (C_b)
R1	19.81	3.2	9	3.2
R2	19.81	3.2	9	3.2
R3	19.81	3.2	9	3.2
R4	19.81	3.2	9	3.2
R5	19.81	3.2	9	3.2
R6	19.81	3.2	9	3.2

Rebar Area & Centroid'!A1

9B. Transverse Reinforcement		
Region	No. of Legs	Stirrup Spacing
R1	5	9.0 in
R2	5	9.0 in
R3	5	9.0 in
R4	5	9.0 in
R5	5	9.0 in
R6	5	9.0 in

9C. Min Horizontal Crack Control Reinforcement	
Crack control rebar area (in^2)	0.44
Spacing (in)	11.0
No of layers of crack control rebar	2
Provided crack control reinforcement	0.11%
Code required crack control reinforcement	0.30%

10. Base Plate Dimensions	
Base plate (bearing plate) length parallel to the pier cap (L_b)	32.0 in
Base plate (bearing plate) width perpendicular to the pier cap (W_b)	32.0 in

11. Reinforcement Development

Horizontal length available (L_d)	27 in

Top Tension Bars	
Enter the diameter of the top longitudinal bar:	1.13 in
Enter the length of the hook provided:	32 in
Basic development length	25 in

It qualifies for 90° hook.

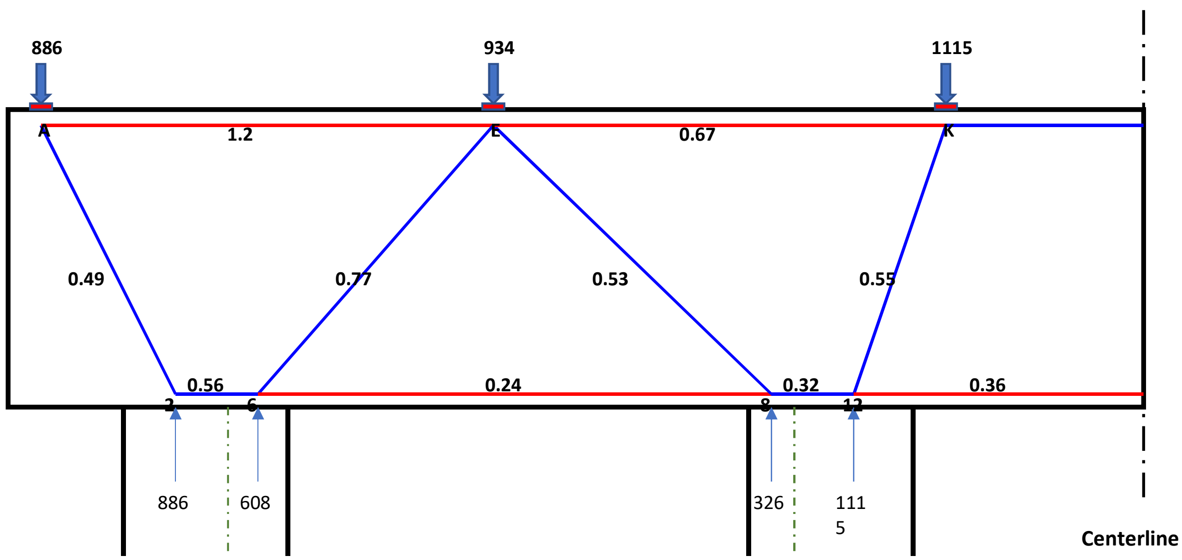
Modification Factor		
1. Are bars epoxy coated?	Yes	1.2
2. Is the side cover for No. 11 bar and smaller, normal to the plane of hook, is not less than 2.5 in, and 90° hook, cover on bar extension beyond hook not less than 2.0 in?	Yes	0.7

Required development length	21 in
Available development length (L_d)	27 in

Reinforcement Capacity Multiplier:	1.00

Analysis Output

12. Generate Output Model



Note: The above figure shows the output model with Utilization Ratio along with the member which are color coded. The node numbers are also printed for every node. This output model is based on below calculation details.

13. Strut and Tie Output Summary

STM Members

Input 0 for "Do not use Tie"
Input 1 for "Use Tie"
Input Your Option Down Here
↓↓↓↓↓↓↓

Option	Member Code	Force (k)	Capacity (k)	Utilization Ratio	Result	
	A-E	521	435	1.20	Flexure Overloaded	Top Members
	E-K	291	435	0.67	PASS	
	K-Q	0	435	0.00	-	
	2-6	-521	-921	0.56	PASS	Bottom Members
	6-8	104	435	0.24	PASS	
	8-12	-291	-921	0.32	PASS	
	12-14	158	435	0.36	PASS	
	B-1	0	-	0.00	-	Vertical Members
	F-5	0	-	0.00	-	
	H-7	0	-	0.00	-	
	L-11	0	-	0.00	-	
0	A-2	-1028	-2098	0.49	PASS	Inclined Members
0	E-6	-872	-1131	0.77	PASS	
0	E-8	-512	-974	0.53	PASS	
0	K-12	-1202	-2182	0.55	PASS	
Bearing Areas — Nodes at ⇉	A	886	1935	0.46	PASS	1
	E	934	1935	0.48	PASS	
	K	1115	1935	0.58	PASS	
	2	886	1585	0.56	PASS	2
	6	608	1088	0.56	PASS	
	8	326	604	0.54	PASS	
	12	1115	2068	0.54	PASS	

Re-Generate Output Model