12-8-2014

A Review of the Design of a Pedestrian Bridge in Cape Girardeau as Part of the Senior Design Requirements of the College of Engineering

Amir Rezaeivahdati
amirr@siu.edu

A Review of the Design of a Pedestrian Bridge in Cape Girardeau

as Part of the Senior Design Requirements of the College of Engineering

Amir Rezaeivahdati

A thesis submitted to the University Honors Program

in partial fulfillment of the requirements for the Honors Diploma

Southern Illinois University

December 2014

Table of Contents

Introduction

This paper analyzes the process, challenges and results of the design of the superstructure of a pedestrian bridge in Cape Girardeau, Missouri, as part of the extension of the existing Riverwalk trail along the Mississippi River. The project was assigned to three groups of students in the Senior Design B class of Fall 2014.

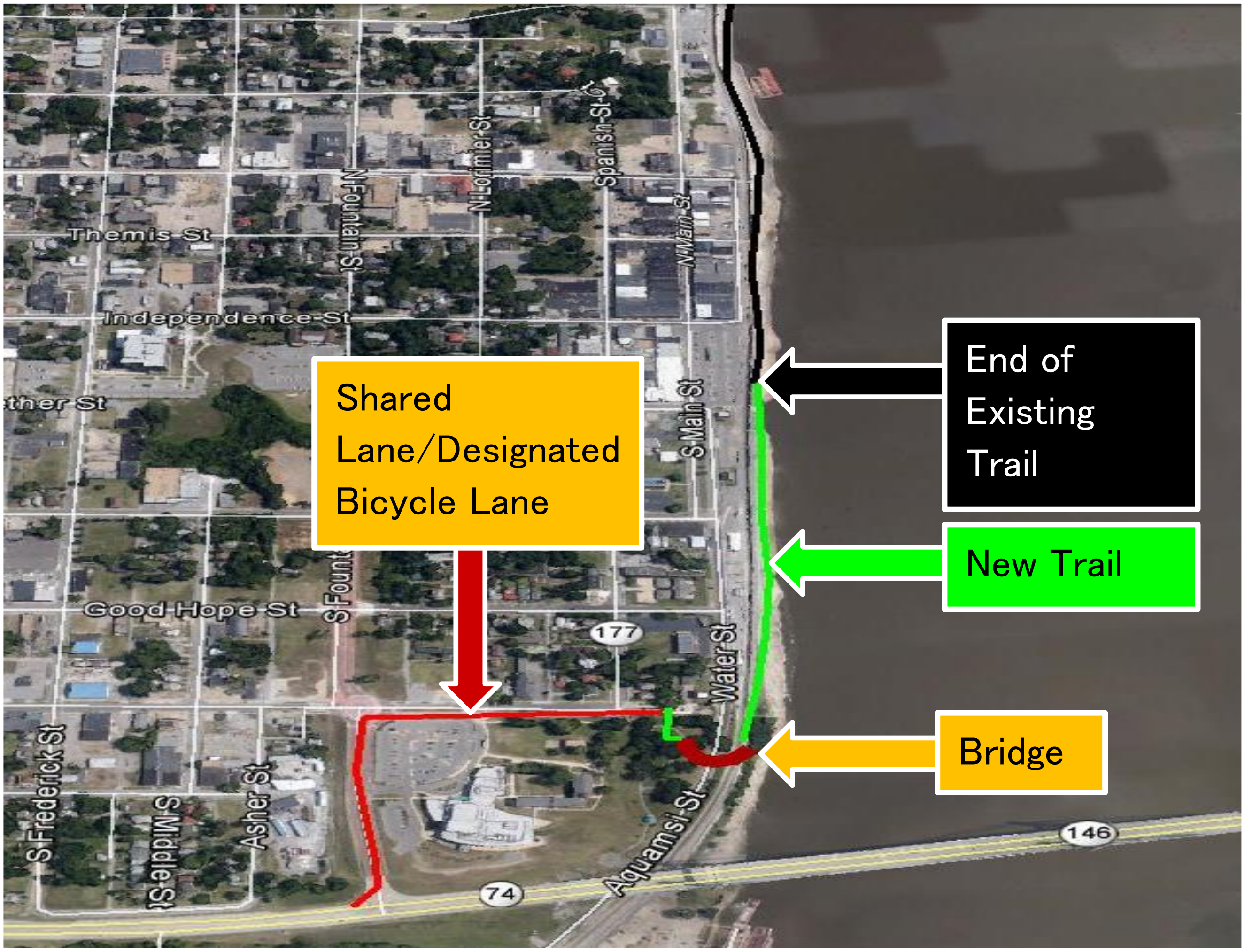
The original project consisted of an approximately 1700 ft pedestrian/bicycle trail along the river, a bridge over Aquamsi street and the railroad track and designing bicycle paths along the Fountain street and Morgan Oak street to connect to the existing trail (Figure 1.) Two groups consisted of six students, and our group was the only group with five students.

I was in charge of the design of the bridge superstructure. We were provided with a geotechnical report making recommendations for design, as well as some maps locating the project site. Additionally, we could consult with the faculty members of the department, members of the professional advisory board, and other professionals and experts in the field with the permission of our supervisor, Professor Eichfeld. This paper will be a depiction of the process of the design of the superstructure, the challenges I faced as a senior civil engineering student and my solutions to them, suggestions to provide a better learning experience to future students of the senior design class, and the final product of the project.

Assignment

The bridge was to be designed to transport the pedestrian and bicyclist traffic over Aquamsi St. and railroad tracks. We were advised to build the bridge in the southern part of Aquamsi St. as not to obstruct the view of the existing overlook which is located at the intersection of Aquamsi St. and Water St. (Figure 1.) The bridge was likely to trespass into the property of the Southeast Missouri State University, which is confined by Aquamsi St. Morgan Oak St, Fountain St. and Shawnee parkway. Therefore, relevant permits would have to be requested from the university for construction purposes. The discussion of the permits that would have to be obtained is beyond the scope of this paper.

Figure 1. Google Earth Image of Riverwalk Trail Extension Cape Girardeau, Missouri



The bridge would have to comply with the Americans with Disabilities Act (ADA) requirements. Additionally, it would have to comply with the International Building Code (IBC), American Association of State Highway and Transportation (AASHTO), and Federal Highway Administration (FHWA) standards. The bridge design would also be compliant with all the standards and requirements of the State of Missouri Department of Transportation (MoDOT.)

The bridge is located in a site class E seismic category. This implies that the soil in the site is mostly clayish and soft, and thus exhibits low resistance to earthquake loads. Additionally, the probability of an earthquake in the site is considerably high, due to its presence near the New

Madrid seismic zone, despite the fact that a 2009 research by a team from Northwestern and Purdue universities postulated that the fault might be shutting down. After studying the fault motion over eight years, the researchers decided that the level of activity has decreased significantly compared to the expectations. “The last large earthquake in the New Madrid seismic zone were magnitude 7- 7.6 events in 1811 and 1812.” (Gardner, 2009) These findings, however, were not implemented by the National Earthquake Prediction Evaluation Council in their 2011 convention. (Monastresky, 2011) This council advises the United States Geological Survey (USGS), which provides the basis of codes and standards for other resources such as IBC and AASHTO. Therefore the IBC 2012 and the USGS seismic hazard maps were used as references for the basic information necessary for seismic calculations and design. Other considerations in the design included the cost of the design and the aesthetic aspects of the design, given the recreational function of the bridge and its proximity to the Bill Emerson Memorial Bridge.

Design of the Superstructure

Enabling the pedestrians and the bicyclists to cross over Aquamsi St and the railroad tracks to the Mississippi River side using a safe and accessible structure was kept in mind throughout the design process of the bridge structure. Eventually, two bridge structures were decided to be designed to address the different issues such as costs, minimum height requirements, zoning requirements of the City of Cape Girardeau, seismic considerations, aesthetic considerations and functionality.

The first bridge structure will be a 120-ft structure, with two simple spans of length 60 ft. The first span will have a 3% slope, such that the rainwater will runoff towards the west side of the Aquamsi St. The other span will have a slope of 5% to direct the rainwater towards the Mississippi River. (The rain and snow water will be collected using the drainage system of the bridge) This bridge structure will have a clearance of 29 ft and 9 inches at the highest point near the Mississippi River (Refer to page 46 for drawing). At this point will also be the new overlook with a wide and clear view of the Mississippi River and the Bill Emerson Memorial Bridge. The bridge will have a clearance of 17ft and 3 inces ft above Aquamsi St. This was due to the fact that our site observations proved that construction trucks used this road, and thus the 14-foot required standard would not be sufficient. Additionally, one foot of clearance was added to the

16’ requirement to meet the *LRFD Guide Specifications for the Design of Pedestrian Bridges (2009.)* The additional three inches was provided to account for future settlements, given the soft nature of soil in the area and insufficient geotechnical data to determine future settlement. Additionally, Mr. Joseph Lenzini, a member of the professional advisory board of the civil engineering department who was acting as our team’s mentor, suggested that the calculations for future settlement is beyond the scope of the project, in particular with the insufficient geological information available. The second Bridge Structure will be directed towards the rest of the trail at a constant slope of 5% to meet all ADA requirements. The bridge will consist of 3 simple spans of 60 ft to form a total length of 180 ft. (Refer to page 47 for drawing)

Various issues were taken into account to determine the bridge type. Primarily, a concrete superstructure was avoided to minimize the weight of the bridge, considering the fact that the bridge is in a site class E seismic category. Among the steel bridges, a three girder I-Girder steel bridge was selected. Federal Highway Administration’s *Steel Bridge Design Handbook, Selectig the Right Bridge Type (2012)* was used as a reference in making this decision. Although the relatively small loads on the bridge would have made a two girder bridge possible, three girders were preferred. According to FHWA’s Steel Bridge Design Handbook, “Two girder bridges are crack fractural critical, meaning that the failure of one of the main girders could lead directly to the failure of the entire bridge. The main girders cannot be made composite with the bridge deck, meaning that the deck offers no strength benefit to the girders. The top flanges of the girders in the compression regions are braced only at the floor beam spacing rather than full length, as would be the case for a composite deck girder bride.” (pg. 6) High seismic loads on this bridge structure called for a composite deck where the lateral strength of the deck could resist the wind and earthquake loads. FHWA states, “In composite decks, the strength of the bridge deck is included in the section properties of the girders. Additionally, shop layout is generally less complex than would be the case for through girders, trusses and arches.“ (pg. 6) Therefore, a three girder steel bridge was selected as the optimum design.

The slab of the bridge was designed to be 7 inches thick, reinforced with #4 bars in the longitudinal and transverse directions, spaced at 16 inches in both directions. The slab was designed using the equivalent strip method and as a one-way slab, where the girders are considered supports and a 12-inch strip of the slab is considered a simple beam. Although it is also possible to consider the strip of the slab as a continuous beam, due to the limits in using computer software in analysis, it was decided to consider the strip of slab as a simple beam to make hand calculations possible. In this design, a 150 pcf concrete with a compressive strength of 4000 psi and steel Grade 60 for reinforcement were used. The overhang, at 1.5 ft, was designed to be within the specified 0.33 times the girder spacing, as suggested by FHWA. The slab was made composite to the girders, using article 6.10.10 of AASHTO LRFD Bridge Design Specifications (2012). To make the deck composite, two shear studs of diameter 0.75" were used in a row, and rows were spaced 10 in. apart along the girder. (Refer to pages 48 and 51 for drawings)

Steel Construction Manual (2011) was used as the reference for the design of the girders. The girders were designed to be W12X53 sections, which were the lightest sections to satisfy the requirements. The moment capacity of the girders was designed to ensure that the girders stay in the elastic range. The girders were designed for bending capacity (Demand to Capacity ratio: 0.87), deflection (Demand to Capacity ratio: 0.79) and shear strength (Demand to Capacity ratio: 0.04) It is clear that the shear strength is by far greater than that required. However the bending capacity and deflection were designed to ensure safety and efficiency.

To transfer the transverse loads, K-type cross frames were selected, as per recommendations of FHWA. Cross frames were chosen over diaphragms, considering that diaphragms consist of a flexural component, cross frames have a truss frame work. The selection between K-type and X-type was done considering the aspect ratio of the girder spacing to girder depth. In this design, this aspect ratio was greater than 1-1.5, and thus a K-type cross frame was preferred, as per recommendations of FHWA. The angles used in the cross frames were L 2X2X5/16. The connections between the angles and the 3/8"-thick plate were done by welding with a thickness of 0.2 inches. 2.53 inches of welding would satisfy the demands, however, 3 inches of welding on each side of the angle was recommended. The angle size provided exceeds the demand in all calculations by far. (Refer to page 52 for drawing)

Two forms of bearing were provided for spans. The ends of the span which require expansion were designed with a PTFE plate to provide longitudinal translation. The fixed ends were simple elastomeric bearing natural rubber bearing pads with steel reinforcement. For the spans exceeding a slope of 3%, which includes all spans except the first span closest to the west side of Aquamsi St, tapered sole plates shall also be used. The bearings were designed, using Method A, as specified in AASHTO LRFD Bridge Design Specifications, article 14.7.6. (2012). This method exempts the design from combined rotations and compressions check. The selection of natural rubber was done with the aid of AASHTO M251 (2006). The design was performed using temperature zone C, with a 50 year low temperature of -30F. Additionally, AASHTO Bridge Construction Specifications, article 18.2 (2004) was used for general specifications. (Refer to pages 49 and 50 for drawings)

The seismic load was calculated using Uniform Load Method, as specified by AASHTO LRFD Bridge Design Specifications article 4.7.4.3.2c (2012). This method is suitable for regular bridges that respond principally in their fundamental mode of vibration, and is known to overestimate the transverse shears at the abutments by up to 100%. Using this method, the equivalent static load on the 120-ft bridge structure was 190 k/ft and on the 180-ft bridge structure it was 179 k/ft. To calculate C_{ms}, the dimensionless elastic seismic response, article 3.10.4.2, as well as information obtained from USGS website was used. Coordinate numbers 37.3092N and 89.5464W were used to locate Cape Girardeau. The site was classified as Class E, Occupancy risk category I, and seismic zone 4. Additionally, in obtaining the information from USGS website, International Building Code (2011) was selected as the design code reference. This was primarily because AASHTO does not provide Seismic Hazard mappings corresponding to 2 percent probability of exceedance in 50 years, but 7 percent probability of exceedance in 75 years.

Throughout the design, load combinations Strength I, Strength III, Service I, and Extreme Event I were considered in the calculations. Load combinations Strength II, IV and V, as well as Service II, III, and IV were not required for the design of this bridge. Factors related to ductility, redundancy and operational classification were taken from article 1.3.2 of AASHTO LRDF Bridge Design Specifications (2012). Load Factors were taken from Table 3.4.1-1 and 3.4.1-2 of the same manual. An H10 maintenance vehicle was considered as live load on the bridge as specified by article 3.2 of LRFD Guide Specifications for the Design of Pedestrian Bridges(2009). The same article exempts the designers from dynamic load analysis for this loading. Article 3.1 of the same manual recommends a pedestrian live load of 90 psf, which was used in design. Article 3.4 of the same manual recommends using AASHTO Signs 3.8, 3.9

(2009) to calculate wind load. This calculation was performed using a basic wind load of 90 mph. The gust effect and drag coefficient were both taken at the maximum allowable of 1.3. This selection was a conservative choice for lack of better information. The importance factor of wind load was taken as 1.15 for a 100 year recurrence interval, as recommended by LRFD guide specifications for the design of pedestrian bridges (2009). All calculations for the design were performed in Microsoft Excel using the manuals and standards mentioned. Screenshots of these Excel sheets are available in the Appendix section of this paper.

Challenges and Suggestions

I faced several challenges to complete this project. These challenges - despite their role in preparing us, the students, to improve our problem solving skills and initiative – could have been planned better to serve a more educational purpose. I will try to analyze the challenges, their causes, and the solutions my team members and I thought to be the best.

One of the most fundamental challenges to complete this project, which all teams equally encountered, was students' lack of experience and exposure to comprehensive design experiences. In most of our classes we learn how to design a single beam or a single column, for instance, at a time, completely isolating it from the system it would realistically be a part of. Furthermore, even such designs were barely holistic, and typically focused on a certain aspect of the design. For example, we would design a column considering buckling only, or compressive strength only. As a result, with the beginning of the semester we found ourselves facing a relatively large project with minimal knowledge and experience in the field. This challenge, in my opinion, was overall a positive experience, as I was able to perform a comprehensive research of the resources available and methods common in practice. However, the amount of time we were given, one semester, was not remotely appropriate to perform an in-depth study of the resources and a subsequently proper design. Therefore, I had to select certain aspects of the design of each part of the superstructure to study more comprehensively, and forego other aspects. For example, the design of the bridge girders has many facets which require much attention and time, and whose study could be very beneficial educationally. However, because of the lack of time, I had to perform the basic deflection, flexure and shear calculations for the girders only. Although the assessment of the design will be consistent with the time and resources given, yet, the expansion of the preliminary study for design into Senior Design A class could provide students a greater chance to learn the real-world challenges and meet the expectations of a project of such a scale.

Another challenge I faced during the design process was the absence of specific instructions as to the means of design we were allowed to use. I started using computer software, CSiBridge by Computers and Structures, Inc, about two months before the official start date of the project in Fall 2014. My logic to do so was that through this, I could learn an essential software in the field of bridge engineering and design, and provide a final product which would be very reliable and realistic. Indeed, the learning of this software was the peak of my learning experience through this project. By the end of September 2014, a model of the bridge superstructure was completed and ready for analysis and design. However, at this time I was notified that the professional advisory board had decided that students should use hand calculations to finish the project, a policy which was later loosened by allowing teams to use pre-fabricated designs. Clearly, the fact that parts of the design were beyond the scope of the project and not expected from us was what we discovered through the course of the semester. As a result, I took a different approach to the design. The new approach had to be significantly efficient, as I had even less time now to finish the project. The most important consequence of changing the design method from computer-based to hand-calculations was the fact that our initial design could not be done with hand calculations, as the calculations would be very extensive and time-exhausting. Our initial design consisted of a continuous span throughout the bridge structures. This would mathematically reduce the required bending moment in the girders, and thus smaller, more economical sections could be used. The new design, however, had to be in the form of simple spans, whose hand calculations were more plausible. The same goes for the design of the slab, where the equivalent strip method had to be used assuming girders were simple supports. This assumption is not very accurate compared to considering the slab as a continuous beam. The reason for that is the composite nature of the deck.

I believe that the above issue could have been avoided if goal-based instructions were provided as to the expectations of the final design and the methods used for design. For example, in the future students could be instructed that a perfectly professional design is not the ultimate expectation of such a project, but rather the importance of the course lies in the educational experience of finding innovative solutions to problems. Or instead, the ultimate purpose of the course could be to practice the combining of the knowledge students gained in previous classes and using that correctly, rather than performing extensive research in other areas.

The other challenge I faced was lack of experience in real-world solutions engineers use in their design. In many instances, I was able to identify the challenge ahead and could provide a functional solution to the problem. However, my lack of work experience coupled with the educational system's focus on theoretical rather than practical knowledge diverted my solutions from those of the actual engineering practice. This challenge, however, was adequately and properly predicted and addressed by the department by appointing members of the professional advisory board who work in engineering firms as mentors of teams. Therefore, I was able to receive helpful and insightful guidance from our mentor, Mr. Joseph Lenzini. The importance of this challenge and the way it was overcome was how this project bridged students' class experience with real-world problem solving skills, an experience that is second only to actual work experience in the field.

Conclusion

The assigned Senior Design Project of the Fall 2014 class included the design of a bridge crossing Aquamsi St. and railroad tracks as specified in Figure 1. The superstructure of the bridge was designed by the author of this paper. The excel files used for design as well as the drawings are available in the appendix of this paper. The final design determined the bridge structures to be of I-girder steel bridge type, with K-type cross-frames at 30 ft and elastomeric bearings with and without PTFE plates for longitudinal translation. The bridge deck was a 7-inch reinforced slab made composite with girders using shear studs.

The project offered many challenges, the most important of which were lack of practical work experience of students in the field, lack of comprehensive design experience in class, insufficient time for a complete design, and unspecific instructions to deliver the project. Lack of work experience was appropriately addressed by the civil engineering department by appointing members of the professional advisory board as student team mentors. However, the other issues could be resolved in the future by allowing the students to start the projects in Senior Design A classes or alternatively providing specific instructions as to the scope of the expected deliverables.

References

AASHTO LRFD bridge design specifications (5th ed.). (2012). Washington, DC: American Association of State Highway and Transportation Officials.

AASHTO LRFD bridge construction specifications. With 2009 interim revisions. (2nd ed.) (2004). Washington, D.C.: American Association of State Highway and Transportation Officials.

Gardner E. (2009) *New Madrid fault system may be shutting down*. Retrieved on Nov 29, 2014 from http://phys.org/news156169464.html

LRFD guide specifications for the design of pedestrian bridges. (2009). Washington, DC: American Association of State Highway and Transportation Officials.

Monastresky R. (2011) *Seth Stein: The quake killer*. Nature, international weekly journal of science. 479, 166-170 (2011) Retrieved on Nov 29, 2014 from http://www.nature.com/news/2011/111109/full/479166a.html

Standard specifications for structural supports for highway signs, luminaires, and traffic signals with 2010 and 2011 interim revisions. (5th ed.). (2009). Washington, D.C.: American Association of State Highway and Transportation Officials.

Steel construction manual (14th ed.). (2011). Chicago, Ill.: American Institute of Steel Construction.

2012 international building code. (2011). Country Club Hills, Ill.: ICC.

Appendix

Loading for the Bridge Deck					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.541666667	0.975	0	0.08	0.01
Span length (ft)	120				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	1.065			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.04				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2	0	0	0	1.065
Column Length (ft)	2				
Column Height (ft)	15	Sub. Tot			
Column Unit Weight (kcf)	0.15	0			
Number of Columns	3				
Bent Capt volume (ft^3)	48				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	3				
Footing Volume (ft^3)	32				
Footing Unit Weight (kcf)	0.15				
Number of footings	3				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*
1.25	1	1.05	1.05		1.378125

Load Combination	*Load Cases*	*VERT [kips]/ft*	*Horizontal [kips]/ft*
UNFACTORED LOADS	DC	1.1	
	LL (H10)	0.0	
	WS Hor		0.1
	PL	1.1	
	E.Q		0.6
Strength I	DC	1.5	

1.75	1	1.05	1.05	1.929375		LL (H10)	0.0	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	1.1	
						E.Q		0.0
					Sum		2.5	0.0

1.25	1	1.05	1.05	1.378125	Strength III	DC	1.5	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		0.1
						PL	0.0	
						E.Q		
					Sum		1.5	0.1

1	1	1	1	1	Service I	DC	1.1	
1	1	1	1	1		LL (H10)	0.0	
0.3	1	1	1	0.3		WS Hor		0.0
1	1	1	1	1		PL	1.1	
						E.Q		0.0
					Sum		2.1	0.0

1.25	1	1	1	1.25	Extreme Ev. I	DC	1.3	
0.5	1	1	1	0.5		LL (H10)	0.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	0.5	
1	1	1	1	1		E.Q		0.6
					Sum		1.9	0.6

Summary		
Strength I	2.5	0.0
Strength III	1.5	0.1
Service I	2.1	0.0
Extreme Even I	1.9	0.6

Bridge Deck Design

Value	Units	Coefficients	Reference	Equation	Notes	d (depth less bc and diam/2) (in)
4000	psi	f'c			Compressive Strength of Concrete	5.75
60000	psi	fy			Yield Strength of Steel	
150	pcf	Uc			Unit Weight of Concrete	di(bmain) (in)
4.5	ft	L			Length of slab (Girder Spacing)	1
7	in	h			Thickness of slab (Look at Genral notes)	di(tmain) (in)
2	in	tc	AASHTO Table 5.12.3-1		Top Cover	1
1	in	bc	AASHTO Table 5.12.3-1		Bottom Cover	
3.652291667	ft	Ln	ACI 8.3.3		Clear Length of Span	m
2.5	kip/ft	Wu			Factored Load (Look at General Notes)	17.64705882
0.5	in	di(bmain)			Diameter of main reinforcement for negative moment	
0.5	in	di(tmain)			Diameter of main reinforcement for positive moement	
12	in	b			width of assumed strip	
0.9		Phi			tension controlled section moment factor	
21		in			Condition 1 for min spacing of As (Satisfied)	
18		in			Condition 2 for min spacing of As (Satisfied)	
0.1512		in^2/ft			As(min) (Satisfied)	
35		in			Condition 1 for min spacing of As(min) (Satisfied)	
18		in			Condition 2 for min spaving of As(min) (Satisfied)	

		1		2			3		4	
ACI MOMENT COEFFICIENT	0.09090909	-0.09091	-0.09091	0.0625	-0.09091	-0.09091	0.0625	-0.09091	-0.09091	0.090909
Mu=Cm*Wu*Ln^2	- 3.03164419	3.031644	3.031644	-2.0842554	3.031644	3.031644	-2.0842554	3.031644	3.031644	-3.03164
Mn=Mu/0.9(k-ft)	3.36849354	-3.36849	-3.36849	2.31583931	-3.36849	-3.36849	2.31583931	-3.36849	-3.36849	3.368494
Rn (psi)	101.882602	101.8826	101.8826	70.0442891	101.8826	101.8826	70.0442891	101.8826	101.8826	101.8826
Ro	0.00172428	0.001724	0.001724	0.00117968	0.001724	0.001724	0.00117968	0.001724	0.001724	0.001724
As =Ro(b)(d)	0.1189751	0.118975	0.118975	0.0813982	0.118975	0.118975	0.0813982	0.118975	0.118975	0.118975
Larger of As, As min	0.1512	0.1512	0.1512	0.1512	0.1512	0.1512	0.1512	0.1512	0.1512	0.1512
Provided As	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16
	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"	#4@16"

Checks										
a	0.23529412	0.235294	0.235294	0.23529412	0.235294	0.235294	0.23529412	0.235294	0.235294	0.235294
Beta (for 4000psi)	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
c	0.27681661	0.276817	0.276817	0.27681661	0.276817	0.276817	0.27681661	0.276817	0.276817	0.276817
epsilon(t)	0.05931563	0.059316	0.059316	0.05931563	0.059316	0.059316	0.05931563	0.059316	0.059316	0.059316
Larger than 0.005?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Shear Resistance	
Phi(V) (kip)	6.54591476
Factored shear at the face	
Vu (kip)	5.25016927
Shear sufficient?	Yes

Loading for Girders

All weight in kip					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	1.05	0.159	0.08	0.01
Span length (ft)	120				
Span Width (ft)	12	Sup. Tot	Sup tot. K/ft		
Concrete Unit (kcf)	0.15	1.299	1.299		
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.04				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)		0	0	0	1.299
Column Length (ft)					
Column Height (ft)		Sub. Tot			
Column Unit Weight (kcf)		0			
Number of Columns					
Bent Capt volume (ft^3)					
Bent Cap Unit weight (kcf)					
Number of Caps					
Footing Volume (ft^3)					
Footing Unit Weight (kcf)					
Number of footings					

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]/ft*	*Horizontal [kips]/ft*
UNFACTORED LOADS	DC	1.3	
	LL (H10)	0.0	

						WS Hor		0.3
						PL	1.1	
						E.Q		1.2
1.25	1	1.05	1.05	1.378125	Strength I	DC	1.8	
1.75	1	1.05	1.05	1.929375		LL (H10)	0.0	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	2.1	
						E.Q		0.0
					Sum		3.9	0.0
1.25	1	1.05	1.05	1.378125	Strength III	DC	1.8	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		0.5
						PL	0.0	
						E.Q		
					Sum		1.8	0.5
1	1	1	1	1	Service I	DC	1.3	
1	1	1	1	1		LL (H10)	0.0	
0.3	1	1	1	0.3		WS Hor		0.1
1	1	1	1	1		PL	1.1	
						E.Q		0.0
					Sum		2.4	0.1
1.25	1	1	1	1.25	Extreme Ev. I	DC	1.6	
0.5	1	1	1	0.5		LL (H10)	0.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	0.5	
1	1	1	1	1		E.Q		1.2
					Sum		2.2	1.2
					Summary			
					Strength I		3.9	0.0
					Strength III		1.8	0.5
					Service I		2.4	0.1
					Extreme Even I		2.2	1.2

Girder Design

Value	Units	Coefficients	Reference	Equation	Notes
7.82		b/t	AISC		Width to thickness ratio for unstiffened elements
28.1		h/tw	AISC		Width to thickness ratio for stiffened elements
50	ksi	Fy	AISC		Yield Strength
29000	ksi	E	AISC		Modulus of Elasticity
9.15161188			AISC		Limiting ratio compact unstiffened
24.08318916			AISC		Limiting ratio non-compact unstiffened
90.55279123			AISC		Limiting ratio compact stiffened
137.2741782			AISC		Limiting ratio non-compact stiffened
2.48	in	ry			
77.9	in^3	z	AISC		Plastic Section Modulus
3893.3	k-in	Mpx	AISC		Plastic Bending Moment
0.9		phi_b	AISC		Coefficient
3503.97	K-in	phi*Mnx =phi*Mpx	AISC		Design Bending Capacity in plastic range (about x-axis) (Lb<=Lp)
30	ft	L			Length of the beam
360	in	Lb	AISC		Lateral Unbraced length
105.12	in	Lp	AISC		Limiting Laterally unbranced length (plastic)
338.4	in	Lr	AISC		Limiting Laterally unbranced length (inelastic) (From Table)
5.5	kips	phi-BF	AISC		from AISC pg 3-24
1.3		Cb	AISC		from ASIC 16.1-46, or Table 3-1 pg 3-18
3503.97		phi*Mnx	AISC		Design Bending Capacity inelastic range (Lp<Lb<=Lr)
3.9	k/ft	wu			The load on beam distributed longitudinally
0.061	k/ft	wbeam			Weight per length of beam
3.961		w_vertical total			Total vertical load on beam per length
2.4	k/ft	w_Service			Service Load for deflection calculations (to include the weight of the beam)
425	in^4	Ix			area moment of inertia about x-axis
1		Cv			
0.6		Coefficient for design			
15.6	in^2	Aw			Cross sectional area of the beam

1		Phi_v			Coefficent for design shear strength

Required Bending Capacity (k-ft)	Notes
148.5375	
Required Shear strength (k)	
19.805	
Deflection of the Beam (in)	
1.18296146	
Acceptable Deflection (in)	For non-plastered roof
1.5	
Deflection satisfied?	
Yes, Acceptable	
Bending Capacity satisfied?	For Plastic range
Yes, Acceptable	
Shear Strength of one girder (k)	
468	
Sufficient Shear Strengths?	
Yes, Acceptable	
Equation to use for Moment capacity:	
Inelastic or Elastic	
Elastic	170.95 k-ft (222.24 with Cb=1.3)
Bending Capacity (k-ft)	
171	
Bending Acceptable?	Final Design: 3 Girders of:
Yes, Acceptable	W 12x53
True for all ASTM W,S and HP Except :	Center-Center Spacing: 4.5'
W44X230, W40X149, W36X135,	Clear Spacing: 3.65'
W33X118, W24X55, W16X26	
W12X14	
AND Fy=50ksi	

AND Fy=50ksi	
and webst of rolled I shape with	
h/tw<=2.24sqrt(E/Fy)	
If these conditions not satisfied, check for other requirements	

Wind Load-General

Value	Units	Coefficients	Reference	Equation	Notes	Wind Pressure (psf)	Note
1		Kz	AASHTO Signs		Heightand Exposure Factor	40.300416	
90	mph	V	AASHTO Signs		Basic Wind Speed		
1.3		Cd	AASHTO Signs		Drag Coefficient	Effective Projected Area (EPA) ft^2	
1.15		Ir	AASHTO Signs		Importance Factor	535.6	
1.3		G	AASHTO Signs		Gust Effect Factor		
412	ft^2	A			Area	Wind Force (kip)	
						21.58490281	applied to center of pressure

Area of girders (ft^2)	120
Area of slab (ft^2)	70
Area of Railings (ft^2)	30
Area of Columns	120
Area of Bent Cap	72
Total Sup. Str	220
Total Sub. Str.	192
Total Bridge	412

Earthquake Load for the 180-foot Bridge Structure (Structure

Value	Units	Coefficients	Reference	Notes
3.0821E-06	ft	Vs(max)	AASHTO C4.7.4.3.2C	deformation corresponding to po (ft)
1	k/ft	Po	AASHTO C4.7.4.3.2C	a uniform load arbitrarily set equal to 1 k/ft
0.1	k/ft	w(x)	AASHTO C4.7.4.3.2C	nominal, unfactored dead load
1017	k	W	AASHTO C4.7.4.3.2C	Total weight of the bridge (Live and footing included)
180	ft	L	AASHTO C4.7.4.3.2C	Total Length of the bridge
32.17	ft/s^2	g		Acceleration due to gravity
3.14159265		PI		
0.9		Fa		Site Amplification Coefficient for Ss
2.4		Fv		Site Amplification Coefficient for S1
35.387	ft/s^2	PGA	Geotechnical Report	Peak Ground Acceleration
39.5691	ft/s^2	Ss	IBC 2012 from USGS 2008	Short Period Spectral Acceleration
13.89744	ft/s^2	S1	IBC 2012 from USGS 2008	1-sec Period Spectral Acceleration
35.61219	ft/s^2	SMS	IBC 2012 from USGS 2008	MCER spectral response acceleration parameters
33.32812	ft/s^2	SM1	IBC 2012 from USGS 2008	MCER spectral response acceleration parameters
23.74146	ft/s^2	SDS	IBC 2012 from USGS 2008	Design Spectral Response Acceleration Parameters
22.22947	ft/s^2	SD1	IBC 2012 from USGS 2008	Design Spectral Response Acceleration Parameters
0.9		F(PGA)	AASHTO Table 3.10.3.2-1	site factor at zero-period on acceleration response spectrum

Bridge Lateral Stiffness K (k/ft)	Notes
58400943.83	
Period of the Bridge Tm (1/s)	
0.003531557	
Ts	
0.936314363	
To	
0.187262873	

As	
31.8483	
Csm	Elastic Seismic Coefficient for Tm<=To
31.69541454	
Csm	when To<=Tm<=Ts
23.74146	
Csm	When Tm>Ts
6294.523479	
Pe (k)	Equivalent Static Earthquake Load
179.0790922	

Column Stiffness (k/in)	
108149.896	fixed-fixed
27037.47399	fixed-pinned
Column Stiffness (k/ft)	
1297798.752	fixed-fixed
324449.6879	fixed-pinned
Displacement (ft)	Due to Po (1k/ft)
7.70535E-07	fixed-fixed
3.08214E-06	fixed-pinned

General Notes
Occupancy (Risk) Category I
Site Class E
Site Coordinates37.3092°N, 89.5464°W
Seismic Zone 4 (0.5<SD1)
http://earthquake.usgs.gov/designmaps/us/application.php
Pages 317 and 148 of AASHTO

Earthquake Load for the 120-foot Bridge Structure (Structure

Units	Coefficients	Reference	Equation	Notes
ft	Vs(max)	AASHTO C4.7.4.3.2C		deformation corresponding to po (ft)
k/ft	Po	AASHTO C4.7.4.3.2C		a uniform load arbitrarily set equal to 1 k/ft
k/ft	w(x)	AASHTO C4.7.4.3.2C		nominal, unfactored dead load
k	W	AASHTO C4.7.4.3.2C		Total weight of the bridge (Live and footing included)
ft	L	AASHTO C4.7.4.3.2C		Total Length of the bridge
ft/s^2	g			Acceleration due to gravity
	PI			
	Fa			Site Amplification Coefficient for Ss
	Fv			Site Amplification Coefficient for S1
ft/s^2	PGA	Geotechnical Report		Peak Ground Acceleration
ft/s^2	Ss	IBC 2012 from USGS 2008		Short Period Spectral Acceleration
ft/s^2	S1	IBC 2012 from USGS 2008		1-sec Period Spectral Acceleration
ft/s^2	SMS	IBC 2012 from USGS 2008		MCER spectral response acceleration parameters
ft/s^2	SM1	IBC 2012 from USGS 2008		MCER spectral response acceleration parameters
ft/s^2	SDS	IBC 2012 from USGS 2008		Design Spectral Response Acceleration Parameters
ft/s^2	SD1	IBC 2012 from USGS 2008		Design Spectral Response Acceleration Parameters
	F(PGA)	AASHTO Table 3.10.3.2-1		site factor at zero-period on acceleration response spectrum

Bridge Lateral Stiffness K (k/ft)	Notes
38933962.55	
Period of the Bridge Tm (1/s)	
0.003639298	
Ts	
0.936314363	

To	
0.187262873	
As	
31.8483	
Csm	Elastic Seismic Coefficient for Tm<=To
31.6907503	
Csm	when To<=Tm<=Ts
23.74146	
Csm	When Tm>Ts
6108.174795	
Pe (k/ft)	Equivalent Static Earthquake Load
190.1445018	
Column Stiffness (k/in)	
108149.896	fixed-fixed
27037.47399	fixed-pinned
Column Stiffness (k/ft)	
1297798.752	fixed-fixed
324449.6879	fixed-pinned
Displacement (ft)	Due to Po (1k/ft)
7.70535E-07	fixed-fixed
3.08214E-06	fixed-pinned

General Notes
Occupancy (Risk) Category I
Site Class E
Site Coordinates37.3092°N, 89.5464°W
Seismic Zone 4 (0.5<SD1)
http://earthquake.usgs.gov/designmaps/us/application.php

Pages 317 and 148 of AASHTO
Uniform Load Method
Suitable for bridges which respond principally in their fundamental mode of vibration
Up to 100% overestimation of transverse shear at abutments

Loading for the Bents

All weight in kip					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	63	9.54	4.8	0.6
Span length (ft)	60				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	77.94			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.04				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2.5	18.75	6.75	0	103.44
Column Length (ft)	2.5				
Column Height (ft)	20	Sub. Tot			
Column Unit Weight (kcf)	0.15	25.5			
Number of Columns	1				
Bent Capt volume (ft^3)	45				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	1				
Footing Volume (ft^3)	32				
Footing Unit Weight (kcf)	0.15				
Number of footings	3				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]*	*Horizontal [kips]*
UNFACTORED LOADS	DC	103.4	
	LL (H10)	20.0	
	WS Hor		20.6

						PL	64.8	
						E.Q		69.0
1.25	1	1.05	1.05	1.378125	Strength I	DC	142.6	
1.75	1	1.05	1.05	1.929375		LL (H10)	38.6	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	125.0	
						E.Q		0.0
					Sum		306.2	0.0
1.25	1	1.05	1.05	1.378125	Strength III	DC	142.6	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		31.9
						PL	0.0	
						E.Q		
					Sum		142.6	31.9
1	1	1	1	1	Service I	DC	103.4	
1	1	1	1	1		LL (H10)	20.0	
0.3	1	1	1	0.3		WS Hor		6.2
1	1	1	1	1		PL	64.8	
						E.Q		0.0
					Sum		188.2	6.2
1.25	1	1	1	1.25	Extreme Ev. I	DC	129.3	
0.5	1	1	1	0.5		LL (H10)	10.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	32.4	
1	1	1	1	1		E.Q		69.0
					Sum		171.7	69.0

Summary	Vertical	Horitonzal
Strength I	306.2	0.0
Strength III	142.6	31.9
Service I	188.2	6.2
Extreme Even I	171.7	69.0

Loading for the Footings

All weight in kip					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	63	9.54	4.8	0.6
Span length (ft)	60				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	77.94			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.04				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2.5	26.25	6.75	33.75	144.69
Column Length (ft)	2.5				
Column Height (ft)	28	Sub. Tot			
Column Unit Weight (kcf)	0.15	66.75			
Number of Columns	1				
Bent Capt volume (ft^3)	45				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	1				
Footing Volume (ft^3)	225				
Footing Unit Weight (kcf)	0.15				
Number of footings	1				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]*	*Horizontal [kips]*
UNFACTORED LOADS	DC	144.7	
	LL (H10)	20.0	
	WS Hor		20.6

						PL	64.8	
						E.Q		44.8
1.25	1	1.05	1.05	1.378125	Strength I	DC	199.4	
1.75	1	1.05	1.05	1.929375		LL (H10)	38.6	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	125.0	
						E.Q		0.0
					Sum		363.0	0.0
1.25	1	1.05	1.05	1.378125	Strength III	DC	199.4	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		31.9
						PL	0.0	
						E.Q		
					Sum		199.4	31.9
1	1	1	1	1	Service I	DC	144.7	
1	1	1	1	1		LL (H10)	20.0	
0.3	1	1	1	0.3		WS Hor		6.2
1	1	1	1	1		PL	64.8	
						E.Q		0.0
					Sum		229.5	6.2
1.25	1	1	1	1.25	Extreme Ev. I	DC	180.9	
0.5	1	1	1	0.5		LL (H10)	10.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	32.4	
1	1	1	1	1		E.Q		44.8
					Sum		223.3	44.8

Summary	Vertical	Horizontal
Strength I	363.0	0.0
Strength III	199.4	31.9
Service I	229.5	6.2
Extreme Even I	223.3	44.8

Total Load for the 120-foot Bridge Structure (Structure 1)

All weight in kip					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	126	19.08	12	1.2
Span length (ft)	120				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	158.28			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.05				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2	45	20.25	90	313.53
Column Length (ft)	2				
Column Height (ft)	25	Sub. Tot			
Column Unit Weight (kcf)	0.15	155.25			
Number of Columns	3				
Bent Capt volume (ft^3)	45				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	3				
Footing Volume (ft^3)	200				
Footing Unit Weight (kcf)	0.15				
Number of footings	3				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]*	*Horizontal [kips]*
UNFACTORED LOADS	DC	313.5	
	LL (H10)	20.0	
	WS Hor		20.6

						PL	129.6	
						E.Q		44.8
1.25	1	1.05	1.05	1.378125	Strength I	DC	432.1	
1.75	1	1.05	1.05	1.929375		LL (H10)	38.6	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	250.0	
						E.Q		0.0
					Sum		720.7	0.0

1.25	1	1.05	1.05	1.378125	Strength III	DC	432.1	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		31.9
						PL	0.0	
						E.Q		
					Sum		432.1	31.9

1	1	1	1	1	Service I	DC	313.5	
1	1	1	1	1		LL (H10)	20.0	
0.3	1	1	1	0.3		WS Hor		6.2
1	1	1	1	1		PL	129.6	
						E.Q		0.0
					Sum		463.1	6.2
1.25	1	1	1	1.25	Extreme Ev. I	DC	391.9	
0.5	1	1	1	0.5		LL (H10)	10.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	64.8	
1	1	1	1	1		E.Q		44.8
					Sum		466.7	44.8

Summary		
Strength I	720.7	0.0
Strength III	432.1	31.9
Service I	463.1	6.2
Extreme Even I	466.7	44.8

Total Load for the 180-foot Bridge Structure (Structure 2)

All weight in kip

		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	189	28.62	18	1.8
Span length (ft)	180				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	237.42			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.05				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2	52.8	27	120	437.22
Column Length (ft)	2				
Column Height (ft)	22	Sub. Tot			
Column Unit Weight (kcf)	0.15	199.8			
Number of Columns	4				
Bent Capt volume (ft^3)	45				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	4				
Footing Volume (ft^3)	200				
Footing Unit Weight (kcf)	0.15				
Number of footings	4				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]*	*Horizontal [kips]*
UNFACTORED LOADS	DC	437.2	
	LL (H10)	20.0	
	WS Hor		20.6

						PL	194.4	
						E.Q		44.8
1.25	1	1.05	1.05	1.378125	Strength I	DC	602.5	
1.75	1	1.05	1.05	1.929375		LL (H10)	38.6	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	375.1	
						E.Q		0.0
					Sum		1016.2	0.0

1.25	1	1.05	1.05	1.378125	Strength III	DC	602.5	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		31.9
						PL	0.0	
						E.Q		
					Sum		602.5	31.9

1	1	1	1	1	Service I	DC	437.2	
1	1	1	1	1		LL (H10)	20.0	
0.3	1	1	1	0.3		WS Hor		6.2
1	1	1	1	1		PL	194.4	
						E.Q		0.0
					Sum		651.6	6.2
1.25	1	1	1	1.25	Extreme Ev. I	DC	546.5	
0.5	1	1	1	0.5		LL (H10)	10.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	97.2	
1	1	1	1	1		E.Q		44.8
					Sum		653.7	44.8

Summary		
Strength I	1016.2	0.0
Strength III	602.5	31.9
Service I	651.6	6.2
Extreme Even I	653.7	44.8

Shear Stud Design

Value	Units	Coefficients	Reference	Notes
4.5	in	h	AASHTO 6.10.10.1	Stud height
0.75	in	d	AASHTO 6.10.10.1	Diameter of stud
6		h/d	AASHTO 6.10.10.1	
2		n	AASHTO 6.10.10.1	Number of transverse studs per row
3	in	S	AASHTO 6.10.10.1	Min center-center distance between studs in a row
10	in	l	AASHTO 6.10.10.1	Longitudinal distance between studs
0.575	in	tf		Thickness of flange of W12X53

h/d acceptable?	General Notes
Yes	The shear stud to go to the middle of the Slab
h>=2*d accepted?	Studs per row on the fland depends on the widths of flange (6.10.10.1.3)
Yes	
tf>=d/2.5 accepted?	
Yes	

Cross-Frame Design

Value	Units	Coefficients	Reference	Notes
0.0403	ksf	w		Unfactored Wind Pressure
30	ft	Lb		Unbraced Length of span
1	ft	d		Depth of Girder
0.583333	ft	dd		Deck Depth
1.4		γ		Load factor for Strength III
1.05		η_i		Load Modifier for Strength III
60	ft	Lb		Span Length
3		n		Number of Longitudinal Members
2.25	ft	Lbc		unbraced length of bottom strut about minor axis (rz)
4.5	ft	Lbc		Unbraced length of bottom strut about major axis (ry)
140		limit		Limiting ratio for slenderness ratio of bottom strut
0.75		K		effective length factor for bolted and welded members
2	in	b		width of angle
0.31	in	t	AASHTO 6.7.3	Minimum structural steel thickness
1.16	in^2	A		Area of L 2 X 2 X 5/16
0.386	in	rz		
0.598	in	rx		
0.598	in	ry		
0.172835	in^4	Iz		
0.655165	in^4	Iw		
0.414	in^4	Iy		
0.414	in^4	Ix		
0.75153	in	rw		

29000	ksi	E		
50	ksi	Fy		
3.141593	rad	PI		
67.72575		KL/ry		Controling ratio of KL/r
62.40079	ksi	Fe		
0.38	in	t		Thickness of gusset plate
2.4	in^2	Ag		Area of the selected member
108	k	Pt		Tensile strength in the gross section
1.11	in	xbar		
3	in	l		The length of the angle on the plate
0.63		u		Reduction Coefficient
68	ksi	Fu		Tensile strength of grade 50 steel
77.112	k	Pr		Tensile Rupture in the net section
2.25	in^2	Anv		Net area subject to shear
2.25	in^2	Agv		Gross area subjected to shear
1.5	in^2	Ant		Net area subject to tension force
1		Ubs	AISC 16.1-412	
145.35	K	Br 1		Block shear rupture 1
127.125	k	Br 2		
127.125	k	Br		Available Block shear rupture
77.112	kip	P		Design Tensile strength of angle
0.19	in	Wmin	AISC 16.1-111 Table J.2.4	
0.25	in	Wmax	AISC 16.1-111 Table J.2.4	
0.2	in	Wselected		
0.1414	in	te		
70		E70		
4.4541	k/in	Pu		Design strength of longitudinally welded member
2.527063	in	Lw		Length of Welding required
6	bolt	n		Number of Bolts
22.5	k/bolt	Phi*rn	AISC Table 7.1	Nominal Shear Strength of a bolt
135	kips	phi*Rn		Design Shear Strength of bolt group
Sufficient				Sufficiency of the shear strength of bolt group

Factored Wind Load (k/ft)	Notes
0.046899125	
Horizontal Windload at brace (k)	
1.40697375	
Maimum Moment for Path I&II (K-ft)	
4.22092125	
Max Moment for Path III (K-ft)	
11.25579	
Minimum rz required (in)	
0.144642857	
Minimum ry required (in)	
0.289285714	
Available Axial Compression (kip)	
23.8	Fy=36 ksi,lbc=3ft (conservative),Table 4-11 AISC Manual
37.32673559	for Fy=50 ksi
Axial Strength Enough?	
Yes	
Limiting ratio for comp. satisfied?	
Yes, Nonslender	
Limiting Ratio for Flex. Satisfied?	
Yes, compact and nonslender	Local Buckling does not apply for compact L-shapes
Fcr (Ksi)	Based on case a, E3, AISC, pg 16.1-33 (controling KL/r)
35.75357815	
Selected Member:	
L 2X2X 5/16	
Plate dimensions: 9X7X 3/8"	Longitudinal and Transverse welding considered

Sufficient Tensile Strength for angle?
Yes
Fracture for longitudinal welds only (k)
91.8
Selected length welding on each side (in)
3
Selected width of welding (in)
3/4 diameter A 325 Standard
Bolts were not used in the final design

Bearing Load

All weight in kip					
		Concrete DC	Steel GirderDC	Railing system	Miscellaneous
Slab Thickness (ft)	0.583333333	31.5	4.77	3	0.3
Span length (ft)	30				
Span Width (ft)	12	Sup. Tot			
Concrete Unit (kcf)	0.15	39.57			
Number of girders	3				
Girder Unit Weight (klf) (W 24 x 162)	0.053				
Unit weight of Railing/fence/light (klf)	0.05				
Miscellaneous Super structure (klf)	0.01				
		Column DC	Bent Cap DC	Footing DC	Bridge Total (kip)
Column Width (ft)	2	0	0	0	39.57
Column Length (ft)	2				
Column Height (ft)	25	Sub. Tot			
Column Unit Weight (kcf)	0.15	0			
Number of Columns	1				
Bent Capt volume (ft^3)	45				
Bent Cap Unit weight (kcf)	0.15				
Number of Caps	1				
Footing Volume (ft^3)	32				
Footing Unit Weight (kcf)	0.15				
Number of footings	1				

γ	η_d	η_r	η_i	*other*	*Total Load Multiplier*

Load Combination	*Load Cases*	*VERT [kips]*	*Horizontal [kips]*
UNFACTORED LOADS	DC	39.6	
	LL (H10)	20.0	
	WS Hor		20.6

						PL	32.4	
						E.Q		44.8
1.25	1	1.05	1.05	1.378125	Strength I	DC	54.5	
1.75	1	1.05	1.05	1.929375		LL (H10)	38.6	
				0		WS Hor		0.0
1.75	1	1.05	1.05	1.929375		PL	62.5	
						E.Q		0.0
					Sum		155.6	0.0
1.25	1	1.05	1.05	1.378125	Strength III	DC	54.5	
				0		LL (H10)	0.0	
1.4	1	1.05	1.05	1.5435		WS Hor		31.9
						PL	0.0	
						E.Q		
					Sum		54.5	31.9
1	1	1	1	1	Service I	DC	39.6	
1	1	1	1	1		LL (H10)	20.0	
0.3	1	1	1	0.3		WS Hor		6.2
1	1	1	1	1		PL	32.4	
						E.Q		0.0
					Sum		92.0	6.2
1.25	1	1	1	1.25	Extreme Ev. I	DC	49.5	
0.5	1	1	1	0.5		LL (H10)	10.0	
				0		WS Hor		0.0
0.5	1	1	1	0.5		PL	16.2	
1	1	1	1	1		E.Q		44.8
					Sum		75.7	44.8

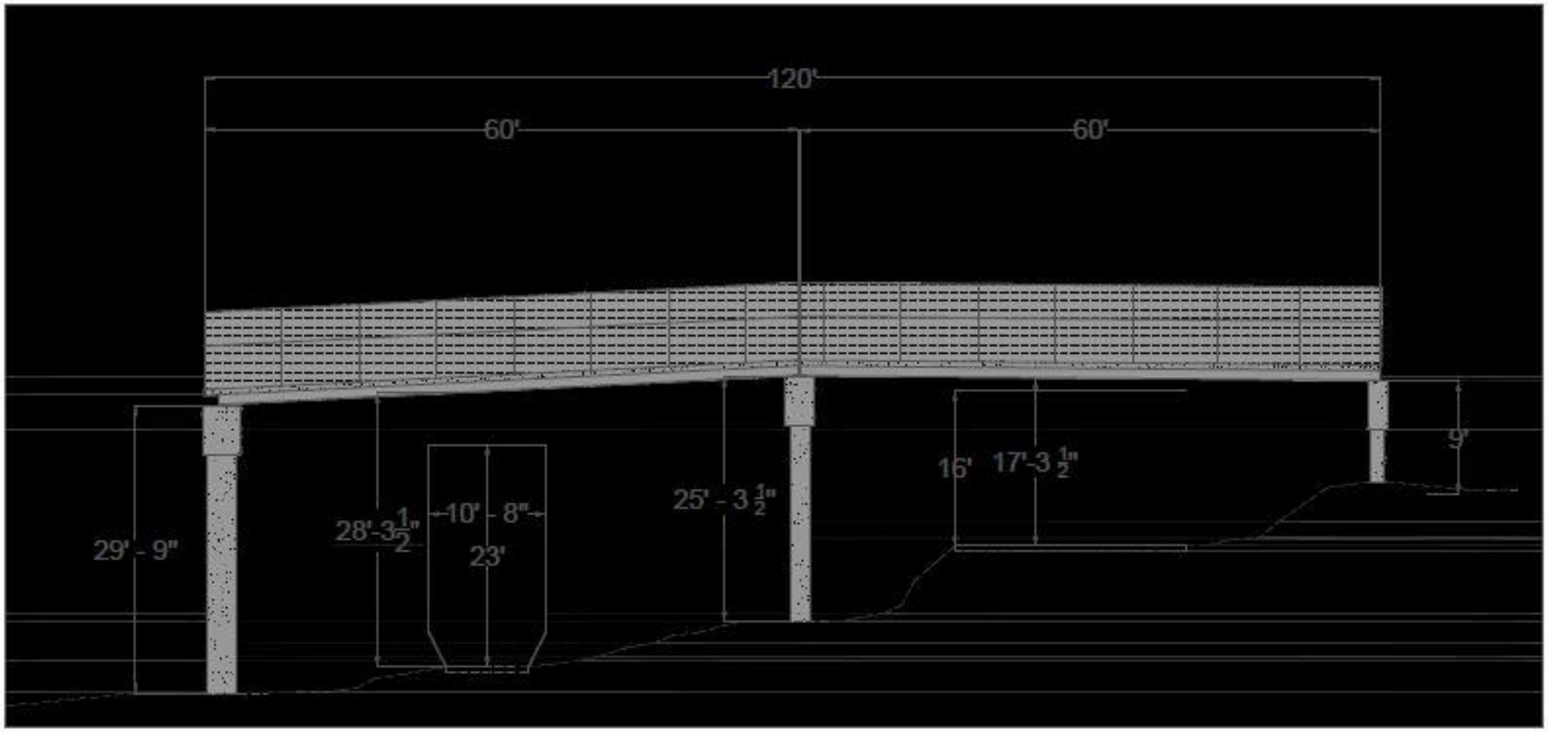
Summary		
Strength I	155.6	0.0
Strength III	54.5	31.9
Service I	92.0	6.2
Extreme Even I	75.7	44.8

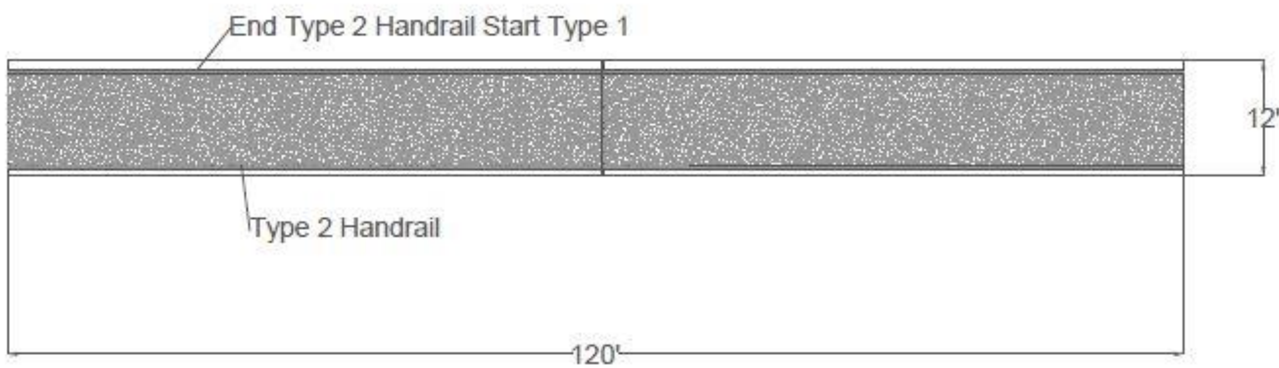
Bearing Design

Value	Units	Coefficients	Reference	Equation	Notes
36	ksi	Fy	AASHTO 14.7.6		As suggested in MiDOT, pg 14-6
3		n	AASHTO 14.7.6		Number of steel reinforcement layers
0.25	in	hc	AASHTO 14.7.6		Thickness of cover layer
0.375	in	hi	AASHTO 14.7.6		Thickness of internal layer
1		PTFE	AASHTO 14.7.6		PTFE used? (1 yes, 0 no)
0.095	ksi	G	AASHTO 14.7.6		Shear Modulus of PEP (0.08-0.25 if PTFE used, or 0.08-0.175
0.357925	ksi	E	AASHTO 14.7.6		Modulus of Elasticity of Elastomer
0.68710526		phi	AASHTO 14.7.6		Elastomer Compressibility Coefficient
8	in	L	AASHTO 14.7.6		Length of Pad
8	in	W	AASHTO 14.7.6		Width of Pad
0.1	in	hrein	AASHTO 14.7.6		Thickness of steel reinforcement
50		HshoreA	AASHTO Table 14.7.6.2.1		
0.25		Cd	AASHTO Table 14.7.6.2.1		Elastomer creep deflection at 25 years divided by ins. Def.
0.47916667	ksi	delta s	AASHTO 14.7.6		Compressive stress due to L and D service I
0.27291667	ksi	delta L	AASHTO 14.7.6		Compressive stress due to L only Service I
			AASHTO 14.7.6		
8		Sc	AASHTO 14.7.6		Shape Factor of cover
5.33333333		Si	AASHTO 14.7.6		Shape Factor of Internal Layers
0.04	in	delta int	AASHTO 14.7.6		Instantaneous Compressive strain
0.065	in	delta total	FHA Example and AASHTO 14.7.6		Total Instantaneous Compressive Strain
0.01625	in	Creep Deflection	FHA Example and AASHTO 14.7.6		
0.08125	in	Total Def.	FHA Example and AASHTO 14.7.6		Total Instantaneous Deflection
1.350000	in	hrt	FHA Example and AASHTO 14.7.6		Height of the Elastomer
0.5	in	cont.	FHA Example and AASHTO 14.7.6		Assumed Contraction due to temperature fall
0.6	in	deltas	FHA Example and AASHTO 14.7.6		
0.0121	rad	theta sz	FHA Example and AASHTO 14.7.6		Service Rotation due to load about transverse axis, assumed

Bridge Lateral Stiffness K (k/ft)	Notes	General Notes
		When the slope of steel beam or plate girder superstructures exceeds 3%, incorporate tapered sole plates into the bearings
		Method A, AASHTO LRFD, 14.7.6, pg 1603
si^2/n<22 ratio met?		**AASHTO M251**
Yes		AASHTO LRFD Bridge Construction Specifications Article 18.2 pg(469)
Ratio of cover to internal thickness met?		Neoprene and Natural Rubber only permitted
Yes		Welding only allowed if 1.5" of steel is between the welding and elastomer
Range of G of PEP satisfied?		Welding to Load plates avoided if possible
Yes		Temperature Zone C, 50 yr Low T=-30 F
No		Design Method A exempts us from combined rotation and compression check
Compressive Strength satisfied?		Page 343 FHA for drawing
Yes		
Temperature Contraction checked and safe?		
Yes		
Rotation Check satisfied?		
Yes		
Stability Check 1 Satisfied?		
Yes		
Stability Check 2 Satisfied?		
Yes		
Steel Reinforcement Height Satisfied?		
Yes		

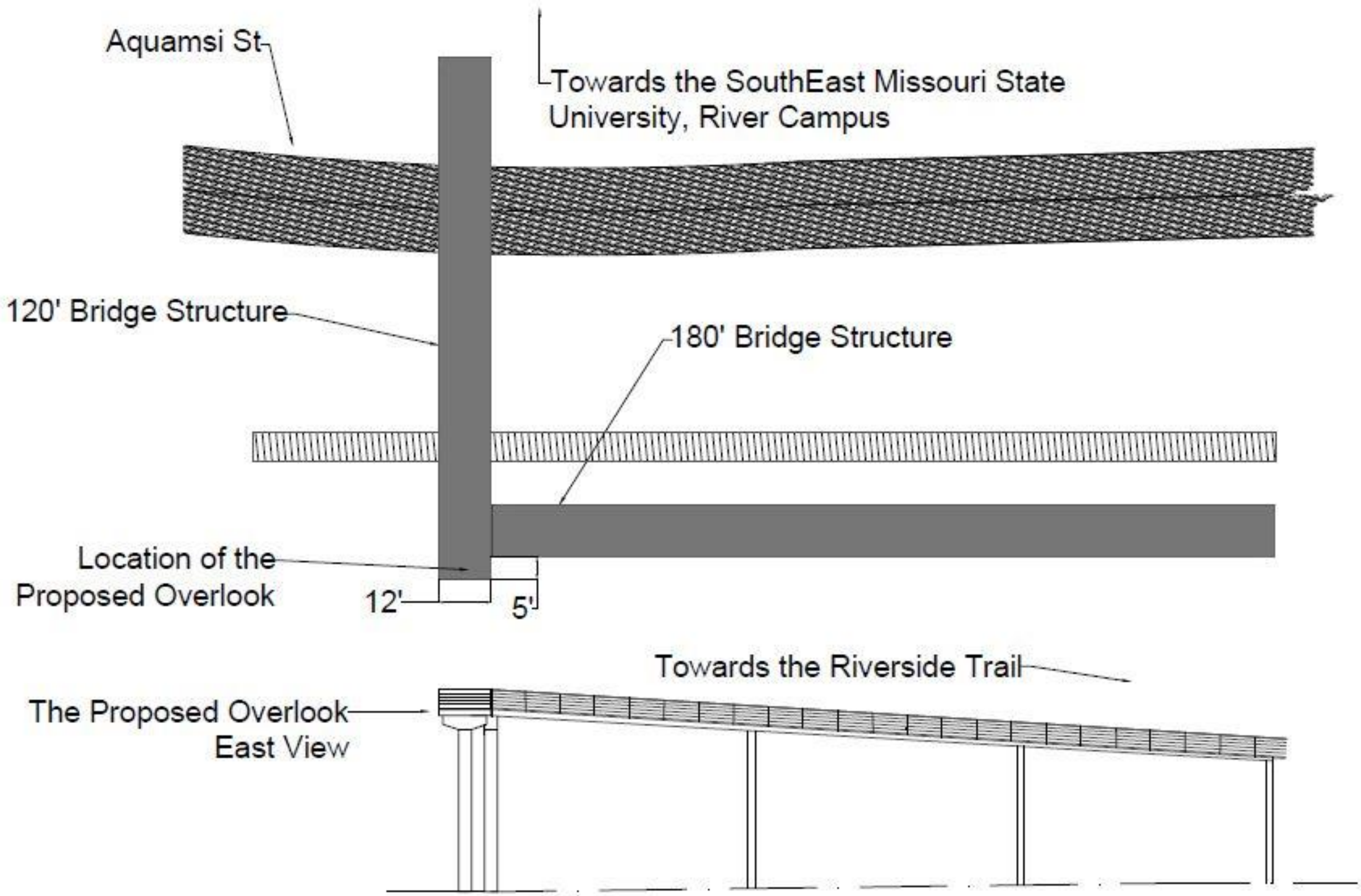
Bridge Structure 1 Side view



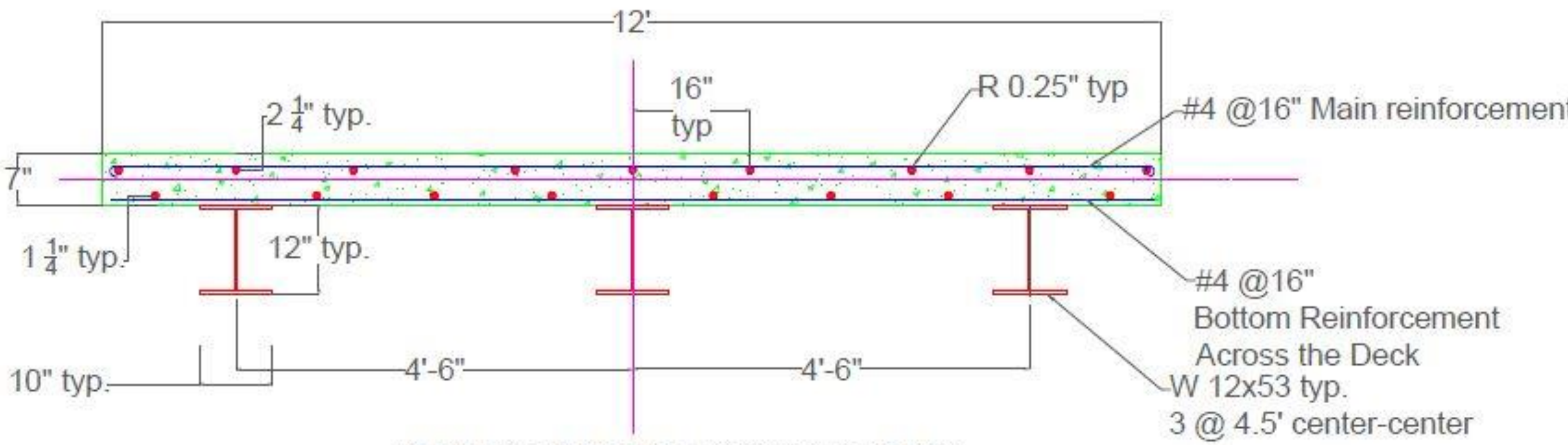


14' Clearance for Local Roads was Increased to 16' for Nearby Standley Ratch Material Concrete Industry's Common use of Construction Vehicles on Aquamsi St.

Side and Top view of Bridge Structure 2

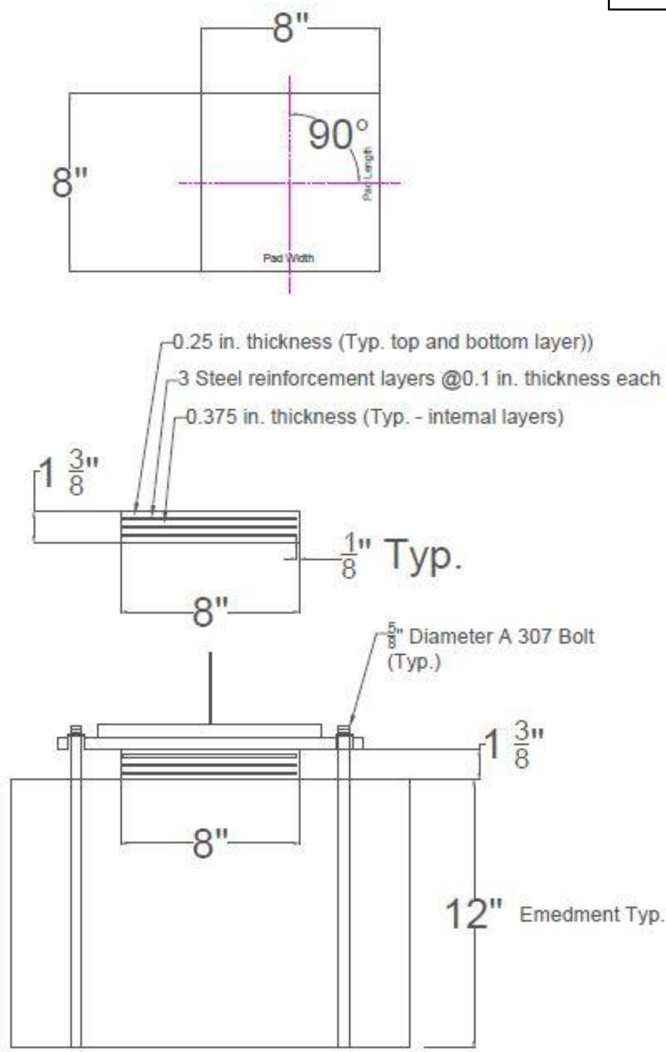


Cross-section of Bridge Deck and Girders



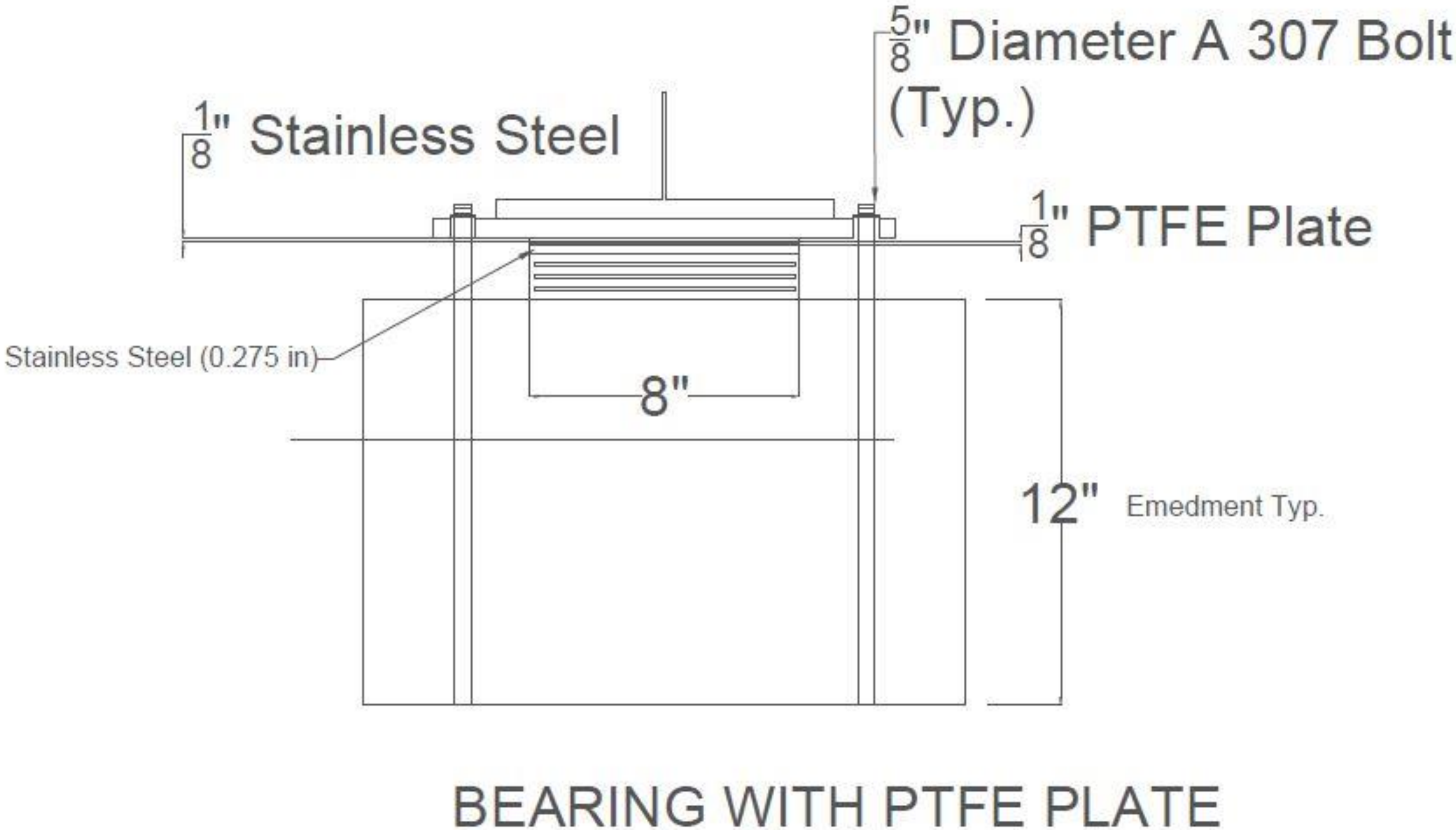
SUPERSTRUCTURE CROSS-SECTION

Typical Fixed Bearing

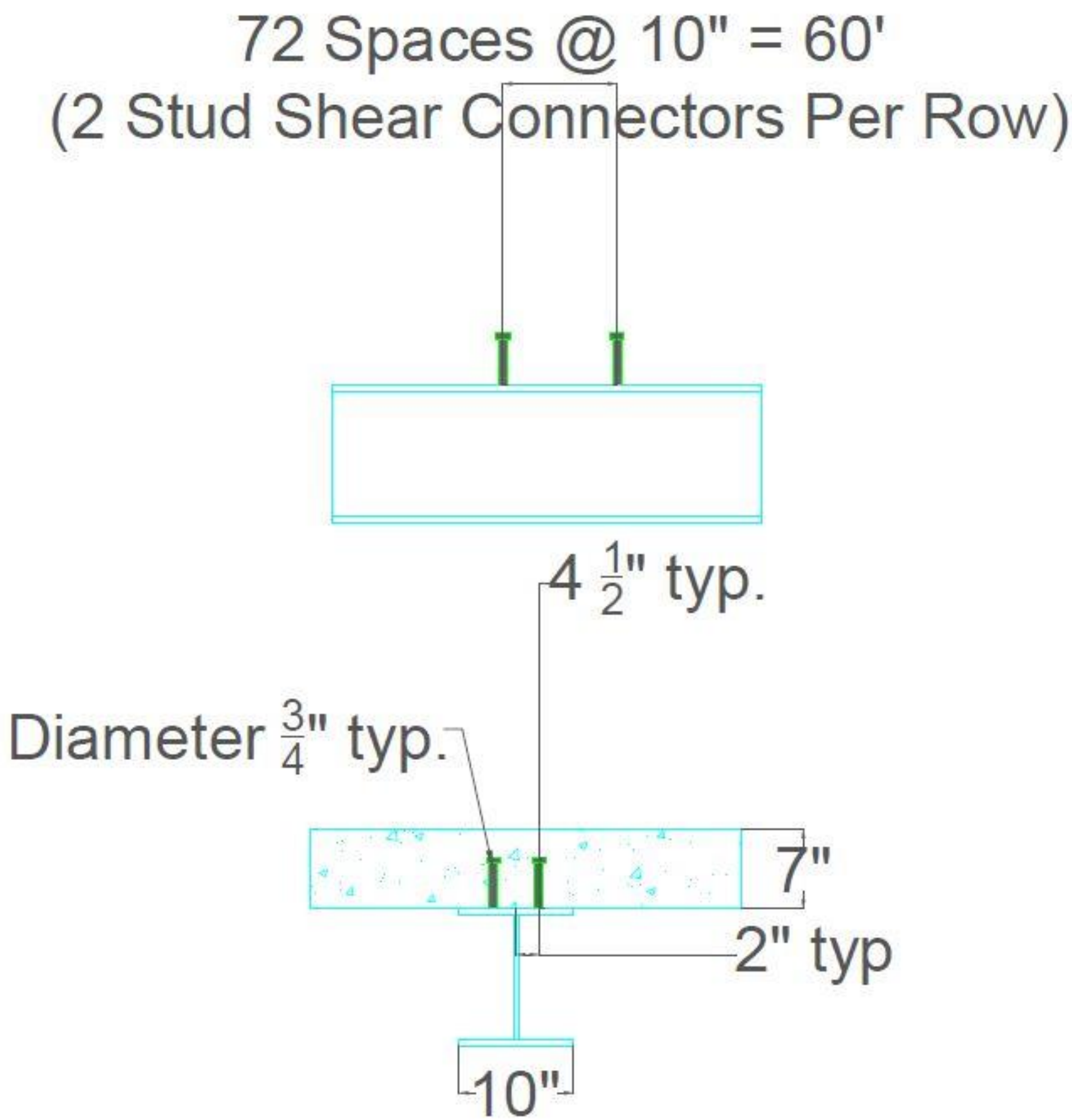


FIXED BEARING

Typical Bearing with PTFE Plate for Longitudinal Translation

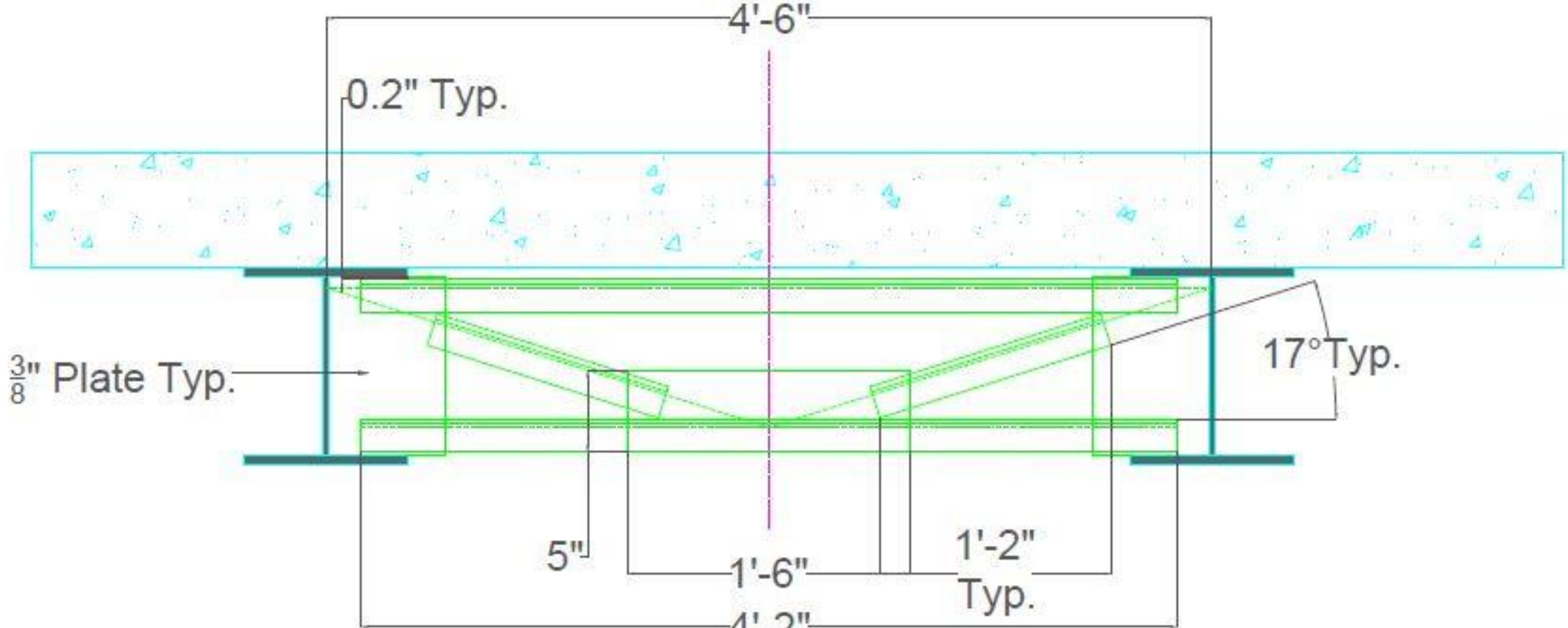


Shear Studs

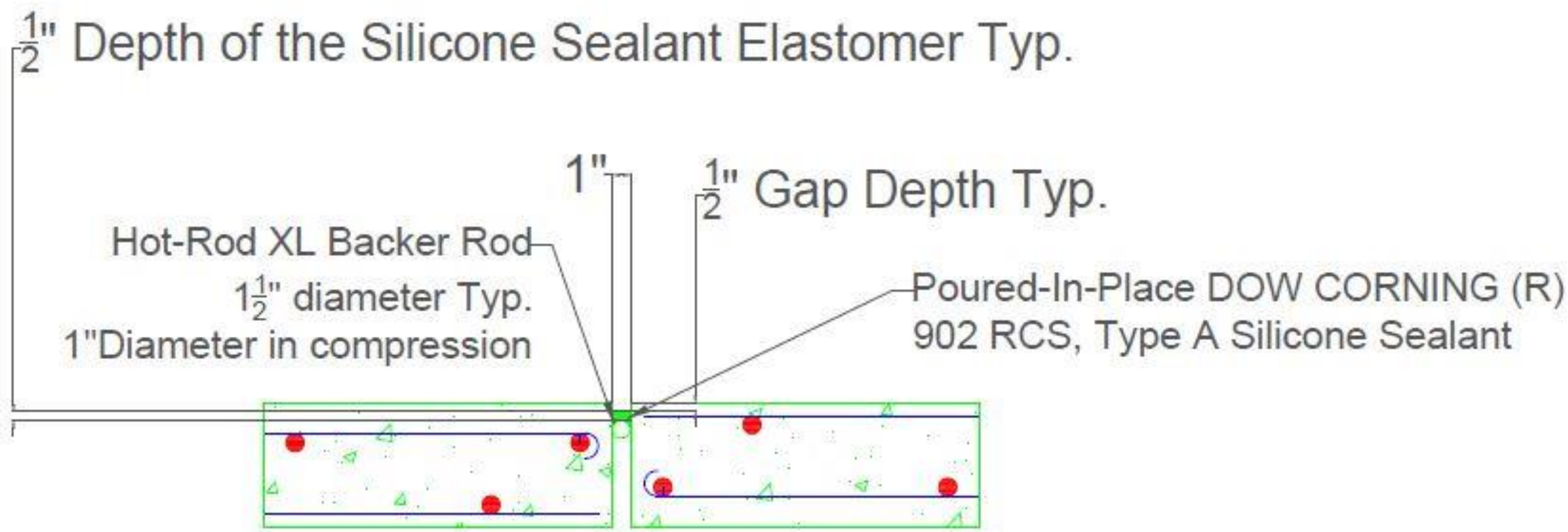


72 Spaces @ 10" = 60'
(2 Stud Shear Connectors Per Row)
4 1/2" typ.
Diameter 3/4" typ.
7"
2" typ
10"

Typical Cross-Frame



Typical Expansion Joint



EXPANSION JOINTS