Design of Highway Bridges

Term Project

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1. INTRODUCTION

Many existing bridge superstructures currently in service consist of steel sections constructed compositely with a reinforced concrete deck. The steel sections can consist of rolled beams or welded plate girders (including box girders). Full depth precast concrete panels are one of the innovative ways for accelerating bridge construction specially for steel girders one. A recent comprehensive study showed that 35% of previously successful projects using FDPC deck panels had been used on steel or steel continuous girders The most economical type of steel section depends on many factors, the greatest contributing factor, however, is the span length. In general, the plate girder is most economical for simple spans within the domain of 65 to 150 feet and continuous spans of 90 feet or more. The longest plate girder in the world is a three-span continuous structure over the Save River at Belgrade, Yugoslavia, with spans of 246, 856, and 246 feet. The welded plate girder I-section utilizes a deep and relatively thin web plate. In many cases, the web plate will buckle elastically under service loads before any yielding of the steel occurs. However, because of the use of intermediate stiffeners and a phenomenon known as "tension-field action" the web has a high post-buckling shear capacity.

This report presents the detailed design of three composite highway bridges. Each bridge is formed by steel girders acting compositely with a reinforced concrete deck slab. The design covers the principal step in the verification of the design in accordance with the AASHTO. The first bridge consists of hot rolled steel beam and the second one is using plate girder, and finally, folded plate girder by using ABC method.

2. DESIGN REQUIREMENTS

For the design of bridges, the most important design objective is safety. Bridges must have adequate strength to support expected and unexpected loads. At service load level, the structure must provide an adequate ride quality, not undergo excessive deflections and be robust enough to withstand repeated applications of live load. Additionally, safety of the structure during construction must be considered. Stability of the main stringers during construction is the primary role of intermediate diaphragms and cross frames in many steel bridges. Other important aspects of steel bridge design are considerations of the design features that positively or negatively impact a structures aesthetic qualities as well as initial and life-cycle costs.

The model bridge was designed using the practices and principles of the last edition of the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges design [1]–[3]. The Load Factor Design (LFD) option was used as required by many states, including Florida. The reinforced concrete deck, excluding the cantilever portion, was designed using the empirical design option of the NCHRP Load and Resistance Factor Design (LRFD) Bridge Specifications and Commentary [4]–[6]. The cantilever portion of the deck was designed using the classical strip method specified in AASHTO (8.16). The design vehicle was the AASHTO HS20-44 truck.

3. BRIDGE DESCRIPTION

In this report three bridges with full scale spanning 90 feet for rolled shape girder and welded plate girder and 60 feet for folded plate girder type are desired. All bridges have same with of 46.7 feet. The superstructure consists of four welded plate girders, four I-girder and four folded plate girders built compositely with a 8.5 inch reinforced concrete deck. The girders are spaced 12.7 feet on center and the reinforced concrete deck has a 4 feet overhang. The concrete barrier structure is a typical open concrete bridge rail, with 11x11 inch posts spaced 8-feet on center. The center girder is elevated 2% inches above the outside girders to produce a crown of .02 ft/ft.

The Figure 1 show the typical cross section of the bridges. Figure 2 and Figure 3 show the cross section for the folded plate and I shape girder bridge, respectively.

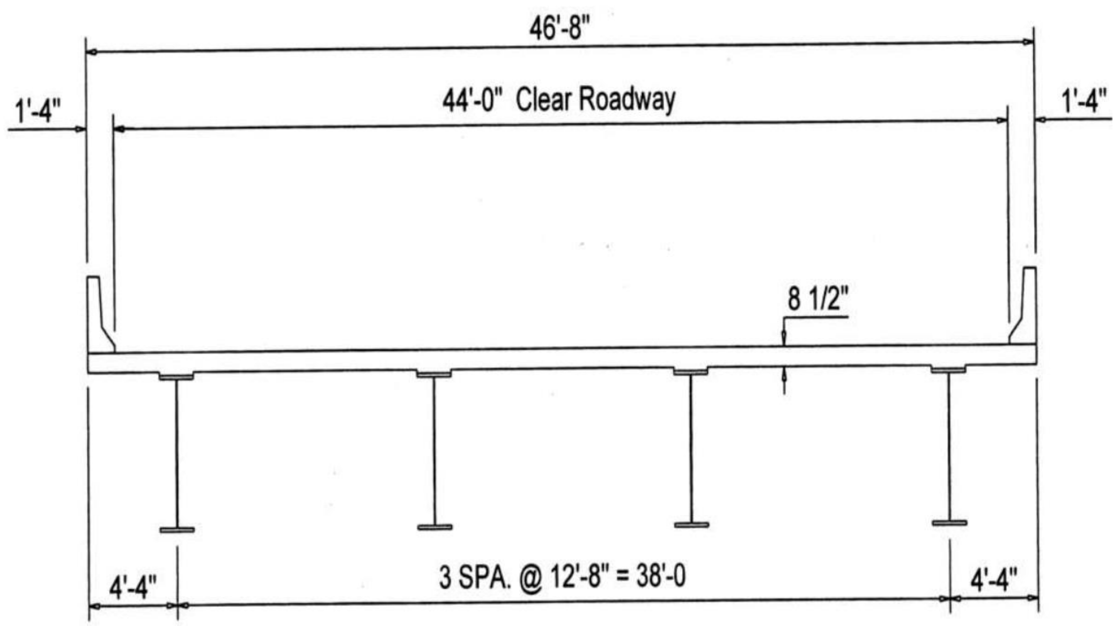


Figure 1: The Bridge Cross Section

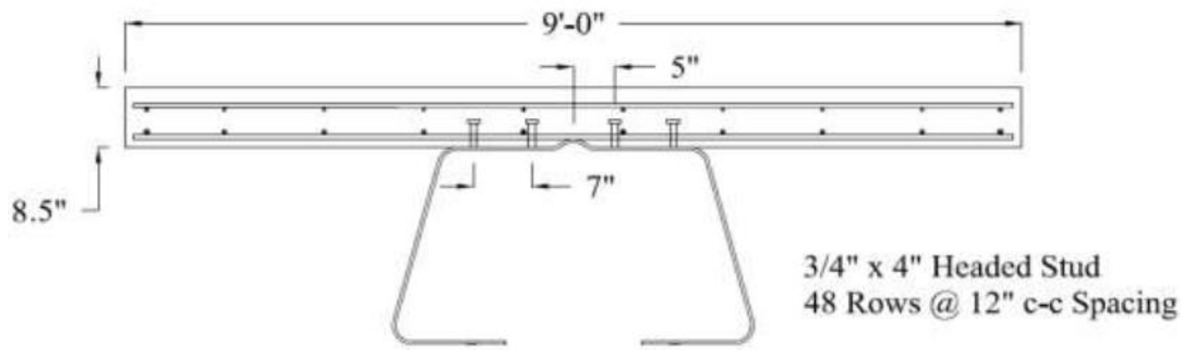


Figure 2. The cross section of folded plate Bridge

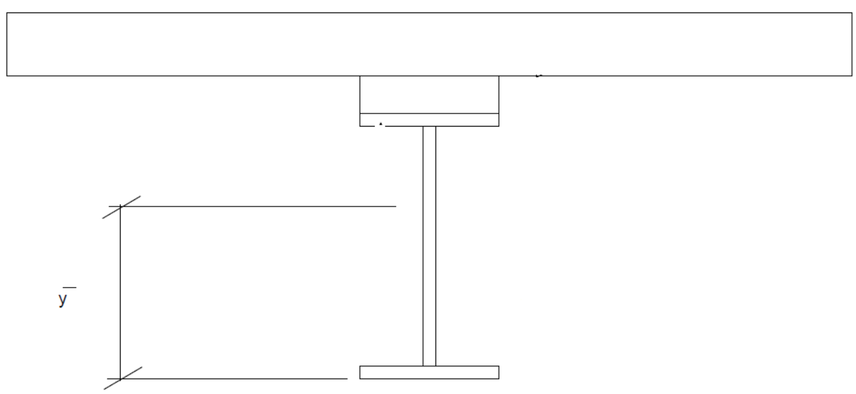


Figure 3. The cross section of rolled and plate girder Bridges

During bridge construction cross-frames consisted of "K-Frames" spaced at 11.2 feet. The cross-frame spacing, and type were modified during live load tests.

When the cross-frames are removed, the angles at the corresponding locations can also be removed. Thus, the cross-frame stiffener will be converted into an intermediate stiffener. design structural elements.

Structural steel yield strength: 50 or 70 ksi for rolled and plate girder and 70 ksi for the folded plate bridge. Concrete with the compression strength of 4 ksi and the reinforcement are grade 60 with 60 ksi yield strength. The module of elasticity of concrete is equal to 57000√ fc and 29000 ksi is considered for the steel materials.

4. AASHTO HS20 LIVE LOADS

There are two classes of design live loads recommended by AASHTO, truck load and lane load. The standard truck load that is used for design is an HS20 Truck. In many jurisdictions and in response to increases in truck weights, an elevated, but not code mandated live load equal to HS25 is used. This live load is simply a 25% increase in axle (wheel) loads over the AASHTO prescribed HS20 loading. The standard HS20 live load consists of three wheels, the loads being 4 kips, 16 kips and 16 kips. This is the load for one wheel ie half of the wheel, i.e. axle load. The distance between the first and second axles is 14 feet. The distance between the second and third axles can vary from 14 feet to 30 feet with the spacing chosen by the engineer to generate maximum force effects. For simply supported bridges, keeping the axle spacing at a constant 14 ft will generate the maximum response.

The second type of mandated live load is a lane load. The lane load consists of a uniformly distributed load with a single concentrated load placed to generate the maximum force. In summary, live load design is done using either the truck load or the combination of the lane load with a concentrated load, whichever one governs the design.

5. LIVE LOADS EFFECT

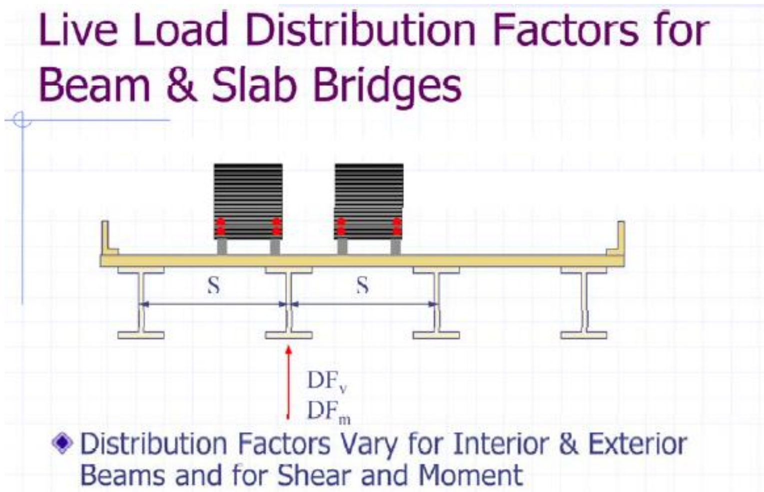
In order to design a main member in a multi-stringer bridge, two important effects must be considered, live load distribution and the dynamic effect known as impact.

Live load distribution involves the use of empirical formulas, based on analytical and experimental methods, that attempt to quantify the total amount of a vehicle load resisted by the most heavily loaded member(s). To design a bridge, we typically do not place discrete trucks on the bridge and compute the load distribution effects because the analysis is highly complicated usually requiring the use of advanced analysis techniques such as the finite element method or other computer based procedures. Instead, empirical approaches such as the AASHTO load distribution factor approach is used. There are separate methods of computing the load distribution effect for exterior and interior stringers and a separate computation as well for the distribution of wheel loads applied at the end of a member over a support. This is known as the distribution effect for end shear.

Similar to load distribution, approximate methods are used to compute the dynamic effects

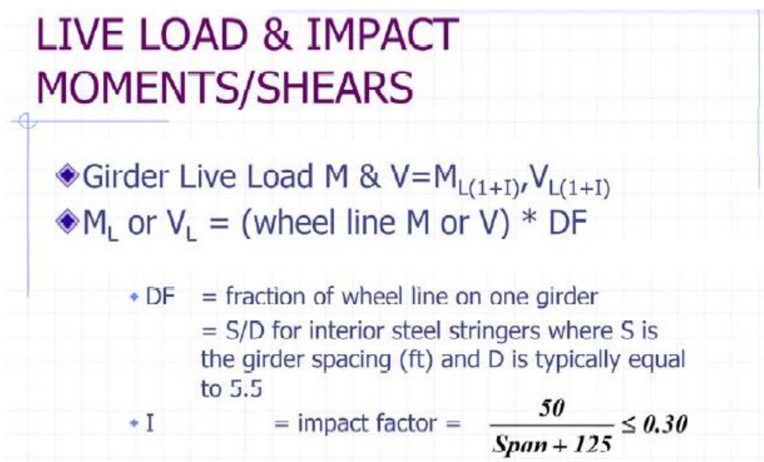
of moving traffic on a bridge. The actual dynamic effect depends on many things such as the vibration characteristics of the bridge, vehicle suspension properties and surface roughness of the bridge deck and approach pavement. Knowing the complexity of such a problem, approximate dynamic amplification factors, known as the impact factor, are used as an increase of the static design load.

6. DISRIBUTION FACTORS



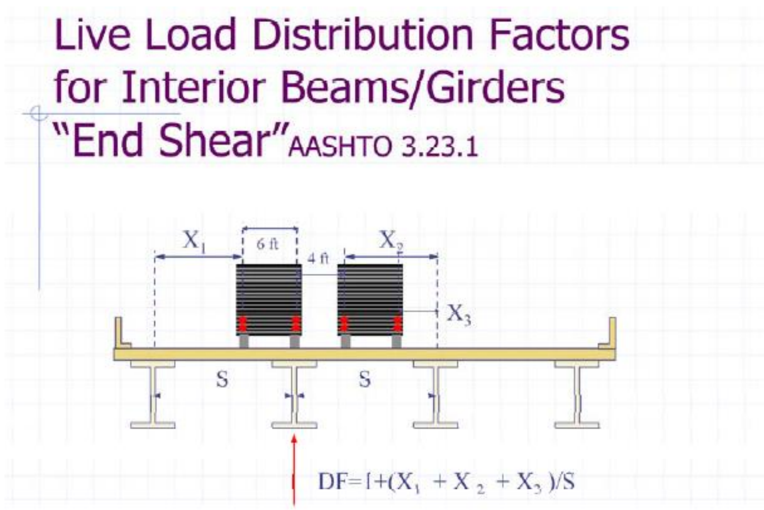
This simple picture illustrates the fact that impact factors attempt to capture the amount of load on the most heavily loaded member from all possible combinations of number of vehicles and truck positions. The actual location and number of lanes is not needed for the typical design of multi-stringer bridges.

7. LIVE LOAD AND IMPACT FACTORS FOR MOMENTS AND SHEARS



The design live loads (HS 20, 25, etc.) are used along with the anticipated dead loads to determine the design moments and shears for the steel beam. ML is multiplied by the impact factor I. ML and VL represent the moment and the shear due to the live loads. ML and VL can be estimated by using wheel line moment or shear multiplied by the distribution factor. The distribution factor determines the fraction of the wheel line resisted by the most heavily loaded member. There are several formulas for the distribution factor depending on whether moment or shear is being considered for either interior or exterior beams. For moment and shear in interior beams with loads applied out in the span, the distribution factor can be estimated by girder spacing S divided by D. D is typically equal to 5.5 for the design of multiple steel stringer bridges supporting a concrete deck. The impact factor **I** corresponds to 50 divided by span length plus 125, but is limited to a value of 0.3. To summarize, the live load effect is estimated from the live load multiplied by the distribution factor which accounts for the lateral distribution of the live load over several beams and the impact factor.

8. DISTRIBUTION FACTORS FOR INTERIOR AND EXTERIOR GIRDERS



The method prescribed by AASHTO for end shear determination uses the simple beam approach. In the simple beam distribution approach, the slab is treated as a simply supported beam spanning between stringers and the reaction at the most heavily loaded stringer is computed. This reaction is the distribution factor for wheel loads applied at the end of the span. Note that for shear at the end of a simple span bridge for instance, the end axle (wheels) are distributed as above while for loads out in the span, the distribution is the same as that prescribed for moment, i.e., the "S Over" approach, typically, S/5.5 for multi stinger bridges where S is the beam spacing in feet.

Similar to interior stringers, which are typically the controlling elements in terms of total design force, empirical formulas also exist for live load distribution to exterior stringers. Although the total dead load and live load to an exterior stringer might tempt one to consider placing a beam of less capacity in the exterior location, such an approach is prohibited by code. The exterior elements, regardless of the lesser design load, must have at least the capacity of an interior stringer.

9. SAFETY

We are talking about load factor design method. In this design, the structural strength must be greater than the factored load effects. AASHTO specifies a number of load combinations that must be considered for bridge design. However, the typical load combination for superstructure design is the Group I load combination. For Group I, the factored load effects use a factor of 1.3 multiplied by the sum of the moments due to dead load plus the factored live load with a live load factor of 1.67. Therefore, the total effect is 1.3*dead loads plus 1.3*1.67=2.17 times the live load plus impact effect. M_u and V_u are nominal bending and shear strength of the beam itself. M_{DL} and V_{DL} are moments and shears caused by the dead load, while $M_{L(1+I)}$ and $V_{L(1+I)}$ are moments and shears caused by the live load and increased by the impact factor.

10.LOADING

In this section calculations related to Self-weight of structural elements has done. The density of reinforced concrete is taken as 150 Ib/ft^3 and the density of steel is taken as 490 Ib/ft^3. The self-weights are based on the nominal dimensions. The weight of railing system is applied to the exterior girders and assumed to be 0.4 kip/ft. Table 1 and 2 present the calculation for dead load for both exterior and interior girders of rolled and plate girder bridges with a 90 ft. span length and folded plate bridge with a 60 ft span length.

Table 1. Dead Loads on Girders for the rolled and plate girder bridges

Design Assumption	Value	Unit
Span Length	90	ft
Girder Spacing	12.67	ft
Number of girders	4	-
Slab Overhang	4.33	ft
Slab Depth	0.708333333	ft
Dead Load		
Slab		
Slab Dead Load	364.39075	kips
Interior Girder Total	121.156875	kips
Interior Girder uniform load	1.3461875	kips/ft
Exterior Girder Total	101.9840625	kips
Interior Girder uniform load	1.13315625	kips/ft
Weight of girder, diaphragms, diaphragms, & details	0.15	kips/ft
Weight of Rail applied on the exterior girder	0.42309585	kips/ft
Total Dead Load		
Interior Girder uniform load	**1.4961875**	kips/ft
Interior Girder uniform load	**1.7062521**	kips/ft

Table 2. Dead Loads on Girders for the folded plate bridge

Design Assumption	Value	Unit
Span Length	60	ft
Girder Spacing	10	ft
Number of girders	4	-
Slab Overhang	4.33	ft
Slab Depth	0.708333333	ft
Dead Load		
Slab		
Slab Dead Load	192.170125	kips
Interior Girder Total	63.75	kips
Interior Girder uniform load	1.0625	kips/ft
Exterior Girder Total	59.47875	kips
Interior Girder uniform load	0.9913125	kips/ft
Weight of girder, diaphragms, diaphragms, & details	0.15	kips/ft
Weight of Rail applied on the exterior girder	0.42309585	kips/ft
Total Dead Load		
Interior Girder uniform load	**1.2125**	kips/ft
Interior Girder uniform load	**1.56440835**	kips/ft

Traffic Loads

The HS20 truck is applied on the bridge as the live load and live load distribution factor is calculated by using Qcon Bridge software as shown in table 3. Impact factor for moving load is calculated based on the equation 1.

Impact (I)= 50/(L+125)= 1.23 for span length of 90 and 1.27 for span length of 60 ft.

Table 3. Live Load Distribution Factor-based on Qcon results

Service Limit State			Live Load Distribution Factor
Interior Girder	Moment	1-Lane	0.57
		2+Lanes	0.84
	Shear	1-Lane	0.84
		2+Lanes	1.099
Exterior Girder	Moment	1-Lane	1.008
		2+Lanes	0.92
	Shear	1-Lane	1.008
		2+Lanes	0.99

Figures 5 to 12 show the moment and shear diagrams for exterior and interior girder of the bridges by considering the impact factor of the moving vehicle.

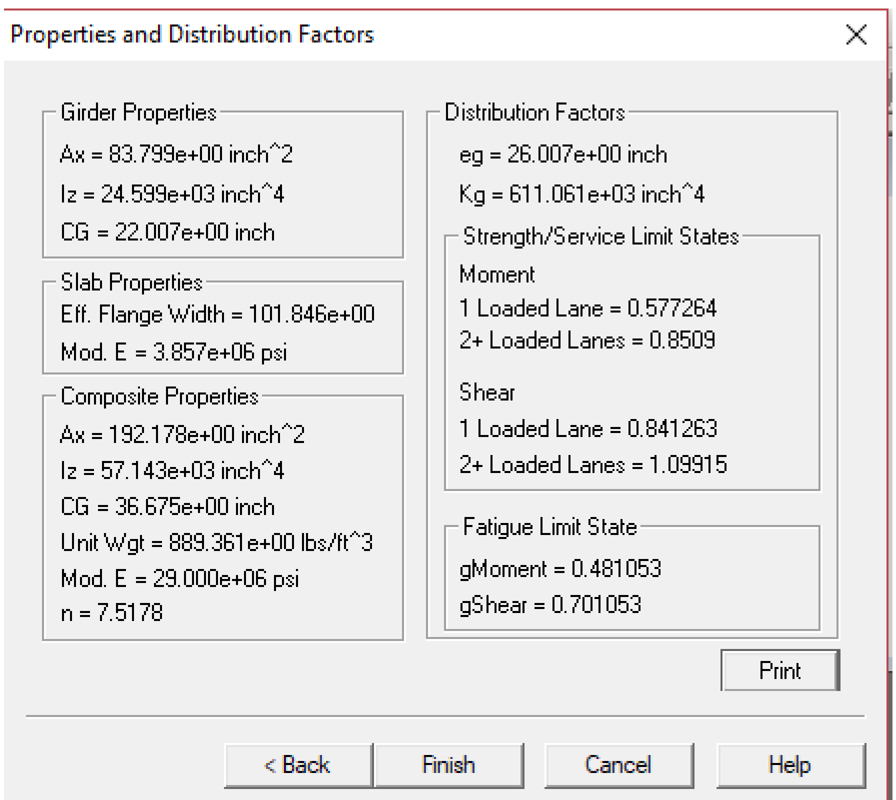


Figure 4: DF for hot rolled shaped girder and steel plate girder

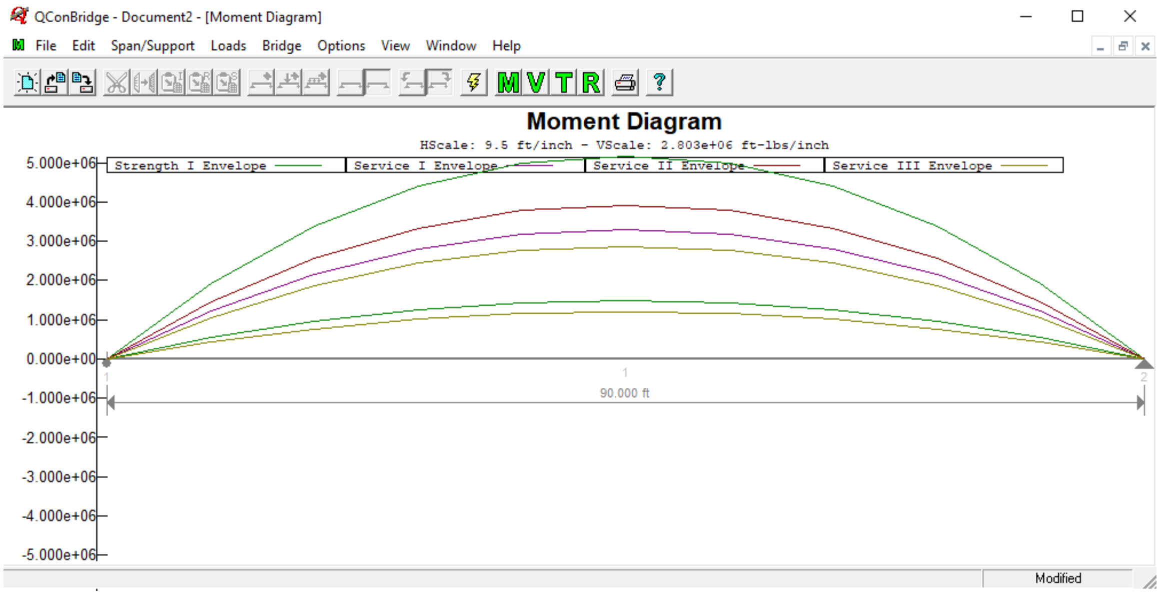


Figure 5: moment envelope diagram for hot rolled shaped girder and steel plate girder-different limit states

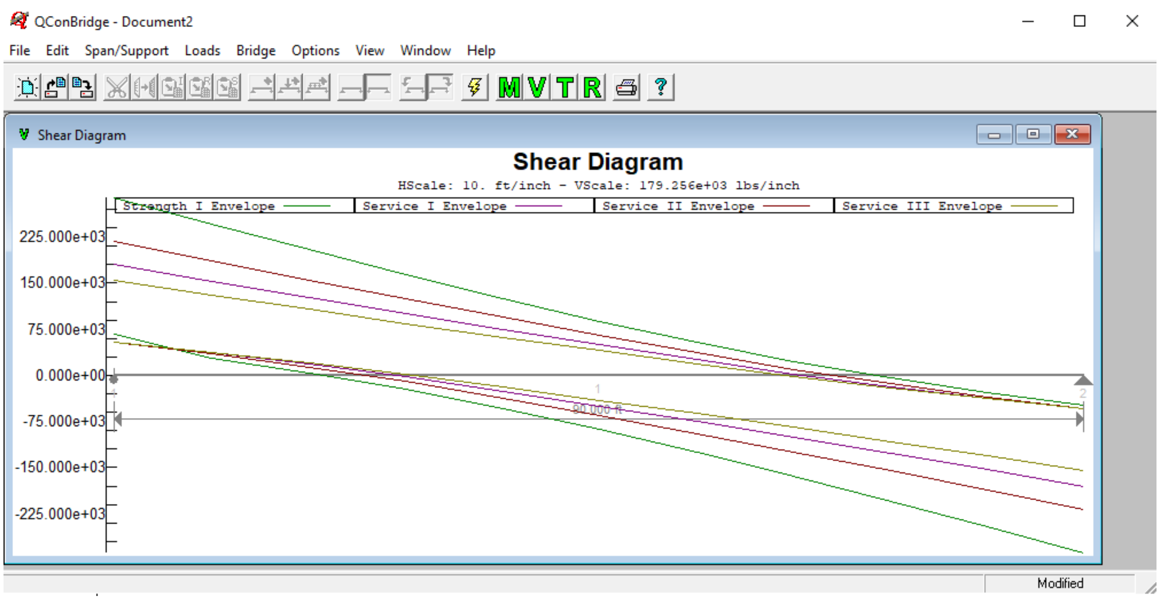


Figure 6: Shear envelope diagram for hot rolled shaped girder and steel plate girder-different limit states

Tabular Results

Strength I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs	Max Moment(ft-lbs)
1	0	66.763e+03	289.425e+03	0.000e+00	0.000e+00
1	1	28.020e+03	246.904e+03	540.787e+03	1.923e+06
1	2	4.101e+03	205.487e+03	961.400e+03	3.390e+06
1	3	-22.694e+03	165.172e+03	1.261e+06	4.398e+06
1	4	-54.586e+03	125.961e+03	1.442e+06	4.994e+06
1	5	-87.852e+03	87.852e+03	1.502e+06	5.153e+06
1	6	-125.961e+03	54.586e+03	1.442e+06	4.994e+06
1	7	-165.172e+03	22.694e+03	1.261e+06	4.398e+06
1	8	-205.487e+03	-4.101e+03	961.400e+03	3.390e+06
1	9	-246.904e+03	-28.020e+03	540.787e+03	1.923e+06
1	10	-289.425e+03	-48.070e+03	0.000e+00	0.000e+00

Tabular Results

Service I Limit State Envelopes

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs	Max Moment(ft-lbs)
1	0	53.411e+03	180.646e+03	0.000e+00	0.000e+00
1	1	36.765e+03	153.296e+03	432.630e+03	1.222e+06
1	2	17.908e+03	126.577e+03	769.120e+03	2.156e+06
1	3	-2.591e+03	100.488e+03	1.009e+06	2.802e+06
1	4	-26.003e+03	75.029e+03	1.153e+06	3.183e+06
1	5	-50.201e+03	50.201e+03	1.201e+06	3.288e+06
1	6	-75.029e+03	26.003e+03	1.153e+06	3.183e+06
1	7	-100.488e+03	2.591e+03	1.009e+06	2.802e+06
1	8	-126.577e+03	-17.908e+03	769.120e+03	2.156e+06
1	9	-153.296e+03	-36.765e+03	432.630e+03	1.222e+06
1	10	-180.646e+03	-53.411e+03	0.000e+00	0.000e+00

Figure 7: maximum and minimum shear and moment for strength I and Service I limit states for hot rolled shaped girder and steel plate girder-different limit states

Tabular Results

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
				Service II Limit State Envelopes	
1	0	53.411e+03	218.816e+03	0.000e+00	0.000e+00
1	1	34.976e+03	186.466e+03	432.630e+03	1.460e+06
1	2	13.667e+03	154.936e+03	769.120e+03	2.573e+06
1	3	-9.777e+03	124.225e+03	1.009e+06	3.339e+06
1	4	-37.009e+03	94.334e+03	1.153e+06	3.792e+06
1	5	-65.262e+03	65.262e+03	1.201e+06	3.914e+06
1	6	-94.334e+03	37.009e+03	1.153e+06	3.792e+06
1	7	-124.225e+03	9.777e+03	1.009e+06	3.339e+06
1	8	-154.936e+03	-13.667e+03	769.120e+03	2.573e+06
1	9	-186.466e+03	-34.976e+03	432.630e+03	1.460e+06
1	10	-218.816e+03	-53.411e+03	0.000e+00	0.000e+00

Tabular Results

Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs)	Max Moment(ft-lbs)
				Service III Limit State Envelopes	
1	0	53.411e+03	155.199e+03	0.000e+00	0.000e+00
1	1	37.958e+03	131.183e+03	432.630e+03	1.064e+06
1	2	20.736e+03	107.671e+03	769.120e+03	1.879e+06
1	3	2.199e+03	84.663e+03	1.009e+06	2.443e+06
1	4	-18.666e+03	62.160e+03	1.153e+06	2.777e+06
1	5	-40.161e+03	40.161e+03	1.201e+06	2.871e+06
1	6	-62.160e+03	18.666e+03	1.153e+06	2.777e+06
1	7	-84.663e+03	-2.199e+03	1.009e+06	2.443e+06
1	8	-107.671e+03	-20.736e+03	769.120e+03	1.879e+06
1	9	-131.183e+03	-37.958e+03	432.630e+03	1.064e+06
1	10	-155.199e+03	-53.411e+03	0.000e+00	0.000e+00

Figure 8: maximum and minimum shear and moment for Service II and Service III limit states for hot rolled shaped girder and steel plate girder-different limit states

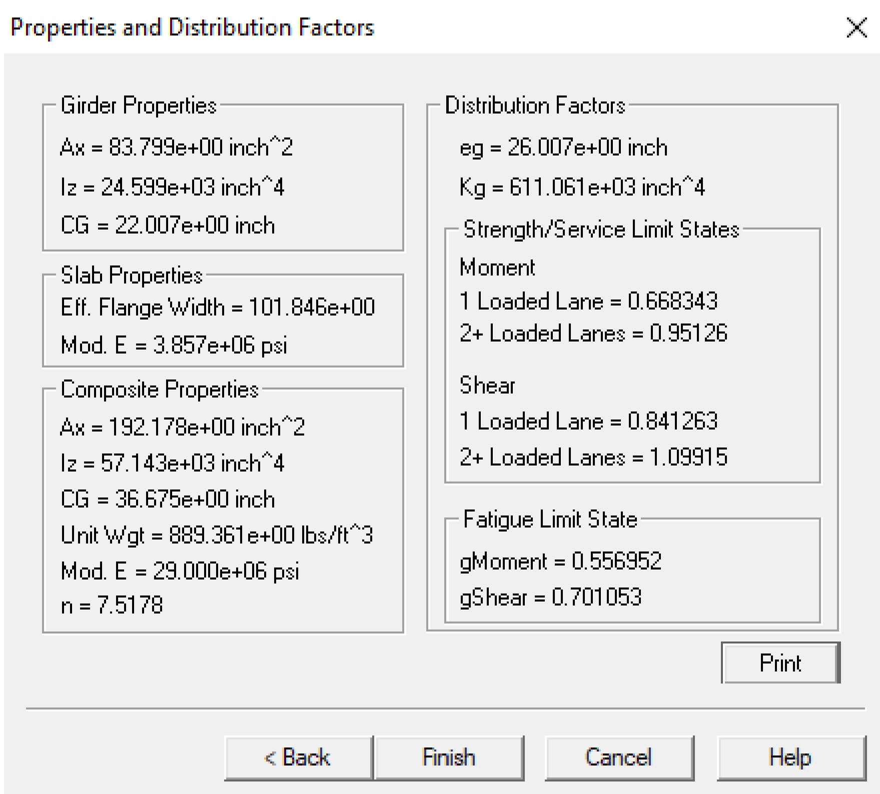


Figure 9: DF for folded plate girder

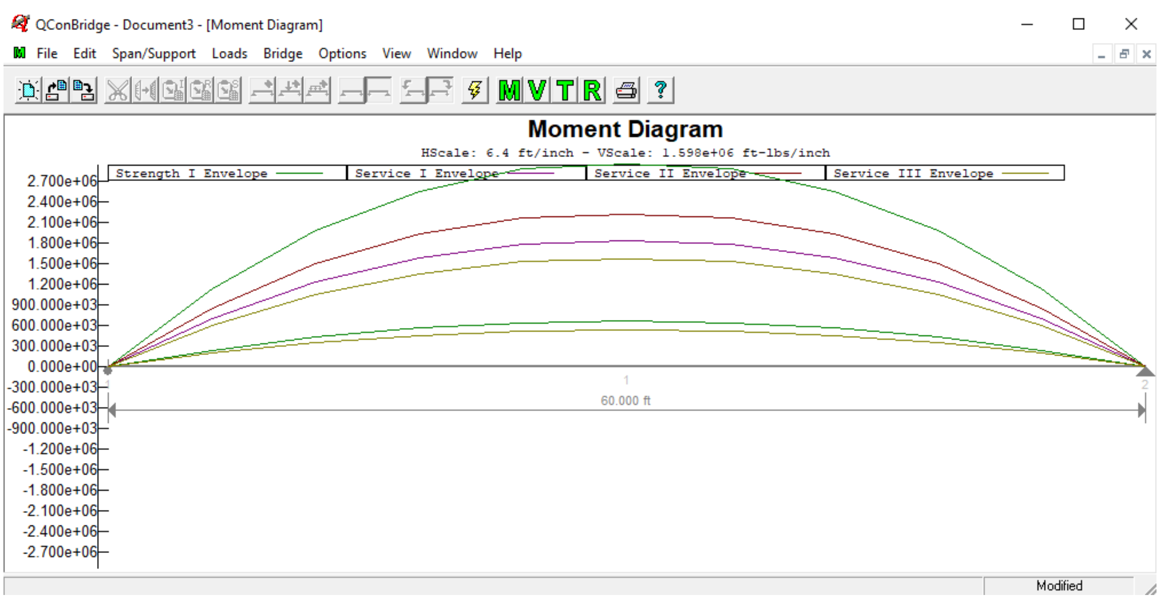


Figure 10: moment envelope diagram for folded plate girder- different limit states

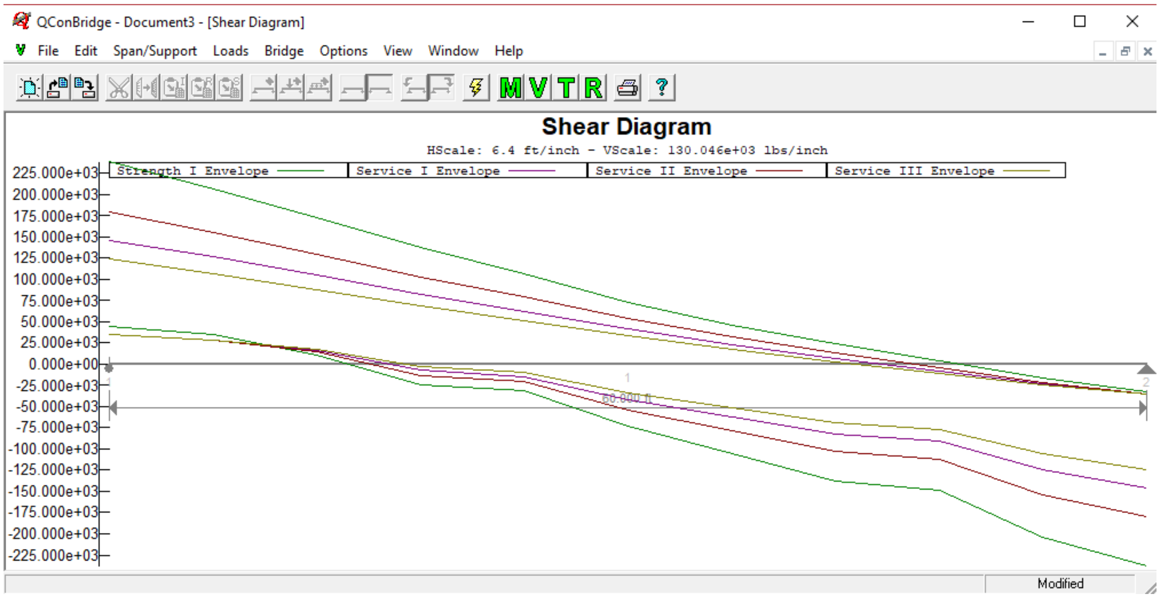


Figure 11: Shear envelope diagram for folded plate girder- different limit states

Tabular Results

				Strength I Limit State Envelopes	
Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lb	Max Moment(ft-lbs)
1	0	44.509e+03	239.096e+03	0.000e+00	0.000e+00
1	1	35.607e+03	207.829e+03	240.350e+03	1.134e+06
1	2	10.319e+03	173.293e+03	427.289e+03	1.983e+06
1	3	-24.292e+03	137.572e+03	560.816e+03	2.546e+06
1	4	-31.069e+03	106.672e+03	640.933e+03	2.873e+06
1	5	-73.566e+03	73.566e+03	667.639e+03	2.938e+06
1	6	-105.202e+03	46.573e+03	640.933e+03	2.873e+06
1	7	-137.572e+03	24.292e+03	560.816e+03	2.546e+06
1	8	-148.476e+03	3.557e+03	427.289e+03	1.983e+06
1	9	-204.519e+03	-16.769e+03	240.350e+03	1.134e+06
1	10	-239.096e+03	-32.046e+03	0.000e+00	0.000e+00

Tabular Results

				Service I Limit State Envelopes	
Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lb	Max Moment(ft-lbs)
1	0	35.607e+03	146.800e+03	0.000e+00	0.000e+00
1	1	28.485e+03	126.898e+03	192.280e+03	703.236e+03
1	2	16.273e+03	105.128e+03	341.831e+03	1.231e+06
1	3	-6.963e+03	82.682e+03	448.653e+03	1.583e+06
1	4	-14.295e+03	62.990e+03	512.746e+03	1.788e+06
1	5	-42.038e+03	42.038e+03	534.111e+03	1.831e+06
1	6	-62.150e+03	23.154e+03	512.746e+03	1.788e+06
1	7	-82.682e+03	6.963e+03	448.653e+03	1.583e+06
1	8	-90.947e+03	-8.344e+03	341.831e+03	1.231e+06
1	9	-125.007e+03	-23.418e+03	192.280e+03	703.236e+03
1	10	-146.800e+03	-35.607e+03	0.000e+00	0.000e+00

Figure 12: maximum and minimum shear and moment for strength I and Service I limit states for folded plate girder -different limit states

Tabular Results

				Service II Limit State Envelopes	
Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs	Max Moment(ft-lbs)
1	0	35.607e+03	180.158e+03	0.000e+00	0.000e+00
1	1	28.485e+03	156.422e+03	192.280e+03	856.523e+03
1	2	14.746e+03	130.258e+03	341.831e+03	1.497e+06
1	3	-13.325e+03	103.214e+03	448.653e+03	1.923e+06
1	4	-20.720e+03	79.751e+03	512.746e+03	2.171e+06
1	5	-54.649e+03	54.649e+03	534.111e+03	2.221e+06
1	6	-78.658e+03	32.237e+03	512.746e+03	2.171e+06
1	7	-103.214e+03	13.325e+03	448.653e+03	1.923e+06
1	8	-111.822e+03	-4.438e+03	341.831e+03	1.497e+06
1	9	-153.963e+03	-21.898e+03	192.280e+03	856.523e+03
1	10	-180.158e+03	-35.607e+03	0.000e+00	0.000e+00

Tabular Results

				Service III Limit State Envelopes	
Span	Point	Min Shear(lbs)	Max Shear(lbs)	Min Moment(ft-lbs	Max Moment(ft-lbs)
1	0	35.607e+03	124.561e+03	0.000e+00	0.000e+00
1	1	28.485e+03	107.215e+03	192.280e+03	601.045e+03
1	2	17.291e+03	88.375e+03	341.831e+03	1.053e+06
1	3	-2.722e+03	68.994e+03	448.653e+03	1.356e+06
1	4	-10.011e+03	51.816e+03	512.746e+03	1.533e+06
1	5	-33.630e+03	33.630e+03	534.111e+03	1.572e+06
1	6	-51.144e+03	17.099e+03	512.746e+03	1.533e+06
1	7	-68.994e+03	2.722e+03	448.653e+03	1.356e+06
1	8	-77.031e+03	-10.948e+03	341.831e+03	1.053e+06
1	9	-105.703e+03	-24.431e+03	192.280e+03	601.045e+03
1	10	-124.561e+03	-35.607e+03	0.000e+00	0.000e+00

Figure 13: maximum and minimum shear and moment for Service II and Service III limit for folded plate girder -different limit states

The final results that gathered from QCon software presented in

Table 4: Factored Design load for span length of 90 ft.

	Strength I	Strength II	Strength III	Strength IV
Maximum Moment (kips.ft)	**6.45E+03**	5.47E+03	2.16E+03	5.90E+03
Maximum Shear (Kips)	**298.45**	252.17	95.97	271.36

Table 5: Factored Design load for span length of 60 ft.

	Strength I	Strength II	Strength III	Strength IV
Maximum Moment (kips.ft)	**3.26E+03**	2.72E+03	8.80E+02	2.89E+03
Maximum Shear (Kips)	**235.59**	195.15	58.66	206.88

After many Iterations using a computer program ETABS the following section was obtained.

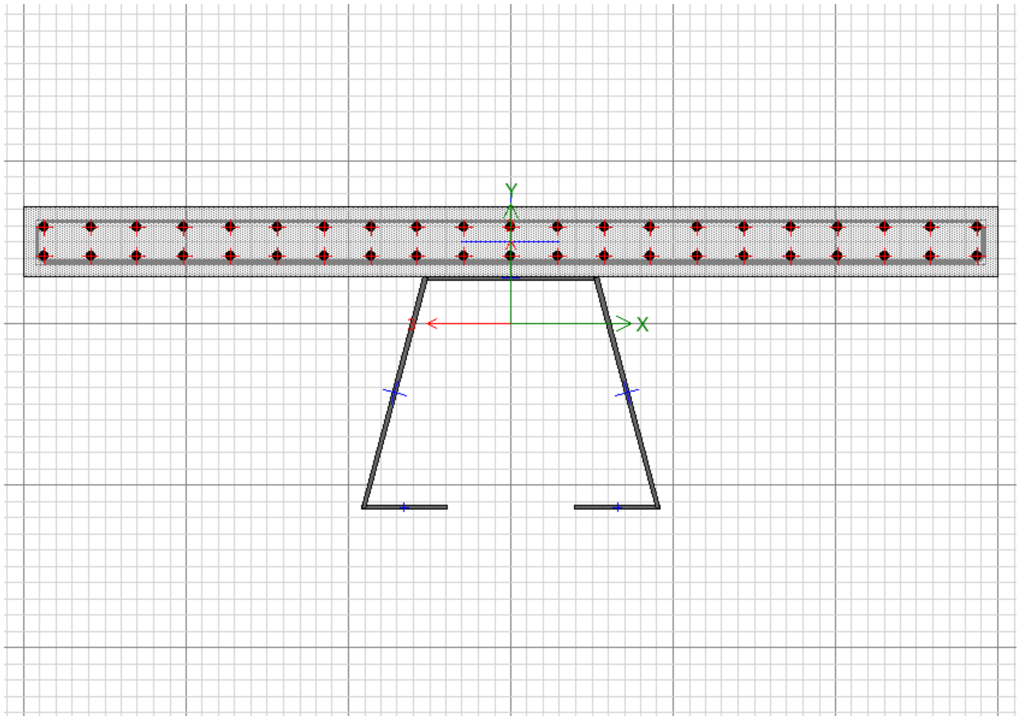


Figure 14: folded plate section in ETABS

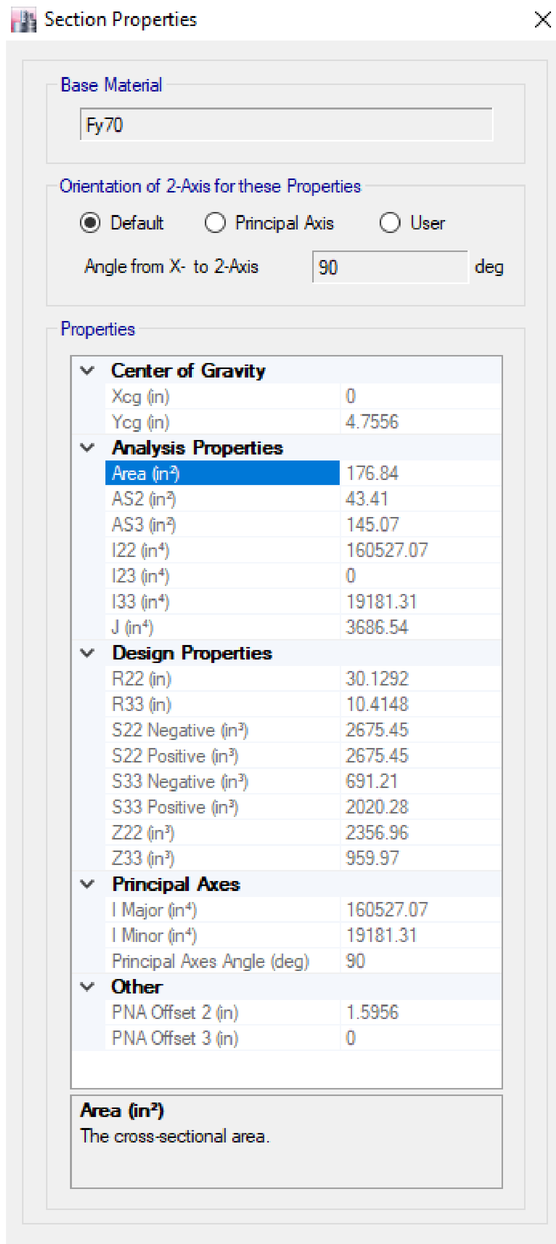


Figure 15: properties foe folded plate section

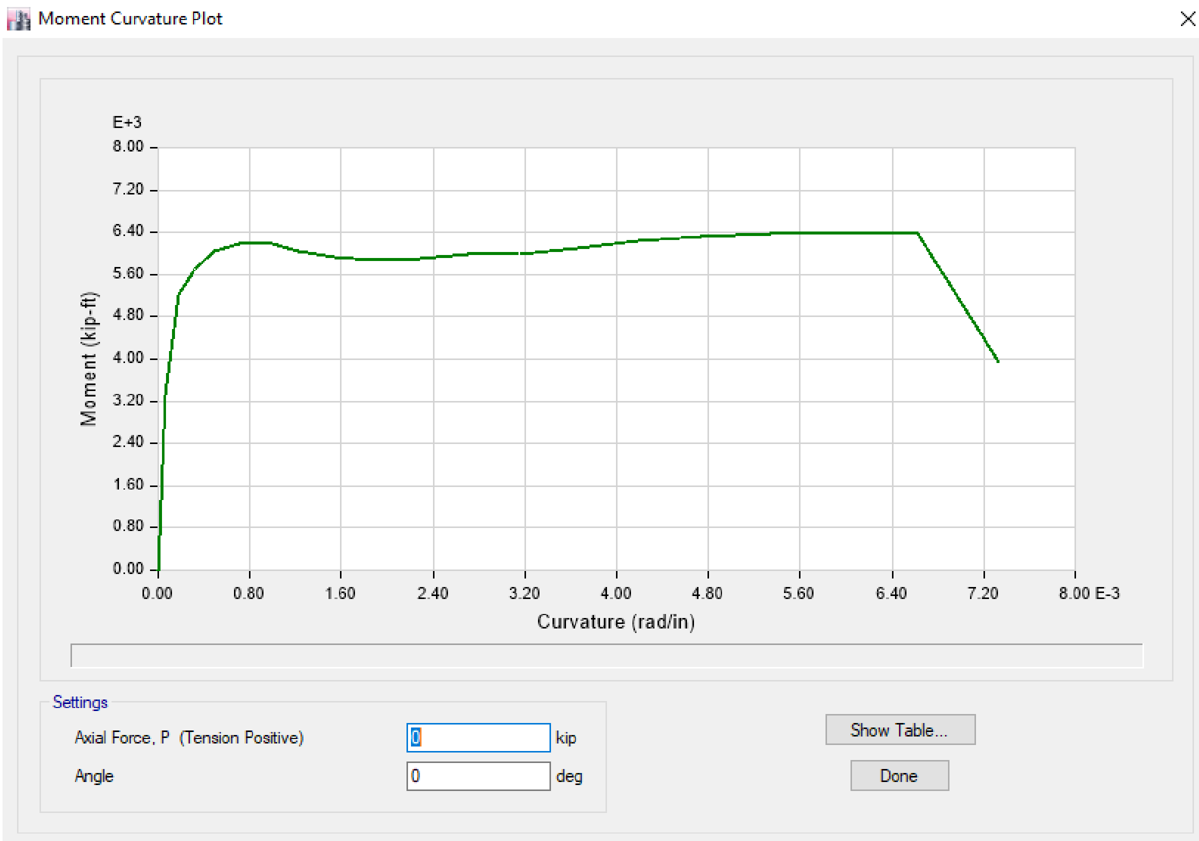


Figure 16: moment curvature diagram for folded plate section

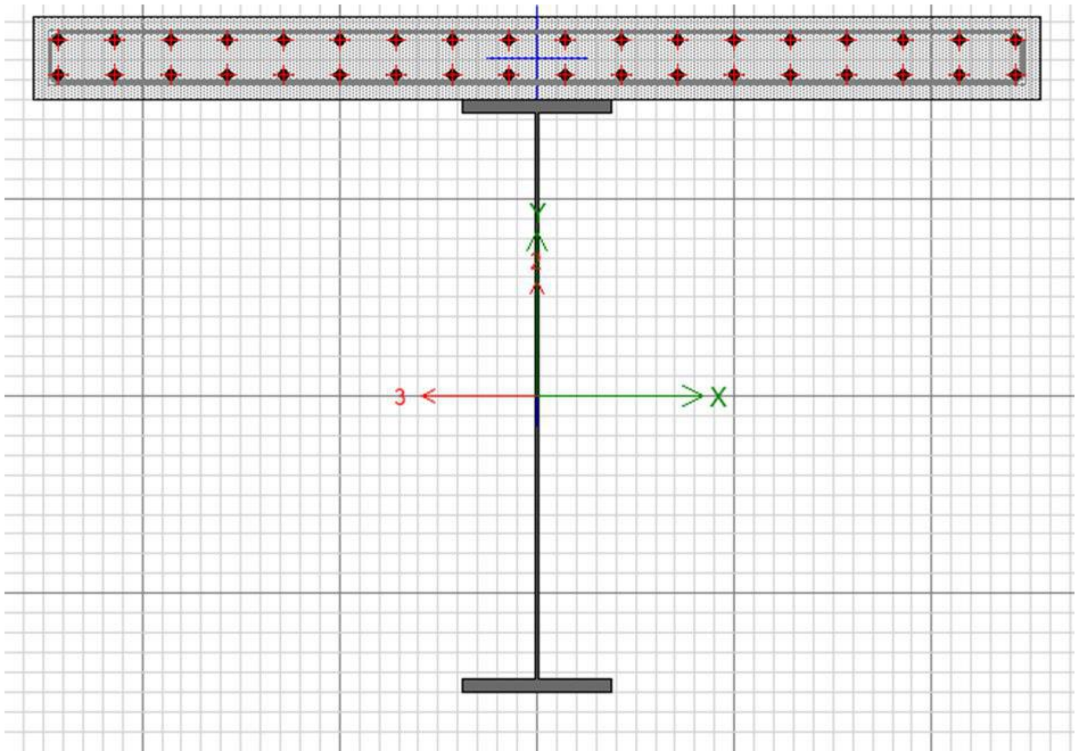


Figure 17: plate girder section modelled in ETABS

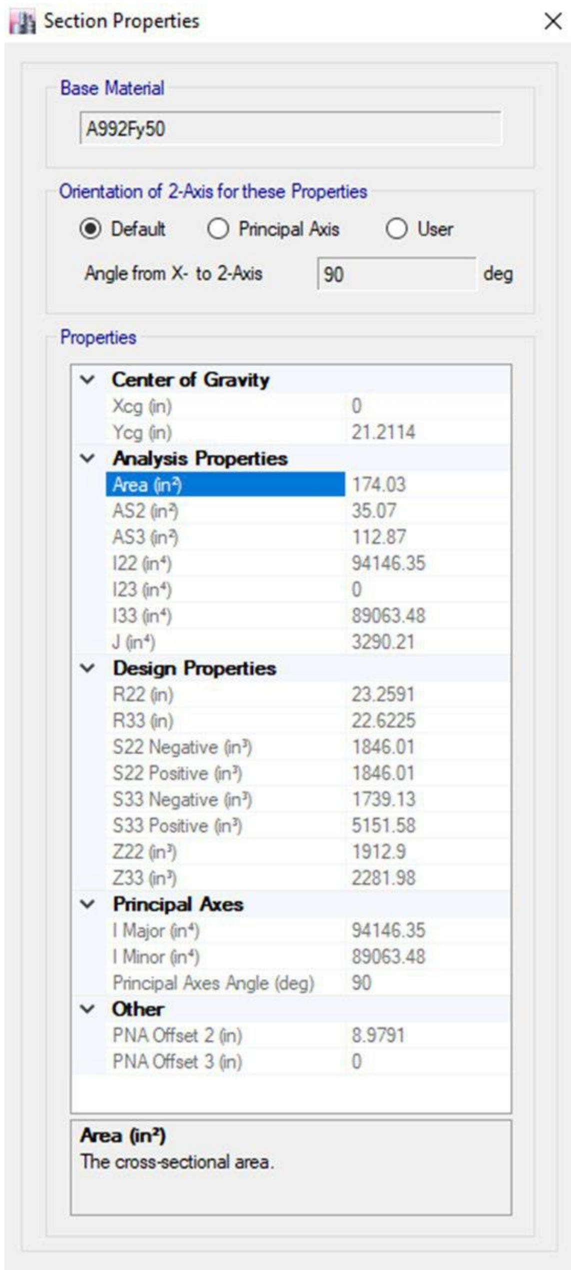


Figure 18: plate girder section properties

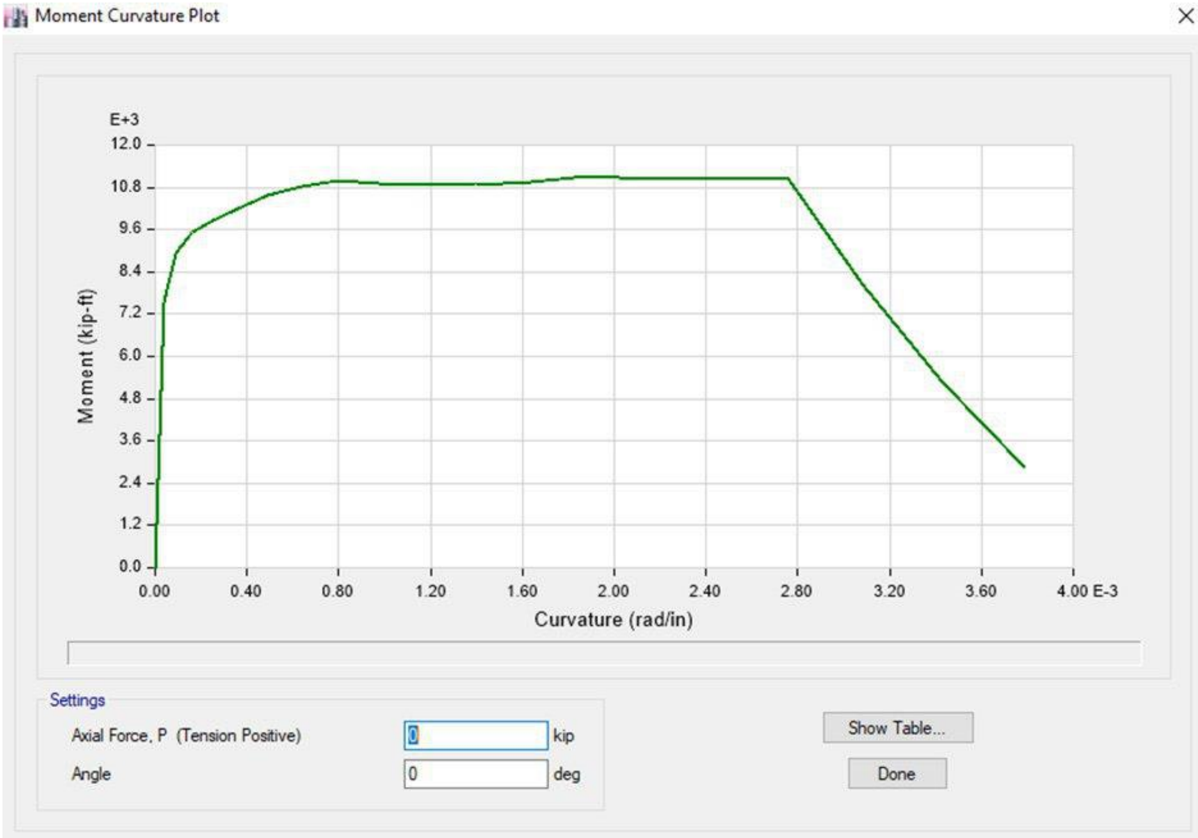


Figure 19: moment curvature diagram for plate girder section

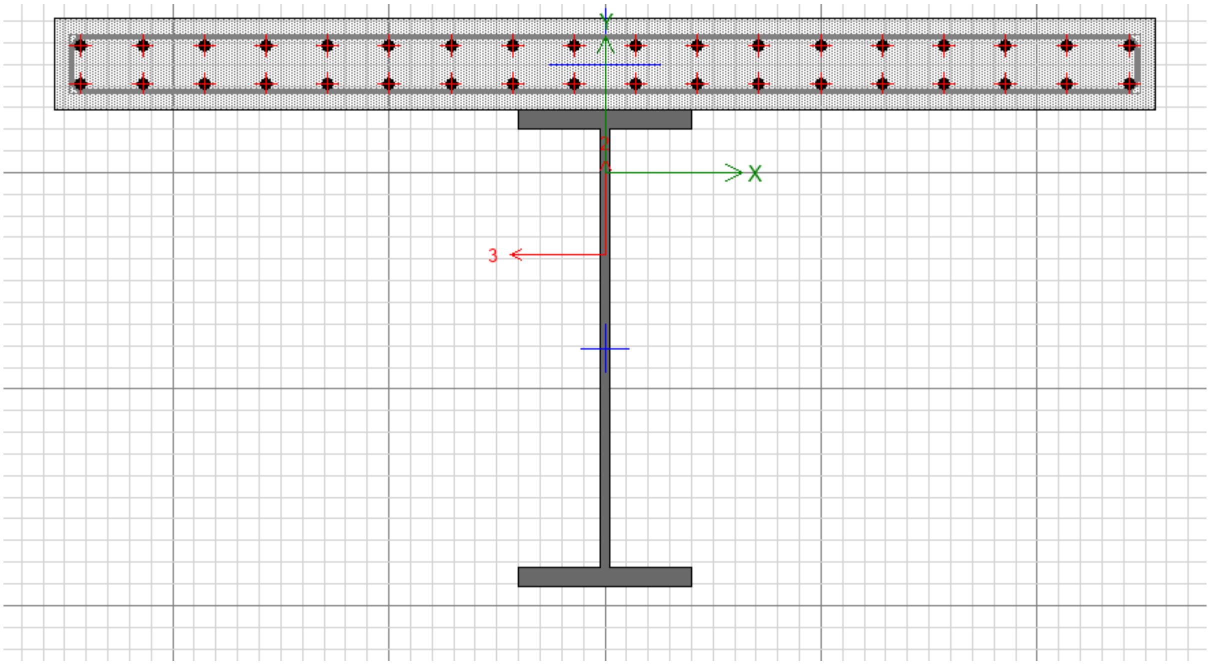


Figure 20: rolled shape girder section modelled in ETABS

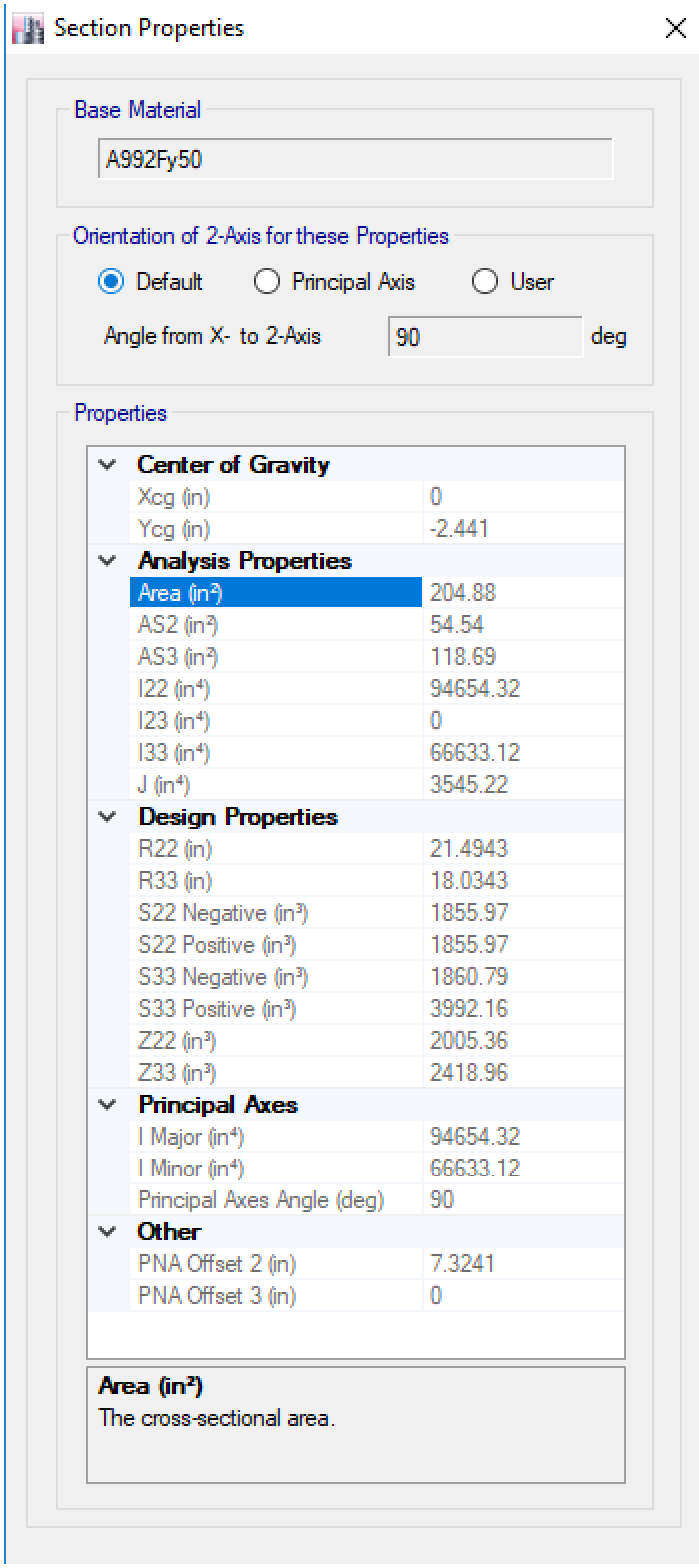


Figure 21: Rolled shape girder section properties

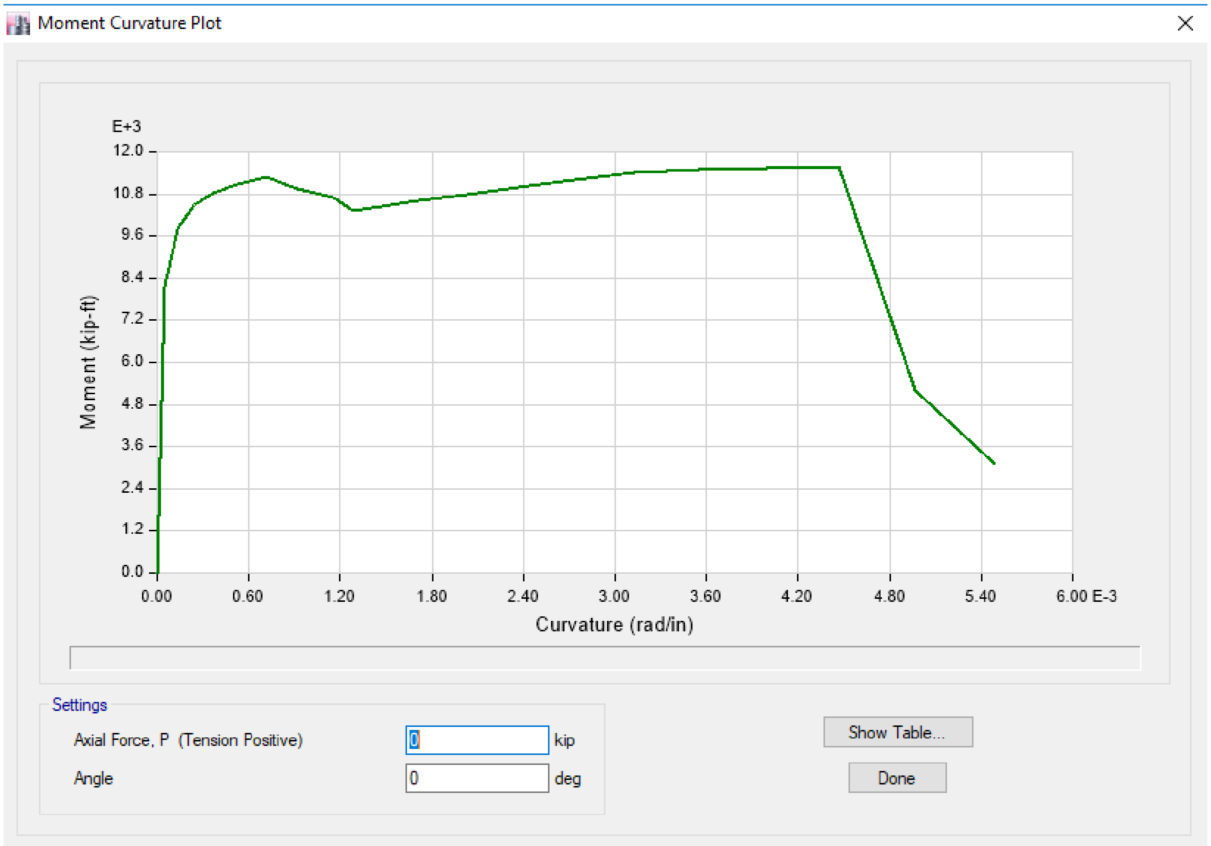
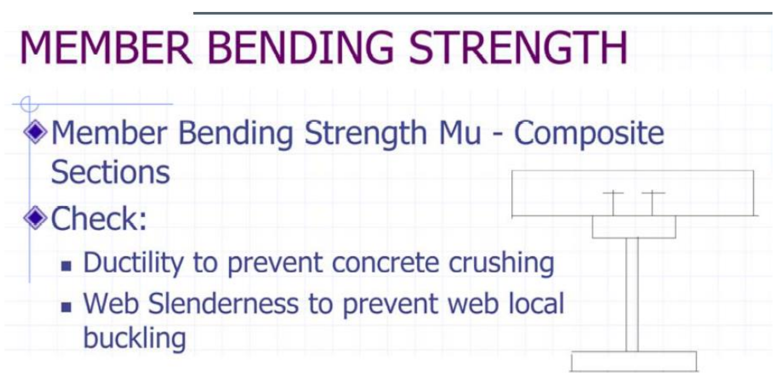


Figure 22: Moment curvature for Rolled shape girder

After completing this section and calculating moment plastic of sections, the next step is the check the sections proportionality and all limit states requirements.

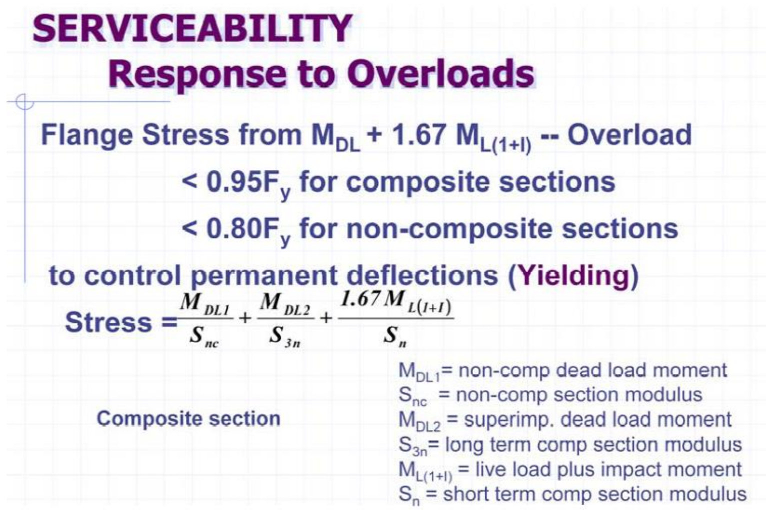
11.MEMBER BENDING STRENGTH

The bending strength for non-composite sections primarily consists of limit states of lateral-torsional buckling, compression flange buckling and web local buckling. These are traditional slenderness checks for steel beam design. The moment capacity of the beam can be equal to M_P if all the adequate slenderness limits are satisfied though this is uncommon for the non-composite condition. More realistically, the moment capacity of a rolled shape or plate girder bridge in the short to medium span range will be at or near to the yield capacity, M_y in the non-composite condition.

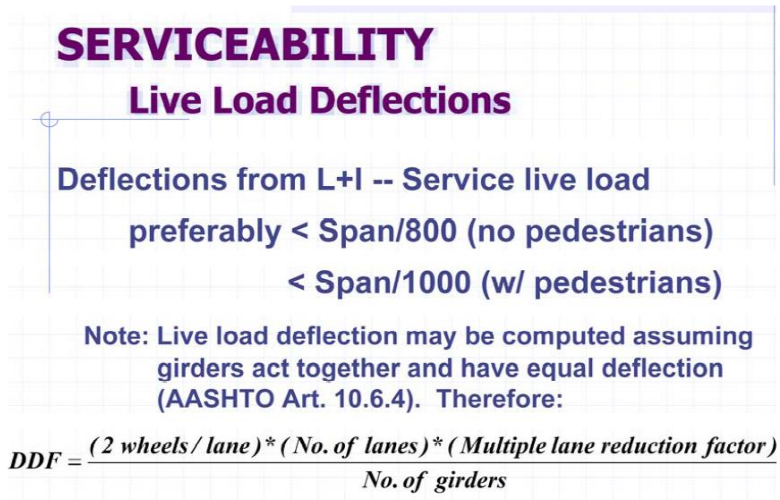


For composite beams, the moment capacity is usually at or very near to the plastic moment capacity of the composite section. Shear studs are required to ensure full composite action. M_P, the plastic moment, corresponds to the crushing of the concrete and significant yielding and strain hardening of the steel. The location of the plastic neutral axis must be determined considering equilibrium of the tensile and compressive plastic forces. The section should be designed in such way so that the concrete crushing occurs after significant steel yielding and strain hardening has occurred. This is to ensure ductility in the event of an extreme overload. As a designer, one should design structures to prevent concrete crushing before steel yielding occurs because that will be a brittle failure mode. The slenderness of the web must be checked to assure that the web does not buckle laterally in the compression region.

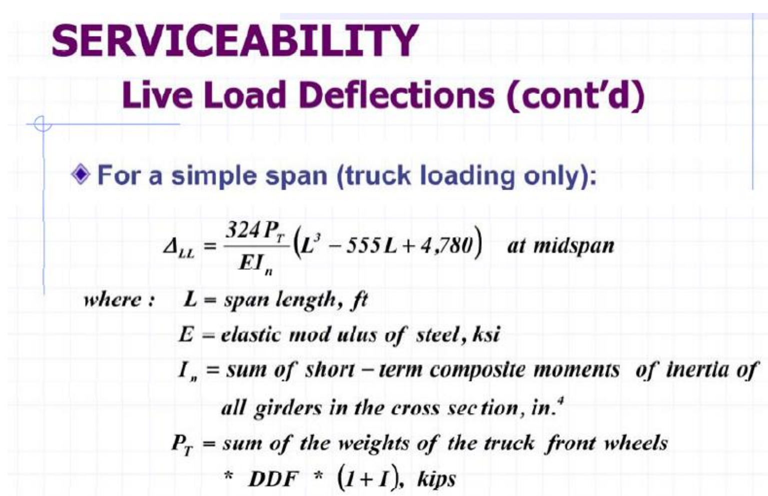
12.SERVICEABILITY



The Overload criteria is a serviceability design requirement for steel beams. For the overload check the stress in the tension flange due to the dead load plus an elevated live load of 1.67 times the live load and impact is limited. The stress caused by the design overload must be less than $0.95F_y$ for composite sections and $0.80F_y$ for the non-composite sections. The purpose of this limit is to control the permanent deflections under the specified load. The stress corresponding to the overload is calculated as shown in the above expression as the elastic superposition of several load effects. The dead loads consist of the non-composite dead loads and superimposed composite dead loads. The steel beam is subject to the dead load caused by its own weight, the weight of the slab and weight of the wearing surface. After the slab hardens, composite action occurs between the steel and concrete. There is additional dead load due to the additional wearing surfaces that are added over the years and the barriers that are placed on the composite section. M_{DL1} corresponds to the non-composite dead load moment and M_{DL2} corresponds to the superimposed dead load moment. Before the concrete hardens, S_{nc} is the section modulus for the non- composite section that is the beam by itself. S_{3n} is the reduced composite section modulus accounting for long term effects, that means creep of the concrete. The composite section is subject to additional dead loads caused by the barriers and wearing surfaces. These are long term loads and the effects of creep are accounted for. Additionally, there is the live load moments and Sn is the short term composite section modulus without consideration of creep because live loads are a transient load.



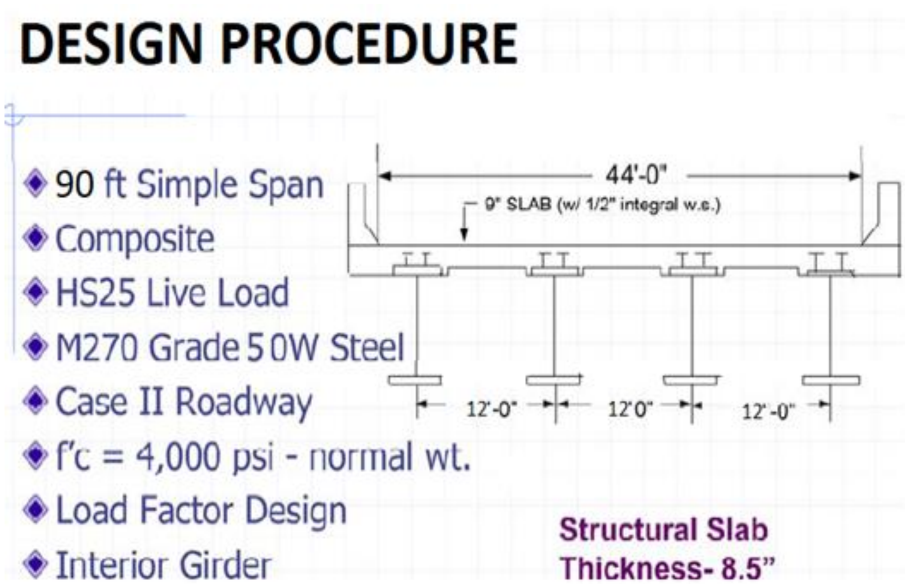
Another serviceability check relates to the deflections caused by the service live load. This is unfactored live load plus impact. The deflection caused by this load must be less than L/800 if there are no pedestrians and L/1000 if there are pedestrians on the bridge. Based on AASHTO specifications, the live load deflection can be computed assuming that the girders act together and have equal deflections. What this means is that the deflection distribution factor can be calculated by 2 wheels per lane multiplied by the number of lanes considering the multiple lane reduction factors. The whole bridge is assumed to act as an unit for the live load to estimate the deflections.



For a simple span with truck loading only, the maximum deflection at the mid span can be estimated using the above equation. The referenced equation is from a series of design examples for steel bridges prepared by the steel industry in the 1970's. In this equation, L is the span length and I_n is the moment of inertia of the completed bridge. The sum of the short term composite moments of inertia of all girders in the cross section is used to estimate the deflection caused by the unfactored live load. P_T is the sum of the weights of the truck front wheels multiplied by the deflection distribution factor and the impact factor. to estimate the deflection for the live load. It is used to estimate the deflection for the live load.

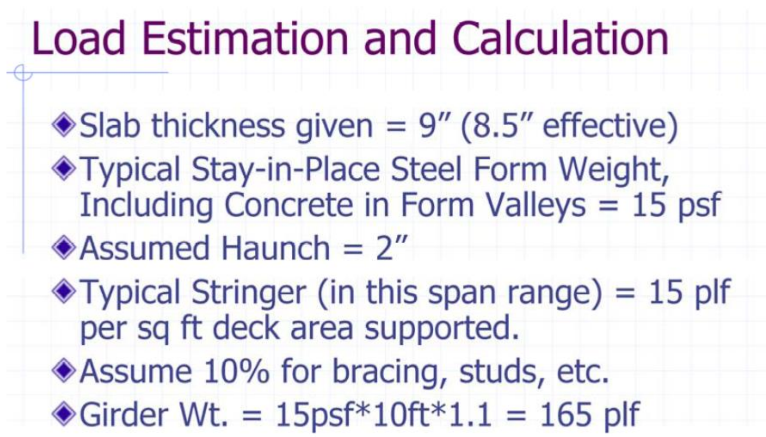
13.DESIGN PROCEDURE

13.1. ASSUMPTIONS



We have identified that in order for us to design a steel bridge, we need to design for strength and serviceability. For strength, we need to compare the moment capacity and the shear capacity of the bridge with the factored loads. For serviceability, we need to look at the overload effects, the deflections caused by the live loads and fatigue stress range. The best way to look at all these checks is through a design example. The example bridge we are going to use is a 90-ft simple span bridge. We will design it as composite steel girder bridge. HS25 will be used for the live load in this design project. We are using 60 ksi steel and Case II roadway. The Case II roadway corresponds to a certain number of cycles for truck loading and a certain number of cycles for lane loading. It corresponds to annual daily truck traffic which is less than 2500. The concrete strength is 4000 psi. We will be using load factor design to design one of the interior girders. The cross section of the bridge is shown above. We have four girders that are spaced at 12 feet each. The slab is 9” thick with ½” integral wearing surface. The structural slab thickness then becomes 8 ½” because the ½” is the integral wearing surface. The roadway width is 44 feet.

13.2. LOAD ESTIMATION AND CALCULATION

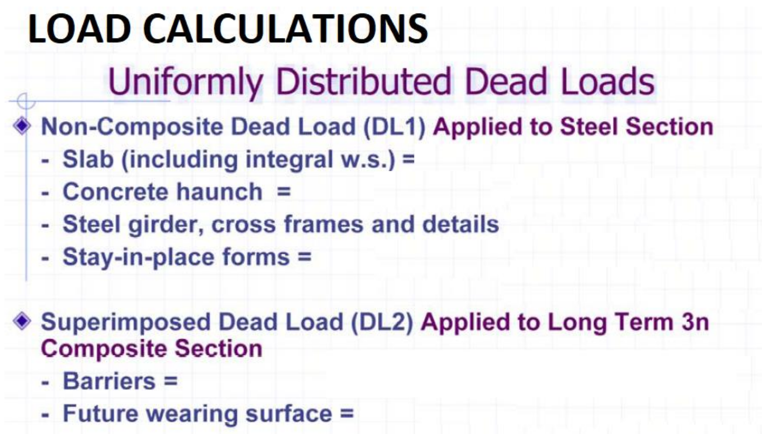


In order to begin our design, some assumptions need to be made regarding the magnitude of some of the loads. Since we have chosen, or previously designed our slab, it has a thickness of 9”. This thickness will be used for self-weight estimation though when we proceed to structural design, the top ½” of the slab will be discounted as it is prone to wear.

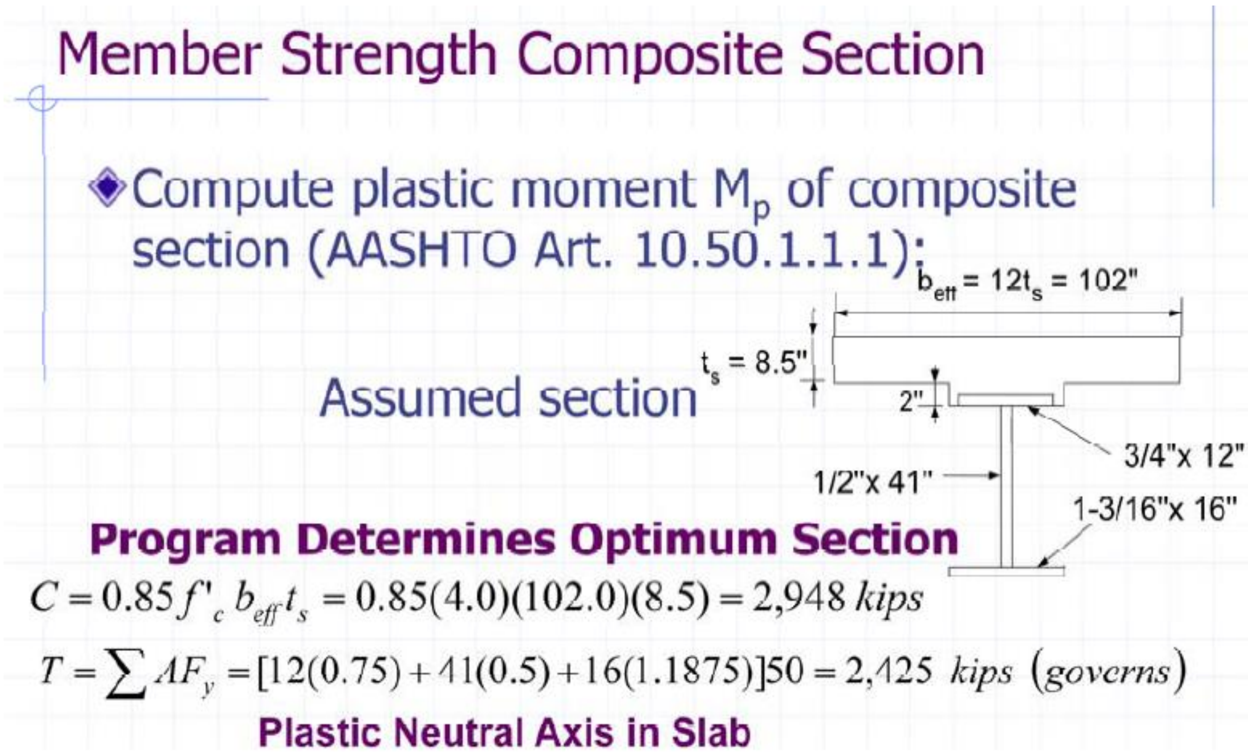
This design presumes the use of stay-in-place steel forms. These forms are typically corrugated and are permanent forms that remain in place following casting of the deck. A reasonable estimate for the self-weight of the form as well as the non-contributing concrete that lays in the valleys of the forms is 15 psf.

A 2” haunch is presumed. This haunch is provided to allow for the cross slope of the deck and as a field adjustment for corrections between the camber of the beam and the vertical profile of the roadway surface. The haunch is not considered as structural concrete but is considered in load calculations.

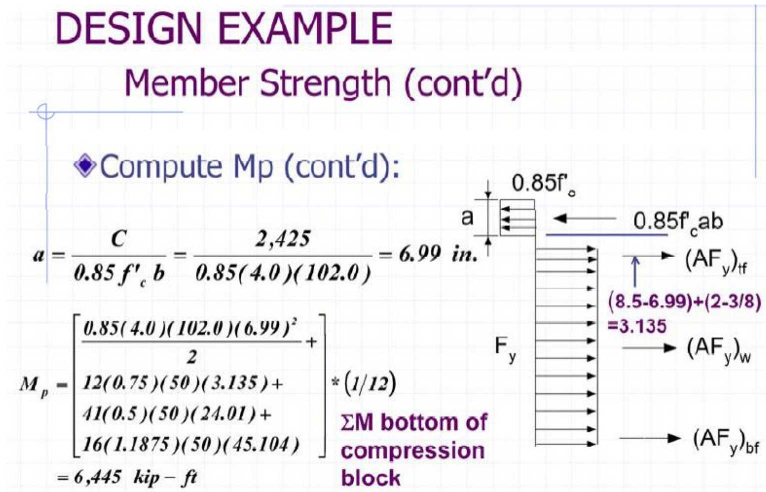
Finally, some estimate must be made of the girders initial weight. In the span range considered for simple span steel bridges, i.e. less than say 120-150 ft, an accurate estimate of plate girder weights can usually be made. The plate girders typically weigh on the order of 15 – 25 plf per sq ft of deck supported. For our bridge we will assume 15 plf per sq ft, thus for a 10 ft beam spacing, we will assume a beam wt of 150 plf. We will allow an additional 10% for miscellaneous steel such as connection plates, shear studs, etc. The initial girder weight is estimated by the **Qcon** software.



The first step is to estimate the loads and the load effects. In order to design a composite beam, the dead load will be estimated in two parts. The first part corresponds to the load that placed on the beam before the concrete hardens. That includes the weight of slab, the weight of concrete haunch, the weight of steel girder, cross frames and details and the weight of stay-in-place forms. The superimposed dead load which applied to long term composite section includes the weight of barriers and the weight of future wearing surface. The weight of barriers and the weight of wearing surface is distributed over all four girders. So the weight of barriers is assumed to be carried by all the girders equally.

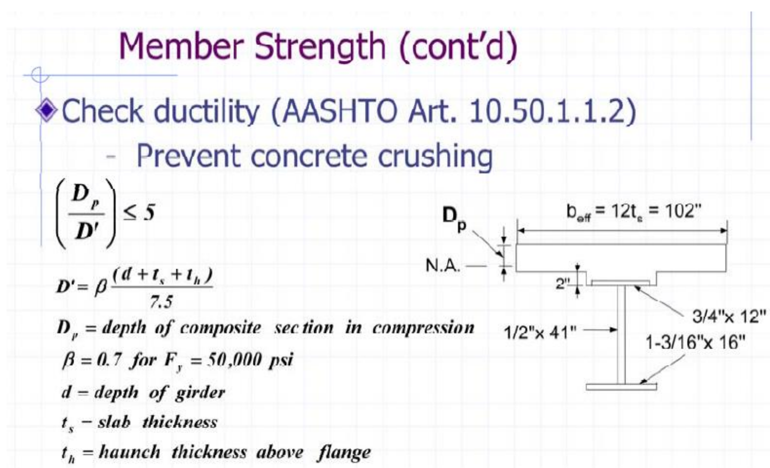


The next step is to estimate the capacity of the beam to carry the loads. We want to compute the plastic moment of the composite section. The first thing will be to estimate the effective deck width which is calculated as the smallest value of three things. It turns out that in this case the 12 times the minimum slab thickness governs the value. The effective deck width is 102”. The structural thickness of the slab is 8 ½” and the haunch is 2”. The girder shape that is used in this design is asymmetrical with a smaller compression flange and a wider tension flange. The plate thickness is ¾” for compression flange, 1 3/16” for the tension flange and ½ for the web. This is reasonably stocky web but results in a fairly simple web to fabricate, one that does not require shear stiffeners, only diaphragm connection plates. This is the section selected by AISIBEAM as an optimum design. The presentation of this computer program is included in the PDHonline Course 115, Computer Aided Design and Detailing of Short Span Steel Bridges. The first order is to calculate the location of the plastic neutral axis through force equilibrium. The compression forces of the section must be equal to the tensile forces. We calculate the compression force assuming the entire slab will be in compression. The compression capacity of the slab is 2,948 kips. If you assume the plastic neutral axis lies at bottom of the slab, the entire steel beam will be in tension, a tensile capacity of 2,425 kips. Since this is less than the plastic compressive load carrying capacity of the concrete, the plastic neutral axis will lie in the slab. With the plastic neutral axis inside the slab and the requirement that compression = tension, the location of the PNA can be determined.

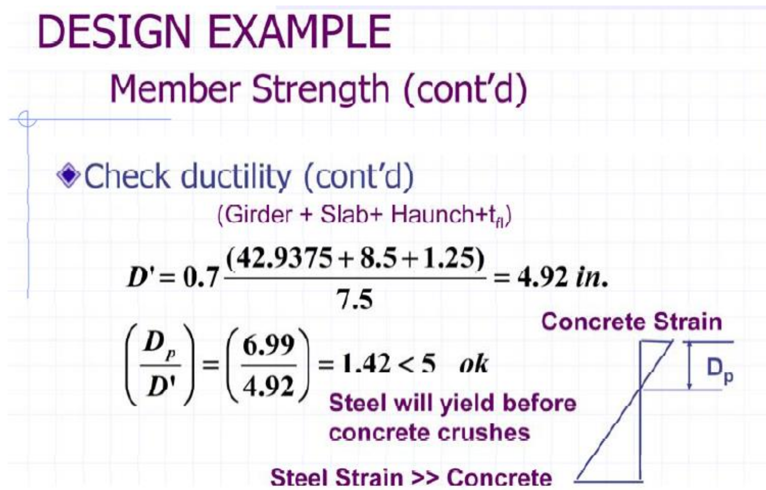


To compute M_p, we need to determine the location of the neutral axis. The neutral axis location is determined as term a equals to C divided by $0.85f'_cb$. This is nothing but a equilibrium check on the cross section. The compression forces must equal to tensile forces, which give us a equal to 7". So the depth of the neutral axis is located 7" from the top of the slab. M_p can be estimated by taking the summation of the moments from the bottom of the compression block. Again the plastic strength of the concrete is $0.85f'_c$ and so this force component equals to $0.85f'_cab_{eff}$.

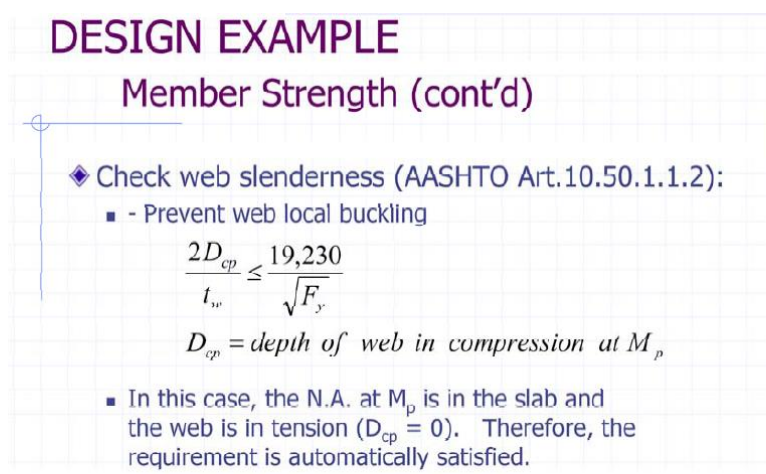
And then we calculate the forces in the top flange, the web and the bottom flange and take moments about the neutral axis. We can obtain the plastic moment of the composite section by summing up all the moments.



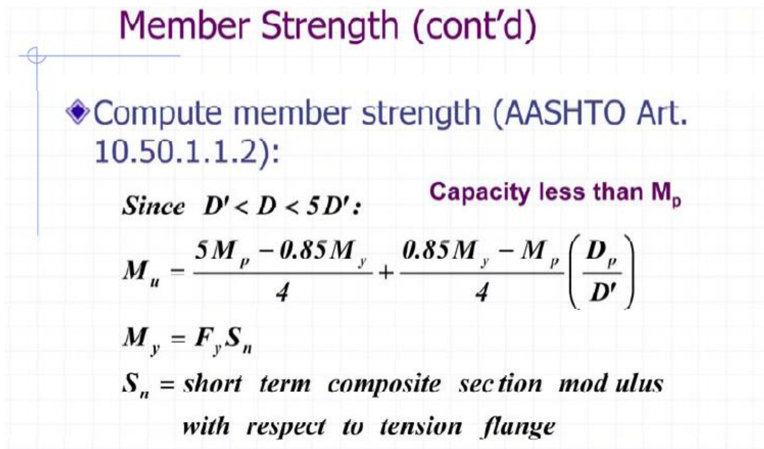
However, when we are calculating plastic capacity of the girder, we are assuming that the concrete can develop compressive strength of $0.85f'_c$ and concrete crushing is going to occur after the steel has yielded. Tu assure this ductile behavior, a check for ductility must be completed. To prevent concrete crashing, D_p /D' must be less than 5. D_p is the depth of the plastic neutral axis calculated as 6.99", say 7". D' is a cross section characteristic whose definition is given in AASHTO and is calculated in the above expression.



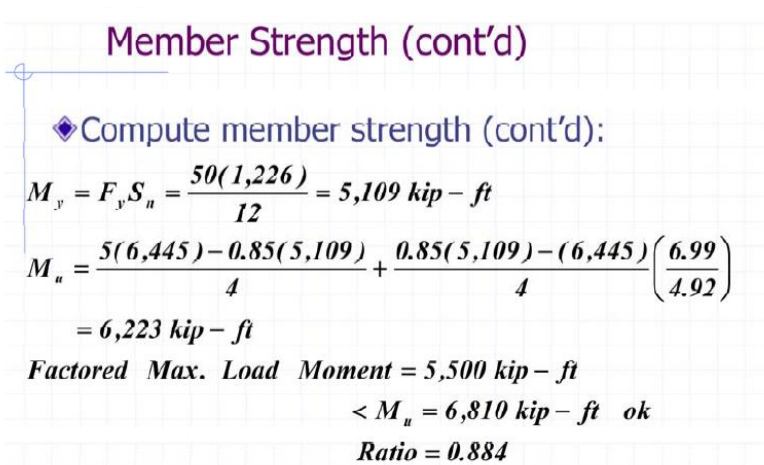
D' is actually the depth of the neutral axis for which the formation of M_p is guaranteed by AASHTO. If the depth of the plastic neutral axis is less than or equal to D', the section will be able to develop its plastic moment capacity. However, if the depth of the plastic neutral axis is greater than D', then the moment capacity will not quite get to M_p and will be limited to a value that will be discussed later. Now, the value of D' turns out to be 4.92" and D_p is 6.99" which is greater than D'. D_p/D' is 1.42 which is still less than 5 and therefore a ductile failure is assured. The impact of this ratio on section strength will be discussed later.



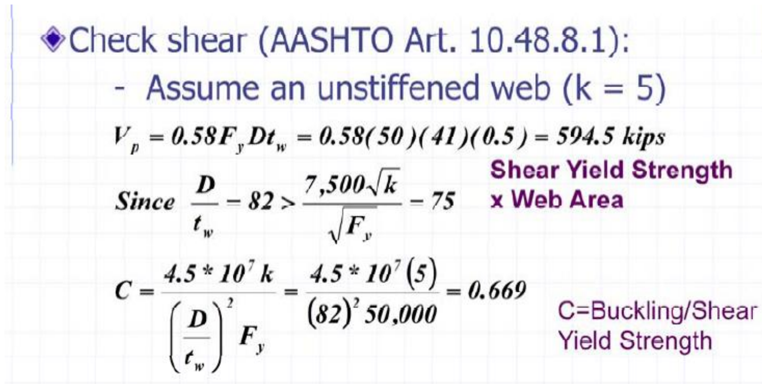
Since the location of the neutral axis has been located, we must check the web slenderness. To prevent the local buckling, this is the equation given by AASHTO. D_{cp} is the depth of web in compression at M_p. In this case, the plastic neutral axis lies in the slab and web is in tension. Therefore, this requirement does not need to be checked.



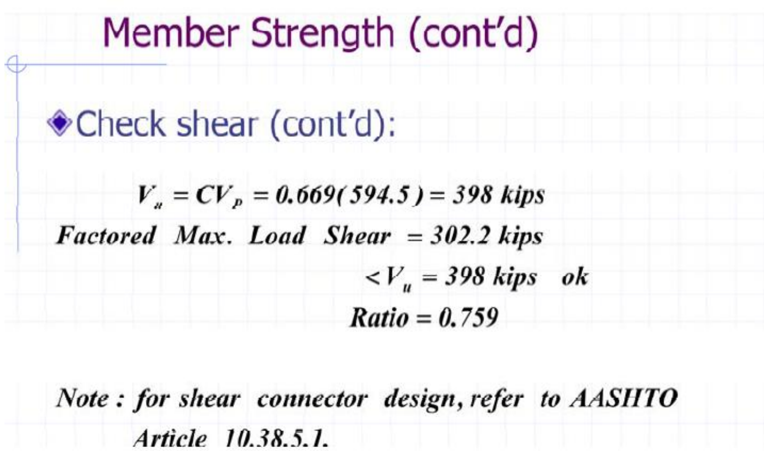
Getting back to computing the member strength. The depth of the plastic neutral axis as we calculated is between D' and 5D'. D' corresponds to the plastic depth where M_p is guaranteed. 5D' corresponds to moment capacity of $0.85M_y$. Since our value lies between these two, linear interpellation is used to calculate the ultimate moment capacity. M_y equals to F_y multiplied by Sn.



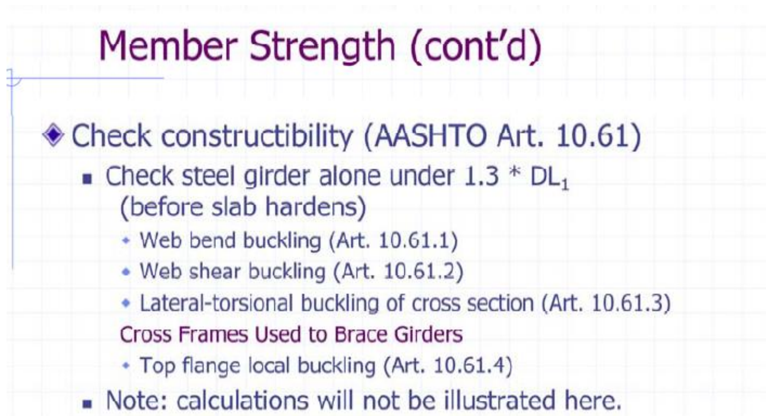
Now we can calculate the moment capacity of the section we are designing here. First we calculate M_y and then M_u. Since the depth of the plastic neutral axis is greater than D', Mu turns out to be 6,223 kip-ft, which is approximately 96 percent of M_p. So the section cannot quite get up to M_p because the depth of the neutral axis is greater than D'. The calculated moment capacity is 6,223 kip-ft, which is still greater than the factored load moment of 5,500 kip-ft. The ratio of flexural demand to flexural strength is 0.884.



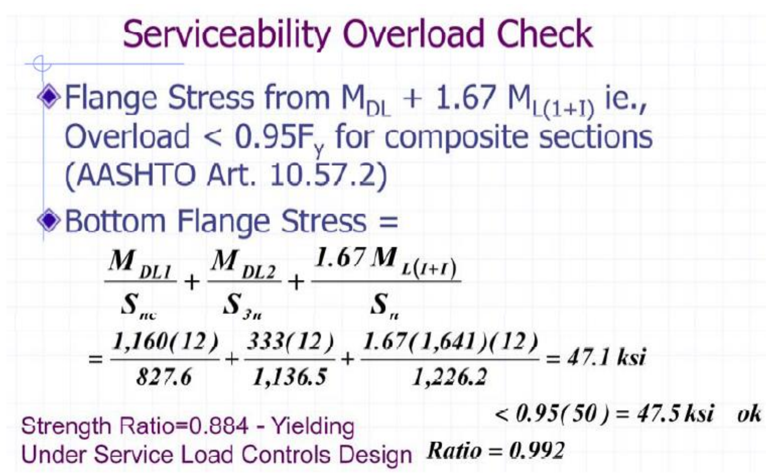
The next is to check for shear. In order to check for shear, we assume an unstiffened web which implies that K = 5. The plastic shear capacity can be calculated as $0.58F_y$ multiplied by depth of the web and multiplied by the thickness of the web. $0.58F_y$ is the shear yield stress derived from the von Mises Criteria. V_p is calculated for this beam to be 594.5 kips. For a D/t_w ratio equal to 82, the ratio of the shear buckling to shear yield strength is computed. We find that shear buckling will govern the strength of the web. It will not develop its yield strength, instead it will undergo shear buckling. Based on the formula for calculating C given by AASHTO specifications, this value turns out to be 0.669. This means that the web will buckle at approximately 67% of the shear yield stress capacity.



V_u is the strength of the beam in shear that is equal to C multiplied by V_p. Vu turns out to be 398 kip and is greater than the factored load shear which is 302.2 kips. The ratio is 0.759. One of the things that we will not go over in this course is the shear connector design. Please refer to AASHTO Article 10.38.5.1.

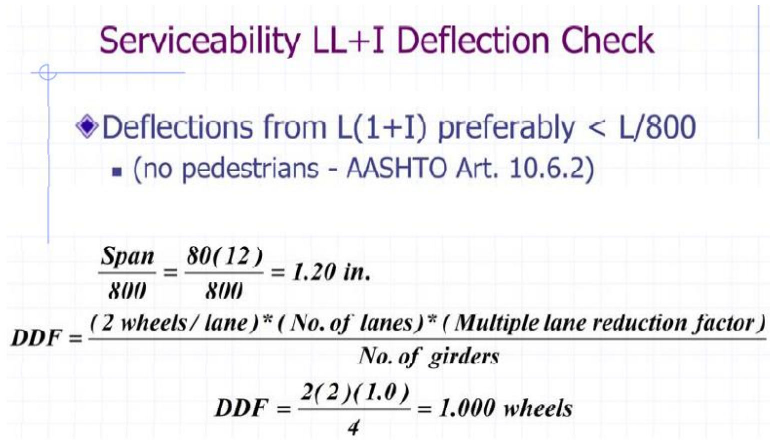


Again the constructability load calculation will not be illustrated here. However, the constructability loads are checked for $1.3DL_1$. DL_1 is the non-composite dead load that is the dead load before the concrete deck hardens. The checks that need to be made are web buckling under flexural, web buckling under shear, lateral-torsional buckling of the cross section where the cross frames are used to brace the girders and compression flange local buckling. The calculations will not be illustrated here, but the final numbers are given in the end of this presentation.

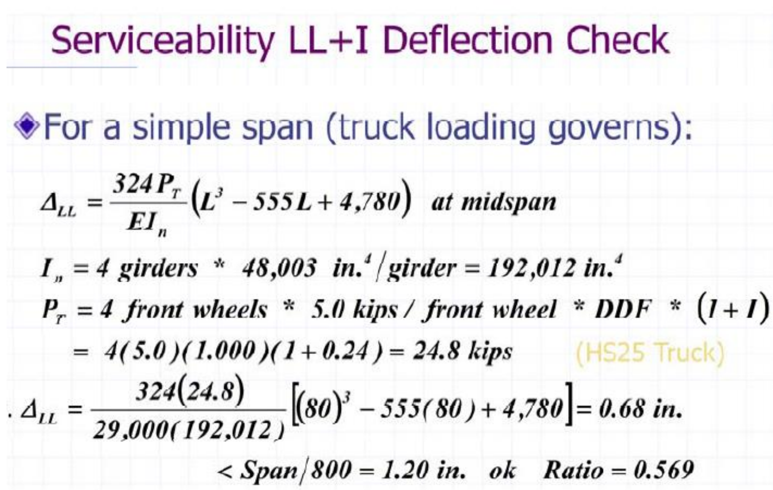


We have done the strength check. We have calculated moment capacity and shear capacity and compared them with the required strength. Now we move on to the serviceability checks, the first one being overload. As you remember, there are three serviceability checks we need to do, overload, deflections, and fatigue. The overload check is performed for the bottom flange. The bottom flange stress from the overload must be less than $0.95F_y$ because this is a composite section. The bottom flange stress can be computed using the above expression. The stress is computed as 47.1 ksi which is less than $0.95F_y$ or 47.5 ksi.

The ratio is 0.992 which is very close to 1. We have calculated the strength ratio which is 0.884. Now we know that the yielding under the service load controls the design because the overload ratio is 0.992. This is fairly common for steel bridges, the overload check frequently controls a design, not strength under factored loads.

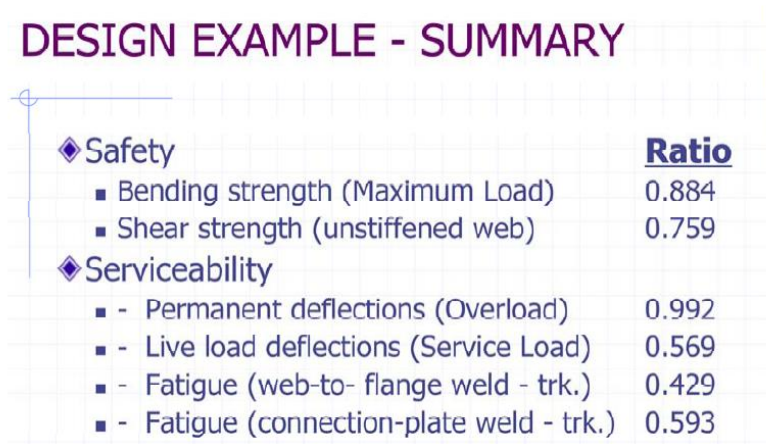


The next serviceability check corresponds to the deflections caused by the live load plus the impact. This is an unfactored live load or service load. For service live load, the deflection must be less than the span over 800, which turns out to be 1.2”. As previously mentioned, the deflection distribution factor can be calculated by the above expression, which yields a value of 1.0 for this design.



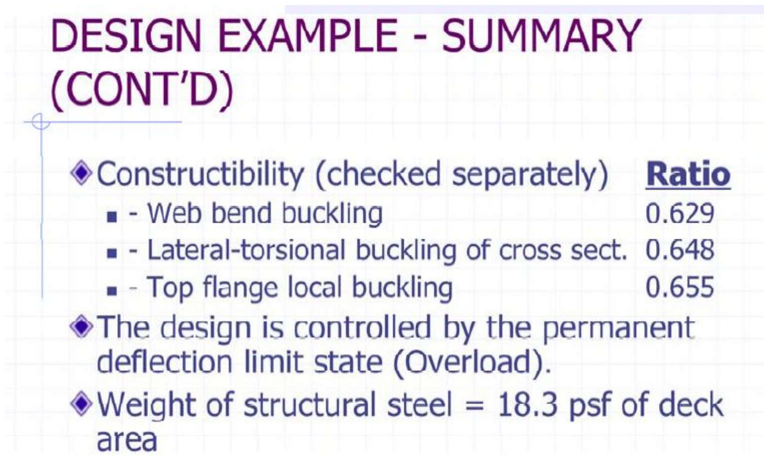
The deflections can be estimated for a simple span using the above expression. I_n is the moment of inertia of the composite section for all the four girders combined. That is 4 multiplied by the moment of inertia of each individual girder. P_T in this expression is the front wheel load. In this case, this is 5 kips because HS25 truck is being used for the design. The live load deflection is calculated to be 0.68" which is less than 1.2" with a ratio of

0.569. So, the second serviceability check is satisfied. The live load deflections is less than span divided by 800.



Now we can summarize the design example. We were illustrating the AASHTO Load Factor design principles for a short span steel bridge. The span was chose to be 90 ft and the design was composite with a concrete deck. Calculations for strength and serviceability limits were prepared. From the strength stand point as far as the bending strength is concerned, the ratio of applied force to capacity was 0.884 and for shear the ratio was 0.759. From the serviceability stand point, three checks were made. One was overload, the second

was deflection and the third was fatigue. For the overload check, the ratio was 0.992. This easily governs the design of the beam for this example. For live load deflections, the ratio turned out to be 0.569. As far as fatigue is concerned, for truck loading and for the web-to- flange weld, the ratio turned out to be 0.429. And for the connection-plate weld under truck loading, the ratio was 0.593.



Constructability, which was not looked into in details here, was checked separately. The ratios for web bend buckling, lateral-torsional buckling of the cross section and local buckling of the top flange were found to be 0.629, 0.648 and 0.655 respectively. As a summary, the design was controlled by the permanent deflection limit state corresponding to an overload truck passing over the bridge. We found that the design weight of the structural steel was 18.3 psf of the deck area, very similar to our estimated weight. This completes the design example, in which the AASHTO load factor design method was utilized to check strength and serviceability of a short span steel bridge.

13.3. GIRDER DESIGN

The AASHTO HS20-44 specifies a 4 kip load for each front wheel, a 16 kip load for each center wheel, and a 16 kips load for each rear wheel. The spacing between the front and center wheels is 14 feet and 14 to 30 feet between center and rear wheels. Analysis using the HS20-44 truck and the AASHTO specifications yielded the following design requirements:

Maximum Factored Bending Moment = 3581.7 Kip-Ft

Maximum Factored Shear = 219.8 Kip-Ft

The section designed for the above load conditions is believed to be an economical configuration given the constraints and objectives of this project. The accuracy of the final design was checked using a CSI bridge software.

13.4. WEB DESIGN

The web is designed to carry all external shear forces. The D/t_w ratio of the web is calculated to be 144. This is less than 162 which is the requirement for no bend buckling to occur (see AASHTO Table 10.48.2.1A). The constant C in AASHTO Eq.10-115, which is a non-linear function of the buckling coefficient, k, is equal to the buckling shear stress divided by the shear yield stress. The D/t_w ratio of 144 falls exactly between the inelastic and elastic buckling curves. Because this ratio is the point in Figure 3.8 where the buckling curve changes from inelastic to elastic, one may argue whether the web will fail inelastically or not. However, AASHTO states in Eq. 10-115 that a D/t_w ratio of 144 will fail inelastically.

Using the equations, the end panel shear capacity is calculated to be 338 kips which is greater than the maximum applied shear (219.8 kips). The end panel has an excess shear capacity of 118.2 kips. This excess shear capacity is a direct result of the relatively small spacing of the first two stiffeners on each end, termed "transition stiffeners". These four stiffeners (two on each end), were spaced accordingly (39.5 inches) to allow for uniform spacing of the intermediate stiffeners.

The intermediate stiffeners are uniformly spaced at 67.2 inches. This translates into a d_0/D ratio of 1.24 inches. Because this ratio is less than 3 and $(260/(D/t_w))^2$, the shear capacity is determined by including post-buckling resistance due to tension-field action using AASHTO Eq. 10-113:

14.CALCULATIONS AND CHECKING SECTIONS

Calculating b_{eff} for both interior and exterior girder:

b_{eff}	¼ Span= 22.5 ft	
	Spacing=12.67 ft	
	12ts=8.5 ft	Control

15.COST ANALYSIS

Based on given data and calculated the section are and length of each girder, cost analysis has been done.

Girder	***Rolled shape***	***plate Girder***	***Folded plate***
Fy(ksi)	50	50	70
Length(in)	1080	1080	720
Section Area(in^2)	98	67	51
Weight(lb)	30799	21057	10686
70 ksi increase cost	0	0	0.15
Cost /lb	0.5	1	1
Ship to site(1.5/lb)	1.5	1.5	1.5
Hot Galvanizing(/lb)	0	0	0.2
Total cost ($)- Each girder	61599	52642	**30454**

16.CONCLUSION

Based on the cost analysis for these three types of girders the best one is folded plate girder and then steel plate girder.

This is completely relying on what assumption considered for cost analysis. The assumption in this report are not up to date and are only for completing the cost analysis section.

17.REFERENCES

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