Structural Analysis

Methodology

In order to determine the safe load carrying capacity of the Bonner Bridge, a structural analysis was performed of typical superstructure and substructure components of the bridge. The structural analysis of the Bridge includes dead, live, pedestrian, wind, and centrifugal loads. Tidal flow, storm surge, and ship impact were not the scope of this assessment and were not included in the analyses.

The original bridge was designed for an H15 design truck. The crutch bents placed in 1980 were designed for HS-15-44 design truck. The design truck for the crutch bents placed in 1978 is unknown; however, it is reasonable to assume based on the date of the plans, the crutch bents were designed for the HS-15-44 design truck. The design truck for the 1986 replacement bents is unknown. Given that the plans are dated December 1985, it is reasonable to assume that the design truck was HS-20. The approach span bents and crutch bents had been analyzed using the current HS-20 design truck.

The original design was performed using the “Working Stress” method based on service load conditions. Current NCDOT policy dictates use of the practices use “Load Factor” design method based on ultimate strength of the materials. This assessment used the Load Factor method as outlined in *AASHTO Standard Specification for Highway Bridges* to determine the capacities of the bridge elements in their original design condition with current HS20-44 loading and in their current deteriorated state with concrete strengths, determined from testing. The specific material strength assumptions are noted in the “Approach Bent Assessment” and the “High Level Bent Assessment” sections of this report.

NCDOT had determined that this bridge is scour critical as evidenced by the numerous crutch bents installed in the past. Critical scour elevations have been established and an action plan is in place if the mud line should fall below the scour critical elevations. Therefore, evaluation of scour was not included in the scope of this assessment.

The analysis was performed using STAAD, MDX, CONSPAN, PCACOL, RC-PIER, MathCAD, and Excel spreadsheets. The high level span bents were divided into six different models to represent the various configurations of the bents. Each model was developed to represent “worst case” geometry and loadings for similar bent configurations. The approach span bents and crutch bents were divided into ten different models to represent the various configurations of the bents and crutch bents. The same methodology was applied to the approach bents as was applied to the high level bents. The complete structural analysis of the bents is included in the “Approach Bents Assessment” and “High Level Bent Assessment” and in Appendices F and H.

Approach Span Bent Structural Analysis per Plans

A structural analysis of the bents in the approach spans (Bents 1 – 128B and Bents 167 – 204) was performed by Ko. Ten different models were created to group bents with similar characteristics. These models were created in STAAD 2005, a structural analysis program. Table 11 provides a summary list of structural models and the bents they represent.

Table 11 – Structural models for the approach span bents and bents represented

Model	Bents
Model 1	1, 2, 3, 5, 6, 7, 9, 10, 11, 13, 14, 15, 17, 18, 19, 201, 202, 203
Model 2	4, 8, 12, 16, 20
Model 3	21, 22, 23, 25, 26, 27, 29, 30, 31, 33, 34, 35, 37, 38, 39, 41, 42, 43, 45, 46, 47, 49, 50, 51, 53, ,54, 55, 57, 58, 59, 61, 62, 63, 65, 66, 67, 69, 70, 71, 73, 74, 75, 77, 78, 79, 81, 82, 83, 85, 86, 87, 89, 90, 91, 93, 94, 95, 97, 98, 99, 101, 102, 103, 105, 106, 107, 109, 110, 111, 113, 114, 115, 117, 118, 119, 121, 122, 123
Model 4	24, 28, 32, 36, 40, 44, 48, 52, 56, 60, 64, 68, 72, 76, 80, 84, 88, 92, 96, 100, 104, 108, 112, 116, 120
Model 5	124, 125, 126, 127, 128, 128A, 128B
Model 6	167, 168, 169, 170, 171, 172
Model 7	175, 179, 183
Model 8	173, 174, 176, 177, 178, 180, 181, 182
Model 9	184, 185, 186
Model 10	187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200

The dimensions used for modeling were taken from the “as built” plans. The original 1962 structure consisted of 4 different approach bent types, which are represented by models 1 through 4. Model 5 represents the bents replaced in 1986. Models 6 and 10 represent two types of crutch bents using subcaps supported by two pairs of 54” AASHTO girders on each side of the original bent cap. Models 7 through 9 represent the various types of crutch bents consisting of subcaps placed longitudinally with the bridge and supported by 20” square piles at each end. The purpose of these crutch bents is to provide a solid foundation for the bent cap where scour depth had reduced the stability of the original piles.

In addition to the crutch bents listed above, Bents 108 through 123, and Bents 187 through 200, have steel H-pile crutch bents supporting the original 1962 bent. There were no plans available for these crutch bents. Since sand has filled in around Bents 108 through 123, the original bents no longer require these steel crutch bents to carry the applied loads. Therefore, the crutch bents were not included in these models.

The compressive testing performed by WJE was used to obtain concrete strengths for use in the structural models. A 4,990 psi compressive concrete strength was selected for Bents 1 through 123 and Bents 201 through 203. (Models 1-4) Since the replacement Bents 123 through 128B are relatively new, no concrete samples were taken. The design concrete strength shown in the “as built” plans was 3,000 psi and was used for this model. (Model 5) A 5,600 psi compressive concrete strength was used for Bents 167 through 200. (Models 6-10) No concrete samples were taken from the 22” octagonal piles, 24” square piles, 20” square piles, 66” cylinder piles, subcaps, struts, 54” AASHTO girders, or pile caps. The design strength shown in the “as built” plans was used for each of these structural members. The 40,000 psi yield strength of the reinforcing steel taken from the “as built” plans was used for all models. For more details on material testing and results, see the “Summary of Concrete Testing” and appendix D.

Analysis of Approach Span Bents per Plans

Model 1 consists of a 42’-0” long bent cap supported by seven 22” octagonal piles evenly spaced at 5’-10”. The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. When constructed in

1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 11" centers between the piles. This model supports the bridge superstructure and a pedestrian walkway on both sides of the bridge.

Model 2 consists of a 42'-0" long bent cap supported by eight 22" octagonal piles evenly spaced at 7'-0". The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. There are a set of brace piles located at 10'-6" from each end of the cap. These piles are battered perpendicular to the bent cap at a 1½ to 12 pitch. These battered piles take the longitudinal loads placed on the structure. When constructed in 1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 13" centers between the piles. This model supports the bridge superstructure and a pedestrian walkway on both sides of the bridge.

Model 3 consists of a 28'-6" long bent cap supported by six 22" octagonal piles evenly spaced at 4'-9". The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. When constructed in 1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 10" centers between the piles. This model supports the bridge superstructure.

Bents 109-111, 113-115, 117-119, and 121-123 had H-steel pile crutch bents supporting the original 1962 bent cap. Since sand has filled in around these crutch bents, they are no longer needed and were not included in the model.

Model 4 consists of a 28'-6" long bent cap supported by seven 22" octagonal piles evenly spaced at 5'-11½". The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. There is a set of brace piles located at 8'-3½" from each end of the cap. These piles are battered perpendicular to the bent cap at a 1½ to 12 pitch. These battered piles support the longitudinal loads placed on the structure. When constructed in 1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 13" centers between the piles. This model supports the bridge superstructure.

Bents 108, 112, 116, and 120 had an H-steel pile crutch bent supporting the original 1962 bent cap. Since sand has filled in around these crutch bents, they are no longer needed and were not included in this model.

Analysis Results: Models 1-4:

The analysis results indicate that the bent caps have the capacity to carry the loads described above. Depending on the load combination group, some piles exceed the 50 ton allowable end bearing capacity. While these design end bearing capacities are exceeded, the structure should still adequately carry the loads described above. The justification for this opinion is described in length under the high level bent report. A summary of the results is shown in Tables 13, 14, and 16 below. Detailed calculations are included in appendix I.

Model 5 consists of the bents replaced in 1986. This model consists of a 38'-6" long bent cap supported by six 24" square piles evenly spaced at 6'-4". The exterior piles are battered parallel to the bent cap at a

1½ to 12 pitch. Piles 2 and 4 are battered perpendicular to the bent cap at a 1½ to 12 pitch to the north, while piles 3 and 5 were battered at the same pitch to the south. These battered piles resist the longitudinal loads placed on the structure.

The end bearing capacity of these piles is unknown. Considering the date of construction and length of pile driven into the sand, it is reasonable to assume these piles were driven to at least 50 tons capacity. The bent cap is a 4'-4" by 3'-0" deep reinforced concrete section with 5 #11 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #5 closed stirrups spaced at 11" centers between the piles. This model supports the cored slab superstructure.

Analysis Results: Model 5

The analysis results show that the bent cap had the capacity to carry the loads described above. A summary of the results is shown in Tables 13, 14, and 16 below. Detailed calculations are included in appendix I.

Model 6 consists of a 28'-6" long bent cap supported by seven 22" octagonal piles evenly spaced at 5'-11½". The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. There are a set of brace piles located at 8'-3½" from each end of the cap. These piles are battered perpendicular to the bent cap at a 1½ to 12 pitch. When constructed in 1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 13" centers between the piles.

Due to concerns about the stability of the bridge, in 1980 crutch bents were added to Bents 167 through 172 to support the bent cap. The crutch bent consists of subcaps supported by two pairs of 54" AASHTO girders on each side and parallel to the original bent cap. The Girders transfers the load to pile caps located at the east and west end of the bent just outside of the superstructure. Each pile cap is supported by two 66" diameter concrete cylinder piles. While the 22" octagonal piles were left in place after the crutch bents were constructed, it is assumed that they are no longer providing any support to the structure and are, therefore, not included in the model. The plan concrete strength of 3,000 psi was used to evaluate the capacity of the subcap, struts and pile caps. In the "as built" plans, a 28-day compressive strength of 6,000 psi was specified for the 54" prestressed concrete beams and 7,000 psi was specified for the 66" diameter cylinder piles.

Analysis Results: Model 6

The results of the analysis indicated that the bent cap, subcap, struts, 54" prestressed concrete girders, pile caps, and 66" diameter cylinder piles had the capacity to carry the loads described above. A summary of the results is shown in Tables 13 through 16 below. Detailed calculations are included in appendix I.

Model 7 consists of a 28'-6" long bent cap supported by seven 22" octagonal piles evenly spaced at 5'-11½". The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. There are a set of brace piles located at 8'-3½" from each end of the cap. These piles are battered perpendicular to the bent cap at a 1½ to 12 pitch. When constructed in 1962, the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3'-0" by 2'-6" deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 13" centers between the piles.

Due to concerns about the stability of the bridge, in 1978 crutch bents were added to Bents 175, 179, and 183 to support the bent cap. The crutch bents consist of subcaps placed longitudinally with the bridge which are supported by 20” square piles at each end. While the 22” octagonal piles were left in place after the crutch bents were constructed, it is assumed that they are no longer providing any support to the structure and are, therefore, not included in the model. The plan concrete strength of 3,000 psi was used to evaluate the capacity of the subcaps and struts.

Model 8 consists of a 28’-6” long bent cap supported by six 22” octagonal piles evenly spaced at 4’-9” constructed in 1962. The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. When constructed in 1962 the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3’-0” by 2’-6” deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 10” centers between the piles. This model supported the bridge superstructure.

Due to concerns about the stability of the bridge, in 1978 crutch bents were added to Bents 173, 174, 176-178 and 180-182 to support the bent cap. The crutch bents consist of subcaps placed longitudinally with the bridge which are supported by 20” concrete piles at each end. While the 22” octagonal piles were left in place after the crutch bents were constructed, it is assumed that they no longer provide any support to the structure and are, therefore, not included in the model. The plan concrete strength of 3,000 psi was used to evaluate the capacity of the subcaps and struts.

Model 9 consists of a 42’-0” long bent cap supported by seven 22” octagonal piles evenly spaced at 5’-10” constructed in 1962. The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. When constructed in 1962 the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3’-0” by 2’-6” deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 11” centers between the piles. This model supported the bridge superstructure and a pedestrian walkway on both sides of the bridge.

Due to concerns about the stability of the bridge, in 1978 crutch bents were added to bents 184, 185 and 186 to support the bent cap. The crutch bents consisted of subcaps placed longitudinally with the bridge and were supported by 20” square piles at each end. For model 9, there was also a subcap placed parallel to and beneath the original bent cap to provide support. While the 22” octagonal piles were left in place after the crutch bents were constructed, it is assumed that they no longer provide any support to the structure and are therefore, not included in the model. The plan concrete strength of 3,000 psi was used to evaluate the capacity of the subcaps and struts.

Analysis Results: Models 7-9

The analysis results show that the bent cap, subcaps, struts, and 20” square piles have the capacity to carry the loads described above. A summary of the results is shown in Tables 13 through 16 below. Detailed calculations are included in appendix I.

Model 10 consists of a 42’-0” long bent cap supported by eight 22” octagonal piles evenly spaced at 7’-0” constructed in 1962. The exterior piles are battered parallel to the bent cap at a 1½ to 12 pitch. There were a set of brace piles located at 10’-6” from each end of the cap. These piles are battered perpendicular to the bent cap at a 1½ to 12 pitch. These battered piles resist the longitudinal loads placed on the structure. When constructed in 1962 the piles were driven to a 50 ton allowable end bearing capacity. The bent cap is a 3’-0” by 2’-6” deep reinforced concrete section with 5 #10 bars in the top and 4 #11 bars in the bottom. The shear reinforcement is single #4 closed stirrups spaced at 13” centers between the piles. This model supports the bridge superstructure and a pedestrian walkway on both sides of the bridge.

Bents 187 through 200 have steel H-pile crutch bents however due to their deteriorated condition they were considered ineffective and were not included in the model. In 1980 crutch bents were added to Bents 187 through 200 to support the bent cap. These crutch bent consist of subcaps supported by two pairs of 54" AASHTO girders on each side and parallel to the original bent cap. The girders transfer the load to pile caps located at the east and west end of the bent just outside of the superstructure. Each pile cap is supported by two 66" diameter concrete cylinder piles. While the 22" octagonal piles were left in place after the crutch bents were constructed, it is assumed that they are no longer providing any support to the structure and were; therefore, not included in the model. The plan concrete strength of 3,000 psi was used to evaluate the capacity of the subcaps, struts and pile caps. In the "as built plans" a 28-day compressive strength of 6,000 psi was specified for the 54" prestressed concrete beams and a strength of 7,000 psi was specified for the 66" diameter cylinder piles.

Analysis Results: Models 10

The results of the analysis indicated that the subcaps, struts, 54" prestressed concrete girder, pile caps, and 66" diameter cylinder piles have the capacity to carry the loads described above. The analysis results also show that for Bents 187 through 200 there was a high shear in the bent cap located at the exterior girders. For Bents 187 through 195 and Bents 199 and 200 there was a 22" octagonal pile located near this point of high shear. Since there are no signs of shear cracking in the bent cap, it is reasonable to assume that these octagonal piles are providing enough support to the bent cap to help carry the shear load. If the mud line were to recede or shear cracks were noticed during future inspections, a repair will be needed. For Bents 196 to 198, the octagonal piles tip elevation is near the current mud line. Although shear cracking in the bent cap was not evident in these three bents, it is unlikely that the octagonal piles are providing any additional support to the substructure.

For bents 196 to 198 it is recommend that an additional concrete subcap be placed beside the existing exterior subcaps at the location of the exterior girder. While this repair is not needed immediately, it is recommended that it be done with the NBIS repairs at these three bents. A summary of the results is shown in Tables 13 through 16 below. A sketch of the proposed repair and Engineer's opinion of construction cost is located in the proposed repair section of this report. Detailed calculations are included in appendix I.

Analysis of Approach Span Bents per Inspection Findings

The inspection findings showed some delaminations and spalling with some exposed reinforcing steel. These areas were minor and were not incorporated into the modeling of the structure. If the recommended repairs in the NBIS reports are made and are of good quality, the structure should perform as analyzed.

The results of the analysis of Models 1 through 9 indicate that Bents 1 through 128B and 167 through 186 are adequate to carry loads described above. Even though these bents were found to be adequate, it should be noted that the loads on some of the piles in Models 1 through 4 exceeded the 50 ton allowable load for end bearing in certain load cases. While these end bearing capacities were exceeded, the structure had historically carried the load described above without subsidence as long as the mud line remained above the critical scour elevation. The analysis results for Model 10 indicated that Bents 187 through 200 experienced high shear stress in the bent cap located at the exterior girders (46% overstress). Tables 13 through 16 provide a summary of these results.

Table 13: Flexural Analysis

	Bent Cap					
	Positive Moment			Negative Moment		
	+ Mu	+ ϕ Mn	Ratio	- Mu	+ ϕ Mn	Ratio
Model	kip-ft	kip-ft	%	kip-ft	kip-ft	%
Model 1	430	490	88%	251	499	50%
Model 2	418	471	89%	400	499	80%
Model 3	178	473	38%	302	500	60%
Model 4	169	473	36%	432	500	86%
Model 5	391	563	69%	347	710	49%
Model 6	302	473	64%	323	500	65%
Model 7	128	473	27%	236	500	47%
Model 8	175	473	37%	241	500	48%
Model 9	302	492	61%	97	500	19%
Model 10	460	473	97%	137	500	27%

Note: "Mu" denotes maximum factored shear applied to cap in analysis.
"ϕMn" denotes ultimate strength (capacity) of cap section.
"Ratio" denotes Mu/ϕMn, ratio of load to capacity.

Table 14: Shear Analysis

	Bent Cap		
	Shear Vu	Shear ϕ Vn	Ratio
Model	Kips	Kips	%
Model 1	136	228	60 %
Model 2	207	263	79 %
Model 3	122	244	50 %
Model 4	174	280	62 %
Model 5	115	322	36 %
Model 6	225	228	99 %
Model 7	225	228	99 %
Model 8	225	228	99 %
Model 9	128	150	85 %
Model 10	307	210	146 %

Note: "Vu" denotes maximum factored moment applied to cap in analysis.
"ϕVn" denotes ultimate strength (capacity) of cap section.
"Ratio" denotes Vu/ϕVn, ratio of load to capacity.

Table 15: Flexural and Shear Analysis - Crutch Bents

54" Prestressed Concrete Girder						
	Mu	**ϕ Mn**	**Ratio**	**Shear Vu**	**Shear ϕ Vn**	**Ratio**
Model	**kip-ft**	**kip-ft**	**%**	**Kips**	**Kips**	**%**
Model 6	1602	3639	44 %	195	234	83 %
Model 10	1701	3637	47 %	247	257	96 %
Concrete Pile Cap						
Model 6	193	1354	14 %	43	459	9 %
Model 10	215	1354	16 %	45	459	10 %
Concrete Sub cap I (Longitudinal)						
Model 6	1231	1767	70 %	221	371	60 %
Model 7	1766	2584	68 %	200	315	63 %
Model 8	794	1505	53 %	174	243	72 %
Model 9	965	1309	74 %	211	265	80 %
Model 10	1856	2472	75 %	337	390	86 %
Concrete Sub cap II (Transverse)						
Model 9	810	1165	70 %	140	311	45 %
Concrete Strut						
Model 7	239	358	67 %	42	104	40 %
Model 8	193	358	54 %	36	104	35 %
Model 9	116	358	32 %	30	104	29 %

Note: See tables 3 and 4 for definitions of symbols.

Table 16: Pile End Bearing Capacity

Model	Pile Type	Max Applied Load, Kips	Capacity, Kips	Ratio %
Model 1	22" Octagonal Pile	110	100	110%
Model 2	22" Octagonal Pile	141	100	141%
Model 3	22" Octagonal Pile	102	100	102%
Model 4	22" Octagonal Pile	135	100	135%
Model 5	24" Square Pile	96	100	96%
Model 6	66" Cylinder Pile	248	500	50%
Model 7	20" Square Pile	141	160	88%
Model 8	20" Square Pile	118	140	85%
Model 9	20" Square Pile	132	140	94%
Model 10	66" Cylinder Pile	288	500	58%

Note: "Ratio" denotes ratio of load to capacity.

High Level Bent Structural Analysis

A structural analysis of the bents in the high level spans (Bents 129 through 166) of the Bonner Bridge was performed by Lochner. As in the approach span bent analysis, concrete strength was determined based on the testing performed by WJE. A concrete strength of 4,400 psi was selected as a conservative value from the concrete testing results. The structural assessment used the current HS20-44 design vehicle rather than the H15-44 design vehicle as indicated on the as-built plans.

The effects of tidal flow, storm surge, and ship impact on the structure were not applied as part of this analysis.

In order to identify and analyze an appropriate number of structural models, six typical configurations (model bents) were chosen that best represent the various bent configurations (see Table 17). Each of these configurations was first analyzed based on the original construction plans. Subsequently, the NBIS inspection report results (A&O) and the concrete strength test results (WJE) were incorporated into the models. In each case, the structure capacity of each bridge component was assessed.

Table 17: Representative Models

Model Bent	No. of Struts	Superstructure Type (North)	Superstructure Type (South)	No. of Piles	Bents Represented
Model 129	0	Cored Slab	AASHTO Girder	10	129
Model 160	0	AASHTO Girder	AASHTO Girder	10	160
Model 137	1	AASHTO Girder	AASHTO Girder	12	130-137, 152-159
Model 142	2	AASHTO Girder	AASHTO Girder	15	138-142, 147-151
Model 143	2	AASHTO Girder	Steel I-girder	21	143, 146
Model 145	2	Steel I-girder	Steel I-girder	40	144, 145

The columns in the high level spans were originally detailed without horizontal tie reinforcement and so were modeled without ties. Columns without ties are not permitted in current design practice. The lack of horizontal ties does not adversely affect the column capacities as long as the longitudinal reinforcing steel remains an integral component of the reinforced concrete column. However, a column failure due to lack of bond or confinement of the longitudinal steel could take place were large spalls on the columns to occur or if a significant length of reinforcing steel was exposed during repairs of spalls & delaminations. The engineer's repair recommendations address concerns pertaining to this potential failure.

Analysis of High Level Bents per Plans

The first phase of the analysis involved modeling the bridge as it is shown in the plans. No field observations were incorporated. The purpose of this step was to set a baseline on which to compare further analyses. The first step in the analysis process was to model the various superstructure types. Lochner modeled the AASHTO girders and the steel I-girders. Ko, which modeled the approach span bents and superstructure, provided the model results for the cored slab units. All but the last half span of the cored slabs were supported by the approach span bents. Once the superstructures were modeled, the six different high level bent configurations were modeled. Finally, the capacity of each component was checked.

Computer Model (STAAD)

The superstructures were modeled using the computer programs CONSPAN (AASHTO girders) and MDX (Welded Steel Plate (WSP) girders). Ko provided the cored slab results from its in-house spreadsheets. The dead and live loads from the superstructure analysis were then applied to the bent models. The live loads were calculated using the HS20-44 truck axle loads of 8 kips, 32 kips and 32 kips respectively. See the Figure 30 below.

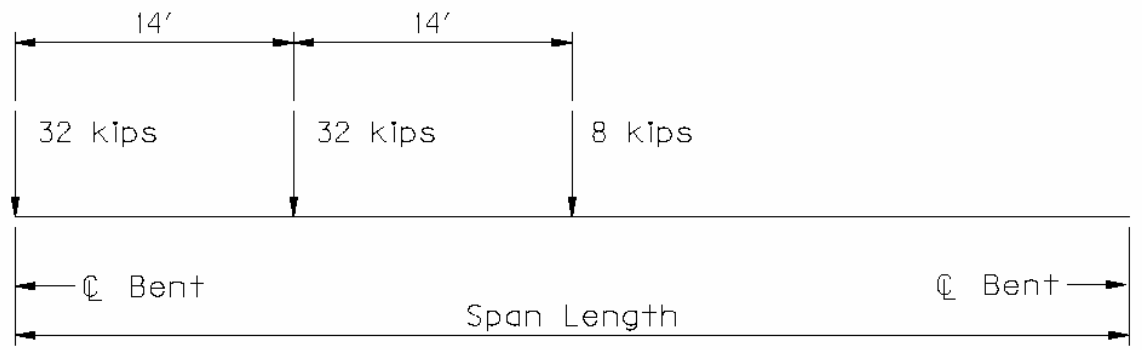

Figure 30. STAAD computer live load model parameters.

The axle loads from each span were resolved into axle load reactions at the bent. The axle load reactions at bents were then divided by two to yield wheel loads. Next, the wheel loads were placed at spacings according to the AASHTO Specifications. The wheel load configurations were placed depending on the desired worst case scenario. For example, for maximum negative moment in the cap, the wheel loads were placed nearest to the curb. For maximum positive moment in the cap, the wheel loads were centered on the roadway. For each case, the live load reactions were determined using a simple beam model and placed on the bent models. See the sketches in Appendix F for each case.

In order to identify and analyze an appropriate number of bent models, one model of each type of high level bent was chosen. The configurations varied by bent height (number of struts), number of piles and superstructure type. As a further refinement of bent selection, the bent numbers above were selected since they represent the tallest of each bent type and, therefore, had the highest loads applied of the bents represented by that model. Drawings illustrating the different types of models can be found in Appendix F.

The first step in analyzing the high level bents was to model them using the structural analysis program STAAD. Next the superstructure loads and hand calculated wind loads were applied to the model. Finally moments, shears, and axial forces generated in STAAD were used to check the capacity of each component of the bents.

The moments and shears for the bent cap and struts were input into an in-house concrete beam design spreadsheet to check capacity. The axial forces and moments for the columns were input into an in-house spreadsheet that generates final axial forces and moments, including slenderness effects. Those axial forces and moments were then input into the PCACOL program to check the capacity of the columns. The columns were modeled without ties as shown in the plans. The last components analyzed were the pile cap and piles. The axial forces and moments at the top of the pile cap were used to analyze the pile cap and piles in RC-PIER. Determination of the lateral capacities of the piles was beyond the scope of this assessment. Therefore, axial loads in the piles were only compared to the minimum bearing capacities given in the original plans.

Analysis Results: High Level Bents per Plans

Each component of each model was checked for its capacity versus the applied loads. The capacity of the caps, columns, struts, and pile caps were adequate to support the applied loads. The one exception was shear capacity of the cap in Bent 129 which supported the cored slab units and AASHTO girders. The shear capacity was roughly 13% lower than that required. All other models had sufficient shear capacity based on the plan dimensions. For the piles, axial loads were compared with minimum bearing capacity (50 tons) given in the original plans. The computed axial loads for the piles for the Bents 129, 137, and 160 ranged from 20% to 35% greater than the 50 ton capacity. For Bents 142, 143, and 145, the computed axial loads for the piles ranged from 2% to 10% greater than the 50 ton capacity.

The reason that the calculated pile loads are greater than the original 50 ton capacity is the heavier HS20-44 live loading used in the analysis. The total axle loads for the HS20-44 truck (72,000 lbs) are more than twice that of the H15-44 truck (30,000 lbs) which was used in the original design. See the Figure 31 below for the truck configurations and loads. For discussion of the shear overstress and pile loads exceeding design capacity, see "Analysis Results: High Level Bents per Inspection Findings" below.

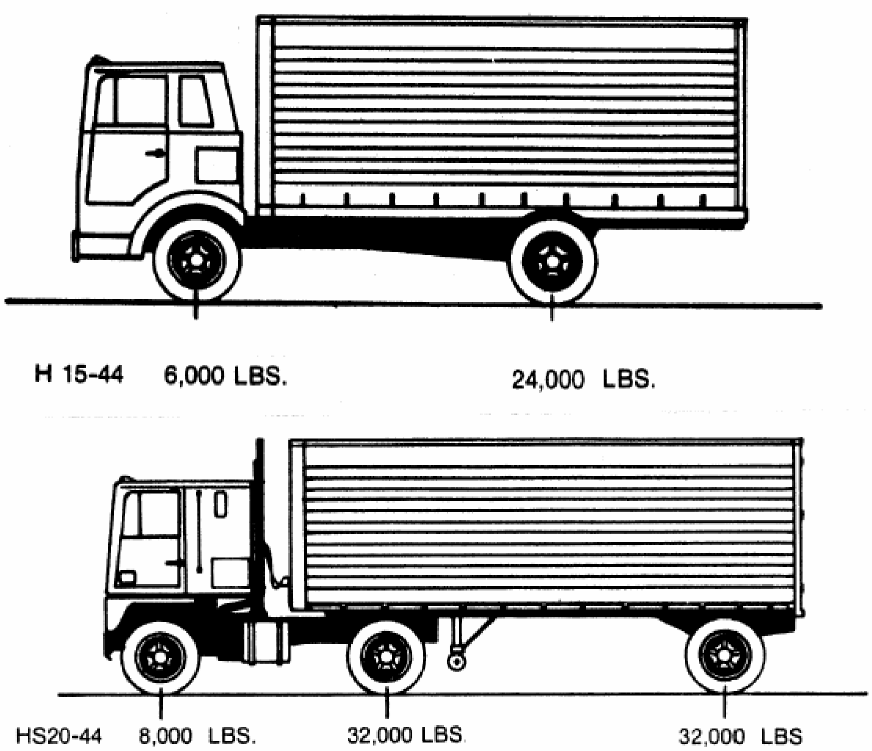

Figure 31. AASHTO standard truck loading.

Analysis of High Level Bents per Inspection Findings

An analysis of the high level bents per inspection findings was performed to give a true capacity of the components under their current conditions. The modeling and analysis procedure was similar to that described above for the analysis per plans.

Model Revised per Inspection Findings

The modeling procedure follows that previously described. Based on the visual field inspection report provided by A&O and discussions with WJE and A&O, the overall dimensions for each component were reduced by one half of the clear cover on each face.

The exception to this modification was the Bent 137 model, which was used to illustrate the conditions for Bent 135. Bent 135 was a single strut high level bent supporting AASHTO girders. Since Bent 135 was completely covered in shotcrete that suffered from heavy delamination, the bent model incorporated total clear cover loss. However, since the deterioration of the pile caps was limited to the edges, no section reduction was modeled for these members.

Based on the concrete compressive test data from WJE, the project team determined that a concrete strength of 4,400 psi was appropriate. While several other tests resulted in higher concrete strengths, the lower value was conservatively used (see Concrete Field Testing Results). The one exception was the Bent 145 model. The compressive test for the core taken from the Bent 145 pile cap resulted in a concrete strength of 3,800 psi. Since all other cores yielded strengths greater than 4,400 psi, the value of 3,800 psi was used only for the pile cap of the Bent 145 model.

For the piles, no section reduction was applied. Instead, piles exhibiting major section losses were removed from the model. Based on the field observations, the pile removal was applied to the models for Bents 129, 143, 145 and 160. The increased axial loads experienced when piles were removed were compared to those from the model with all piles intact.

Analysis Results: High Level Bents per Inspection Findings

The results of the structural analysis based on field observations were generally favorable overall (see Tables 19-23). The results indicate that the capacities of the bent caps, struts, columns, and pile caps are adequate to handle the loads modeled. There were two exceptions to the overall favorable results of these analyses: One is an inadequate shear capacity in the bent cap of Bent 129. However, the shear overstress is less than 4% and can be considered negligible for the purposes of this assessment. The second exception is that the calculated axial loads for the piles are greater than the pile capacity listed in the original plans.

Table 19: Flexural Analysis

Model Bent	Bent Cap					
	Positive Moment			Negative Moment		
	+ Mu	+ ϕMn	Ratio	- Mu	- ϕMn	Ratio
	kip-ft	kip-ft	%	kip-ft	kip-ft	%
Model 129	502	853	59%	1060	1177	90%
Model 160	637	853	75%	796	1177	68%
Model 137	634	818	78%	793	1128	70%
Model 142	623	853	73%	806	1177	68%
Model 143	680	1365	50%	1061	1365	78%
Model 145	934	2709	34%	1555	2709	57%

Note: "Mu" denotes maximum factored moment applied to cap in analysis.
"ϕMn" denotes ultimate strength (capacity) of cap section.
"Ratio" denotes Mu/ϕMn, ratio of load to capacity.

Table 20: Flexural Analysis

Model Bent	Bent Strut					
	Positive Moment			Negative Moment		
	+ Mu	+ ϕMn	Ratio	- Mu	- ϕMn	Ratio
	kip-ft	kip-ft	%	kip-ft	kip-ft	%
Model 129	-	-	-	-	-	-
Model 160	-	-	-	-	-	-
Model 137	43	670	6%	117	670	17%
Model 142	42	706	6%	112	706	16%
Model 143	55	1190	5%	139	1190	12%
Model 145	38	1486	3%	142	1486	10%

Note: "Mu" denotes maximum factored moment applied to cap in analysis.
"ϕMn" denotes ultimate strength (capacity) of cap section.
"Ratio" denotes Mu/ϕMn, ratio of load to capacity.

Table 21: Shear Analysis

Model Bent	Bent Cap			Bent Strut		
	Required Stirrup Spacing	Provided Stirrup Spacing	Ratio	Required Stirrup Spacing	Provided Stirrup Spacing	Ratio
	in	in	%	in	in	%
Model 129	5.8	6.0	103%	-	-	-
Model 160	6.7	6.0	90%	-	-	-
Model 137	6.1	6.0	98%	24	13	54%
Model 142	6.7	6.0	90%	24	14	58%
Model 143	6.2	5.0	81%	24	14	59%
Model 145	5.5	5.0	91%	22	15	68%

Note: "Ratio" denotes ratio of load to capacity.

Table 22: Column Analysis

	Model Bent					
	Model 129	**Model 160**	**Model 137**	**Model 142**	**Model 143**	**Model 145**
Controlling Mu/ϕMn ratio (%)	68%	66%	87%	96%	88%	76%

Table 23: Pile End Bearing Capacity

Model Bent	**Pile Type**	**Load Kips**	**Capacity, Kips**	**Ratio %**
Model 129	22" Octagonal	150	100	150%
Model 160	22" Octagonal	134	100	134%
Model 137	22" Octagonal	120	100	120%
Model 142	22" Octagonal	109	100	109%
Model 143	22" Octagonal	131	100	131%
Model 145	22" Octagonal	91	100	91%

Note: “Ratio” denotes ratio of load to capacity.

The 50 ton pile capacity shown in the plans is the original design capacity based on non-factored dead and live loads. At the time of the bridge’s construction, the piles would have typically been driven to provide for the planned capacity with a factor of safety of three (3). According to Chris Kreider, NCDOT Geotechnical Engineer, the pile driving equations used at the time of the original construction differ from those that are currently in use at the time of this assessment. The factor of safety would have fluctuated between two (2) and six (6). Based on this consideration, the piles were likely driven to between 100 and 300 tons (ultimate capacity). In addition, the piles would have likely been designed based upon a known or assumed ground line elevation to provide stability of the piles for the design loads.

The affects of scour on the bridge have been addressed by NCDOT. Prior to this assessment, NCDOT addressed scour activity at the bridge and prepared an action plan that included critical scour elevations that would trigger corrective and/or preventative actions. The pile loads determined by this assessment’s analysis were up to sixty-nine (69) tons using the HS20-44 live load test. The piles are resisting this load without apparent distress, based on current observations and the results of the A&O NBIS inspection report. Calculating the sufficiency of the piles to support the 69 ton load during a scour event, however, is beyond the scope of this assessment.

Based on the structural assessment analysis, repairs to various bridge elements are recommended primarily to reduce the rate of advancement of deterioration, to maintain the structural capacity of various deteriorated elements, and finally as preventative maintenance. In general, the repairs include shotcrete patching, epoxy resin/chemical grout crack injection, epoxy mortar patching, penetrant sealer application, and installation of pile jackets.

Prestressed Concrete AASHTO Girders (Spans 1-143 & 147-204)

As with the bent analysis, the analysis of the superstructure was completed using the 17th Edition of the *AASHTO Standard Specification for Highway Bridges* (2002 - Load Factor Design method). Also, as before, the live loading in the original plans was the H15-44 truck loading while the loading used for the girder assessment was the current HS20-44 truck loading. For the slab assessment, the heaviest wheel load of the largest North Carolina legal truck (11 kips) was used.

The first phase of the analysis involved modeling the bridge as it was shown in the plans. No adjustments were made for field observations. The purpose of this step was to set a baseline on which to compare further analyses. The superstructure was broken down into two components for analysis: the slab and the girders. The structure modeled was 45" prestressed concrete AASHTO girders at 8'-0" spacing supporting a 7¼" reinforced concrete deck with a concrete parapet and aluminum railing. Figure 32 below shows the typical prestressed girder section.

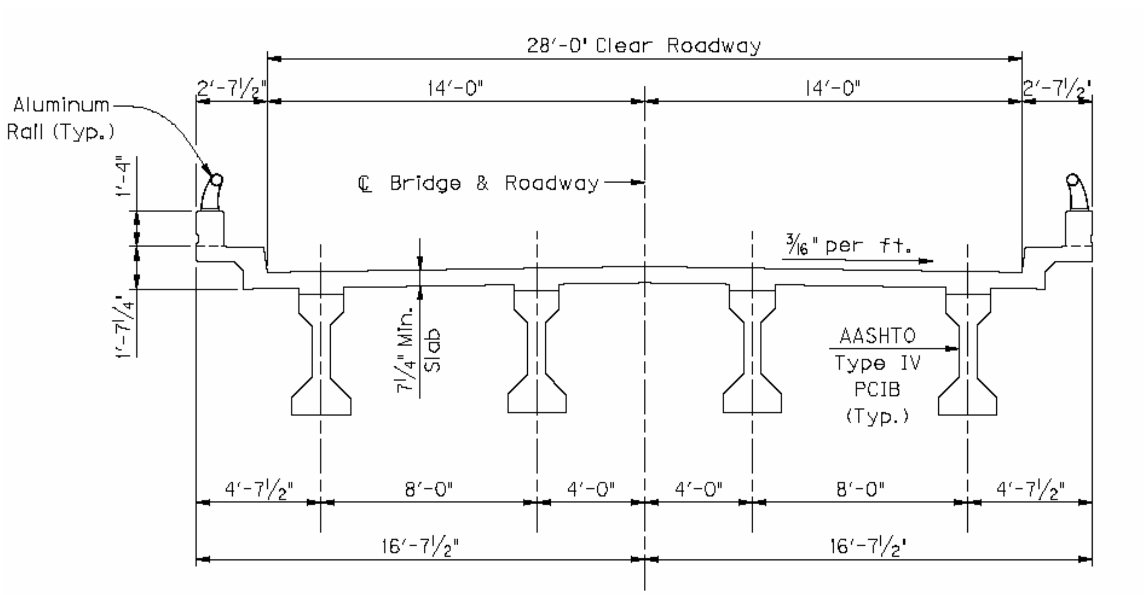

Figure 32. Typical Cross section of AASHTO girder span section.

Concrete Slab

The North Carolina legal truck wheel load used for the slab analysis (1/2 of 22 kip axle load = 11 kips) is shown below (Figure 33).

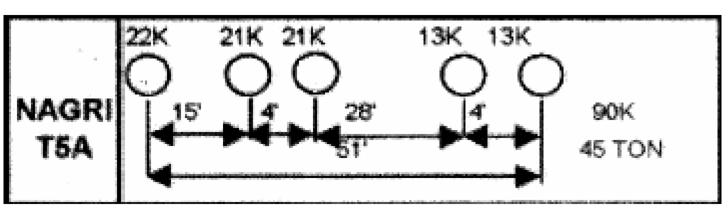

Figure 33. NCDOT Standard Truck T5A.

For the slab, the dead and live loads were applied to a transverse strip between girders. Based on the design concrete strength of 3,000 psi and the reinforcement shown in the plans, the slab's capacity was checked via an in-house spreadsheet.

45" AASHTO Girders

The first step in the analysis was to calculate the dead loads and live loads that the girders support. The dead loads consisted of the concrete slab/parapet, concrete diaphragms, and the aluminum rail. The live load used for the girders was HS20-44 truck. The HS20-44 truck consisted of three axles (Figure 33). The dead loads and live loads were input into the computer program CONSPAN, which checked the structural capacity of the girders. The plan concrete strength of 5,000 psi was used.

Results per Plan

The capacity of the slab and girders are adequate to support the applied loads.

Analysis per Inspection Findings

The analysis of the superstructure per inspection findings were performed to give a true capacity of the components. The modeling and analysis procedure was similar to that described above.

Model Revised per Inspection Findings

The modeling procedure followed that described above. Based on the visual field inspection report provided by A&O, the depth of the slab was reduced by the bottom clear cover of 1½". Since broken prestressing strands were observed in the field, the girders were checked to determine how many strands could be lost before a failure occurred. No modification to the plan concrete strength of 5,000 psi for the girders was made. However, a concrete strength of 4500 psi was used for the slab assessment, based on concrete core testing by WJE.

Results per Inspection Findings

Unlike with the plan modeling described above, the capacity of the slab was inadequate to support the applied loads once the field observations were incorporated (describe the field observations). The moment capacity available was 27% less than that required. The shotcrete repairs described in the Repair Recommendations will restore the slab to sufficient strength. For the girders, prestressing strands were removed from the bottom up until failure occurred. Failure of the girder occurred once nine strands were removed.

Welded Steel Plate Girder Spans

The analysis of the superstructure was completed using the 17th Edition of the AASHTO Standard Specification for Highway Bridges (2002 - Load Factor Design method). As with the concrete superstructure analysis, the HS20-44 truck was used for the girder assessment while the North Carolina legal load was used for the slab assessment.

The first phase of the analysis involved modeling the bridge as it was shown in the plans. No field observations were incorporated. The purpose of this step was to set a baseline on which to compare further analyses. The superstructure was broken down into two components for analysis: the slab and the girders. The structure modeled was welded steel plate (WSP) girders at 8'-0" spacing supporting a 7¼" reinforced concrete deck with a concrete parapet and aluminum railing. Figure 34 below shows the typical steel girder section.

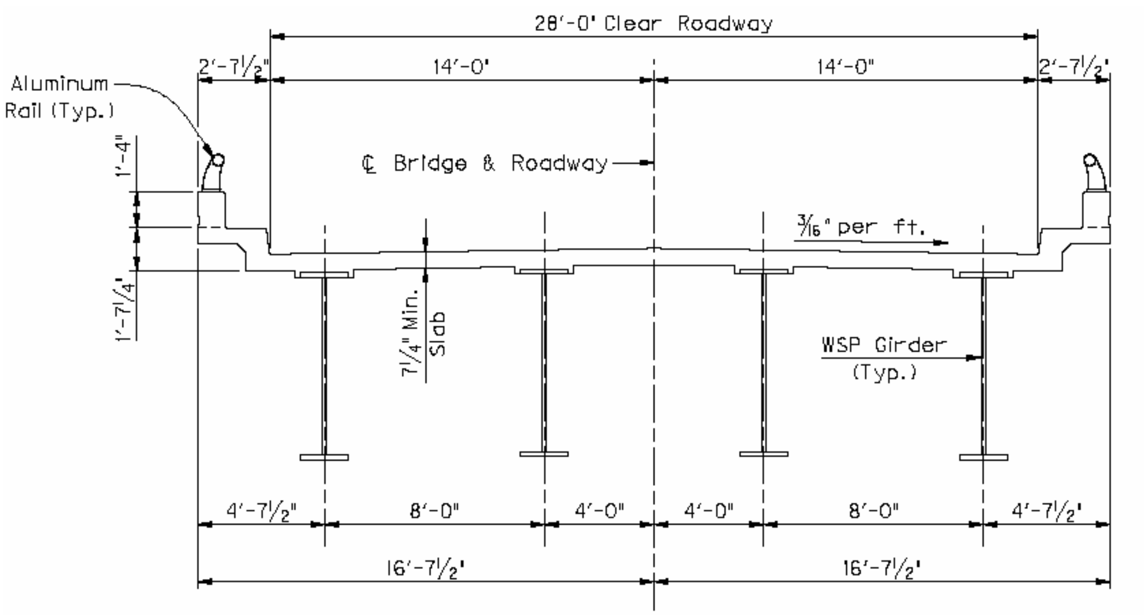

Figure 34. Typical welded steel plate girder section.

Welded Steel Plate Girders

The first step in the analysis was to calculate the dead loads and live loads that the girders support. The dead loads consist of the concrete slab/parapet, steel girders and miscellaneous steel, and the aluminum rail. The live load used was a HS20-44 truck. The HS20-44 truck consists of three axles. Figure 31 provides a comparison of the HS20-44 truck to the H15-44 truck.

For the girders, the dead loads and live loads were input into the computer program MDX, which evaluated the structural capacity of the girders. The steel for the girders was listed on the plans as ASTM A373. ASTM A373 had a yield strength of 32 ksi. As a note, the lateral bracing at the bottom of the girders was not included in the model because of conflicts in MDX. This did not have an effect of the girder analysis for the purposes of this task.

Concrete Slab

For the slab, the dead and live loads were applied to a transverse strip between girders. See the North Carolina legal truck wheel load used for the slab analysis (1/2 of 22 kip axle load = 11 kips) in Figure 33. Based on the design concrete strength of 3,000 psi and the reinforcement shown in the plans, the slab's capacity was checked via an in-house spreadsheet.

Results per Plan

The capacity of the slab and girders are adequate to support the applied loads.

Model Revised per Inspection Findings

Based on the visual field inspection report provided by A&O, the depth of the slab was reduced by the bottom clear cover of 1½" for analysis bsed on inspection findings. A concrete strength of 4,500 psi was used for the slab assessment based on concrete core testing by WJE.

Results per Inspection Findings

The inspection of the steel girders did not reveal any significant section loss of the griders; section loss was limited to the bracing members. Therefore the results of the analysis based on plan dimensions are applicable and the welded steel plate girders are adequate to support the applied loads.

The capacity of the slab was inadequate to support the applied loads once the field observations were incorporated; the moment capacity available was 36% less than that required. The shotcrete repairs, described in the Repair Recommendations section, will restore the slab to sufficient strength.