Determination of the Structural Stability of the Proposed Indian River Inlet Bridge

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August 15, 2003

TABLE OF CONTENTS

ABSTRACT

Results of a complete investigation of the stability and strength of a single rib concrete arch are presented. The arch was designed as a single rib concrete arch with a span length of 1,000 ft and a span rise of approximately 230 ft. Through an elastic stability analysis it was determined that the lateral-torsional bucking of the arch would govern the design. Various stability equations were used to produce critical buckling loads for both circular and parabolic arch types. In addition to checking the stability limit state, a computer model was constructed to obtain support reactions and member stresses in order to check the strength limit state. Through the determination of these stresses, in association with various load factors, it was possible to assess the safety of the arch in terms of strength. Final conclusions suggest that the single rib concrete arch can safely meet both stability and strength design requirements and effectively span the required length.

INTRODUCTION

Our nation's bridge infrastructure is extremely diverse. From large scale suspension bridges to small simply supported timber bridges. One bridge type of particulate interest here is the arch. Tied, through, single ribbed, and double ribbed are just some examples of arch structures that can safely and effectively span their required lengths. Arches are also constructed out of various materials such as steel, concrete, and masonry. No matter what the magnitude or material all arches have one thing in common: their main structural members are under compression.

Because of this compression, both the stability and strength limit states need to be closely examined. Both in-plane and out-of-plane buckling must be considered. Often buckling occurs without warning and far below the ultimate strength of the material. Therefore, it is very important to design the structure properly to ensure this problem does not exist. Designers typically handle this problem by creating two ribs with lateral bracing connecting them. This lateral bracing not only helps reduce the chance of buckling but it also allows heavier loads to be carried by the structure.

However, due to aesthetic reasons, a double ribbed arch is not always desired by the design engineer. More recently single ribbed arches, both constructed of steel and concrete, have been chosen for design (Damen Avenue Bridge, Chicago, Illinois, J. Muller International; Presidents JK Bridge, Brasilia, Brazil, Alexandre Chan, Mario Jaime dos Reis Vila Verde, Filemon Botto de Barros, Piotr Slawinski). In these cases both the structural rigidity of the rib member and the hanger cable arrangement play a large role in determining the resistance to buckling.

This paper begins with a brief overview of the Indian River Inlet project. A brief history of the existing structure is presented, along with the proposed replacement of the structure. Once the proposed structure is presented, an extensive stability analysis of the structure is preformed. Next the strength of the rib arch is evaluated. Finally, the arch design is evaluated in terms of both stability and strength limit states.

BACKGROUND

Extensive studies have been conducted on the stability of arches by Timoshenko and Gere 1961 [1] and Tokarz and Shandhu 1972 [2]. Both in-plane and out-of-plane buckling behavior has been evaluated. Based on the results of these studies it is possible to determine the critical loads at which an arch will buckle. This paper will attempt to apply these studies to the Indian River Inlet Bridge and determine its structural stability.

EXISTING BRIDGE

The existing Indian River Inlet Bridge is located in southeastern Sussex Co., Delaware. The inlet was formed between 1928 and 1937. There have been several bridges used to span the inlet. The first bridge was constructed in 1934. This bridge had a timber superstructure. In 1938 a replacement concrete/steel swing bridge was constructed. However, due to large ice flows, this bridge was destroyed in 1948. The swing bridge was finally replaced in 1952 with a similar structure. Due to tidal flow and an increase in traffic the existing two lane northbound bridge was built in 1965. The northbound bridge is a 5 span steel I-girder design. An additional sister bridge was built in 1976 to once again ease traffic congestion. Currently, the bridge carriers State Route 1 traffic north and south between the various Delaware beaches and state parks. The present traffic volumes are estimated between 16,000-18,000 vehicles per day.

Due to the location of the inlet and the presences of two piers in the water an extensive scouring problem has arisen. During the inlets inspection, between 1928 and 1937, the maximum depth was measured at approximately 28 ft. Today, due to heavy scouring, the maximum inlet depth has been measured to be 100 ft. In 1989 an extensive rehabilitation project was undertaken to help stop the scouring problem. Rip-rap was added around the steel piles to control the scouring. In addition to the scouring problem the existing north bound bridge is approaching its design life of 50 years. Due to the immense scouring problem, the high cost of maintaining the bridge as it approaches its design life, and the continued need for monitoring of the rip-rap, it has been determined that a new structure should be built that completely spans the current inlet width of 500 ft.

PROPOSED STRUCTURE

Due to the severity of the scouring problem, the proposed bridge will span the entire inlet width, thus eliminating the scouring problem. The lead design firm, Figg Engineering Group, conducted two design charettes in order to choose a design for the proposed structure. These design charettes allowed for the community to have an active role in the choice of the deigned structure. At the first design charette a brief overview of the project was given. Additionally, several design options were presented and initial feedback was gathered on the design of the bridge. The second design charette allowed Figg Engineering to solidify a specific structural design based on community input. The design chosen was a single ribbed concrete tied arch structure (see figure). One of the primary reasons the single rib arch option was chosen was due to its uniqueness.

The concrete structure will have a main span length of 1,000 ft and a rise of approximately 230 ft. The structure will also include two 150 ft open back spans (see figure). A vertical clearance over the water of 45 ft will be provided. This increases the vertical clearance of the existing structure by 10 ft.

Proposed Indian River Inlet Bridge (Figg Engineering Group)

Figg Engineering is proposing to use 8,000 psi concrete on all arch and tie beam segments, 6,500 psi concrete on the remaining superstructure, and 4,500 psi concrete on all abutments and footings. The cross section of the arch is tulip shaped in nature and is 20 ft tall by 16 ft wide with 4 ft thick walls.

The overall roadway width is approximately 108 ft. The roadway consists of four 12 ft traffic lanes, two 10 ft shoulders, two 4 ft shoulders, and one 12 ft pedestrian sidewalk adjacent to the north bound lane.

The construction of the arch is proposed to take two and a half years to complete and will begin in the fall of 2004. During construction temporary towers will be built to erect and stabilize the arch until its completion. The arch and tie beam segments will be cast in place using a form traveler. The roadway box sections will be cast off site in segments and be post tensioned in the field.

RESEARCH TOPIC

Due to the compressive nature of an arch, a long design span length, large dead loads, one plane of hanger cables, and no lateral bracing, the chosen design (a single ribbed concrete arch) is quite prone to structural instability. The limit states that will be examined are buckling and strength. Buckling will be examined using theoretically derived analytical equations. Strength is based upon the compressive strength of the material and therefore, an extensive computerized stress analysis is used in this examination.

STABILITY MODEL OF THE PROPOSED STRUCTURE

THEORY OF ELASTIC STABILITY

The theory used to determine the stability of the long span concrete arch is the Theory of Elastic Stability. Timoshenko and Gere derived several equations that can be used to determine the uniform critical load at which a circular or parabolic arch will buckle [1]. This buckling was determined to take place in the vertical plane. Additionally, Tokarz and Shandhu derived several equations that can be used to determine the uniform critical load at which a parabolic arch will buckle laterally or torsionally [2]. Both theories of stability are similar in that the critical load was based upon several parameters. These parameters are; span length, span to height ratio, modulus of elasticity of the arch material, moment of inertia of the arch cross section, polar moment of inertia of the arch cross section, support conditions, and a numerical constant. The basic equation that governs the critical load is given as follows [1], [2]:

$$q_{cr} = \gamma_i \frac{EI}{L^3} \qquad (1)$$

wherre, q_{cr} = critical uniform buckling load

E = Young's modulus of elasticity

I = moment of inertia

L = span length

γ_i = numerical constant, i depends on the case being studied

The numerical constants for both in-plane and lateral-torsional buckling were determined using mathematical techniques. Further laboratory experiments by Tokarz and Shandhu verified these numerical constants [2].

CRITICAL BUCKLING LOADS

An analysis of both in-plane buckling and lateral-torsional buckling was performed. First in-plane buckling was considered followed by out-of-plane buckling. Both buckling analysis were governed by the theory of elastic stability.

In-Plane Buckling

For in-plane buckling, it was assumed that the arch has a circular shape with a constant cross section. To evaluate the buckling load for this case the equations derived by Timoshenko and Gere were used [1]. Another assumption that was made in conducting the in-plane structural analysis of the arch was that the uniformly distributed load, which was initially normal to the arch, remains normal to the buckled axis of the arch. An additional assumption was that the arch was either supported with two hinges or supported with two fixed ends. Results for each condition are presented. Due to the geometric complexity of the proposed cross section a simpler cross section was chosen for the analysis. The cross section chosen, its geometric properties, and the modulus of elasticity are given in Table 1.

Table 1. Summary of Properties Used

Moment of Inertia	Value
	(ft^4)
Ix =	10,954.82
Iy =	4,407.09
Iz =	6,547.73

Modulus of Elasticity	Value
	(ksi)
E =	4,000

The mathematically derived numerical constant, γ_i, depends on the height to span ratio and the support conditions. The span length of the structure is 1,000 ft and the height of the structure is approximately 230 ft. This gives a height to span ratio of 0.23. Values for γ_i were interpolated from Table 2 for both fixed-fixed supports and hinged-hinged supports. Values for γ_i for their respective cases are given in Table 3.

Table 2. Values of the Factor γι for Uniformly Compressed Circular Arches of Constant Cross Section (Timoshenko and Gere, 1961)

h/l	No hinges	One hinge	Two hinges	Three hinges
0.1	58.9	33	28.4	22.2
0.2	90.4	50	39.3	33.5
0.3	93.4	52	40.9	34.9
0.4	80.7	46	32.8	30.2
0.5	64.0	37	24.0	24.0

Table 3

Table 3. gi Values

h/l	No hinge	Two Hinges
0.23	91.3	39.78

Once the above γ_i values were calculated it was possible to determine the critical loads at which the arch would theoretically buckle. Equation (1), the general critical load equation stated earlier was used to calculate the critical loads.

$$q_{cr} = \gamma_i \frac{EI}{L^3} \qquad (1)$$

Fixed – Fixed

$$q_{cr} = \frac{91.3 * 4{,}000\text{psi} * 6{,}547.73\text{ft}^4}{(1{,}000\text{ft})^3}$$

$$q_{cr} = 344\text{k/ft}$$

Hinged – Hinged

$$q_{cr} = \frac{39.78 * 4{,}000\text{psi} * 6{,}547.73\text{ft}^4}{(1{,}000\text{ft})^3}$$

$$q_{cr} = 150\text{k/ft}$$

The above calculated critical loads of 344 k/ft and 150 k/ft represent the maximum allowable uniformly distributed load the arch can withstand without buckle in the vertical plane.

Both end support conditions were examined. However, in the investigation of the stability of the structure it was assumed the fixed-fixed case would govern the design. The fixed-fixed end support conditions were chosen because it was determined that fixed-fixed end supports were a better representation of the actual structure.

Lateral-Torsional Buckling

The second case that was analyzed was the case of lateral-torsional buckling of a parabolic arch as derived by Tokarz and Shandhu [2]. The same assumptions and cross sectional properties were used for this analysis as were used in the previous in-plane analysis.

Table 4. Smallest Critical Buckling Parameter for Various Values of h/l and C/A for Free Standing Arch with Fixed End Supports (Tokarz and Shandhu, 1972)

h/l	C/A					
	0.6	0.8	1.0	1.2	1.4	1.6
0.1	27.7	27.87	27.97	28.044	28.09	28.13
0.2	39.16	39.84	40.29	40.6	40.84	41.03
0.3	37.17	38.7	38.72	39.2	39.58	39.88
0.4	30.7	31.49	32.08	32.55	32.93	33.21
0.5	24.32	24.9	23.36	25.74	26.05	26.32
0.6	19.2	19.61	19.94	20.21	20.45	20.33
0.7	15.31	15.59	15.82	16.02	16.2	16.36
0.8	12.38	12.58	12.75	12.9	13.03	13.14
0.9	10.18	10.32	10.45	10.55	10.65	10.74
1	8.508	8.616	8.71	8.793	8.869	8.939

Table 5. Smallest Critical Buckling Parameter for Various Values of h/l and C/A for Free Standing Arch with Hinged End Supports (Tokarz and Shandhu, 1972)						
h/l			C/A			
	0.6	0.8	1.0	1.2	1.4	1.6
0.1	5.76	5.903	5.992	6.052	6.097	6.13
0.2	5.334	5.754	6.051	6.245	6.401	6.523
0.3	3.183	3.603	3.913	4.151	4.339	4.492
0.4	1.726	2.027	2.263	2.545	2.611	2.743
0.5	0.9494	1.145	1.306	1.441	1.556	1.655
0.6	0.5463	0.6713	0.7781	0.8705	0.9511	1.022
0.7	0.3308	0.4123	0.4838	0.5471	0.6036	0.6539
0.8	0.2114	0.2665	0.3156	0.3589	0.3999	0.4363
0.9	0.1458	0.1851	0.2205	0.2528	0.2823	0.3096
1	0.1176	0.1486	0.1765	0.207	0.2256	0.2476

Once again the numerical constant γ_i depends on several factors. In this case the γ_i value depends on the height to span ratio and on the torsional rigidity (C) to flexural rigidity (A) ratio. The values for C and A respectively are 812.77 lb*in^2 and 1263.07 lb*in^2. The height to span ratio and the torsional rigidity to flexural rigidity are 0.23 and 0.64 respectively. The γ_i value, from Table 4, was interpolated to be 38.71. The γ_i value from Table 5 was interpolated to be 4.77. Using the critical load equation given earlier (Equ. 1), the critical loads for both end support conditions are given.

$$q_{cr} = \gamma_i \frac{EI}{L^3} \qquad (1)$$

$$q_{cr} = \frac{38.71 * 4000\text{psi} * 6547.73\text{ft}^4}{(1000)^3} \qquad q_{cr} = \frac{4.77 * 4000\text{psi} * 6547.73\text{ft}^4}{(1000)^3}$$

$$q_{cr} = 146\text{k/ft} \qquad q_{cr} = 18\text{k/ft}$$

Once again both end support conditions were examined. However, in the investigation of the stability of the structure it was assumed the fixed-fixed case would govern the design. The fixed-fixed end support condition was chosen because it was determined that fixed-fixed end supports were a better representation of the actual structure.

Upon completion of the two elastic stability analyses, it can be seen that lateral-torsional buckling of the arch will govern the design when stability is considered. This result was expected due to the asymmetrical cross-sectional shape of the arch. The corresponding uniformly distributed load is 146 k/ft.

Arch Computer Model of the Proposed Structure

After the determination of the critical loads for the stability limit state using the above equations, it was desired to create a computer model to analyze the structure for strength. A computer model was used to determine dead load and live load stresses and compare these stresses to the ultimate strength of the arch. The computer-modeling program that was used in the determination of these stresses was STAAD.Pro [3]. The arch was modeled by replicating construction drawing in AutoCAD and transferring the geometric data into STAAD.Pro. The arch was modeled using 38 straight segments positioned in a circular arrangement.

Dead Load Stresses

Dead load stresses were estimating based upon the construction drawings [3]. The cross section areas of each member were needed to accurately compute a dead load. Table 6 describes each member, number of members in the structure, weight of each member, and total dead load of the structure.

Table 6. Estimated Dead Load Weights

Member	Individual Weight	Number of Members	Total Weight
`	(kips)		(kips)
Northbound Typ.	104.86	74	7,760
Northbound Anchor Typ.	148.41	37	5,491
Southbound Typ.	87.61	74	6,483
Southbound Anchor Typ.	120.07	37	4,443
Tie Beam Typ.	120.22	74	8,896
Tie Beam Anchor Typ.	178.56	37	6,607
Concrete Railing	600(lb/ft)	3	1,800
		Total Dead Load =	41,480

The total dead load of 41,480 kips was then evenly distributed over the 37 cables that support the roadway. As such, the vertical dead load on each cable was approximately 1,117 kips. Due to the cable arrangements, horizontal dead loads were also modeled. The vertical dead loads (tie beam, roadway box sections), subsequent horizontal loads (due to inclined cables), and the self-weight (of the arch itself) were then used to calculate dead load stress for various models. Each model differed only in the support conditions. The cases that were model were as follows: fixed-fixed, fixed-pinned, and pinned-pinned.

Because the final design of the arch was determined to be a tied arch, dead load stresses were also computed for a tied arch. The loads and geometry were identical to that of the free standing arch with the difference being that a 20 inch diameter solid circular steel tie member was placed in the structure. A 20 inch diameter tie member was chosen because of the uncertainty of the actual geometry of the tie beam. The 20 inch tie member is meant to represent both the pre-stressing strands and the post tensioning strands that will run through the tie beam.

As expected, stress results of the tied arch model were very similar to that of a free standing arch. The only difference between the free standing arch and the tied arch is in the horizontal reactions at the supports. Horizontal reactions at the supports for the free standing arch approached 40,000 kips, while there are virtually no horizontal reactions at the supports in the tied arch case because the steel tie member takes the horizontal thrust of the arch. Tables 7 and 8 present the results of the dead load stress analysis.

Table 7. Dead Load Stresses

Support Condition	Dead Load Axial Compressive Stress
	(ksi)
Fixed - Fixed	1.975
Fixed - Pinned	1.961
Pinned - Pinned	1.964
Tied Arch	1.924
20 inch tie member	8.845 (tensile)

LIVE LOAD STRESSES

Live load stresses were also determined for the arch structure. The proposed structure is designed to carry four 12 ft traffic lanes, two 10 ft shoulder, two 4 ft shoulders, and one 12 ft pedestrian walkway. AASHTO specifications require that a 640 lb/ft load must be applied to every 12 ft traffic lane [4]. The combined shoulder length of 28 ft is equivalent to two additional traffic lanes. Therefore, the corresponding load for the six traffic lanes is 3,840 kips. Additionally, a load of 85 lb/ft^2 must be applied to the pedestrian walkway [4]. The pedestrian walkway is 12 ft wide and 1,000 ft long. Given these dimensions the pedestrian live load total is 1,020 kips. The total combined vehicular live load and pedestrian live load is 4,860 kips. This combined load is then uniformly distributed over the entire length of the arch. This uniformly distributed value is 4.31 kips/ft. Once again STAAD was used to accurately evaluate the live load stresses in the bridge. Similar to the dead load analysis a free standing arch with varying end supports and a tied arch were modeled. Results of the live load stress analysis are given in Table 9. Once service dead load stresses and service live load stress were obtained it was possible to determine the service dead load to live load ratio. The calculated service dead load to live load ratio is 0.069.

Table 8. Live Load Stresses	
Support Condition	Live Load Axial Compressive Stress
	(ksi)
Fixed - Fixed	0.137
Fixed - Pinned	0.134
Pinned - Pinned	0.135
Tied Arch	0.133
20 inch tie member	0.620 (tensile)

ANALYSIS OF PROPOSED ARCH, IN TERMS OF STABILITY AND STRENGTH

After performing the desired theoretical and computer analysis, a design evaluations of the arch can be conducted. The design evaluation involves checking both the stability and strength limit states. These limit states are based on the LRFD design equation given below.

$$\sum_i \gamma_i Q_i \leq \Phi R_n \qquad (2)$$

where, γ_i = load factor

Q_i = unfactored stress or load

Φ = reduction factor

R_n = ultimate stress or load

STABILITY ANALYSIS

From the elastic stability calculation it was determined that the critical uniformly distributed load that would cause the arch to buckle was 146 k/ft. According to AASHTO, a reduction must be applied to concrete members that are in compression. For tied elements, this reduction factor is 0.75 [4]. Applying the reduction factor of 0.75 the critical buckling load is reduced from 146 k/ft to 109.5 k/ft.

$$\Phi R_n = 0.75 * 146 \text{k/ft}$$
$$\Phi R_n = 109.5 \text{k/ft}$$

The uniform dead load, including the dead load of the roadway and the self-weight of the arch, was calculated to be approximately 64 k/ft. According to AASHTO a dead load factor of 1.25 must be applied to this value [4]. Upon application of the 1.25 dead load factor, the uniform dead load value was increased from 64 k/ft to 80 k/ft.

$$\gamma_{DL} Q_{DL} = 1.25 * 64\text{k/ft}$$
$$\gamma_{DL} Q_{DL} = 80\text{k/ft}$$

The uniform live load calculated during the theoretical analysis was 4.31 k/ft. According to AASHTO an impact factor of 33% is applied to the live load stress [4]. Applying the impact factor of 33% increases the uniform live load from 4.31 k/ft to 5.75 k/ft. This load must also be multiplied by a live load factor of 1.75, as per AASHTO [4]. After application of the live load factor the uniform live load is increased from 5.75 k/ft to 10 k/ft.

$$\gamma_{LL} Q_{LL} = \gamma_{LL}[LL(1+I)] \qquad (3)$$
$$\gamma_{LL} Q_{LL} = 1.75 * [4.31 * (1+0.33)]$$
$$\gamma_{LL} Q_{LL} = 10\text{k/ft}$$

where, $I = \text{impact factor}$
$LL = \text{unfactored live load stress}$

Having applied the appropriate load and resistance factors it is possible to evaluate the design in term of stability. Combining the factored dead and live loads one gets a final factored load of 90 k/ft. This factored load is lower than the reduced ultimate load of 109.5 k/ft. Since the capacity exceeds the demand, the design is adequate in terms of the stability limit state.

$$\sum_i \gamma_i Q_i \leq \Phi R_n \qquad (2)$$
$$\gamma_{DL} Q_{DL} + \gamma_{LL} Q_{LL} \leq \Phi R_n$$
$$80\text{k/ft} + 10\text{k/ft} \leq 146\text{k/ft}$$
$$90\text{k/ft} \leq 109.5\text{k/ft}$$

STRENGTH ANALYSIS

The design strength of the concrete arch, f'c, is equal to 8,000 psi. According to AASHTO, a reduction must be applied to concrete members that are in compression. For tied elements, this reduction factor is 0.75 [4]. After applying this reduction factor, the maximum ultimate capacity becomes 6,000 psi.

$$\Phi R_n = 0.75 * 8{,}000\text{psi}$$
$$\Phi R_n = 6{,}000\text{psi}$$

Because of the uncertainty of the dead loads and live loads, load factors are applied to the calculated service design stresses. For dead load, the load factor that is applied is 1.25, as per AASHTO 1998 [4]. This load factor is multiplied by the final calculated dead load stress from the computer model. The highest calculated dead load axial compressive stress was 1.975 ksi. This dead load stress occurred in the member located at the support of the arch. This stress corresponds to the fixed-fixed end support conditions. After applying the dead load factor the final design dead load stress becomes 2.469 ksi.

$$\gamma_{DL} Q_{DL} = 1.25 * 1.975 \text{ksi}$$
$$\gamma_{DL} Q_{DL} = 2.469 \text{ksi}$$

A maximum live load axial compressive stress of 0.137 ksi was obtained in the initial stress analysis. This maximum stress again occurred in the member located at the support of the arch. Live load factors must also be applied to the structure. The first factor that must be applied to the live load stresses is the impact factor. An impact factor of 33%, as per AASHTO 1998, is applied to the live load stress [4]. This factor increases the live load stress from 0.137 ksi to 0.183 ksi. After the impact factor is applied a live load factor of 1.75, as per AASHTO 1998, must also be applied [4]. This factor once again increases the final design live load stress from 0.183 ksi to 0.319 ksi.

$$\gamma_{LL} Q_{LL} = \gamma_{LL}[LL(1+I)] \qquad (3)$$
$$\gamma_{LL} Q_{LL} = 1.75 * [0.137 * (1 + 0.33)]$$
$$\gamma_{LL} Q_{LL} = 0.319 ksi$$

where, $I = \text{impact factor}$
$LL = \text{unfactored live load stress}$

Having applied the appropriate load and resistance factors it is possible to evaluate the design in terms of compressive strength. After combining the dead load stress and the live load stress a final factored axial compression stress of 2.8 ksi was obtained. This factored stress is significantly below the reduced ultimate stress of 6 ksi. Therefore, the arch design is adequate in terms of the strength limit state.

$$\sum_i \gamma_i Q_i \le \Phi R_n$$
$$\gamma_{DL} Q_{DL} + \gamma_{LL} Q_{LL} \le \Phi R_n$$
$$2.469 \text{ksi} + 0.319 \text{ksi} \le 6.000 \text{ksi}$$
$$2.8 \text{ksi} \le 6.0 \text{ksi}$$

SUMMARY OF ANALYSIS

After conduction both strength and stability analyses, one can determine which limit state, stability or strength, governs the design of the arch. The ratio of factored designed strength to factored ultimate strength for buckling and crushing are 0.822 and

0.465 respectively. This indicates that buckling of the arch is much more of a concern than strength of the arch.

To maximize the design of the structure it is desired to obtain approximately the same values for the two ratios. One way to try to alleviate this discrepancy between the two ratios is to change the cross section of the arch. Because the cross section is asymmetrical, lateral or torsional buckling will govern the design of the arch. Perhaps by obtaining a symmetrical cross section it would be possible to obtain values for these ratios that are in close proximity to one another. Future work in the analysis of the proposed structure might involve such calculations. However, for aesthetic reasons, such a shape may not be desirable. In this case, aesthetics is an important design criterion.

CONCLUSION

The Delaware Department of Transportation has decided to replace the current Indian River Inlet Bridge with a new structure. To avoid scour problems, the new bridge will completely span the inlet. Figg Engineering Group has proposed to build a 1,000 ft span single ribbed concrete arch with a single line of vertical hangers. This design was developed with substantial input from the local community.

This report has presented the proposed design and has evaluated the two most important limit states for the arch design; stability and strength. The theory of elastic stability has been used to show that the proposed design has adequate resistance to both in and out of plane buckling. LRFD capacity equations have been used to show that the proposed design is also sufficient in term of compressive strength of the arch. Of the two limit states that have been evaluated, stability is more critical for the proposed design.

ACKNOWLEDGEMENTS

This material is based on work supported by the National Science Foundation under Grant No. EEC-0139017, Research Experience for Undergraduates in Bridge Engineering, at the University of Delaware. The writer would also like to acknowledge Prof. Michael Chajes and Prof. Dennis Mertz for their knowledge, expertise, and advice with respect to the project and Diane Kukich for her assistance in the preparation of this report. Thanks also to Figg Engineering and DelDOT for discussions and for design information provided in "Indian River Inlet Design Charette II," May 7, 2003.

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APPENDIX

I. AASHTO LOAD FACTOR TABLES

Section 3 - Loads and Load Factors

SPECIFICATIONS

Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, φ, may be taken as:

- When the geotechnical parameters are well defined, and the slope does not support or contain a structural element 0.85
- When the geotechnical parameters are based on limited information, or the slope contains or supports a structural element 0.65

For structural plate box structures complying with the provisions of Article 12.9, the live load factor for the vehicular live loads LL and IM shall be taken as 2.0.

COMMENTARY

another permanent load reduces the uplift, it would be multiplied by the minimum load factor, regardless of the span in which it is located. For example, at Strength I Limit State where the permanent load reaction is positive and live load can cause a negative reaction, the load combination would be 0.9DC + 0.65DW + 1.75(LL+IM). If both reactions were negative, the load combination would be 1.25DC + 1.50DW + 1.75(LL+IM). For each force effect, both extreme combinations may need to be investigated by applying either the high or the low load factor as appropriate. The algebraic sums of these products are the total force effects for which the bridge and its components should be designed.

Water load and friction are included in all strength load combinations at their respective nominal values.

For creep and shrinkage, the specified nominal values should be used. For friction, settlement, and water loads, both minimum and maximum values need to be investigated to produce extreme load combinations.

Table 3.4.1-1 - Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS EL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

3 - 10

SPECIFICATIONS

COMMENTARY

Table 3.4.1-2 - Load Factors for Permanent Loads, γ_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.0	1.0
EV: Vertical Earth Pressure		
• Overall Stability	1.35	N/A
• Retaining Structure	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

The load factor for temperature gradient, γ_{TG}, and settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{TG} may be taken as:

- 0.0 at the strength and extreme event limit states,
- 1.0 at the service limit state when live load is not considered, and
- 0.50 at the service limit state when live load is considered.

The load factor for temperature gradient should be determined on the basis of the:

- Type of structure, and
- Limit state being investigated.

Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e., γ_{TG} = 0.0.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL \qquad (3.4.1\text{-}2)$$

The load factor for live load in Extreme Event Load Combination I, γ_{EQ}, shall be determined on a project-specific basis.

Past editions of the Standard Specifications used γ_{EQ} = 0.0. This issue is not resolved. The possibility of partial live load, i.e., γ_{EQ} < 1.0, with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that γ_{EQ} = 0.50 is reasonable for a wide range of values of average daily truck traffic (ADTT).

SPECIFICATIONS

Table 3.6.2.1-1 - Dynamic Load Allowance, IM

Component	IM
Deck Joints - All Limit States	75%
All Other Components • Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

The application of dynamic load allowance for buried components, covered in Section 12, shall be as specified in Article 3.6.2.2.

Dynamic load allowance need not be applied to:

- Retaining walls not subject to vertical reactions from the superstructure, and
- Foundation components that are entirely below ground level.

The dynamic load allowance may be reduced for components, other than joints, if justified by sufficient evidence, in accordance with the provisions of Article 4.7.2.1.

COMMENTARY

- Hammering effect is the dynamic response of the wheel assembly to riding surface discontinuities, such as deck joints, cracks, potholes, and delaminations, and
- Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 1 is the product of 4/3 and the basic 25 percent.

This article recognizes the damping effect of soil when in contact with some buried structural components, such as footings. To qualify for relief from impact, the entire component must be buried. For the purpose of this article, a retaining type component is considered to be buried to the top of the fill.

3.6.2.2 BURIED COMPONENTS

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

$$IM = 33\,(1.0 - 0.125 D_E) \geq 0\% \qquad (3.6.2.2\text{-}1)$$

where:

D_E = the minimum depth of earth cover above the structure (FT)

3.6.2.3 WOOD COMPONENTS

For wood bridges and wood components of bridges, the dynamic load allowance specified in Article 3.6.2.1 may be reduced to 50 percent of the values specified for IM in Table 3.6.2.1-1.

C3.6.2.3

Wood structures are known to experience reduced dynamic wheel load effects due to internal friction between the components and the damping characteristics of wood.

3.6.3 Centrifugal Forces: CE

Centrifugal forces shall be taken as the product of the axle weights of the design truck or tandem and the factor C, taken as:

C3.6.3

Lane load is neglected in computing the centrifugal force, as the spacing of vehicles at high speed is