Static Ice and Observed Damage to the Lake Champlain Bridge

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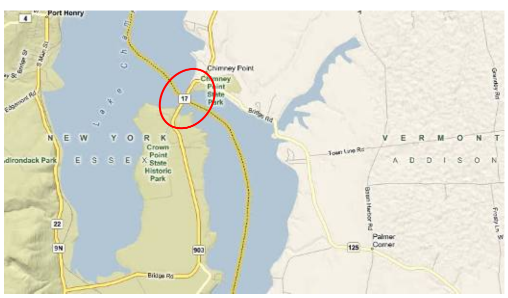
On December 28th, 2009, the Lake Champlain Bridge between New York and Vermont was explosively demolished given extensive pier damage and the potential for catastrophic failure. Visible pier damage was consistent with ice abrasion and freeze thaw damage at the waterline. Diving inspections indicated large cracks around the entire perimeter of these large unreinforced substructures, even for expansion piers, which suggest that static forces associated with lake ice contributed significantly to observed damage. Lake ice is often considered benign and does not warrant consideration in pier design. Thermal movements of a large intact ice sheet induce sizeable forces on pier elements and contributed strongly to the observed damage. This paper explores design issues associated with static ice for bridges that traverse lakes in cold regions.

Introduction

The Lake Champlain Bridge connects Crown Point, NY and Chimney Point, VT. Comprised of 14 steel spans with an overall length of nearly 670m, the two-lane bridge was opened to traffic in 1929. Of the 14 spans, five of the spans are deck trusses, one span is a half-through truss, and the remaining are steel girder superstructures. The combination of deck and through trusses at the midspan of the bridge (Spans 6-8) has been noted for its historic significance and its iconic form.



The bridge serves two sparsely populated regions, with average daily traffic on the order of 3500 vehicles. However, there are few alternative crossings for Lake Champlain, with ferry and fixed crossings resulting in detour lengths that exceed 150km, with ferry operations compromised during the winter months. The regions on either side of the lake are economically interdependent such that the regional importance of the crossing cannot be underestimated.

Lake Champlain Bridge Main Span Unit

The bridge's iconic form was conceived by Charles M. Spofford, an early pioneer in design methods for continuous trusses. The bridge's form is a particularly elegant application of Spofford's ideas that demonstrate the advantages in efficiency of continuous structural systems. This bridge has an important

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place in the evolution of continuous trusses and the practice of bridge engineering in the United States. In February 2009 the Lake Champlain Bridge was granted approval to be entered in the National Register of Historic Places. While not formally listed yet, the structure has met the eligibility requirements regarding "age, integrity, and significance".

There has been a significant amount of rehabilitation and retrofit work on the bridge over its 80 year life, with the most extensive work completed in the early 1990's. This rehabilitation included replacement of the existing concrete deck with a concrete filled steel grid deck, bearing rehabilitation, post tensioning pier retrofits, and structural steel repairs. However, deterioration has continued to progress rapidly both for the superstructure and the substructure. The most severe substructure damage occurred in the pier shafts near the waterline and is attributable to the effects of lake ice. The significant pier deterioration represented a considerable risk to the overall safety of the structure and ultimately led to its replacement.

The Lake Champlain Bridge is an early design of a continuous truss and its chief designer, Charles Spofford, was influential and active in the analysis and design and construction of such structures, authoring a book entitled *Theory of Continuous Structures and Arches* published in 1937. This structural form was a clear early innovation in the design of continuous trusses, and Spofford's role in its development is well understood. One of the challenges with continuous truss design is that forces in the truss system are dependent upon support geometry and must be prescribed. After closure of the main span superstructure and prior to installation of the bearings, the structure must be jacked into its final geometry. This process is described by Griggs (Griggs, 2007) as well as Spofford in his writings on continuous truss bridges, and is a critical aspect of the design and construction of the structure. The superstructure's sensitivity to pier movement is a key concern.

The use of plain rather than reinforced concrete for the piers was clearly the most unusual aspect of the design of a bridge in the late 1920's, particularly given the pier heights and associated slenderness. There were no explicit considerations in the pier design for the potential for concrete damage at the waterline associated with ice abrasion. Caisson and pier concrete placed below water in the deep open cofferdams, the use of a 1-yard dump bucket instead of by tremie pipe.

In the discussion of Spofford's ASCE paper entitled *Lake Champlain Bridge* (Spofford, 1931), a noted expert in concrete and bridge design, Jacob Feld questioned whether there were any

> *"special precautions in protecting the surface of the piers at the waterline to take care of ice pressure and wear during the winter seasons… undoubtedly, this item was considered in the design, and the omission of special protection for the concrete must have resulted from definite reasons. It would be interesting to have those arguments on record."*

Spofford's response to Feld's inquiry is telling:

"the reason for not protecting the pier concrete against abrasion and deterioration, it may be pointed out that the piers are in a fresh-water lake with little current and are practically free from danger of abrasion from ice and floating objects".

In fact, significant damage to the piers occurred over time, with deterioration and cracking a nearly constant concern through the life of the bridge. Pier damage at or near the waterline, was ultimately a major issue in the overall safety of the structure.

<u>Past Inspections & History of Repairs</u>

Many structural repairs were implemented throughout the bridge's lifetime: a summary of several major repair projects under both the Lake Champlain Bridge Commission (through 1987) and NYSDOT/VAOT (1987 – Present) are given in the adjacent table. It is evident that the piers have been repaired numerous times over the life of the structure. Pier and bearing repairs in 1945 should be regarded as unusual, given that the bridge was in service for only 15 years, and is no doubt associated with the lack of reinforcement. Since the 1970's major pier rehabilitation has been required each decade. A problematic aspect of pier rehabilitation activities is that many of the repairs have been non-structural, masking the seriousness and degree of pier deterioration.

Date	Description
1945	Repairs to Bearings and Piers
1972	Replacement Bearings
1974-1975	Repairs to Piers 6 and 7
1974-1977	Repair of Concrete Bridge Deck and Curbs
1978-1979	Repair of Concrete Bridge Deck and Curbs
1982-1984	Repairs to Piers 5 and 8
1990-1991	• Maintenance Painting & Bearing Rehabilitation • Deck Replacement with Lightweight Grid Deck • Concrete Repairs to Piers and Abutments • Addition of Pier 4 Post-Tensioning Bands
1995	Replacement of Vermont Abutment Bearings
2005	Pier Post-Tensioning Band Replacement
2008	Pier Concrete Repairs
2009	Steel Repairs and Strengthening

Recent Biennial Inspections

The most recent biennial inspection of the Lake Champlain Bridge occurred in spring 2009 and was performed by Chas H. Sells, Inc. During this inspection the number of reported red, yellow, and safety flags increased dramatically. The yellow flag identified in 2007 for concrete deterioration of pier 3 was repaired prior to the 2009 inspection. All 2009 flags were conditions not previously flagged during inspections, illustrating an increased rate of deterioration of the bridge. Two of the yellow flags were directly related to the conditions of the piers, including the deterioration of piers 6 and 7 post-tensioning bands added in a previous repair contract to address vertical cracking of the existing piers.

Year	Yellow Flags	Red Flags	Safety Flags
2005	1	1	0
2007	1	0	0
2009	20	4	1

In addition to the recent biennial inspections, a diving inspection was performed in summer 2005 and an in-depth inspection was performed in fall 2007. The diving inspection investigated the conditions of piers 4, 5, 6, 7, and 8 below the water line. Widespread deterioration was noted, including map-cracking, scaling, and spalling. Deterioration at the water level was noted up to approximately 4 inches deep. The report recommended repairs to the cracks identified at piers 6 and 7 and repairs of the abrasion damage to all piers at the waterline. These repairs were anticipated as part of the ongoing project to rehabilitate the bridge. Diving inspections are required every five (5) years; therefore, the next diving inspection was scheduled for the summer of 2010.

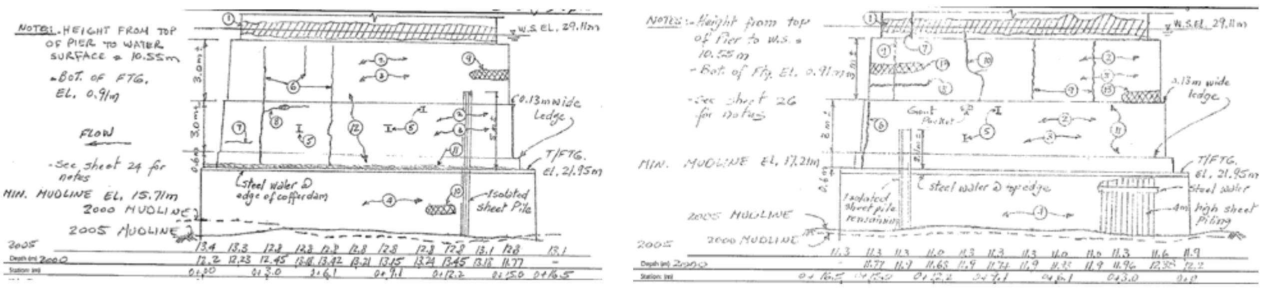
At the time of the bridge closure on October 16th, 2009, the bridge was open to a single lane of traffic and posted for 40 tons, in order to address the many yellow and red flags issued during the 2009 biennial bridge inspection. With closure to the bridge on October 16th, 2009, these repairs were never completed.

Safety Assessment & Basis for Bridge Closure

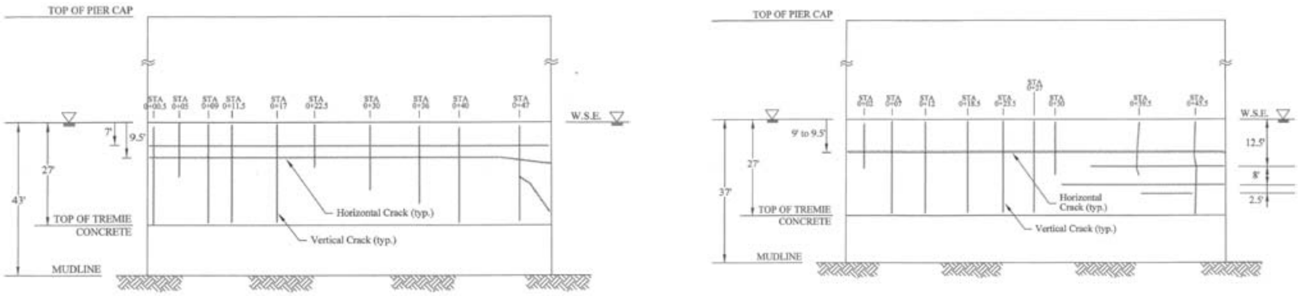
Deterioration at Pier 8

As had been highlighted in previous inspection reports, piers 5, 6, 7, and 8 exhibited severe deterioration of the existing concrete at the water level. A drop in the water level in mid September of 2009 exposed surface deterioration that was much worse than had been previously noted. Upon further investigation, together with concrete coring in this region, the water piers exhibited approximately 30% section loss (a depth of nearly 460mm, as compared to 10mm noted in previous inspections). Cores taken at the waterline also indicated the presence of horizontal and vertical cracks deep into the piers (between 750mm to 900mm from the face of concrete). This degree of deterioration, together with an evaluation of the safety of the piers under live load and temperature/wind loads described in more detail below, the bridge was closed to traffic on October 16th, 2009, and the design of emergency repairs begun. In concert with this design effort, a dive inspection and pier monitoring were implemented to further assess the degree of damage to identify the extent and degree of proposed repairs.

Consistent with the increase in waterline deterioration, the diving inspection completed in late October 2009 revealed similar signs of advanced distress below the waterline. The sketches in the figures below compare the results of the 2005 diving inspection with emergency inspection. The sketches are for Pier 7, and similar conditions were observed at Pier 5. The increase in degree and severity of the vertical and horizontal cracks below the water level was particularly troubling and reconfirmed our decision to recommend bridge closure. This deterioration has occurred in a very short period of time, with significant degradation in the past 5 years, particularly the depth of abrasion damage at the water line and the degree, together with the seriousness and distribution of concrete cracking below the water line.



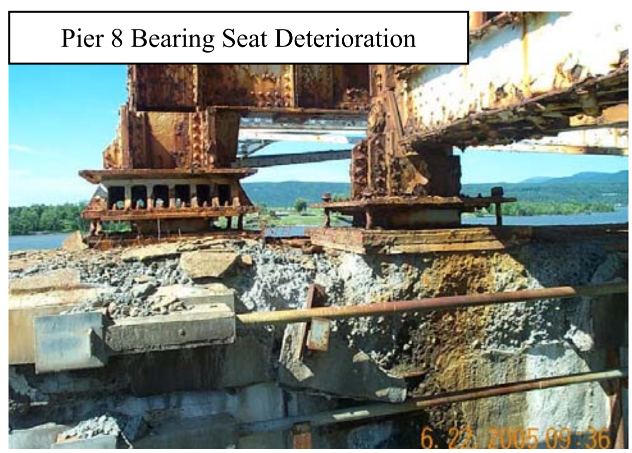
Pier 7 - 2005 Diving Inspection



Pier 7 - 2009 Diving Inspection

To complicate circumstances, most of the bearings for the water piers were not functioning as intended, with bearings frozen and severe deterioration at the bearing seats. Of particular concern was the potential for restraint forces associated with temperature change to further induce damage to the deteriorated substructure elements. Pier monitoring installed at Pier 5 (expansion pier) confirmed that the expansion bearings were frozen and that significant displacements were associated with temperature change. Subsequent analyses outlined below, considered the impact of frozen bearings and pier force demands.

Pier 8 Bearing Seat Deterioration



Structural Evaluation - Caissons

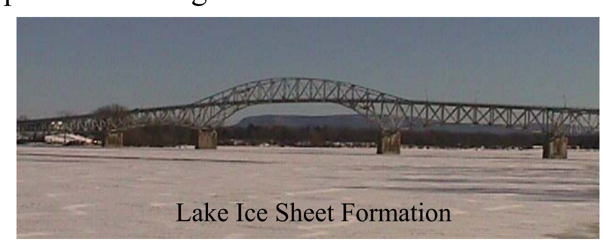
The water piers of the bridge, piers 4 through 8, are founded on plain (unreinforced) concrete caissons bearing on bedrock at varying elevations. As a point of reference, the approximate mean water elevation of the lake is 29.3m. The tops of the caissons are typically located 1m below the lake bottom, ranging in elevation from 22m for piers 6 and 7 to 27m for piers 4 and 9, such that the top of the caissons range from 3m to 7.5m below water and are not visible during a diving inspection.

Concrete for the caissons was placed in the wet using a drop bottom bucket; an unusual technique that warranted a detailed description by Spofford. The aggregates (both coarse and fine aggregates) used in the pier and caisson concrete came from nearby iron mine tailings, in Mineville, NY. Iron ore was separated from the surrounding rock using a magnetic separator, with less than 10% of the iron remaining in the rock after processing. While testing of the resulting concrete showed it to be unusually strong, the use of iron mine tailings for coarse and fine aggregate is also extremely unusual.

Given the drop bottom bucket technique, a significant portion of the concrete must be exposed to water during placement. This would result in the localized formation of a thin layer of laitance during each bucket placement which would then be buried by subsequent placement of concrete. Reportedly, underwater inspections performed during construction showed little or no laitance (Spofford, 1931). It is difficult to assess how this placement technique impacted concrete strength and the potential for the formation of cracks or zones of weakness in the caisson concrete.

To assess the capacity of the caissons, lateral support due to the soil was neglected due to its poor quality (Spofford, 1931). The capacity of a pier, comprised of a pier stem and caisson, will be controlled by one or the other. The ability of the caissons to resist design loads is particularly critical in evaluating the safety of the structure, given that caissons in deep water below the mudline are extremely difficult and expensive to retrofit.

In their as-built condition, with the bearings functioning as designed, the pier caissons were found to have sufficient capacity for gravity (dead) loads. It is important to note that, under gravity loading the piers are subjected to axial compression alone. Their fragility as unreinforced concrete elements becomes apparent only under lateral loads. In terms of longitudinal loading, only pier 6 of the water piers supports fixed bearings and must resist all longitudinal loads transferred from the superstructure (with the assumption that the bearings are functioning as designed). These longitudinal loads result from wind, seismic and braking forces. With the bearings performing as originally designed, there are minimal longitudinal forces from temperature resulting from friction on the adjacent piers. The remaining caissons that support expansion bearings are most sensitive to loads applied longitudinally directly to the piers, such as thermal ice and vessel collision.

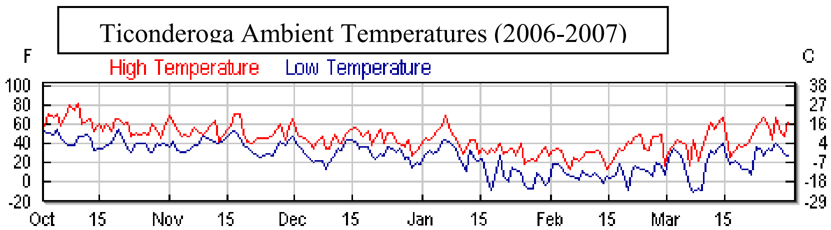


Lake Ice Sheet Formation

Static pressure due to thermal movement of ice sheets is relatively unusual and is associated with the behavior of ice in lakes or reservoirs. The more common case for bridge pier design is dynamic ice loading which is associated with flow in rivers. In accordance with current AASHTO LRFD specifications,

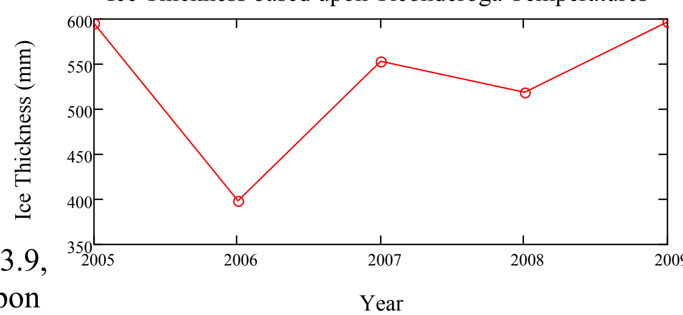
3.9.3 Static Ice Loads on Piers – Ice pressures on piers frozen into ice sheets shall be *investigated where ice sheets are subject to significant thermal movements relative to the pier where the growth of shore ice is on one side only or situations that may produce substantial unbalanced forces on the piers.*

Under thermal movement of ice sheets, the pressures generated can be large, on the order of 100kN/m to 150 kN/m based upon ice sheet measurements for dams (G. Comfort, et al, 2000) that are consistent with ice thicknesses experienced on Lake Champlain. For dams, static ice forces that exceed 300 kN/m are possible, particularly in circumstances where water level fluctuations result in cracking and refreezing of the ice sheet.



The graph depicts temperatures for winter months in 2006-2007. The high variability illustrates conditions conducive for ice formation followed by a rapid thaw period in which ice can break apart and ice floes form. For the static ice condition, the formation of a large thick ice sheet between the shore and piers represents the most severe case.

Ice Thickness based upon Ticonderoga Temperatures



Average ice thicknesses calculated per AASHTO LRFD Section 3.9, are depicted in the figure below based upon temperatures in Ticonderoga for the last four (4) years. Ice thicknesses computed in this manner are approximate; it is preferable to use local measurements where available. Given that this portion of Lake Champlain is a popular ice fishing location, we are fortunate to have independent measurements of ice thickness. Local accounts observe maximum ice thicknesses on the order of 600mm to 750mm of ice encountered each year during peak fishing season.

Ice Fishing at Bridge Site

Another key consideration is the shoreline configuration and the presence of approximately 100 ft of mud flats along both the Vermont and New York shorelines. In many cases, static ice pressures are limited by localized failure of the ice at the shoreline, particularly where the banks slope gradually upward, with resulting shoreline modification. In the case of mudflats, it is highly likely that the ice sheet and shoreline bond remains intact, and the ice sheet is thereby able to impart significant forces to the piers under thermal expansion. There are no indications

Shoreline Modification (Comfort, 2006)

of shoreline modification, consistent with an effective ice-sheet shoreline bond. This is consistent with the observations for Lake Champlain shoreline modifications associated with thermal ice expansion (Wagner, 1970).

Given the slenderness of the caissons, the type of failure they are most susceptible to in their original condition is overturning. This type of failure is addressed in the current AASHTO Specifications (both Standard and LRFD) by limiting the eccentricity of the axial compression at the bottom of the footing.

Section 10.6.3.3 of the AASHTO LRFD Specifications requirements gives a maximum eccentricity (e) for spread footings on rock of 3/8xB, with B taken as the length of the footing in the direction under consideration. From this requirement, the maximum longitudinal loading can be determined based on the axial force in the caisson due to dead load (P), the distance from applied load to the bottom of the foundation (d), and the maximum eccentricity (e).

$F_{capacity} = Pe/d$, where e is taken as 3/8B

The table below shows the maximum applied loads at the top of the substructure in the case where the bearings are functioning (free) and not functioning (frozen). Loads shown in red are instances when the caisson capacity has been exceeded. The AASHTO LRFD Strength III load case, dead load + wind load, governs for F_{top} when the bearings are free; the Strength I load case, dead load + live load + temperature load, governs when the bearings are frozen. The magnitudes of the F_{top} values correspond to roughly 10-15% of the dead load reaction.

Capacity (kN)	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8
F_{top}	1010	1730	2650	2650	1850
$F_{waterline}$	1520	2970	3760	3760	3260

Two different ice loadings were considered: a lower level loading corresponding to moderate level of thermal ice and a higher level loading corresponding to a peak thermal ice loading. The caissons in their original condition have inadequate capacity under factored dead and live load when the bearings are frozen. Pier 4 has inadequate capacity for both levels of ice loading.

Design Load (kN)	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8
F_{top}- free	0	0	1860	0	0
F_{top} - frozen	1410	1930	2760	2760	950
$F_{waterline}$ – moderate	1560	1560	1560	1560	1560
$F_{waterline}$ –	2850	2850	2850	2850	2850

Since the top of the caissons are generally below the lake bottom, evaluating their condition requires extensive excavation. At pier 7, however, the caisson extends about 1.5m above mudline. An inspection of this caisson performed on Oct. 30, 2009 indicated that there are vertical cracks ranging in width from hairline to 5mm. Laitance was also observed, conflicting with the diving inspection performed during construction.

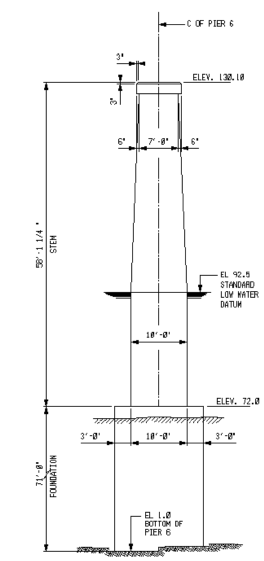
The current capacity, if the conditions at pier 7 are assumed to be representative of all the caissons, is impossible to predict. Given as-built conditions for the caissons, a rehabilitation of the pier stems could only increase the capacity of the pier to match that of the caissons. Prior to such an undertaking, some assessment would be required of the caissons to evaluate their current condition. This would require excavating to the extent possible, inspecting and mapping of cracks, and coring the caisson concrete. Due to the drop bottom bucket method of placement, the quality of the concrete at the time of construction was likely inferior to that of the pier stems. Based on the observed condition of the caisson at pier 7, it is believed that the level of deterioration of the caisson concrete is comparable to that of the pier stems, with the exception of the localized damage at waterline.

Unreinforced Pier Shafts

The use of unreinforced concrete piers for major truss bridges was an unusual practice by the late 1920's, with many bridges of similar size and span incorporating a minimum amount of reinforcement. Two such examples that were contemporary with the construction of the Lake Champlain Bridge, the Pulaski Skyway in New Jersey and the Cape Girardeau Bridge in Missouri, both continuous truss bridges with similar spans over water, have piers constructed of reinforced concrete.

The American Association of State Highway Officials (AASHO, a precursor to the present day AASHTO) provisions were not adopted until the early 1930's, however, development of the specifications began in 1921 and they were widely distributed by 1931. Therefore, they are representative of design practices at the time of the design of the Lake Champlain Bridge and are consistent with other relevant handbooks on concrete construction that pre-date the formal adoption of AASHO provisions. Below are a number of excerpts from the 1935 AASHO Standard Specifications for Highway Bridges (Second Edition):

- *3.4.12 Concrete Exposed to Sea Water – Concrete exposed to the action of ice, drift, or other forces producing shock and abrasion shall be protected by encasing that portion of the surface so exposed with a special sheathing or protective armor as shown on the plans or as noted in the supplemental specifications, and provision shall be made in the size of the original cofferdam for sufficient clearance to permit access to the concrete surface for the installation and effective anchorage of this sheathing.*



PIER 6 - EXISTING SIDE ELEVATION

- *5.5.5 Piers – Piers shall be designed to withstand dead and live loads, superimposed thereon; wind pressures acting on the pier and superstructure; and forces due to stream current, floating ice and drift; and tractive forces at the fixed end of spans. Where necessary, piers shall be protected against abrasion by facing them with granite, vitrified brick, timber, or other suitable material within the limits of damage of floating ice or debris*

- *5.7.10 Columns – The ratio of the unsupported length of a column to its least* *dimension shall not exceed 4 for unreinforced and 15 for reinforced concrete sections… The reinforcement of columns shall consist of at least 4 longitudinal bars tied together with lateral ties or hoops enclosing the longitudinal reinforcement. The longitudinal reinforcement shall not be less than 1 inch in diameter and shall have a total cross sectional area of not less than 0.7% of the total cross-sectional area of the column.*

Pier slenderness for all the pier stems of the Lake Champlain Bridge exceeds the slenderness limit of 4 for unreinforced concrete. Even the use of standard batters of 1/2" per ft (~40mm/m) which was consistent with the practice of the time for highway bridges was not strictly followed, with the batter stopping at elevation +28.2m, when a 3m width in the longitudinal direction was achieved and remained constant to the top of caisson (elev. 22m for piers 6 and 7, and 24.4m and 26.8m for piers 5 and 8 respectively). Of particular concern is pier 6, where all longitudinal forces from wind, live load, and seismic are transmitted via fixed bearings for the main span unit.

The pier stems are sensitive to the same load cases as the caissons. As with the caissons, the as-built capacity of the piers under gravity loading was found to be sufficient, and the concerns regarding the plain concrete construction apply to the piers as well. The capacity of the piers was calculated in the same manner as the caissons. Although they are not spread footings, the fact that there is no positive connection between the pier and the caisson makes them susceptible to the same type of failure (overturning). The application of this approach produces the capacities shown below.

Capacity (kN)	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8
F_{top}	680	890	1220	1220	980
$F_{waterline}$	2160	2560	3450	3460	5600

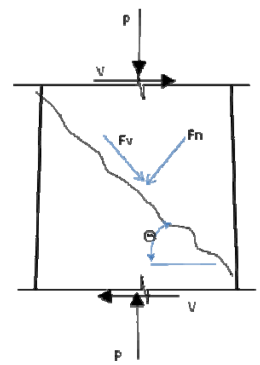
Again, the table below shows loads applied to the substructure when the bearings are free to function as designed. Loads shown in red are instances when the pier stem capacity has been exceeded. Similar to the caissons, the AASHTO LRFD Strength III load case, dead load + wind load, governs for F_t when the bearings are free; the Strength I loadcase, dead load + live load + temperature load, governs when the bearings are frozen.

The piers are therefore under capacity for dead and wind loading when the bearings are operating in their original condition, as well as dead and live loading with the bearings frozen. In addition, piers 4 and 5 are inadequate for the higher level ice forces. This evaluation assumes as-built conditions and does not account for the significant waterline deterioration and freeze thaw damage which has resulted in loss of sound concrete to depths that exceed 450mm around the entire pier perimeter.

Loading (kN)	Pier 4	Pier 5	Pier 6	Pier 7	Pier 8
F_{top}- free	0	0	1860	0	0
F_{top} - frozen	1410	1930	2760	2980	950
$F_{waterline}$ – moderate	1560	1560	1560	1560	1560
$F_{waterline}$ – maximum	2850	2850	2850	2850	2850

Cracking and the Impact on Pier Capacity

The typical approach for calculating the shear capacity of concrete members, by summing the individual contributions to shear capacity made by both the concrete and steel, cannot be applied to unreinforced members. The concrete contribution is based on empirical data obtained from the testing of reinforced members. To estimate the shear capacity of unreinforced members, the principle tension is calculated and compared with the modulus of rupture. This approach does not apply to members once they have cracked. To estimate the post-cracked shear strength of the piers, a shear friction approach based on Section 8.16.6.4 of the AASHTO Standard Specifications was adopted. This approach is used for reinforced members in determining shear capacity across an interface.



The shear friction equation is:

$V_r = \phi_v V_n = A_{vf} f_y \mu$ (8-56A)

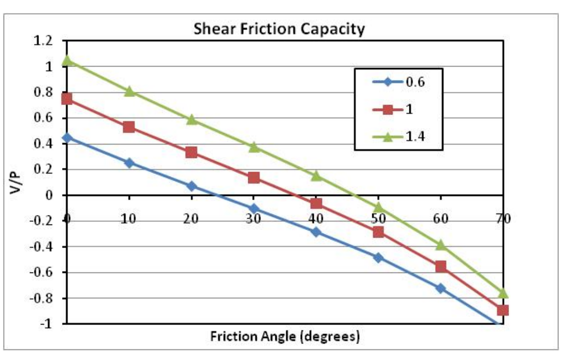
In the case of the piers and caissons, the normal force ($A_{vf}f_y$) of the above equation) is provided solely by axial compression. The code specifies friction coefficients ranging from 0.6 for construction joints where the concrete surface has not been intentionally roughened, to 1.4, to be used for post-cracked, monolithic concrete.

To apply this approach, a crack angle of inclination is assumed and the forces normal and parallel to the crack calculated from the applied loading, as shown. From this, equation 8-56A can be rewritten as: $F_v = \phi_v F_n \mu$ where $F_n = P\cos\Theta - V\sin\Theta$ and $F_v = P\sin\Theta + V\cos\Theta$. Substituting and solving for the ratio of shear capacity to axial compression gives the following:

$$V/P = (\phi_v \mu \cos\Theta - \sin\Theta)/(\phi_v \mu \sin\Theta + \cos\Theta)$$

A graph of V/P for varying crack angles and friction coefficients is shown below (only values up to 70 degrees shown):

The influence of both the friction coefficient and the crack angle is clear. Notice that if the crack angle is great enough, V/P becomes negative, which means the pier no longer has adequate capacity under dead load alone and collapse is imminent, noting that this assessment applies only to cracks extending through the pier cross section. The cracks that are vertical or near vertical in the transverse direction are not directly at issue.



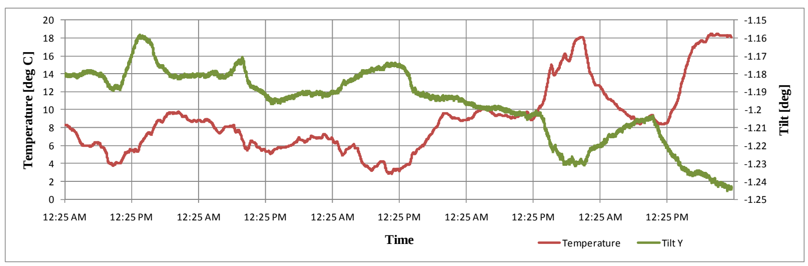
From the underwater inspections, horizontal cracks were found along both faces of all piers. From the locations of the cracks, it can be implied (though not definitively) that any through-thickness cracks are relatively flat. However, pier 7 in particular shows a number of horizontal cracks at varying elevations, making it difficulat to predict at what angle a through thickness crack may develop. Another uncertainty is the friction coefficient, though a lower bound of 0.6 is recommended given the size, distribution, and severity of the cracking together with the marine environment. From the above graph, it is clear that for cracks that exceed an angle of 20 degrees, failure can be anticipated for nominal lateral loads. Given that applied shears under static icing events approach 20% of the pier dead load (V/P=0.2) this is particulary concerning, as the through cracks at an angle as shallow as 10 degrees is at the strength limit.

Impact of Frozen Bearings

Tilt-meter / Accelerometer

Frozen bearings at the expansion piers, given the overall articulation of the structure, substantially increase the potential for pier instability. In addition, the presence of a robust ice sheet further exacerbates restraint and load demands on critical portions of the deteriorated piers. Restraint from the ice sheet, in addition to loads associated with thermal expansion of the ice sheet is of particular concern.

To assess the degree to which frozen bearings were influencing pier behavior, a triaxial accelerometer / bi-directional tilt meter was installed at pier 5. Installation of this remote sensor system was completed on November 4th, 2009. The monitoring system confirmed the top of pier 5 translates approximately 30mm for every 30°C of temperature change, and tracks . This behavior is strongly indicative of poor performance of the expansion bearings intended to isolate the piers from the thermal behavior of the superstructure.



Risk Assessment

The deterioration of the piers represents a significant decrease in the overall safety of the structure, particularly given the potential for localized failure to generate a catastrophic collapse, which could engage not only the main span unit but the approach spans as well. Given the structure's height above water, the depth of water for the main span unit and the lack of emergency equipment and personnel, catastrophic collapse would most likely result in multiple fatalities, even though the average daily traffic for the facility is less than 4000 vehicles per day.

One measure of ensuring the safety of bridges to the travelling public is the federal bridge inspection program, implemented after the Silver Bridge collapse in West Virginia in 1967. While bridge inspection and the use of federal funding to rehabilitate aging bridges has enhanced the safety of U.S. bridges, major bridge collapses still do occur, as evidenced from recent occurrences including the De La Concorde Overpass Collapse on September 30, 2006 in Quebec, Canada and the I-35 bridge collapse on August 1, 2007 in Minneapolis, Minnesota. Both bridge collapses are relevant to the safety assessment of the Lake Champlain Bridge, as both structures received in-depth inspections and were deemed safe and remained open to live load. Both bridges collapsed abruptly, and were not exposed to any unusual environmental loads or heavy traffic loads at the time of collapse.

The use of reinforcement in concrete dates back to the late 1800's, and by the time of the design of the design of the Lake Champlain Bridge, the use of reinforcement in concrete was typical of pier construction. The designer's choice of plain concrete, particularly for such slender piers, is difficult to justify. One of the key advantages of reinforcement in concrete is the ability to develop flexural strength and ductility (i.e. to avoid brittle failure). Minimum reinforcement requirements were specified very early in the code development of reinforced concrete, where sufficient reinforcement to develop the cracking moment is typical. Otherwise, under flexure, abrupt failure can occur. Scale effects, whereby larger concrete members show more brittle response as compared to smaller beams, have been the focus of recent research and modern codes, recognizing the need for additional reinforcement in large members to avoid explosive behavior.

The collapse of the De La Concorde Overpass near Montreal was an abrupt shear failure of a thick slab without shear reinforcement. The cantilever slab was over 4 ft thick and supported a precast concrete drop in span. Forensic investigations demonstrated the brittle nature of the shear failure of the slab as well as freeze thaw deterioration of the concrete in the vicinity of the failure plane. The cantilever slab met the applicable design requirements at the time of construction.

For the unreinforced concrete piers of the Lake Champlain Bridge, the potential for similar abrupt failure cannot be ruled out. Freeze thaw deterioration is continuing to damage the piers at water level. Lake icing and thrust associated with thermal movements of the ice sheet can produce large horizontal loads in the piers, well beyond their design capacity. Additionally, frozen bearings are introducing longitudinal forces into piers 5 and 8, which have the most serious deterioration at the water level. The potential for any of these loads, individually or in combination, to precipitate pier failure and collapse of the structure cannot be ruled out.

At the time, one of the innovative features of the superstructure design was the use of a continuous system, which has a number of advantages from the perspective of structural efficiency. A clear down-side to this structural system is its sensitivity to damage, even localized damage, which could result in destabilizing the entire superstructure system. The dramatic failure of the I-35 Bridge in Minneapolis on August 1, 2007 due to the buckling and localized failure of a single gusset plate is clear evidence of the fragile nature of continuous truss systems. It is interesting to note that frozen bearings and shifting piers were evaluated as potential contributors to the I-35 collapse (though found to be relatively unimportant, in comparison to the gusset plate mechanism). Both frozen bearings and the potential for pier movement are significant concerns for the Lake Champlain Bridge.

The deterioration of the piers, resulting in relative instability as compared to modern reinforced construction, represents a major safety issue for the structure. Relative movement and/or localized settlement as a result of continued pier deterioration, is sufficient to cause collapse of the structure under its own weight, without the presence of live load or other lateral loads that might serve as triggering events.

Recommendation for Bridge Replacement

The deterioration and lack of safety at pier 5 under thermal loads with the bearings assumed frozen was the basis of closure of the bridge to live load on October 16th, 2009. At that time, the potential for pier 5 failure and subsequent collapse of a significant portion of the structure could not be ruled out. Further investigative work was conducted to assess the degree of deterioration below the water line, including a dive inspection and water line cores were taken from the remaining water piers to assess degree of deterioration, together with the implementation of a tilt-meter/accelerometer at pier 5 to assess behavior under thermal loads. These investigations, together with strength evaluation of the main substructure units pointed to a fragile structure, which had deteriorated rapidly over the past 4 years, and may not survive another winter.

Given the potential fragility of the bridge under wind, temperature and static ice loads, it was clear that restrictions to work activities would be necessary and the cost and complexity of any interim emergency repairs would put engineers and contractors at risk, particularly given that ice was beginning to form on Lake Champlain. In late November of 2009, it was decided that the proper course of action was to demolish the bridge in a controlled manner and to expedite replacement. The bridge was explosively razed on December 28th, 2009 and a replacement bridge is currently under construction on the existing alignment. The replacement bridge substructure elements incorporate granite to resist ice abrasion at the waterline and are sloped to reduce forces induced by ice sheet thermal expansion.

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