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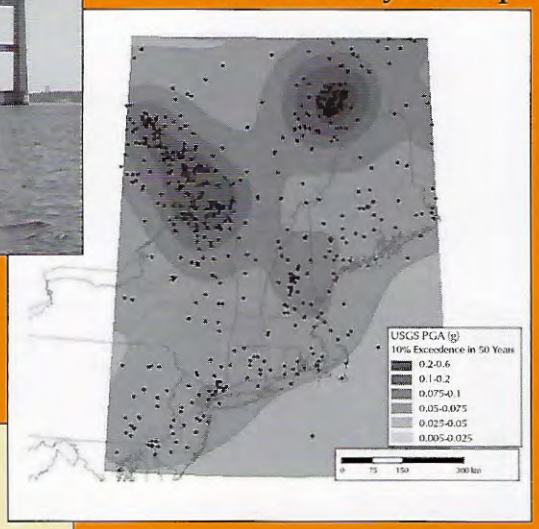
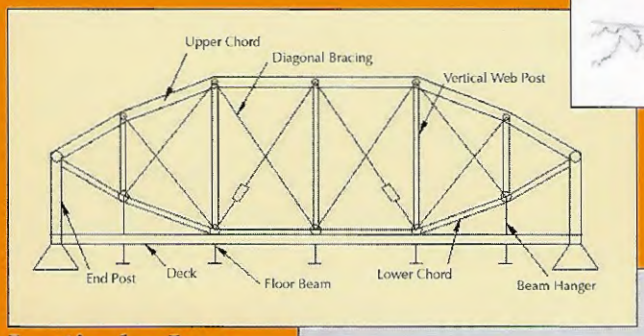
Suspension Bridges of New England

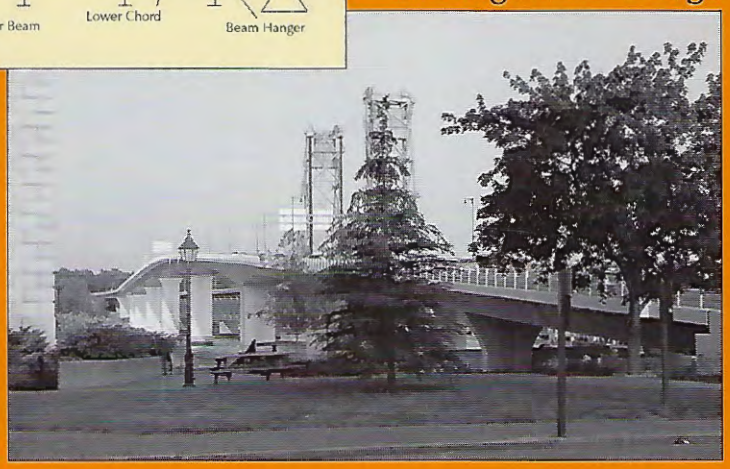
Using Custom Probabilistic Seismic Hazard Analysis Maps





Lenticular Iron Truss Bridges

Designing & Building the Sagadahoc Bridge



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Erratum:

The last paragraph at the end of the first column of Dr. Charles Ladd's article, "Anatomy of a Court Trial on Tank Settlements," in Vol. 19, No. 2 (Fall/Winter 2004) should read, "After being cross-examined for the entire next day, I was again put on the stand for redirect testimony. This time the defense lawyer posed a series of specific questions, mostly regarding how I obtained the E/s_u of 39, why this was so unusual, and why deleting the piezometers contributed to the problem. This testimony was, I believe, more effective in defending D&M."

Using Custom Probabilistic Seismic Hazard Analysis Maps Based on U.S. Geological Survey National Seismic Hazard Mapping Procedures

When site conditions fail to conform to an applicable design code, or when a variance to the code is desired, a customized probabilistic seismic hazard analysis map can be of great use.

RICHARD J. DRISCOLL & LAURIE G. BAISE

In seismically active regions, earthquake effects are often the controlling lateral load in the design of structures. There are a variety of methods used to determine the seismic forces that a structure may have to withstand. However, for many simple, non-critical and non-monumental bridges and buildings, design codes provide simplified procedures that engineers can use. Central to code-based seismic design are seismic hazard maps, which provide a ground motion parameter, usually an acceleration value, which is applied in some form as a lateral force to the structure.

Cornell introduced methods for evaluating seismic hazard probabilistically.[1] Uncertainty in the quantity, magnitude and source of earthquakes can be considered by expressing seismic risk in terms of the return period, in

the same way that wind and flood effects are evaluated in civil engineering. An estimated return period for a particularly damaging natural event can be used to rationally balance the risk of economic loss from that event and the cost of mitigating that risk. This approach typically results in more economical designs than the use of a "maximum credible" event as a design criterion.

Like meteorological events, the earthquake recurrence for a given source can be estimated based on the historical record. Earthquake prediction, however, involves the uncertainty of the temporal distribution of the event magnitude, as well as spatial distribution on multiple sources. The effect on a particular site of a given earthquake at some point on a source is also uncertain. Cornell developed solutions for evaluating the seismic hazard from a source using statistics.[1] By allowing the risk from all potential earthquakes on all potential sources to contribute to the seismic hazard at a site by weighting them according to probabilities, a rational, and possibly more realistic, risk analysis could be made. This type of analysis is called probabilistic seismic hazard analysis (PSHA).

The United States Geological Survey (USGS) developed its first national seismic hazard maps in 1976 and provided peak ground acceleration with a probability of exceedance of 10 percent in fifty years, which corresponds to a 474-year return period. Derivative forms of this map were found in the 1985 and 1988 editions of the Recommended Provisions for the Development of Seismic Regulations for New Buildings of the National Earthquake Hazard Reduction Program (NEHRP). The NEHRP provisions are the basis for seismic design provisions in most model building codes. Later USGS efforts included maps of peak acceleration and peak velocity, as well as design response spectra ordinates. These efforts were incorporated in subsequent editions of the NEHRP provisions, usually a few years after their development.[2]

Procedures used in developing the maps evolved between editions, with additions and refinements made after a consensus review process. The most recent procedures are described by Frankel *et al.*[3] and are based on previous studies,[4,5] with Frankel *et al.*'s 2002 maps forming the foundation of seismic design for the next several years.[3] The contiguous forty-eight states are split roughly at the Rocky Mountains and different procedures are used for the Central and Eastern United States (CEUS) and the Western United States (WUS). Earthquake causative mechanisms and wave propagation vary considerably between the active plate boundary areas of the WUS and the more stable interplate crust of the CEUS. WUS earthquakes are larger and more frequent, and they are more likely to occur on mapped geologic features than CEUS earthquakes. However, CEUS earthquakes are felt over a much larger area because the crust in that region is older and more intact. Multiple models incorporating major known faults, spatially smoothed historic seismicity and background seismicity zones are combined with multiple ground motion estimators to account for uncertainties in modeling.

The procedure used to develop the national seismic hazard maps can be adapted to form the basis for a customized PSHA. Ground motion parameters can be calculated to correspond to the probabilistic criteria that are unavailable in published maps. In addition, instead of using the generalized CEUS or WUS modeling assumptions, adjustments can be made to be more representative of a specific region. Due to the facility with which the USGS procedures can be modified, they can be used for regional- or project-specific analysis.

Procedure

National PSHA Maps. Seismic hazard maps are created by calculating the ground motion parameter with a certain rate of exceedance at discrete points across an area of interest. Figure 1 shows the logic tree for the CEUS seismic hazard calculation as it pertains to this study. Earthquakes are assumed to follow a Poisson probability distribution (in which the probability of some number of events occurring can be determined within a given time and can be determined with an assumed rate

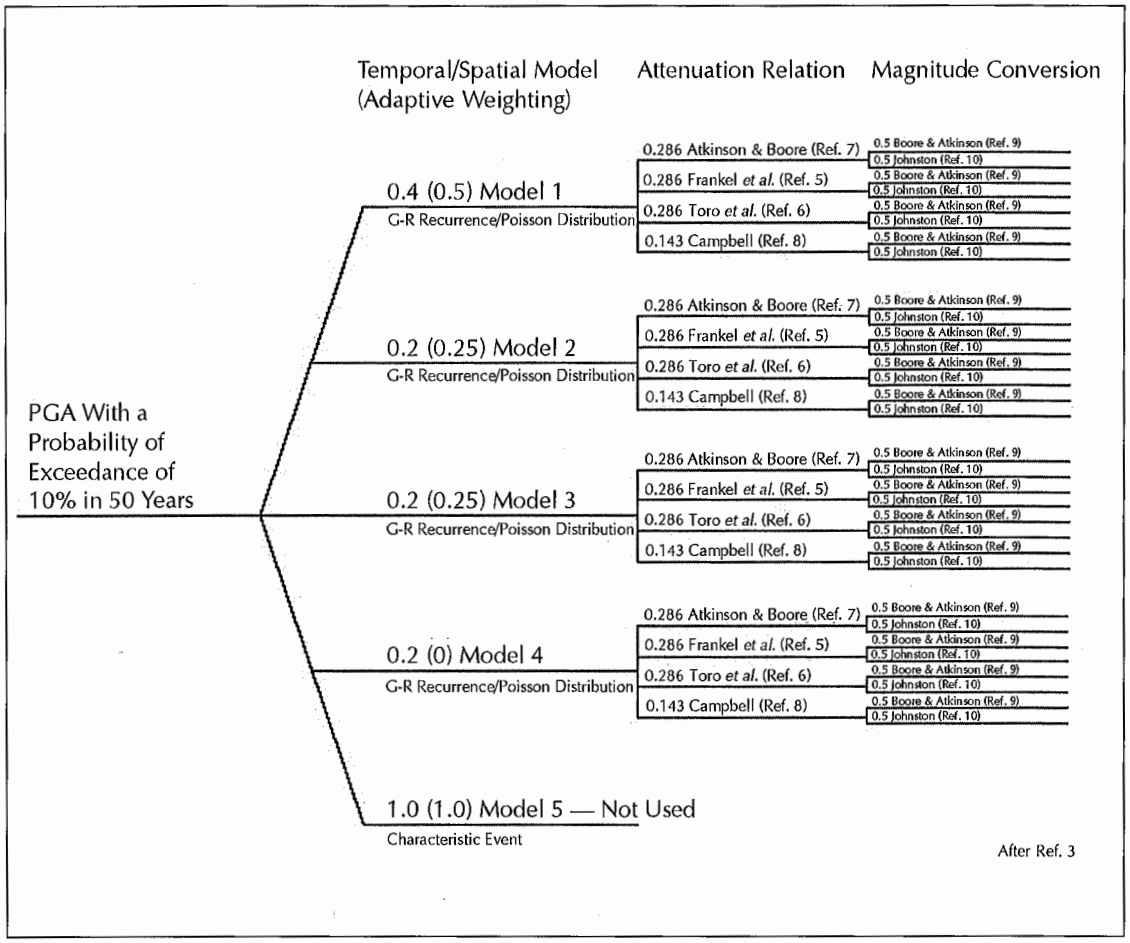

FIGURE 1. Logic tree for seismic hazard maps.

of occurrence). This distribution can be rearranged to define the probability of the exceedance of a ground motion value within a certain number of years, given an annual rate of exceedance (*i.e.*, the probability of exceedance of 10 percent in fifty years). Conversely, an annual rate of exceedance can be calculated for a specific probability of exceedance over a period of time. The peak ground acceleration (PGA) corresponding to this annual rate of exceedance can be calculated by iteration and can be plotted in order to create the seismic hazard maps.

The most recent version of the national seismic hazard maps, produced in 2002, used an earthquake catalog that included events through December 2001 to develop three historic seismicity models.[3] Model 1 consists of the smoothed, historic rate of events greater than *Lg* magnitude (m_{bLg}) 3.0 since 1924. Model 2 includes earthquakes greater than m_{bLg} 4.0 since 1860. Model 3 is based on m_{bLg} 5.0 and greater events since 1700. Model 4 is the background seismicity model that represents the hazard from moderate (m_b 5.0 to 7.0) earthquakes in recently inactive areas.[4] Background seismicity is considered only when the inclusion of Model 4 results in higher ground motion estimates than the historic seismicity models alone. The earlier Frankel *et al.* study superimposed a Model 5 with full weight to account for characteristic earthquakes of moment magnitude 7.0 and greater from four areas of high hazard, none of which would be expected to pose a significant risk to New England;[5] therefore, this model was ignored. The later study by Frankel *et al.* used a finite fault model for magnitudes of 6.0 and greater in calculating gridded seismici-

ty.[3] However, this procedure only affected very low probabilities of exceedance (less than 2 percent in fifty years), was not well described and was, therefore, neglected.

To account for catalog omissions and uncertainty in earlier earthquakes, the 2002 Frankel *et al.* study adjusted the historic seismicity rate for each model to the presumed "complete" seismicity rate.[3] This adjustment was made by multiplying the gridded seismicity by a seismic rate adjustment factor (SRAF) calculated by Mueller *et al.* for various areas, equal to the presumed complete rate divided by the observed or "counted" rate.[11]

The 2002 Frankel *et al.* study used four attenuation relations to estimate ground acceleration given an earthquake magnitude and distance.[3] A fifth relation was used for Model 5 for the USGS maps. Each attenuation relation used slightly different assumptions, particularly the choice of magnitude scale and site conditions. The 2002 Frankel *et al.* study applied a weight of 0.286 to Toro *et al.*,[6] the earlier Frankel *et al.* study[5] and Atkinson and Boore,[7] while a weight of 0.143 was assigned to the Campbell attenuation relation.[8] The Campbell attenuation relation attempts to correct errors in near source ground motions predicted by CEUS attenuation relations by using scaled WUS ground motion estimates near source.[8] Near source data are more abundant for WUS earthquakes because they are more frequent and better instrumented. This approach is novel and untested; therefore, it warrants a lower weight. For the attenuation relations requiring moment magnitude (M_w) as an input parameter, conversions by Boore and Atkinson[9] and Johnston[10] were used with equal weights to convert m_{bLg} or m_b to M_w.[3]

Custom PSHA Maps. Since this type of analysis makes intensive use of spatial data as well as involved, complicated and iterative calculations, computer software was necessary to automate the custom PSHA mapping process. Hundreds of events in the catalog had to be processed and data plotted at various stages. Therefore, a geographic information system (GIS) package was selected to process input and output data from the seismic hazard analysis. GIS allows information in databases to be plotted and manipulated spatially, making it very useful in plotting earthquakes, developing the catalogs for each model, checking processes based on spatial data and plotting results. To perform all calculations, programs were developed using a variant of the BASIC programming language, which is less powerful than other programming languages, but is more convenient to learn. These programs were executed in a commonly used spreadsheet program, thereby simplifying input and output. (While the BASIC variant was available for the GIS software, its use in the spreadsheet program was better documented, and it represented a more expedient choice.)

The maps in this study were based on a grid with 0.1° longitude by 0.1° latitude cells. The grid cell value was the weighted average of four recurrence models utilizing different sources and catalogs (as shown in Figure 1). The basic algorithm used in this study for developing the seismic hazard maps was as follows:

> For each historic seismicity model, the proper events were selected from the earthquake catalog. Using GIS, the number of events falling into each grid cell was counted. The cell counts were spatially smoothed using a programming subroutine. For the background seismicity model, the count over the entire source area was divided among all cells. The seismic hazard calculation program used the smoothed counts for each model to determine the rate of exceedance of a trial PGA value. The trial PGA was adjusted until the rate of exceedance converged upon that corresponding to 10 percent in fifty years. This process was repeated for all cells in the selected source region and for all models. The PGA values for all four models contributed to a total value, which was plotted as a map in GIS.[12]

The most recent earthquake catalog available from the USGS website was that used for the 1996 maps.[13] Aside from a few minor modifications mentioned by the 2002 Frankel *et al.* study, the only difference between the 1996 and 2002 catalogs is the inclusion of events from 1996 to 2001. Therefore, the 1996 USGS

catalog was used for this study with events added from the northeastern United States catalogs available from Boston College's Weston Observatory. This approach allowed the custom PSHA to consider earthquakes between January 1996 and April 2003.[14]

Due to the limited coverage of the Weston catalogs and the need for computational efficiency, a source area enveloping all events expected to contribute to seismic hazard in New England was defined. A large source area was desirable because seismic waves travel well in stable crust. In addition, excluding the hazard from events beyond the source area could cause the expected ground motion to be underestimated, particularly near the boundaries of the source area. Expanding the source area to include regions with an incomplete catalog would also result in errors. For convenience, a 10° longitude by 10° latitude area was used and was centered so as to include all of New England, as well as surrounding regions of Québec and New York. The source area was defined between –77° and –67° longitude and between 39° and 49° latitude.

The cell counts for the spatially smoothed historic seismicity models were determined using GIS. A grid was constructed using a computer-aided drafting program. This grid consisted of a square array of 10,000 squares. Longitude and latitude were used as the x and y axes, respectively, and the squares were sized just under 0.1° by 0.1° to prevent overlap (which would confuse the analytical abilities of GIS). The grid was imported to the GIS package in the same coordinate system and converted to a shape file that consisted of a database of polygons. A point file was created that defined the coordinates of the midpoint of each cell. This point file was joined with the grid shape file to define the coordinates (in longitude and latitude) of the midpoint of the grid as an attribute of the cell. Finally, the grid was shifted an incremental amount to the southwest because a large number of earthquakes were falling on the boundaries of the polygons and were either double counted or ignored by GIS.

The grid defining the source area and the events selected for each historic seismicity model are shown in Figure 2. Events greater than magnitude 3.0 since 1924 were selected for Model 1. The selection was exported as a separate database file. This process was repeated for magnitude 4.0 and greater events since 1860 for Model 2 and magnitude 5.0 events since 1700 were compiled for Model 3. The database files for each model were joined to the grid to create a shape file with the number of events from the database within each cell. The event counts for each historic seismicity model were exported to the spreadsheet where they were manipulated by the programming modules.

A subroutine was written (after Frankel[4]) to spatially smooth the raw counts for each model. For every cell, the subroutine calculated a new value for the cell count that was effectively an average of the raw cell counts for the surrounding cells within a certain radius weighted according to the distance to the original cell. If the distance from one cell to another was less than or equal to the correlation distance, then the count for that nearby cell was included in calculating the smoothed count. Frankel used the following Gaussian smoothing function to calculate the count for each cell:[4]

$$n_i = \frac{\sum_j n_j \bullet e^{\frac{-\Delta_{ij}^2}{c^2}}}{\sum_j e^{\frac{-\Delta_{ij}^2}{c^2}}}$$

The smoothed count for cell i, n_i, is therefore the sum of the raw count in cell j for all values of j, multiplied by the exponential term and divided by the sum of the exponentials, where Δ_{ij} is the distance between cells i and j and where c is the correlation distance. Following the earlier Frankel *et al.* study,[5] a correlation distance of 50 kilometers was used for Model 1, while a correlation distance of 75 kilometers was used for Models 2 and 3. This count was printed along with the cell's longitude and latitude on a new worksheet

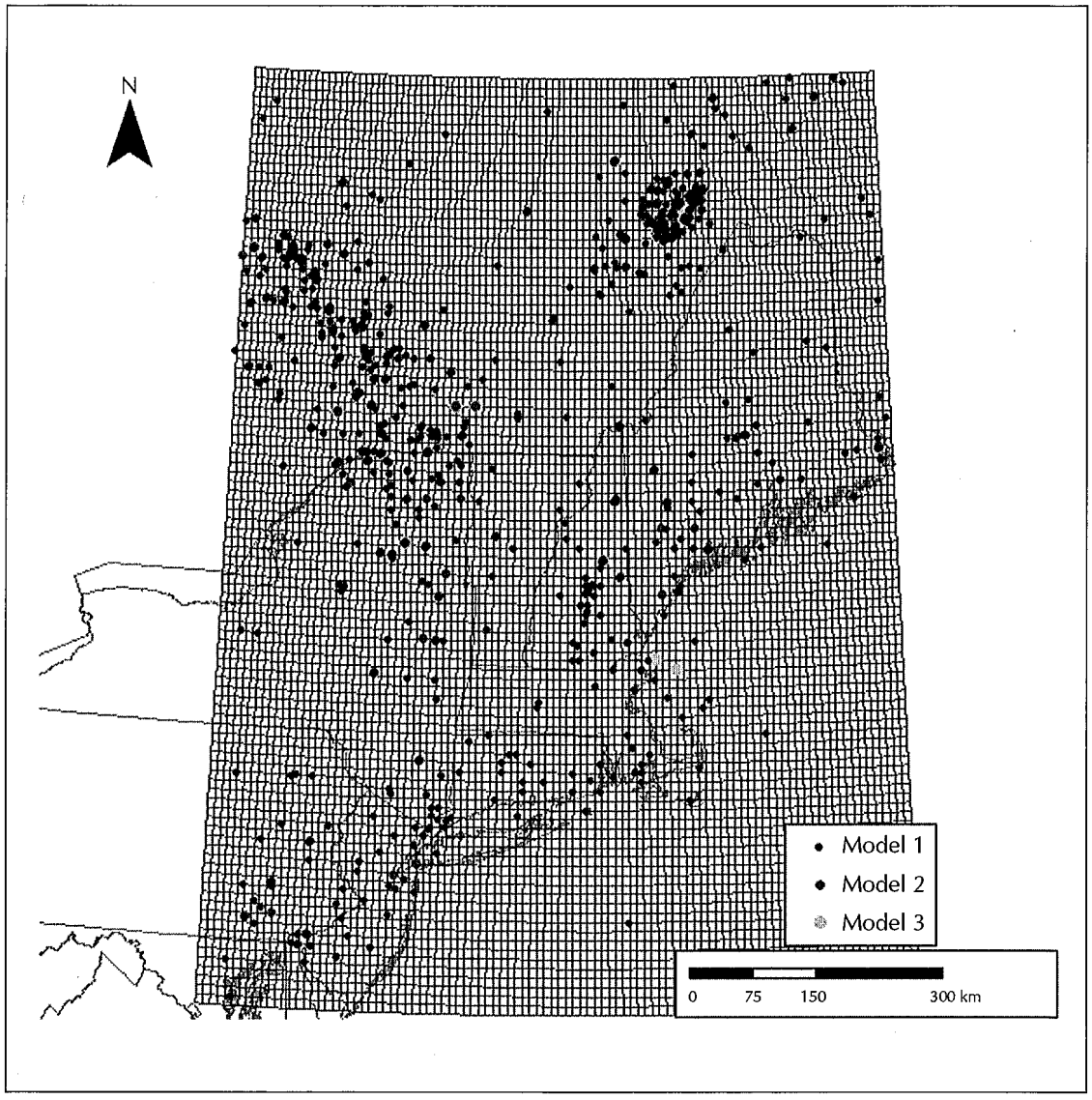

FIGURE 2. Events included in historic seismicity models.

for later use by the seismic hazard subroutine.

The seismic hazard subroutine was run for each model to collect the cell coordinates and smooth the count. The user was prompted for the time period and the minimum magnitude of the catalog for the model and for the SRAF. The source area for this study overlaps regions with different SRAFs. For this study, the SRAFs derived by Mueller *et al.* for the east coast of the United States were used for the entire source area.[11] Factors of 1.27, 1.15 and 1.58 were multiplied by the smoothed counts for Models 1, 2 and 3, respectively, in order to adjust the seismicity. Different factors were used by the 2002 Frankel *et al.* study north of the St. Lawrence River.

The subroutine looped through all 10,000 grid cells and called a function that determined the PGA value with a probability of exceedance of 10 percent in fifty years at each grid cell. The function selected trial ground motions and calculated its rate of exceedance, which was then compared to the rate of exceedance corresponding to 10 percent probability of exceedance in fifty years using a

Poisson distribution. The annual rate of exceedance, λ, for a ground motion, u_0, was calculated for a specific cell after Frankel, as follows:[4]

$$\lambda(u>u_0) = \sum_l \sum_k 10^{\left(Log\left(\frac{N_k}{T}\right) - b(M_l - M_{ref})\right)} \times P(u>u_0 \mid D_k, M_l)$$

This equation represents the sum of the product of the annual rate of exceedance and the probability of a ground motion exceeding u_0, for discrete distance and magnitude bins represented by distance k and magnitude l, respectively. The annual rate of exceedance is a form of Gutenberg-Richter recurrence law, which expresses the logarithm of the rate of exceedance as a linear function. The "*a* value" is the intercept calculated by taking the logarithm of the adjusted, binned count, N_k, divided by the catalog time period, T. The "*b* value" in the recurrence law is the slope of the equation and is independent of the magnitude used to calculate the *a* value. The early Frankel *et al.* study used a *b* value of 0.95 for the CEUS, except for a 40- by 70-kilometer ellipse around Charlevoix, Québec, where the value of 0.76 was used.[5] Since ground motions for Charlevoix were not of interest to this study and the area in which the lower *b* value was used was not defined, a constant value of *b* equal to 0.95 was used. Three magnitude bins were used to consider events between m_{bLg} 4.5 and 7.5, represented by magnitudes of m_{bLg} 5.0, 6.0 and 7.0. Distance bins of 10-kilometer increments were used to cover distances up to 500 kilometers and represented by the midpoints of the 10-kilometer intervals.

For Model 4, the entire source area was treated as a uniform source zone.[4] Therefore, the ground motion for every cell from this model is equal. The 1996 Frankel *et al.* study constructed this model by calculating the *a* value from all CEUS m_b 3.0 and greater events since 1924 adjusted to the post-1976 seismicity rate.[5] That study normalized this count by area and disaggregated so an existing seismic hazard code could be used without modification. For this study, the number of m_{bLg} 5.0 and greater events were counted since 1924 for the New England source zone. This reference magnitude was selected following the suggestion by Frankel that the reference magnitude and time interval selected by the 1996 Frankel *et al.* study for Model 4 underestimated the hazard of large events. Theoretically, there should be no difference between the two approaches to Model 4 because the logarithm of the annual rate of exceedance for earthquakes of a given magnitude varies linearly with magnitude. These nine unadjusted events, normalized by dividing by 10,000 cells and the average cell area (which was estimated to be 88 square kilometers) represented the cell count. Since the count was constant for all cells, no smoothing was required. The same general procedure used for the historic seismicity models was used for Model 4, but calculations were only performed at two locations to ensure that the values were the same. In addition, the distance bins were replaced with a function that multiplied the normalized cell count by a function that estimated the area of the bin.

The gridded ground motions for each model were factored according to Figure 1 to create the largest weighted average, which was plotted spatially in GIS. This plot resulted in contour maps that could be directly compared with the 2002 USGS data at the same locations, allowing an analysis of the procedure.

Results

Comparison of the New England map with the published USGS map demonstrated the conformance of the procedure used in this study with that used to develop the national seismic hazard maps, as well as the effect of differences between the two procedures. The gridded seismicity database from the 2002 USGS data was utilized to provide the primary graphical and numerical comparisons with the results from this study.[15] Figure 3 is the total seismic hazard map for New England as calculated in this study.

Figure 4 is a plot of the 2002 gridded PGA values from the national seismic hazard maps.

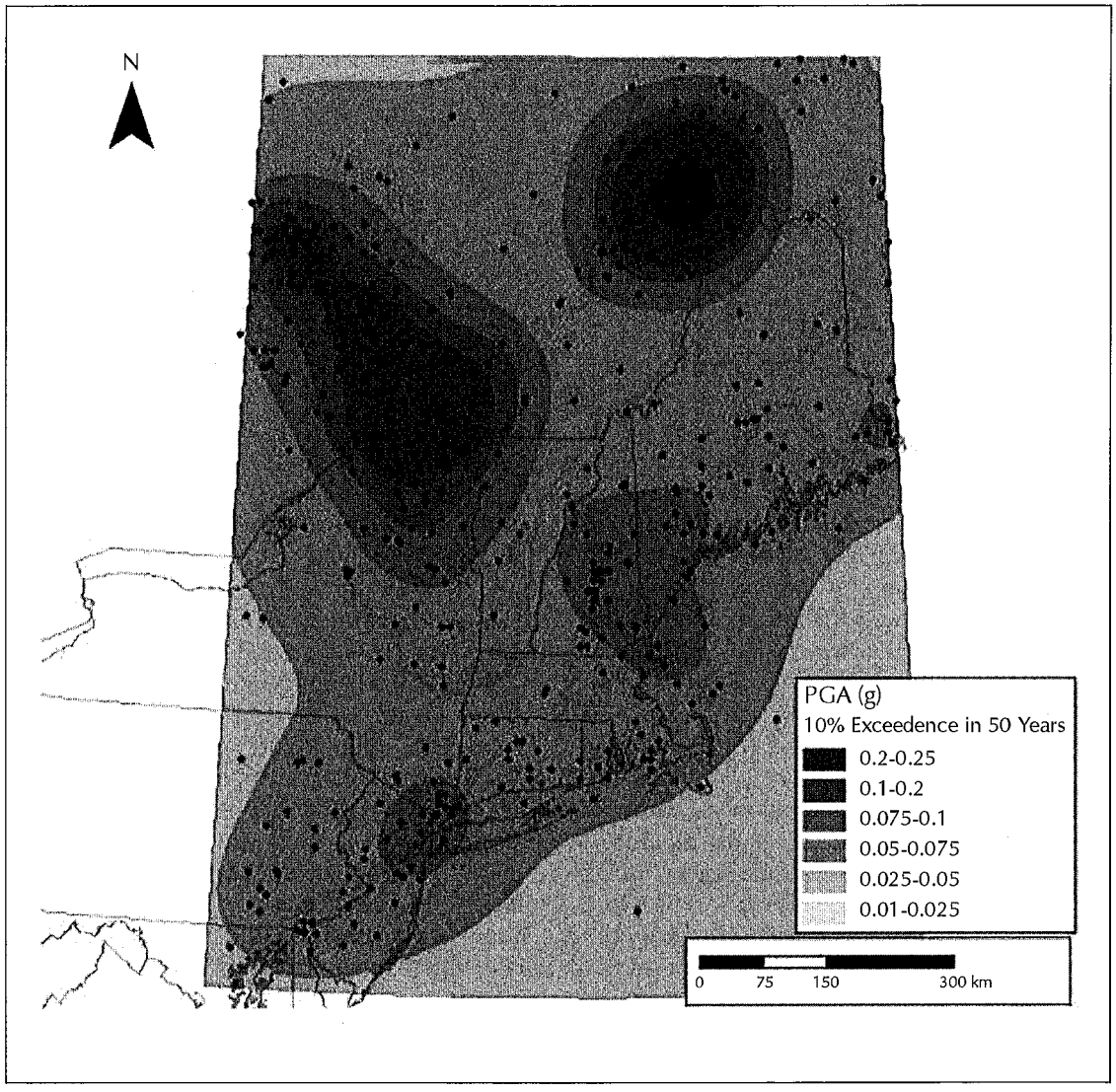

FIGURE 3. Total seismic hazard, adjusted historic seismicity.

The similarity between the maps shows that the procedure used to develop the New England map produced results very similar to the 2002 USGS map. The peaks centered in southern New York and New Hampshire shown in Figure 3 are reflected by similar peaks in Figure 4 of slightly different size and shape. The New York peak is smaller in radius in Figure 4. The 5 percent g contour of the New Hampshire peak does not extend as far east or southeast in Figure 4 as in Figure 3, but it does extend northwest to connect with the northeast-southeast peaks from the northern Adirondacks into Québec. In Figure 4, this peak area is connected to the Charlevoix peak by an area between 5 percent g contours. The Charlevoix peak has a large area of PGA greater than 27 percent g in Figure 4 (which is not observed in Figure 3). Figure 4 lacks a peak at Passamaquoddy Bay, Maine. The general trend is that the maps from this study, as seen in Figure 3, show slightly higher PGA in the southern portions of the map and lower PGA in the northern parts of the source area compared to the 2002 USGS maps.

The differences between the 2002 USGS gridded PGA values and the cell values plotted in Figure 3 were calculated and are plotted

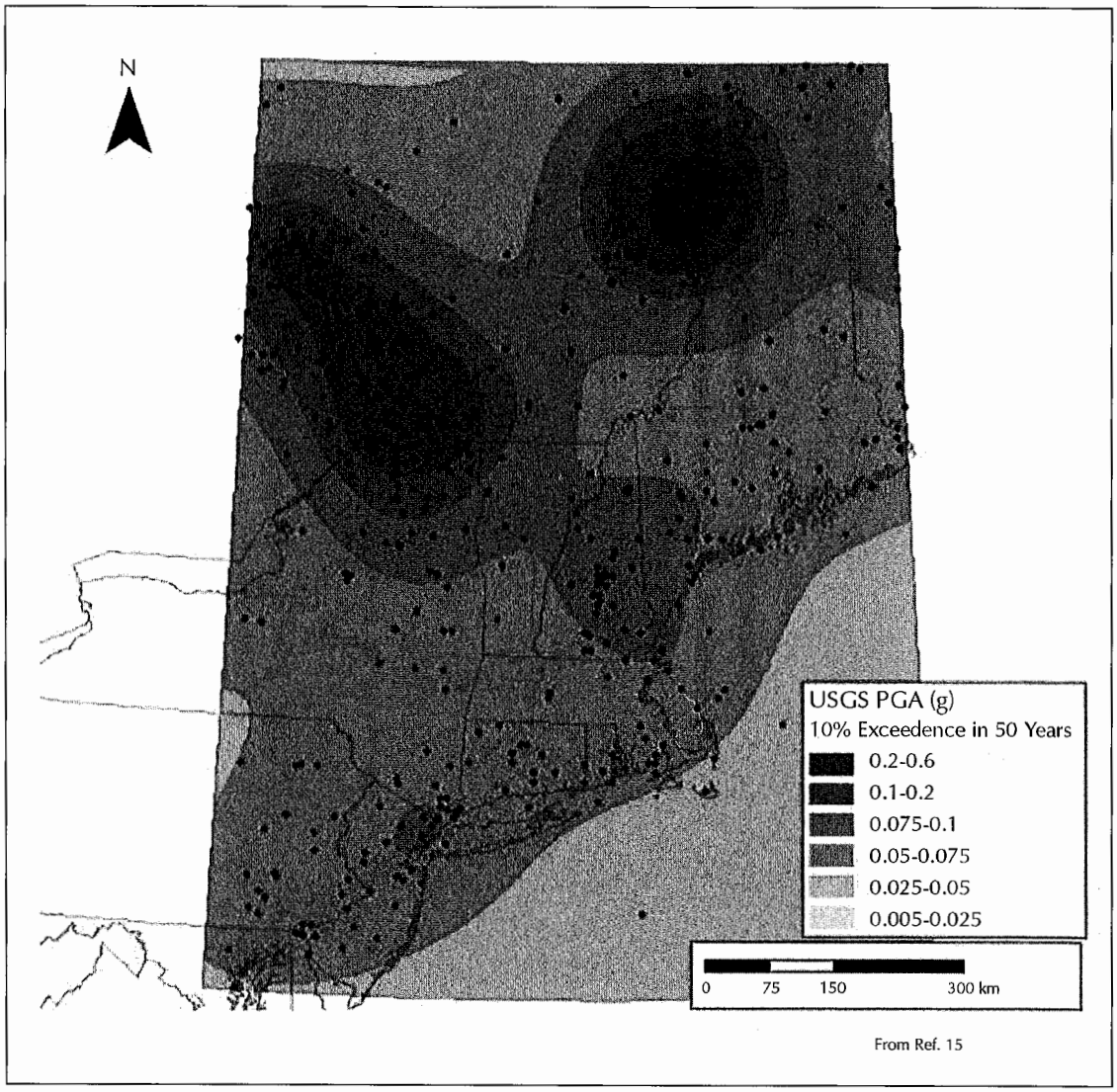

FIGURE 4. Total seismic hazard.

in Figure 5. This plot supports the observation that the southern part of the map is more consistent with the USGS values. Most of the map shows a difference of ±1 percent g. Since ground motions are typically expressed with a precision of whole percents of gravitational acceleration, this interval was judged to be effectively in "agreement." By inspection, over 75 percent of the map is within the "agreement" interval. The PGA off Cape Ann, Massachusetts, on the New York-Québec border and in the New York metropolitan area was calculated to be as much as 2.25 percent g greater in this study. Patches of Québec and Ontario fall in the +1 to +5 percent g region, especially in the northeast corner of the map, indicating higher ground motions from the USGS maps relative to this study. Differences as high as 40 percent g are observed around Charlevoix, where a different *b* value and different SRAFs were used. Interestingly, it is surrounded by an "agreement" region, which is surrounded by an area of +1 to +5 percent g differences between the maps.

Discussion

Having directly compared a custom seismic hazard map for New England with the 2002

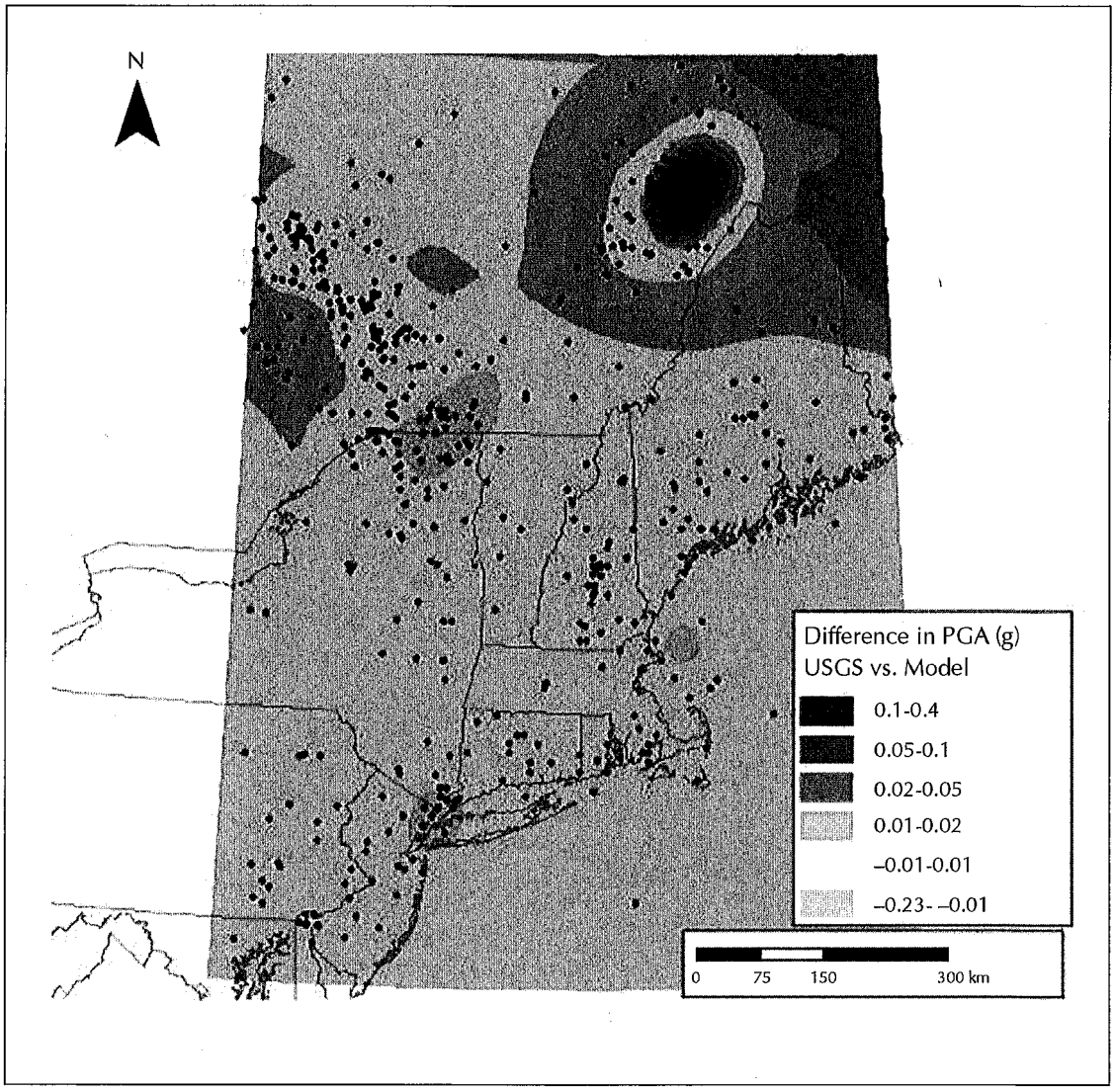

FIGURE 5. Comparison of total hazard maps with USGS data, adjusted seismicity.

national seismic hazard maps, the effects of different modeling assumptions can be discussed. The effect of catalog changes, source area, ground motion models and seismicity rates can be discussed in some detail, since these parameters have relatively observable effects and are well documented. Less obvious effects, including changes in the logic tree and the internal workings of the hazard calculation, can be considered with less certainty.

The single largest modeling assumption encountered in this study is the use of SRAFs. SRAFs of 1.27, 1.15 and 1.58 were applied to the spatially smoothed historic seismicity of Models 1, 2 and 3, respectively. Much of the significant differences between the New England and USGS maps appear to be related to the choice of SRAFs. The SRAFs used in this study were those from Mueller *et al.* for the east coast of the United States.[11] The source area for the New England maps, however, included areas north of the St. Lawrence River (where Mueller *et al.* defined factors of 1.35, 1.41 and 1.94[11]) and Charlevoix (where factors of 1.80, 1.80 and 1.36 were provided for Models 1, 2 and 3, respectively, and a lower *b* value of 0.76 was used). Using the generally

smaller east coast factors resulted in a serious underestimation of PGA north of the St. Lawrence, which appears to be the cause of most of the disagreement between the northern portions of Figures 3 and 4.

The effect of the earthquake catalog used for a seismic hazard analysis can be evaluated by examining the contour shapes of maps using different catalogs. Seismic hazard is based on the rate of events occurring on a source. Adding years to a catalog time period will generally make an analysis more representative, but it will also locally redistribute hazard, thus reducing the seismic hazard near temporally and spatially isolated events. Adding events from 2002 and early 2003 to the catalog in this study produced no perceptible effect.

The spatial limitations of the catalog were expected to affect the seismic risk towards the edges of the source zone, necessitating a source zone enveloping significant areas beyond New England. The selection of the source area is a significant potential source of error at the edges of the source area. If clusters of events beyond the source area presented a hazard to cells within the source area, then the hazard for those cells would be logically underpredicted. A region of underprediction relative to the USGS map would be therefore expected along the perimeter of the source area. The plots of the difference between the USGS map and the New England maps from this study do not reveal such a trend. Figure 2 shows few events on the perimeter of the source area that could be part of an earthquake cluster of significant hazard. Most of the perimeter of the map appears to be with ±1 percent g of the 2002 USGS values.[15] The only areas of the map affected by the selection of source area are therefore those for which a cluster of increased hazard beyond the source area contributes significantly. For example, the decreased PGA in northern Maine relative to the 2002 USGS map may be related to the exclusion of events near Miramichi, New Brunswick, although it is not specifically indicated on any specific model.[15] The exclusion of a cluster near Buffalo was observed to result in an underprediction over a small area of Model 3, although this underprediction did not appear on the averaged maps. Therefore, it would appear that the size of the source area is unimportant as long as it is located correctly. The source area must include a representative area that is likely to contribute significantly to the hazard, but need not include all areas of low hazard.

The choice of ground motion estimators is not expected to be a significant source of difference between the New England map and the 2002 USGS map because the same attenuation relations were used. The choice of attenuation relation is not, however, insignificant. The Atkinson and Boore relation significantly overpredicts ground motions of the scale typically found in New England because, for simplicity, a quadratic form, rather than a fifth order polynomial, was used to describe the modeled data.[7] Although attenuation relations are generally consistent for moderate to large earthquakes, they may be extremely inaccurate close to the epicenter, particularly for small events. The Campbell attenuation relation was designed to address this problem by using scaled west coast attenuation relations near the source, but this approach is relatively untested.[8] It is necessary to understand the limitations of any attenuation relation used in a specific analysis.

It would be expected that 1995 Frankel and the 1996 Frankel *et al.* studies used a consistent estimate of uncertainty for each map edition.[4,5] Frankel *et al.* used a lognormal standard deviation of 0.75 for the attenuation relations used for the 1996 map. However, it was not clear what standard deviations were made in the 1995 Frankel or 2002 Frankel *et al.* studies, requiring assumptions to be made for this study.[4,3] The assumption was made to keep the constant estimate for the attenuation relations used in 1996 and use uncertainty estimates for the other relations from the primary sources. Some of the differences between the maps in this study and in other publications likely resulted from the choice of standard deviation, but these variances could not be quantified. Such error would be expected to appear more randomly than the effects of other assumptions and would be difficult to identify.

The remaining sources of error remain buried in the procedure and the programming subroutines, where detailed definition was required in the absence of published guidance. An example of the assumptions made in developing the process is the use of magnitude and distance bins. The range of distances and magnitudes used to develop exceedance rates were consistent with the USGS maps. However, it is unknown how well the distance bins used by the 2002 Frankel *et al.* study or this study approximated the integral form of the exceedance rate.[3] It appears likely that this sort of error is responsible for some of the small, unexplained differences between the maps.

Conclusions & Extensions

A custom regional seismic hazard analysis was performed for the New England area following the procedures from the 2002 Frankel *et al.* study and was compared with the 2002 national seismic hazard maps.[3] The maps compare favorably, in spite of the fact that the analysis required assumptions and simplifications to account for incomplete documentation of the procedure and to facilitate programming. Both sets of maps share similar contours and PGA magnitudes, indicating that the same procedure effectively was used. The grid cell values for most of the custom New England map fall within ±1 percent g of the cell values from the 2002 USGS maps.[15] For the less than 25 percent of the cells that are not in the "agreement" range, much of the differences can be attributed to specific modeling assumptions and parameters.

The assumption resulting in the greatest difference between the New England map and the USGS map was the choice of SRAFs. The use of factors developed for the United States eastern seaboard for events north of the St. Lawrence River (where Mueller *et al.* used higher factors[11]) resulted in different cell counts and, therefore, different recurrence rates. Errors of up to 5 percent g were attributed to the selection of SRAFs. Selection of the *b* value for the recurrence law proved to be even more critical (as shown by the high error in Charlevoix), but its effect was more localized and did not seem to affect New England hazard estimates.

Other errors (such as the effect of the source area boundaries, the coarseness of rate of exceedance estimate discretization and the assembly of the total hazard maps based on PGA rather than seismicity rates) were found to result in divergences of up to ±1 percent g. Although this amount is significant in terms of percent, it is insignificant to engineering applications. Therefore, minor errors due to assumptions that were required to complete the published procedure did not result in significant errors.

Due to the resilience of the hazard calculation procedure, minor changes in the procedure (such as the assumptions made due to incomplete documentation) did not produce significant results. Larger changes were visible, but did not radically alter results. This lack of effect on results means that the procedure may be reasonably simplified, refined or otherwise altered with low risk of unrealistic results because the procedure is inherently stable. Therefore, this procedure lends itself to customization to further refine the maps to account for region- or project-specific requirements.

A customized PSHA (such as one applied in this study) can be performed when site conditions fail to conform to the applicable design code, or when a variance to the code is desired. A complete seismic hazard analysis can be customized for the specific requirements of a given site or project, with the seismic loads determined directly from seismological data and models. Ground motions could also be calculated for probabilistic criteria that are unavailable in published maps. Although it would be prudent to run the analysis with a common rate of exceedance to check against published maps, once the analysis works properly any rate of exceedance could be selected to reflect the risk tolerance and design life of any project.

The 2000 International Building Code (IBC), which is intended to replace the model building codes used in jurisdictions across the United States, allows site-specific ground motion analysis.[16] A site-specific study is required to account for regional geology, seismicity and maximum magnitudes of events on known faults and source zones relative to

the site-to-source distance. Near source effects and subsurface characteristics must be considered. The required design event is the maximum considered earthquake (MCE), which has an average return period of 2,500 years (2 percent probability of exceedance in fifty years). The design ground motion parameters are the 5 percent damped design spectral response accelerations at short period (SDS) and 1-second period (SD1) that can be used to define the design response spectrum. The design spectral response acceleration is two-thirds of the spectral response acceleration from the seismic hazard analysis and it must be greater than 80 percent the spectral response from the code procedure.[16,17] This custom PSHA methodology could be easily adapted to comply with IBC requirements by using the correct attenuation relations, probabilistic criteria and additional post-processing.

National seismic hazard maps will evolve as seismologists and engineers better understand the mechanics of earthquakes both at the edges and interiors of tectonic plates. It is hoped that the calculation procedure would continue to incorporate an extensive, but practical set of models to predict ground motion parameters. As shown in this study, a large number of models prevents bias and decreases the sensitivity to small and large modeling assumptions. Too many models, however, will increase the computational power required to perform this type of analysis and would prohibit the type of custom hazard calculation that this study represents. The current maps balance these needs very well. Future maps should attempt to maintain this balance.

NOTES — *The GIS software selected for this study was ArcMap by ESRI. Visual Basic for Applications (VBA) programs were developed and run in Microsoft Excel to perform calculations. AutoCAD was the computer-aided drafting software used.*

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Suspension Bridges of New England

New design innovations have made the suspension bridge a rare bird. Before they all disappear from the landscape, it might be best to review and honor those that still exist.

DAVID LATTANZI & DEREK BARNES

Despite all of the infrastructure development in New England, there are only five automotive suspension bridges in all of Massachusetts, Maine, New Hampshire and Vermont. Three of them were built in the 1930s and none have been built in over thirty-five years. Since the success of the Zakim Bridge has begun the process of replacing aging suspension bridges with cable stayed systems, it is time to look back and celebrate the suspension bridges that still grace the New England landscape.

The Claiborne Pell Bridge

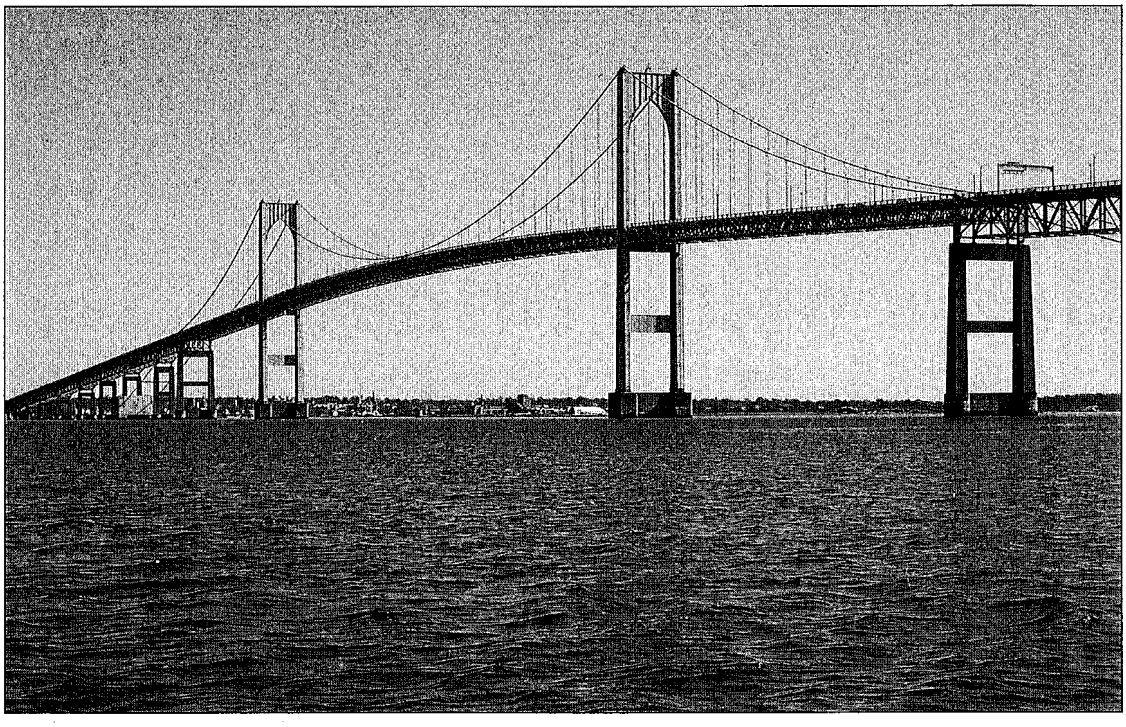
The Claiborne Pell Suspension Bridge is located in Newport, Rhode Island. It stretches a total of 11,248 feet, carrying State Route 138 across the Narragansett Bay and linking Jamestown to Newport.[1] It is the longest suspension bridge in New England, and the first to use prefabricated parallel-wire strands.

The early planning for the bridge design was initiated by the State of Rhode Island in 1934 when it sought federal aid to build bridges across the west and east passages of the Narragansett Bay. The new bridge would replace the ferry servicing the Conanicut and Aquidneck islands. The west passage was designed, and built by 1940; it was later called the Jamestown Bridge.[2]

The onset of World War II put a hold on the east passage project, and planning did not resume until 1944.[3] The newly formed Newport-Jamestown civic commission finally convinced the state legislature to approve the Newport-Jamestown Bridge Proposal in 1948.

Extensive engineering studies were then performed for the proposed crossing. There were thirty-two different studies done that evaluated possible locations of a suspension bridge, cantilever bridge, tunnel or possible bridge-tunnel combination. Factors that narrowed down the selection of location were: higher construction costs for greater water distances, extensive relocation of U.S. Navy property, disruptions to boat traffic in heavily navigated areas and strong local opposition from residents.

It was not until the mid- to late 1950s that the project gained momentum. The Rhode Island Turnpike and Bridge Authority



Claiborne Pell Bridge, Rhode Island.

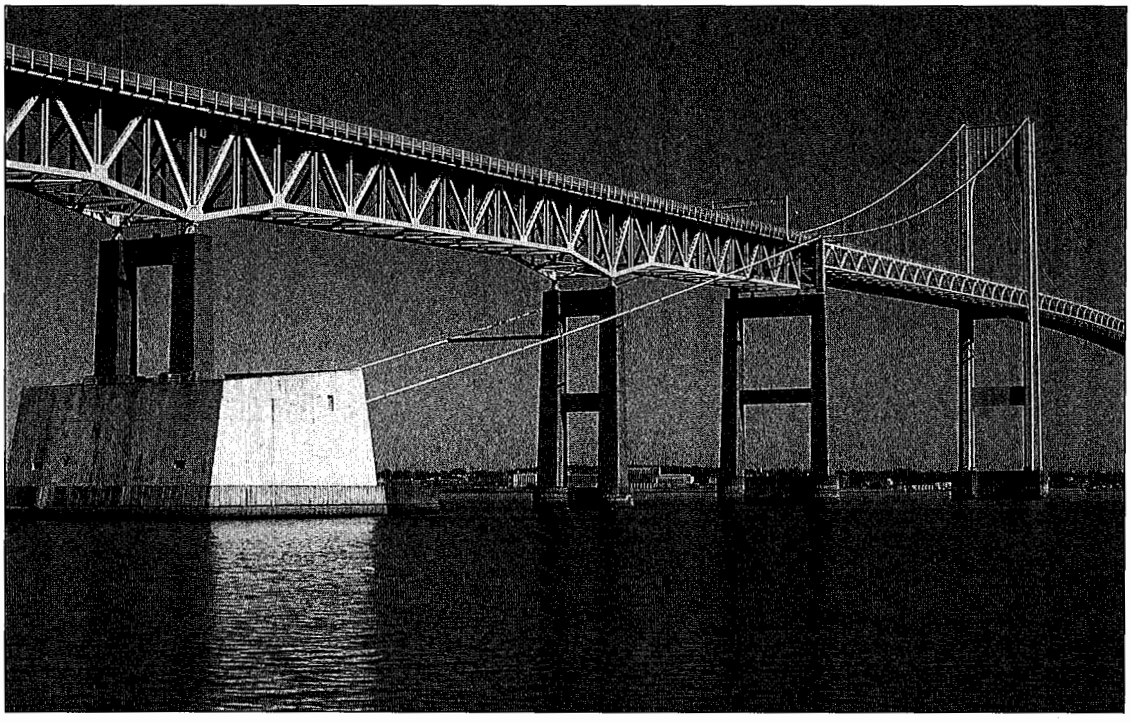
(RITBA) was created in 1954. This organization would eventually finance, build and maintain the proposed east passage crossing and its immediate approaches. The Rhode Island Department of Public Works included the east passage in its plans for a statewide expressway network.

Despite the gains in support, the project was not scheduled for construction until the mid-1960s. In 1960, RITBA secured a firm to design the bridge. The chief design engineer assigned to the project was Alfred Hedefine. The firm recommended a bridge location that was a variation of the earlier proposed Taylor Point–Admiral Kalbfus Road alignment. This variation addressed the U.S. Navy's expensive relocation concerns by veering the alignment south toward Washington Street before turning west, thus avoiding the Coasters Harbor Island Naval Station altogether.

The chosen alignment would connect Taylor's Point in Jamestown to Washington Street in Newport. The next obstacle that stood in the path of the project was a statewide referendum that would give RITBA bond-selling power. It was rejected by voters in 1960, but it was finally ratified in 1964 (and later re-approved in 1965).

RITBA received final approval for the bridge location and clearances from the U.S. Army Corps of Engineers, and the U.S. Navy in 1965. Construction on the bridge started on April 5, 1966. A total of 838 steel piles were driven down to bedrock under 162 feet of water (the deepest ever attempted) to support the concrete foundation blocks. Working at this depth proved difficult for divers. Productivity initially was limited to one pile per day because divers could only stay under for thirty minutes at a time while they cut off the tops of the piles. However, a diving tank was later shipped to the site, and productivity increased significantly to fifteen piles per day.[2]

Prefabricated forms were used to build the piers. They were brought to Newport by barge because of their immense size (the largest weighing more than 400 tons and standing an astounding ten stories high). One of the world's largest floating lifters, "the Avondale Senior," was used to remove the forms from the barges and set them in place. It traveled all the way from New Orleans. The vessel was



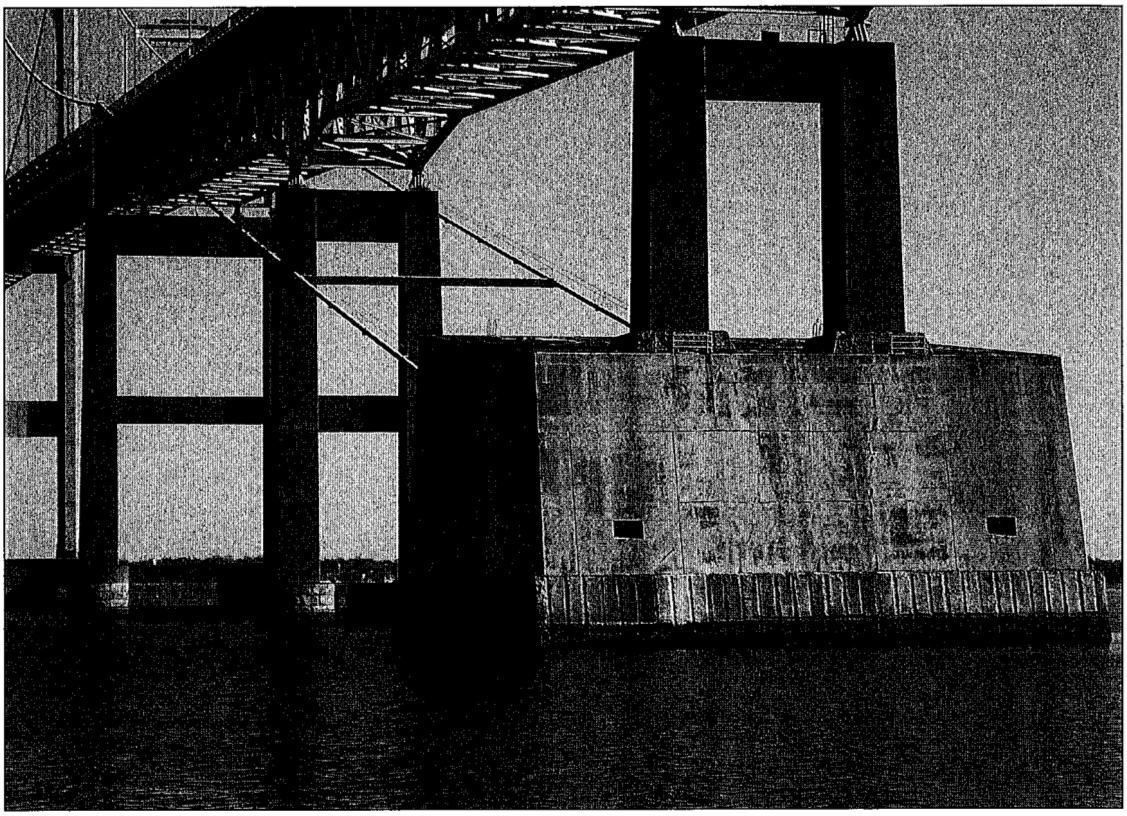
Approach spans of the Claiborne Pell Bridge.

309 feet long, and had twin booms that could handle 500 tons. During placement of the forms, two powerful storms struck Narragansett Bay. The high winds and pounding waves caused significant damage to the forms. Workers had to straighten them out before the concrete could be poured.

It took an estimated of 90,000 cubic yards of concrete to pour the piers and anchorages. This amount was the largest ever structural concrete placement under water. The tremie method was used, which resulted in reduced pressure exerted on the sides of the pier walls. The steel reinforcing used in the concrete weighed 4,000 tons. The piers were finally completed by 1967.

During the summer and fall of 1967, the 400-foot steel towers were erected. The cable steel manufacturer developed a new technique in stringing the main suspension cables using prefabricated parallel wire strands. This major innovation was used in place of the conventional cable spinning process, which involved spinning each strand wire by wire while stringing it from one anchor, over each tower and down to the other anchor. This process was repeated thousands of times for each cable. The prefabricated strands saved time and money. They were made in Pennsylvania and sent to Newport as a package. Each cable measured 15.0625 inches in diameter, and was coated with a protective plastic sheath. Once the bridge construction was complete, it opened to traffic on June 28, 1969. It was named simply "The Newport Bridge."

The over two-mile bridge consists of a 1,600-foot suspended span, 688-foot approach spans, eleven deck truss spans totaling 3,450 feet, fifteen girder spans totaling 2,524 feet, 300 feet of multi-girder spans and 2,000 feet of prestressed concrete beams. The bridges roadway carries four lanes of traffic with a total width of 48 feet over the Narragansett Bay at 225 feet above the water line. The maximum grade of roadway is 4.8 percent. The maximum gross weight for vehicles is 40 tons (80 tons with a permit). Its towers rise 400 feet above mean high water. The deck is stiffened by a lateral truss running longitudinally along the bridge that protects against failure due to lateral wind forces.[4]



Claiborne Pell Bridge abutments.

The two main suspension cables each have a 15.0625-inch diameter that supports 4,636 suspender cables on either side. Each cable contains 76 strands, and each strand has 61 wires. Each wire has a diameter of 0.2 inches, and measures 4,516 feet long. The total length of wire if laid out end to end is an astounding 8,000 miles long. The total combined weight of the cables is 2,280 tons.

The substructure contains 136,000 cubic yards of concrete. The roadway and approaches contain an additional 17,500 cubic yards of concrete. The concrete deck is 7.5 inches deep. Once completed, the total cost of the original structure came in at $54.742 million.

The bridge's strength and reliability were tested in February 1981. A tanker carrying 50,000 barrels of oil collided with one of the main piers of the bridge. The pier suffered only a smear of grey paint. No structural damage was found whatsoever. The tanker was not so lucky — its bow was impacted ten feet. Even though the bridge sustained the blow from the tanker without damage, RITBA is conducting continued investigations for the need for pier protection. Currently, there are no fenders protecting the bridge's piers.[2]

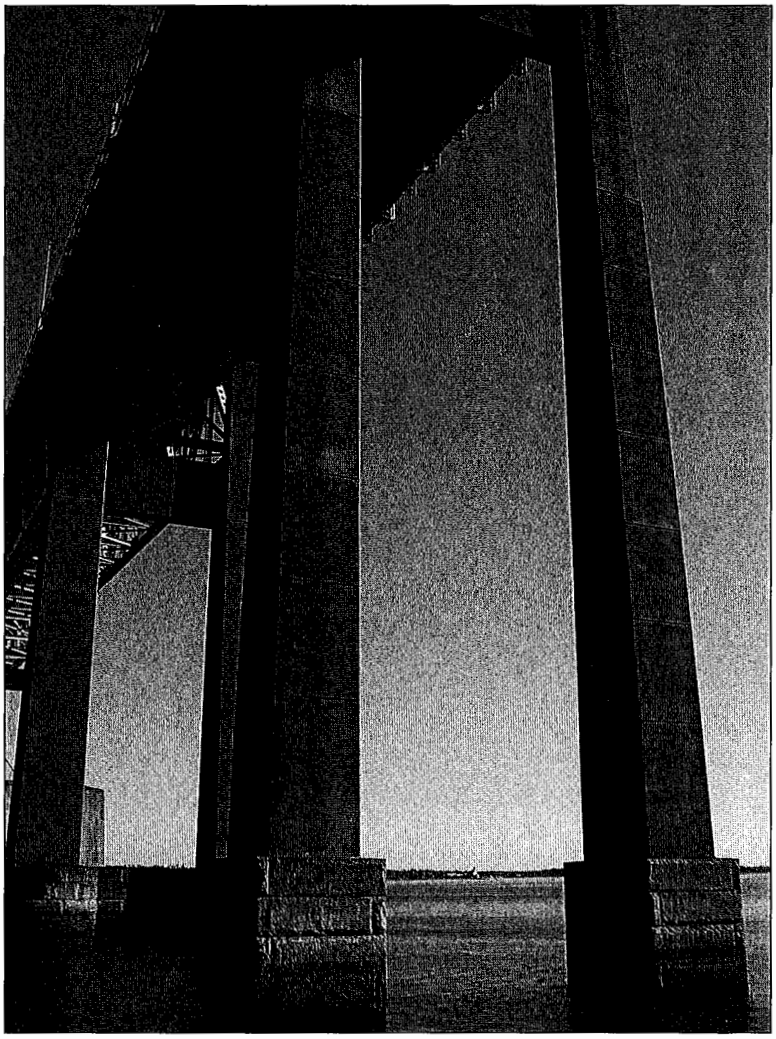
In 1997, the bridge was rededicated and renamed in honor of retired six-term U.S. Senator Claiborne Pell of Newport. Senator Pell was the longest serving senator in Rhode Island history. He is currently serving on the Board of Governors of the National Parkinson Foundation.

RITBA places travel restrictions, and sometimes closes the bridge, when winds reach certain speeds. Travel is restricted once winds reach a sustained 50 knots coming from a North, North-East-South or South-West direction. If the winds sustain 60 knots, RITBA considers closure as a safety precaution.

The bridge receives regular maintenance and repairs. Recent repairs have been made to the concrete slab, piers, steel catwalk and other miscellaneous steel repairs. In 1997, another engineering firm was contracted by

bridge's original design group to provide consultation services in the rehabilitation of 720 linear feet of reinforced concrete deck. The deteriorated concrete was removed by hydro-demolition methods, and replaced with high-performance microsilica modified concrete. Because of the cold New England weather conditions, relatively new techniques were used in the placement of the new concrete. Specifically, the adaptations of ground-thawing techniques were used to maintain pre- and post-placement temperatures to ensure quality concrete placement in the cold weather. The rehabilitation was completed in the spring of 2001.

In 2003, repairs and/or replacement of deteriorated steel catwalks were performed. The concrete piers also received repair work during that time. These repairs consisted of crack repair, shallow and deep spall repair, repointing of granite facing at water piers and anchors, and cleaning and protective sealing of the anchorages. Only limited short-term lane closures were allowed for mobilization of equipment throughout the construction.[2]

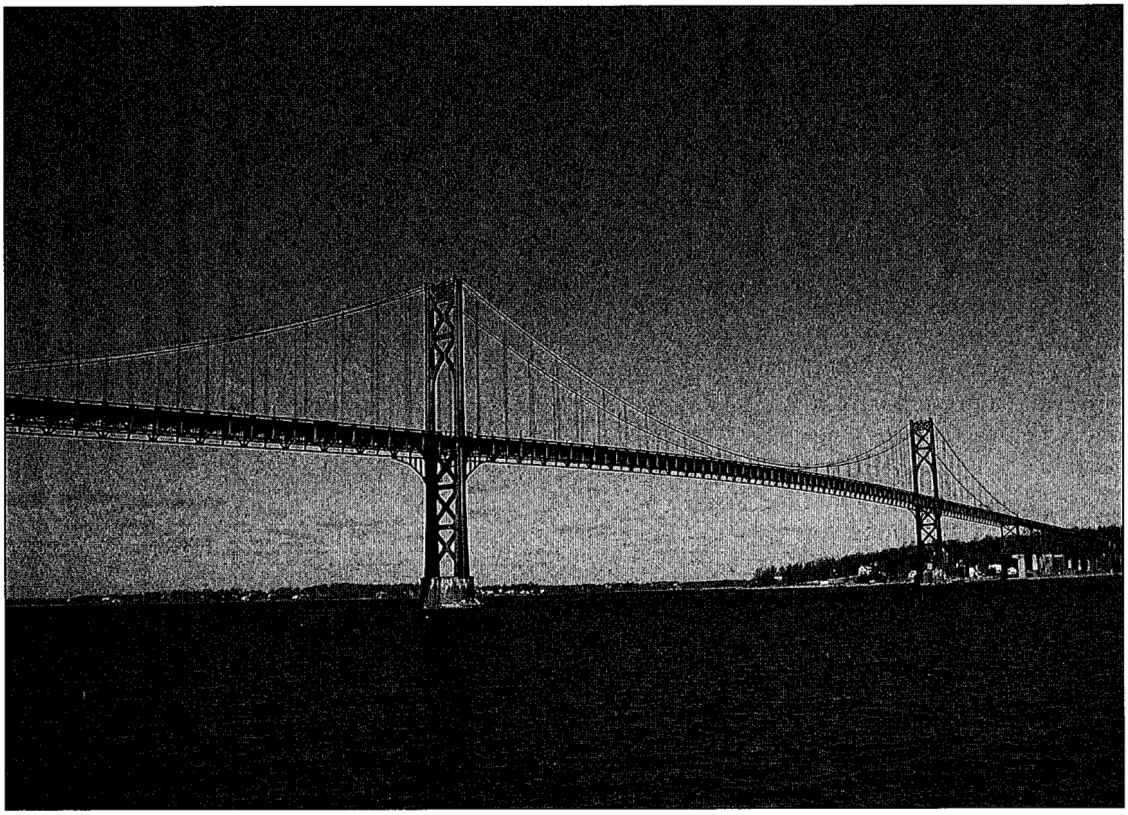


Claiborne Pell Bridge piers.

The Claiborne Pell Suspension Bridge faced many challenges during its construction. Engineering innovation and creativity in design proved to be invaluable in making the crossing possible. The bridge won awards for excellence in engineering design from the New York Association of Consulting Engineers, the Consulting Engineers Council, the American Iron and Steel Institute and the American Society of Civil Engineers. Today, it carries an average of 27,000 vehicles per day.[2] The surrounding areas have benefited from the bridge over the past thirty years. The Claiborne Pell Bridge continues to encourage trade, travel and tourism by providing an efficient and aesthetically pleasing gateway over Narragansett Bay.

The Mount Hope Bridge

The Mount Hope Suspension Bridge was originally built as a private toll bridge. It carries State Route 114 over the small area of water between Mount Hope Bay and Narragansett Bay in Rhode Island and connects the towns of Bristol and Portsmouth. The bridge is stiffened with a deck truss and is gravity anchored. It is known for its stability and

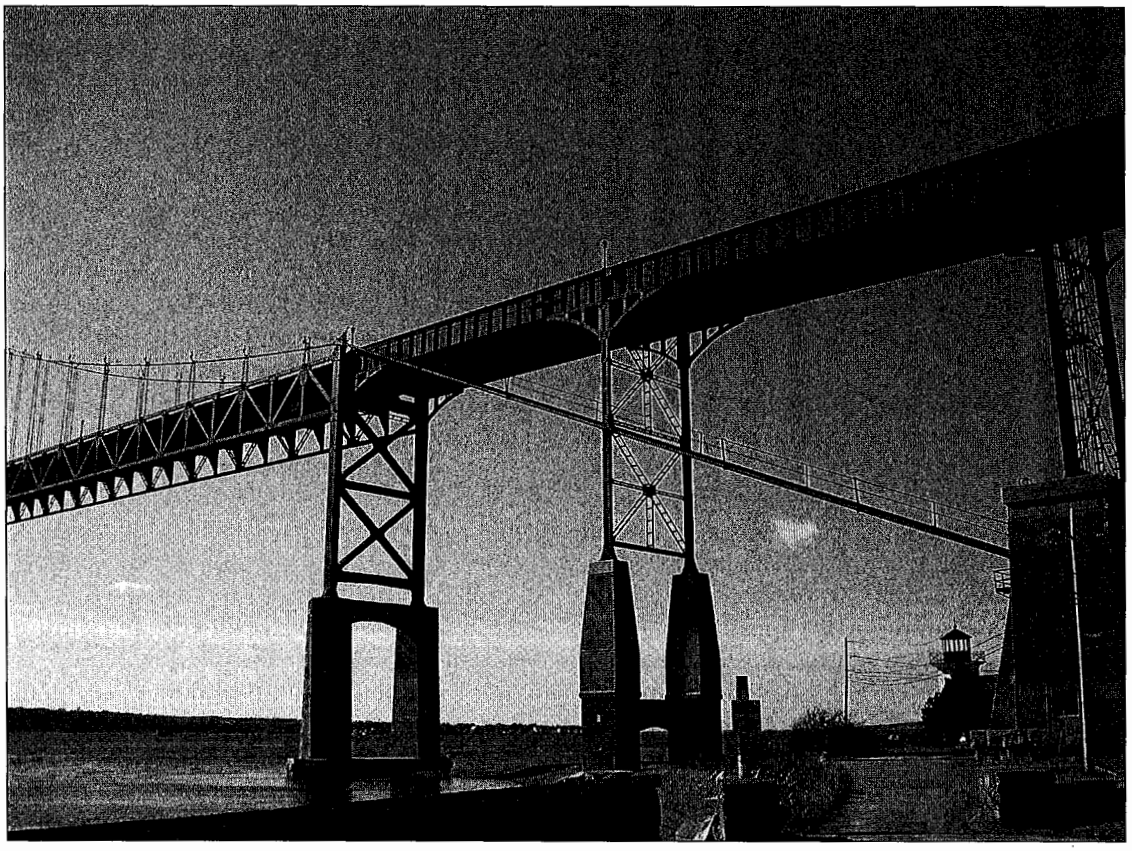


Mount Hope Bridge, Rhode Island.

strength since it has survived seventy winters and three hurricanes. It was the largest suspension bridge in New England before the construction of the Newport (Claiborne Pell) Bridge. It was also the first suspension bridge built in Rhode Island, and it was known for taking "the Island out of Rhode Island."

The site of the Mount Hope Bridge is historically significant. In 1636, Roger Williams settled the Providence Plantations to the north of the bridge area. John Clarke settled the Island of Rhode Island in 1638 to the south of the bridge. The bridge got its name from the nearby Mount Hope, which is located within the town of Bristol. Mount Hope is where the Indian attacks of 1675–76 where planned by the Wampanoag chief, King Philip. Captain Benjamin Church and his men used the ferry services that ran where the bridge now stands to gain access to the mainland in order to quell the Wampanoag uprising.[5]

The cold winters of Rhode Island would cause the waters of Bristol Harbor to freeze, thus preventing the ferries from running. This isolation was hard for the people on the island of Rhode Island (which includes Newport) to endure. Legislators from Newport County would occasionally be late to, or miss, sessions of the Rhode Island General Assembly due to the harbor freezing. A resolution sponsored by William L. Connery of Bristol in the Rhode Island General Assembly on March 9, 1920, formed a joint committee to investigate the need for a bridge linking Bristol to Portsmouth. Ultimately, the committee decided that the financial burden to build a bridge would be too great for the state to shoulder at the time. However, by the mid-1920s there was increased need for a direct link between the two cities due to the rapid influx of tourism from the west and due to the increase of vehicular traffic because of greater automobile use. The expense of building the bridge was still too high for tax payers, so the General Assembly decided to hand over building rights and ownership to a private company.



Mount Hope Bridge abutments and approach spans.

The Mount Hope Bridge Company would finance the project and operate the bridge as a private toll bridge.[6]

David B. Steinman, a well-known structural engineer, was put to the task of designing the Mount Hope Bridge in 1927.[7] He designed over 400 bridges in his lifetime. He also founded the National Society of Professional Engineers and served as its first president. Steinman was first informed of the Mount Hope Bridge project in 1926. He quickly turned in a proposal to the Rhode Island State Highway Engineer only to learn that a commission was already appointed to the task of designing the bridge. However, the state rejected the commission's proposal, and turned to Steinman, asking if he could design and build the bridge for less than $4 million. Steinman replied, "I can do better than that. I can build it for three million, and maybe I can shave that."[??] Steinman was awarded the job and completed it for $2.5 million.[6]

Excavation began in December 1927. The deck and foundation construction began in 1928. The deepest foundation was poured 54 feet below sea level. A total of 40,000 cubic yards of structural concrete were used on the project. Since the bridge marked the first time that 150-foot continuous steel girders were used, they had to be custom made. The steel spans were completed in only thirteen days. The suspension cables were constructed using the conventional cable spinning process.

During construction a problem was discovered with the suspension cables. It was only four months from opening day when the cables were declared unsafe. The problem was the result of using heat-treated steel even despite Steinman's objections. The cables were breaking and crews had to work around the clock to take down the roadway and replace the cables with new cold-drawn steel cables at a cost of an extra $1 million. The completion date was pushed four months past the dead-

line. The contractor paid for the changes and construction delays. The total cost of construction was $3,897,820 million. Today, it would cost an estimated $33.7 million to erect the same bridge.

The bridge opened to traffic on October 24, 1929, only two years after construction began. The Mount Hope Bridge was purchased by the State of Rhode Island on November 1, 1955. Eventually, RITBA purchased the bridge on June 1, 1964. The bridge became toll-free on May 1, 1998.

The bridge is 6,130 feet long and consists of a 1,200-foot suspension span. The total water span is 3,000 feet. Each 150-foot girder span contains four steel girders. The bridge's roadway carries two lanes of traffic with a total width of 27 feet over 135 feet above the water. The maximum grade of roadway is 3.8 percent. The maximum gross vehicular weight is 42 tons. Its steel towers rise to 285 feet above mean high water. The deck is stiffened laterally by a deep truss running longitudinally along the bridge that protects against failure due to wind forces.

The two main suspension cables each have an 11-inch diameter. Each cable contains 2,450 wires. The total length of wire if laid out end to end is 2,620 miles long. The total loading capacity of the cables is 5,600 tons. The use of cable bents with straight backstays at the ends of the side spans were a major innovation, and resulted in an economic benefit.[8]

Recent repairs were performed on the main suspension cables. The repairs were completed in two phases from April to December 1999, and April to October 2000. The scope of work included rehabilitating the main cable by replacing the hand rope system, or conventional cable spinning method. Also new wire rope suspenders were installed at designated locations along the main of the bridge.

The main cable wire wrapping was removed and disposed of. Then 4,400 linear feet of main cable were repaired, compacted, waterproofed and rewrapped with new galvanized steel wire. The cable bands were recaulked and repainted. New re-tensioned cable band bolts were fabricated and installed. A new "bridge necklace" lighting system was also installed.[9]

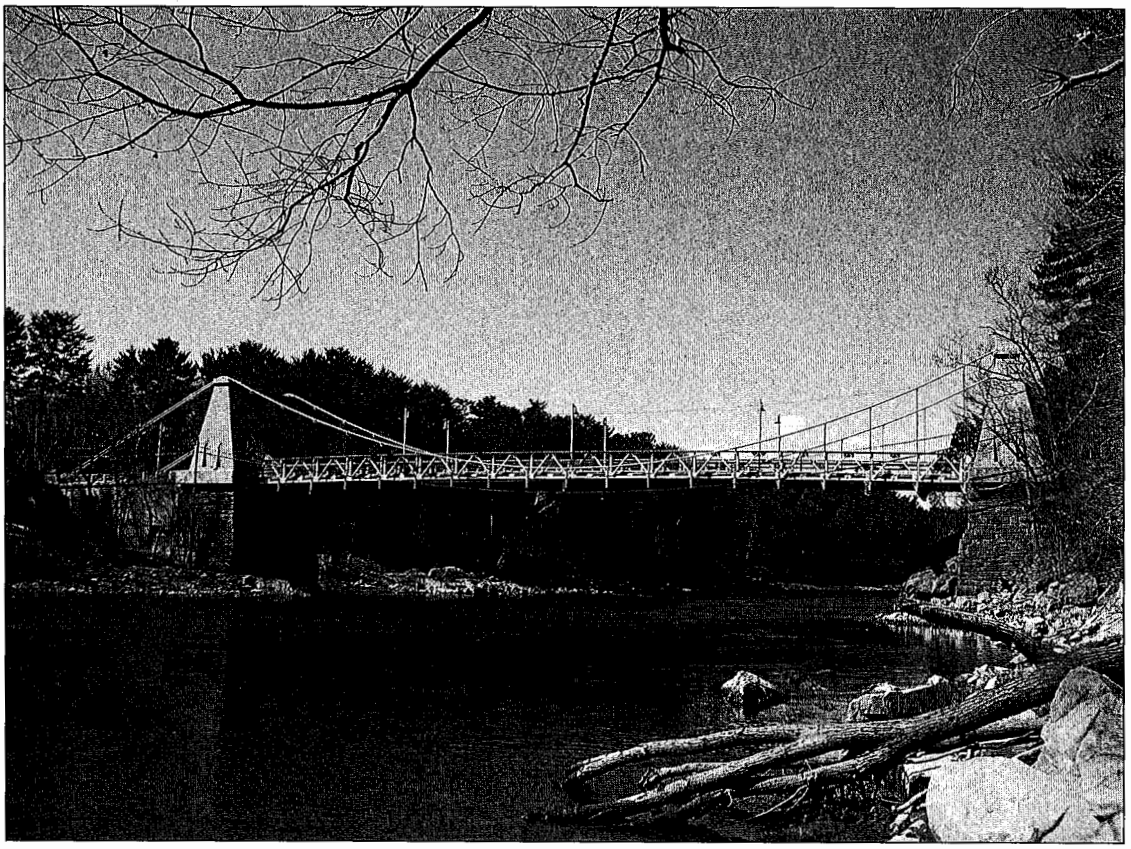
The Mount Hope Bridge is known for its beauty. Its light green tint helps the bridge blend into its surrounding environment. This color represented a significant departure from other bridges at the time, which were usually painted battleship grey or black. The design also incorporated artistic lighting, a new concept at the time. The combination of the lighting and the green hue enables the suspension cables to be clearly visible at night. Extra funding was obtained to landscape the areas adjacent to the bridge access roads. Various flowers and trees were planted to help integrate the bridge into its environment. In 1929, the Mount Hope received the Artistic Bridge of the Year Award by the American Institute of Steel Construction.[8]

The building of the Mount Hope Suspension Bridge greatly benefited the surrounding community as did the building of the Claiborne Pell Bridge. Easier access has financially boosted the Newport area due to increased tourism and trade. The Mount Hope Bridge is one of the oldest long-span suspension bridges in the United States.

The Merrimack River Chain Bridge

The Merrimack River Chain Bridge is the only standing vehicular suspension bridge in Massachusetts. It is also the site of the oldest suspension bridge in the United States. The present-day structure still stands on the original bridge's foundation. The Chain Bridge is located in Newburyport just a sort distance south of Interstate 95. The Merrimack River is separated into two channels by a small body of land known as Deer Island. The Chain Bridge carries Main Street in Newburyport across the southern channel of the Merrimack River onto Deer Island.

The crossing was originally built as a long-span timber truss bridge, known as the Essex Merrimack River Bridge. The bridge consisted of two structures that were combined to form a total length of 1,030 feet. Each structure contained one long span timber arch truss. The northern structure consisted of two abutment approach spans, three watercourse spans, three pier sections and a 113-foot timber truss arch span. The southern structure, currently known as the Chain Bridge, contained two



Merrimack River Chain Bridge, Massachusetts.

abutment approach spans and a 160-foot span timber truss arch. The southern structure was the longer of the two arch spans.[10]

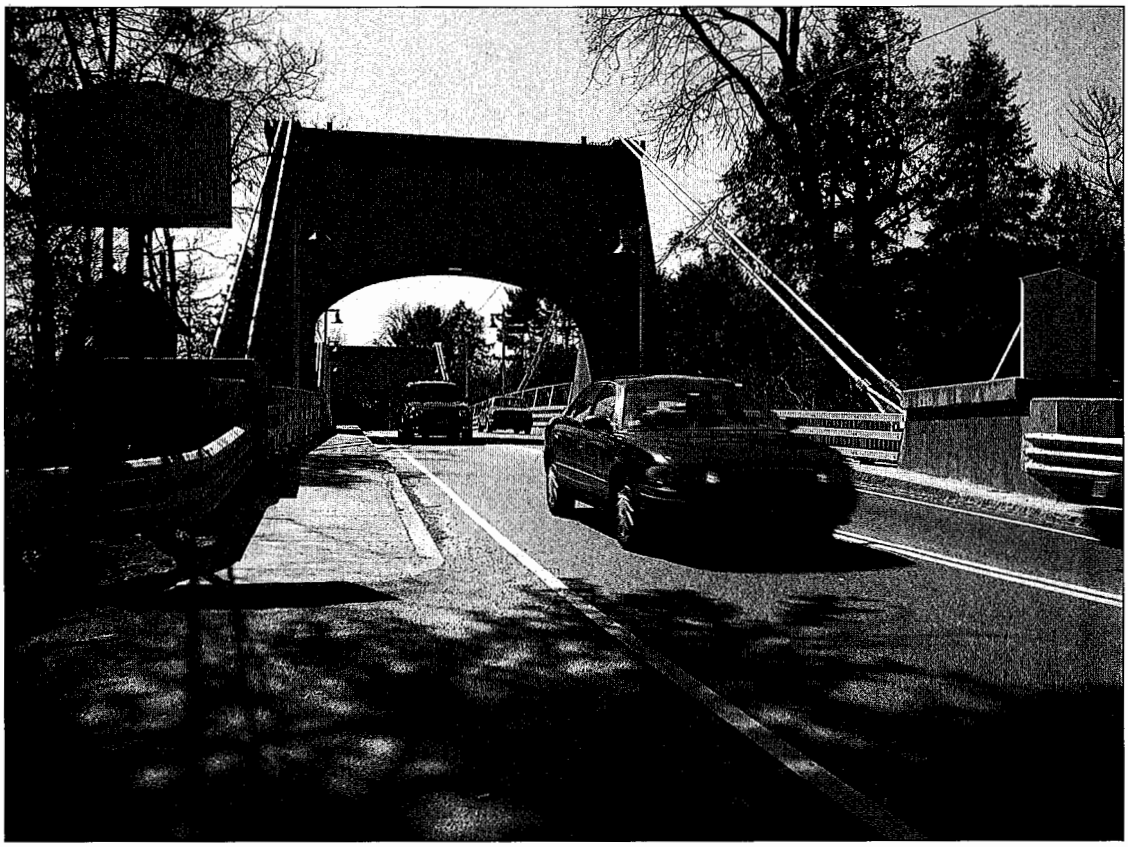
This timber truss bridge was believed to be the first of its kind in America, and the first to cross the Merrimack River. It was designed under the direction of Timothy Palmer in just seven months. The bridge was opened to traffic on November 26, 1792. This structure stood for almost two decades. However, boaters found it to be a hindrance to navigation, so the southern section of the bridge was eventually replaced with a bridge suspended by wrought iron chains in 1810.[11]

This replacement for the southern section was the original Chain Bridge. It was designed by Judge James Finley (1756-1828) of Pennsylvania, and built by John Templeman. The wooden stiffening truss provided a level and rigid roadway, which was uncharacteristic of bridges at the time. Most were not stiff and deflected significantly. The northern portion of the Essex Merrimack River Bridge was replaced with an iron bridge that was erected at a later date.

The Chain Bridge remained in service for seventeen years until 1827 when one of the supporting chains failed due to corrosion and the bridge fell into the river. It was rebuilt according to its original design in 1828 where it remained as a toll bridge until 1868. It then became a public highway.

It was rebuilt again under the authority of an 1908 act of the General Court by the County Commissioners of Essex County in 1909. The bridge was redesigned by consulting engineer George F. Swain. The county engineer Robert R. Evans also played a role in the design of the bridge. The new design took the form of the original Chain Bridge, but added some then state-of-the-art materials.

From 1909 on, the Chain Bridge has been rehabilitated a number of times. The original wooden deck was replaced in 1922, and again



Merrimack River Chain Bridge roadway.

in 1931. The hinges on the stiffening truss were rebuilt in 1935. In 1938, all of the timber decking was replaced by steel grid decking.[12]

The structure now consists of a deck truss-gravity anchored suspension bridge that measures 384 feet between cable anchors (which also includes the 70-foot retained earth approaches). The suspension span is 244 feet (not including the approaches). Instead of wood, the deck truss was constructed with a 7.25-foot-high single-intersecting steel pony stiffening truss hinged at the pier ends and at the center. The roadway width is 24 feet with a 4-foot-wide timber sidewalk.

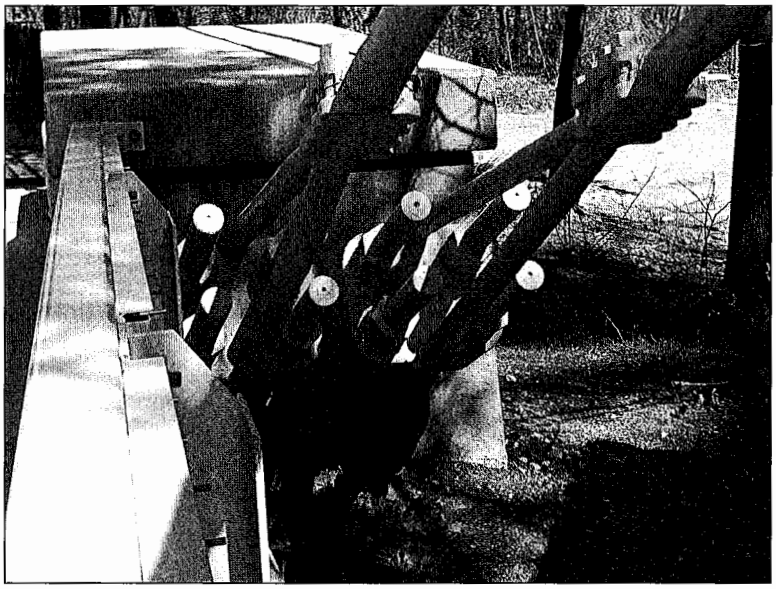
The problem of those corroding wrought iron chains was solved by using four 3.5-inch-diameter cables containing bundled steel wires. There are two cables on each side that are supported by castings and rollers situated at the top of two hollow reinforced concrete towers. The main cable ends are linked to groups of 1-inch by 5-foot steel eye bars in massive concrete anchorages. The towers stand about 34 feet above the roadway surface and are supported on the original granite piers. The granite block piers are founded on rock, but are interestingly not bonded with mortar. Instead, wooden dowels were placed in some of the larger blocks to give the blocks strength.

The suspender cables consist of 2.75-inch steel spaced approximately every 12.33 feet along the deck. They connect from the four main suspension cables and attach to the built-up steel riveted truss floor beams using saddle plate connections. The shorter suspender cables are actually steel rods connected to the floor beams, and the longer cables wrap around a circular plate connecting to the saddle plate.

In 2003, the bridge received major renovation that was estimated to cost $3.26 million (year 2000 dollars). The renovations included replacing the existing steel grid deck system with an orthotropic deck system. The floor

beams and connections were repaired, along with damaged cables. The substructure was patched, and retrofitted, to meet current seismic code requirements.

A major component of the seismic retrofit consisted of providing a system that would transfer the load path around the granite piers. There were concerns that a seismic shear transfer would displace blocks along the pier walls, and cause the structure to collapse. Therefore, reinforced concrete collars were wrapped around the base of the towers. They were then connected with the anchor slabs at each approach span. This design was adopted so that the lateral shear forces would transfer directly to the anchor slabs, and away from the "shaky" granite block piers.



Merrimack River Chain Bridge cables.

Another key feature of the seismic retrofit introduced transverse restraint of the suspension cable saddle castings and rollers at the top of each tower. The concern was that a seismic event could potentially move the fixtures off the tower, thus causing the bridge to collapse.

The orthotropic deck system was installed for two reasons: it is a very lightweight deck system; and it adds protection to the floor beams from roadway surface constituents. This deck system consists of a steel deck plate longitudinally stiffened by trapezoidal-shaped steel ribs. It acts as a composite by using an extension plate to connect the existing floor beams with the steel ribs. By connecting the floor beams, added torsional stiffness to the deck system is achieved. Also, a special 0.375-inch-thick polymer concrete wearing surface was placed on top of the orthotropic steel plate deck.

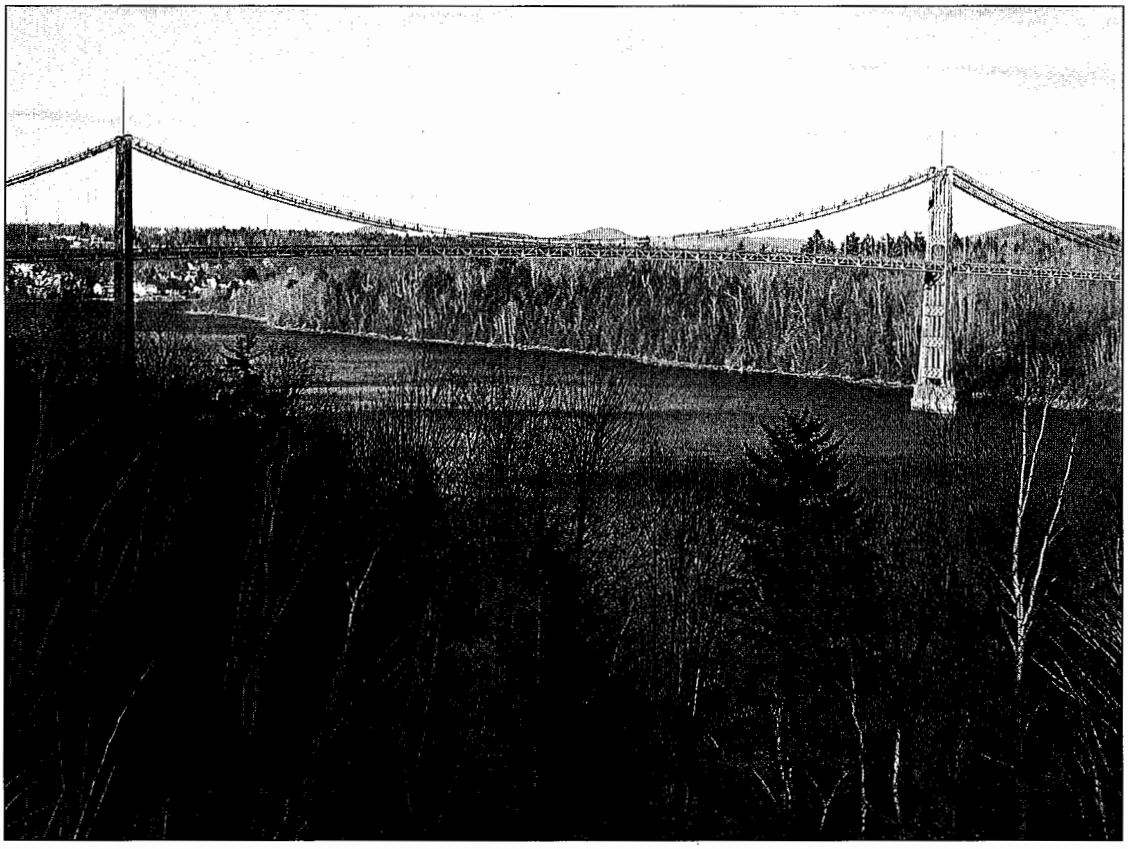
The stiffening trusses were also rehabilitated. The middle hinge on either side was removed for improved live load capacity. A cover plate was then added to the top chord in the middle four bays of the truss to prevent rotation at mid-span, reduce deflections and eliminate the need for a joint in the deck.

Other repairs included the replacement of suspender cables, cable saddle plates and hanger plate connections at a few locations along the bridge. Wingwall caps were also added, along with repairs of spalled concrete along the face of the towers. Navigational lights and roadway light posts were also added. The old x-lattice rail was replaced with crash-tested Massachusetts S3-TL4 bridge rail, which has a similar appearance to the original rail. The new rail uses a vertical post and horizontal tube rail configuration.[13]

After all renovation work was completed the bridge opened again to traffic on June 28, 2003. With all of the replacement and repair work performed on the Chain Bridge, it still maintains its unique original appearance in accordance with its historical significance. Today, it carries an average daily traffic of 23,120 vehicles.

The Waldo-Hancock Bridge

Located in Verona, Maine, the Waldo-Hancock Bridge spans the Penobscot River. Originally designed to connect the towns of Verona and Prospect, it has become a major thoroughfare for travelers on Route 1 in Maine. Up until the construction of the bridge,



Waldo-Hancock Bridge, Maine.

the only crossing of the Penobscot was done by ferry, which severely limited travel between the two towns.

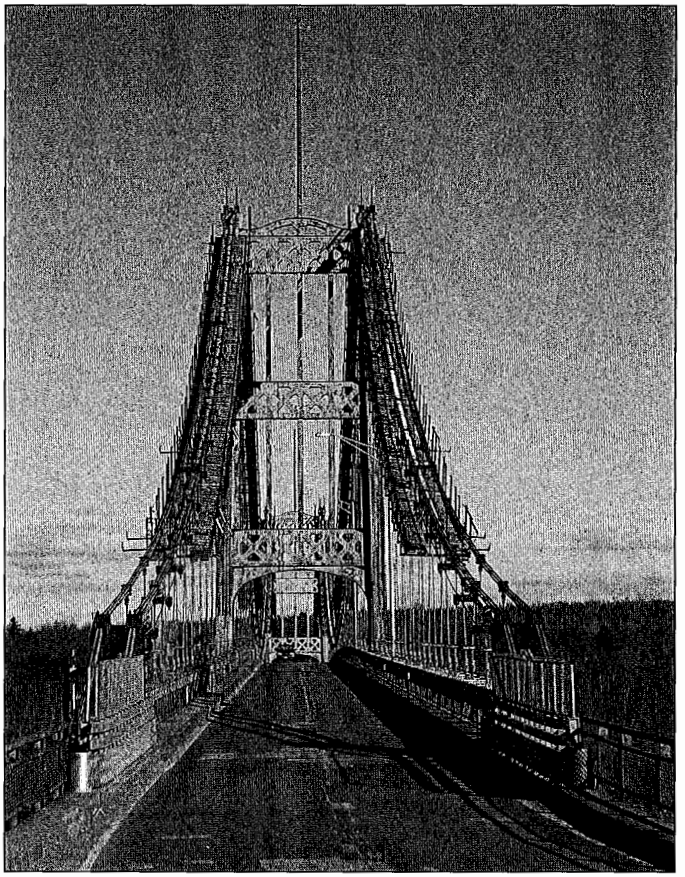
Construction took place from 1929 until 1931. The bridge was one of several New England suspension bridges designed by David B. Steinman, whose firm also managed the construction of the bridge. The severely depressed economy forced the designer to alter designs in several ways and to stretch the budget for the bridge to an extreme. The overall cost of the bridge was roughly $850,000, leading to its nickname as "The Million Dollar Bridge." The bridge was awarded by the American Institute of Steel Construction the title of most beautiful and artistic span of all bridges constructed in 1931. It was also (for a time) the longest span in Maine. The cost was to be paid for by tolls on the bridge. Twenty-two years later, the cost was recovered and the tolls from the bridge were removed from the bridge.[14]

The Waldo-Hancock Bridge is a gravity-anchored suspension bridge with concrete abutments. Like Steinman's other suspension bridges, it has steel pylons and piers supporting the spans. The main span is 800 feet long, and the overall length including approaches is 2,040 feet. The towers reach a maximum height of 236 feet. The deck has deep stiffening trusses, although wind vibrations are still perceptible while crossing the bridge. The bridge has one lane in each direction, which has had a substantial impact on traffic patterns along the Atlantic coast route. Economic concerns led to a helical design of the cable strand system. Helical strands are capable of spanning longer distances and carrying added strength. However, splicing broken helical wires is next to impossible, and would eventually force the replacement of the span.[15]

The bridge underwent major renovations from 1959 to 1961 after engineers discovered severe cracking in the deck and slab, as well as

corrosion in the steel superstructure. Work was also performed on the cable wraps after they were gouged by a passing truck.

A rehabilitation study was performed in 1988 that found that the bridge was in need of massive rehabilitation, and would cost roughly $20.6 million to restore. After debating the merits of a total replacement, a $5.3 million rehabilitation project was chosen. Early analysis reduced the load rating of the bridge from 50 to 12 tons. This reduction was due to a substantial amount of corrosion in the cable strands. The first step in the rehabilitation was to reinforce the piers in order to prevent erosion behind them. Environmental concerns slowed down this portion of the project substantially, but it was eventually completed in May 2001. A massive cable strand replacement effort began in 2002. Halfway through the project it was discovered that several helical strands in the main span had broken, and it was determined that rehabilitation was no longer an feasible option. The immediate replacement of th bridge was deemed critical.[16]

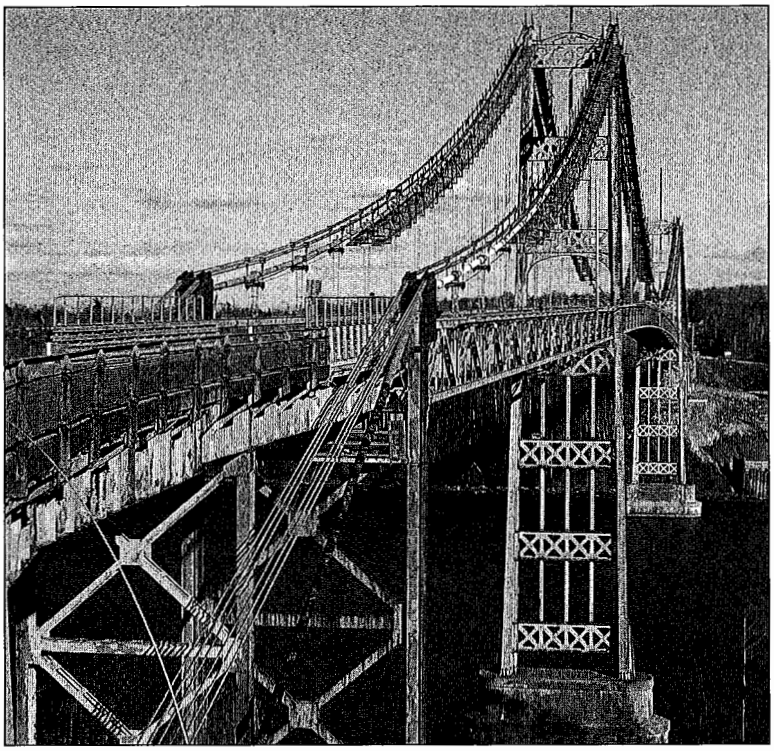


Waldo-Hancock Bridge roadway.

Traffic and economic requirements have had a huge impact on the rehabilitation and replacement efforts. Alternate crossings of the Penobscot River are forty miles away and the bridge needs to remain open to local traffic. Concerns from both towns about the economic impacts of closing the crossing have prevented a complete closure of the structure.

These problems led to emergency rehabilitation work, and an expedited replacement timetable.[17] Additional work was done in order to support the sagging spans. A highly complex and innovative load transfer procedure was performed in 2003 in order to reduce the load on the corroded cables. At an overall cost of $2.5 million, new abutments and a new cable system were linked to the entire structure via steel plates, which were welded to the original system. The new system now supports 50 percent of the dead load, and allowed the Maine Department of Transportation (DOT) to increase the load rating to 40 tons. Furthermore, the immediate need for a new bridge has increased the cost to build it, which is now estimated to be roughly $75 million.[16]

Construction has now begun on the new cable-stayed replacement, although several points of the design are still under consideration. The scenic views and historic nature of the original Waldo-Hancock Bridge have dictated that the new design replace the existing structure not only in function, but also as a landmark. In order to maintain the feel of the original structure, the piers were placed on the banks of the Penobscot River, not in the river itself. The piles for the new structure have already been placed, and the replacement bridge is being quickly built. Sadly, the old Waldo-Hancock Bridge will be demolished when construction is complete. But perhaps



Waldo-Hancock Bridge primary and secondary cable systems.

the current aspect of the Waldo-Hancock Bridge is even sadder. Severe rusting on the towers and strands is visible from far away, and the rushed rehabilitation work has completely masked the elegance of the original span. What was once an award-winning and beautifully slender span has become a hulking mess of rusted steel. While many local residents might mourn the loss of such a beautiful and historic landmark in coastal Maine, the deteriorated state of the bridge has many people eagerly awaiting the new structure that will grace the Penobscot River.

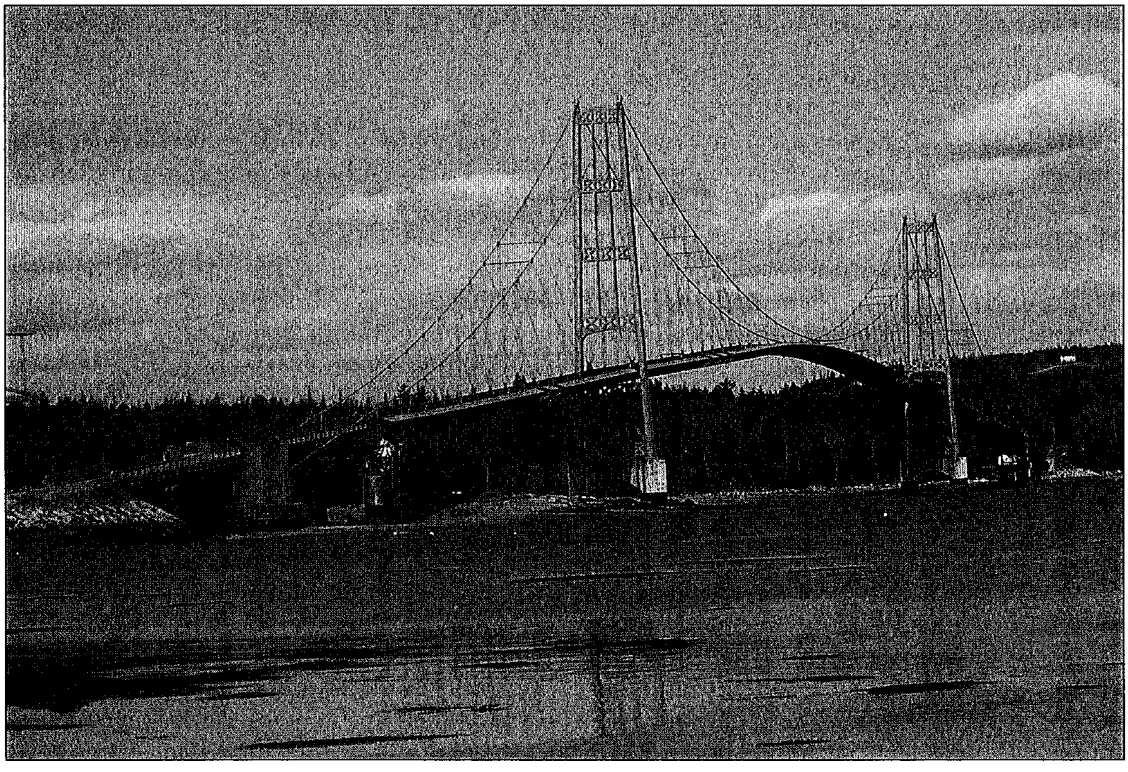
The Deer Isle Bridge

Located between Sedgwick and Deer Isle, Maine, the Deer Isle Bridge carries State Route 15 across Eggemoggin Reach. This crossing of Penobscot Bay is the primary point of entry to the sleepy resort area of Deer Isle, and is a well-known crossing for anyone visiting Acadia.

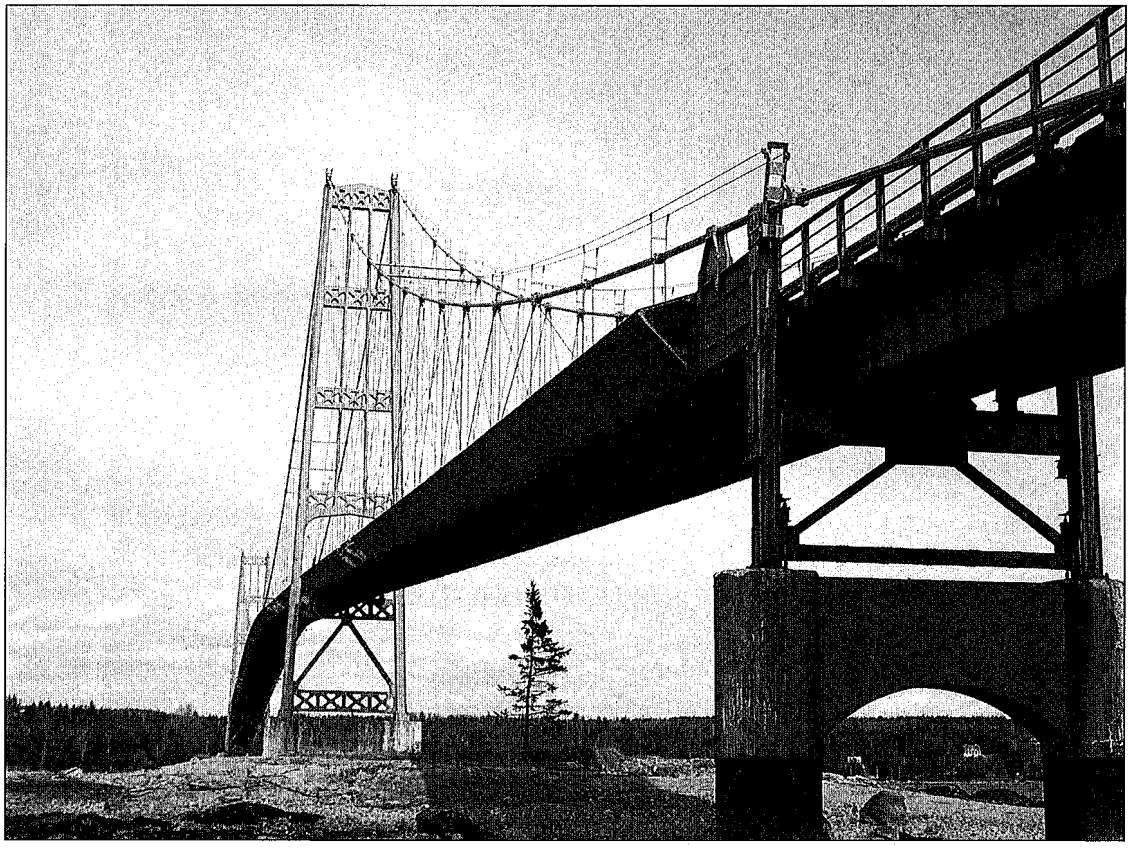
The residents of Deer Isle have led a relatively secluded life for centuries. Since the island was too small to be economically influential, bridge crossings were not considered vital. Ferry crossings were the only means of access for residents and travelers. But Deer Isle's one major export eventually helped bring about its first bridge crossing to the mainland. Deer Isle granite was highly valued because of its stability and aesthetic appeal. Demand for the stone reached all the way into New York. In the 1920s and 1930s, Deer Isle granite was extremely prized since steel and reinforced concrete construction had not yet overtaken stone as construction materials. The high cost of the granite was further exacerbated by the difficulty in obtaining it. The only access to the island was by ferry — not the most efficient system for transporting large amounts of stone. The need for better access to the island was growing rapidly.

Demand for a bridge increased as the rise of automobile use in the United States heightened travelers' frustrations with the ferry crossing. This last of the three suspension bridges in New England designed by David B. Steinam's firm was constructed in 1939. The construction of the bridge was considered the most important moment in the history of the island by its residents. It was not uncommon to have several boatloads of observers floating around the newly formed piers of the bridge during construction. For residents who, for generations, had been isolated from the rest of the United States, simple and direct access to the mainland brought the promise of many changes. Hope ran high that automobile access to Deer Isle would bring additional income to the island so there would not be so much reliance on the granite trade. In the end, the bridge became a major reason why Deer Isle is a quiet and readily accessible vacation spot for many on the east coast today.[18]

Deer Isle Bridge, Maine, from the west.



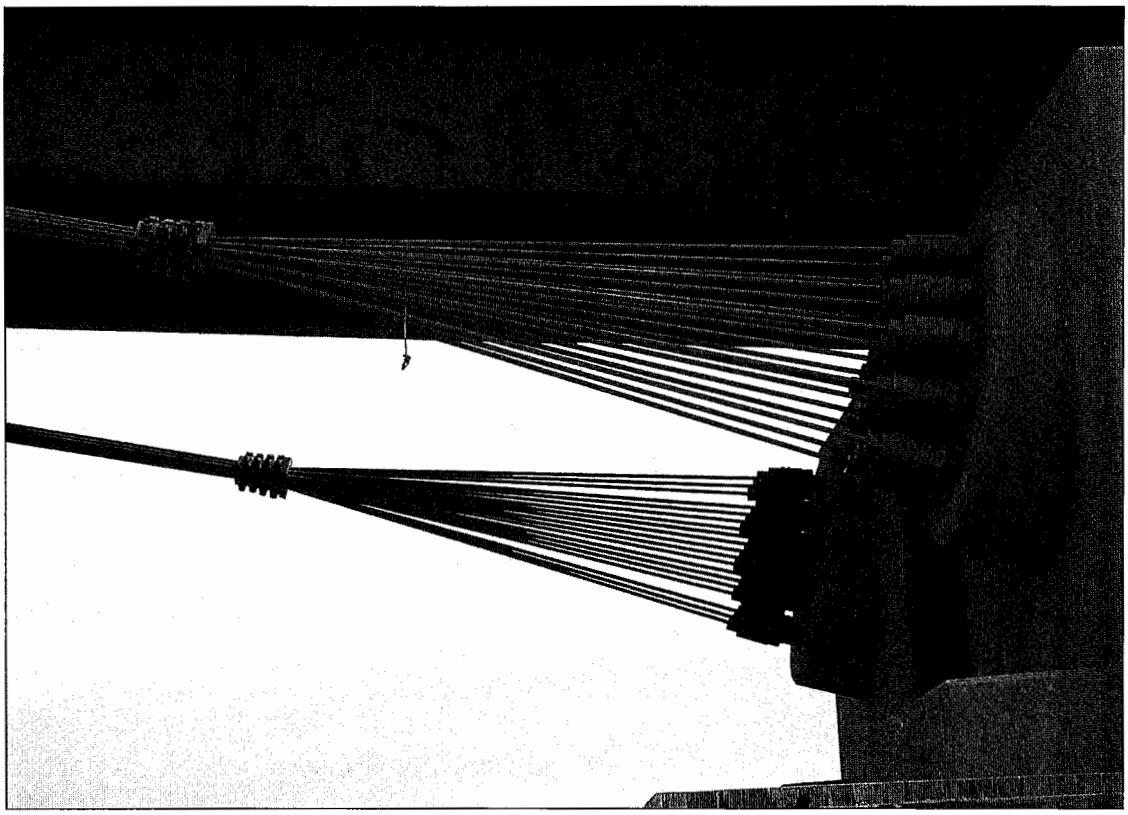
Deer Isle Bridge from the east.



Deer Isle Bridge orthotropic deck retrofit.

The Deer Isle Bridge is a gravity-anchored suspension bridge with a main span of 1,081 feet. It has steel pylons and piers and it reaches a maximum height of 210 feet. The cable system is unwrapped (meaning that there is no protective casing system and the cables themselves are exposed). Each individual cable was coated to protect against corrosion. The cables run into concrete abutments at the ends without any end casings. The bridge has a thin plate girder deck, one of several that were designed during this period. This method of deck construction led to elegant, slender designs, but also made the bridge susceptible to failure from lateral wind loads. The Tacoma Narrows Bridge is the most famous example of this type of deck design. The pitch of the Deer Isle Bridge is dramatic and it borders on parabolic when seen in elevation. The bridge has one lane in each direction.

Almost immediately after its construction travelers began to report heavy wind-induced loadings of up to 10 inches even in even moderate wind conditions. These reports led the designer/contractor to retrofit the bridge with diagonal cable stays in order to prevent major vibrations. After the Tacoma Narrows disaster, the Deer Isle Bridge was heavily monitored. A minimum spacing of 500 feet. between heavy trucks was ordered, as well as load ratings and closings in high winds. Several major wind studies were undertaken during the 1980s, with the end result being a retrofit of aerodynamic fairings to the plate girder deck in order to counteract wind forces. The bridge is now constantly monitored via wind testing mechanisms attached to the deck that were installed in 1981. These mechanisms send real-time data to research and monitoring facilities in Virginia for further analysis. Despite the retrofit, many locals still comment on uncomfortably high vibration levels during high winds, although designers claim a safety factor of 2.5 for the current bridge.[19]



Deer Isle Bridge cables.

The Deer Isle Bridge has become a significant landmark in the Maine landscape over the years. With its extreme grade and slender profile — not to mention the misadventures of crossing in high winds — it had become a local treasure. The profile of the bridge easily blended with the breathtaking landscape that it spanned. Passengers were treated to stunning views while making their crossings. Unfortunately, the retrofitted diagonal cables obscure those views now, and the graceful profile of the deck is concealed behind the aerodynamic fairings.[20]

Aside from the vibration problems, a recent rehabilitation study found the bridge to be in good condition considering its age. The unwrapped cables were very thoroughly inspected and they were rated to be in "good" condition. The lack of corrosion is most likely due to fact that the unwrapped strand system did not have the same issues that plagued the Waldo-Hancock Bridge further south where the wrapping trapped water and accelerated cable corrosion. The inspection also found moderate to heavy local corrosion in the steel superstructure, which is to be expected considering the saltwater environment of the bridge. Currently, the main issue with the bridge is substantial cracking in the deck. While there is no major spalling of the concrete yet, the deck is delaminating and it needs substantial repair. Work is scheduled to begin in 2005.[15]

Perhaps in large part due to the problems associated with the Waldo-Hancock Bridge, there is growing local sentiment to replace the bridge completely in order to avoid safety issues as well as the high cost of rehabilitation. If this movement goes forward, there might be substantial resistance from other residents who place more value on the bridge's historical significance.

NOTES — *RITBA awarded the task of designing the Claiborne Pell Bridge to Parsons, Brinckerhoff, Quade, and Douglas. The chief design engineer*

assigned to the project was Alfred Hedefine, principal engineer and also a partner at Parsons. Bethlehem Steel developed the new technique in stringing the main suspension cables using prefabricated parallel wire strands. Dupont supplied the protective plastic cable sheathing. The Maguire Group, Inc., was contracted by Parsons & Brinckerhoff to provide consultation services in the rehabilitation of the reinforced concrete deck. Design of the Mount Hope Bridge was contracted to Dr. Steinman's firm, Robinson & Steinman. The contractor was McClintic-Marshall. Cianbro carried out the repair work on the Mount Hope Bridge. The rehabilitation study for the Waldo-Hancock bridge was performed by the Parsons Transportation Group. The new cable-stayed replacement for the bridge was designed by the FIGG Engineering Group. Cianbro, Reed, and Reed is the contractor in charge of replacing the Waldo-Hancock Bridge. This article was written as part of a Masters degree program at Tufts University.

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DEREK BARNES *received his masters in civil engineering from Tufts University.*

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Designing & Building the Sagadahoc Bridge Between Bath & Woolwich, Maine

Using a carefully planned and implemented design-build approach led to on-time project completion and with costs on-budget.

GEORGE R. POIRIER, BRUCE VANNOTE, R. KENT MONTGOMERY, WILLIAM J. ROHLEDER, JR., & C. ERIC BURKE

In 1927, the Carlton Bridge was constructed as a steel truss lift bridge to carry U.S. Route 1 and adjacent railroad lines (owned by the Maine Department of Transportation) over the Kennebec River between Bath and Woolwich, Maine. Vacationing motorists seeking Maine coastal tourist destinations and the growth of Bath Iron Works (located in Bath, adjacent to the bridge site — a major shipbuilder contractor and Maine's largest employer) contributed significantly to the 25,000 vehicles per day that crossed the Kennebec River on the Carlton Bridge. Seasonal traffic flows, daily shift changes at Bath Iron Works and frequent openings to allow marine traffic to pass in the river caused significant traffic congestion and unacceptable traffic delays on the narrow two-lane bridge.

To alleviate the congestion and the delays, the state began planning in the early 1990s for a wider structure capable of carrying four lanes of traffic. Through a consensus-building process with the neighboring communities, it was agreed in 1996 that a new bridge would be built parallel to the Carlton Bridge. By that time, $38 million in federal funding that had been targeted for the project was at risk. The funding would expire by October 1997 if it were not committed to the project. There was insufficient time for a conventional design, bid and build process; in addition, it was understood that delays would contribute to high user costs associated with the severe traffic congestion in Bath and Woolwich. As a result, the Maine Department of Transportation (DOT) decided to explore a design-build method of project delivery in order to obtain the $38 million in federal funding and to quickly resolve the increasing traffic burden.

This project would be Maine DOT's first use of the design-build approach.

At that time Maine law required low-bid competition. The Maine legislature, however, at the request of Maine DOT, approved a statutory change to allow design-build for the new bridge project in Bath–Woolwich. Maine DOT, with direct involvement of the Federal Highway Administration (FHWA), reviewed the design-build selection processes that had been used by other state DOTs. Maine DOT used this information to create a state-of-the-art design-build selection process that incorporated the best features from other states and added special improvements to fit its project expectations. A key improvement was the idea of a "value based selection." This improvement was intended to incorporate elements of owner control over the product by weighting the technical merit of a design-builder's concept with price.

During construction, Maine DOT engaged the local community to name the new bridge. Through a public voting process, the new bridge was named the Sagadahoc Bridge, for the county where the bridge is located and for the Native American tribe that historically lived in the area.

Maine Design-Build Approach

Selection of a design-build team for the new Sagadahoc Bridge started in the fall of 1996 with a national advertisement for expressions of interest (EOI) from interested parties. Seven teams responded. Four of those teams were selected to further respond to a defined request for proposal (RFP). Maine DOT offered a $60,000 stipend to each team that submitted a responsive proposal. Each team was to submit a price bid and an enhanced technical proposal that included a set of preliminary bridge plans.

Maine DOT identified key required design criteria in its RFP, including:

- established horizontal and vertical alignments;
- deck/roadway configuration (number of lanes, shoulders, sidewalks, barriers);
- approximate abutment stations (with defined allowable deviation);
- allowable superstructure types (steel or concrete box girders);
- allowable river foundation types (piles or drilled shafts);
- maximum number of allowable piers in the river and area of allowable river bottom impacts (based on pre-determined hydraulic, environmental and navigational requirements); and,
- maximum number of expansion joints.

Prior to advertising this project, Maine DOT concluded all environmental clearances and obtained permits from the United States Coast Guard and the Army Corps of Engineers. These efforts included establishing minimum required navigational clearances under the bridge and maximum allowable area of river channel impacts due to construction and permanent foundations. By establishing a higher vertical profile for the proposed alignment, the need for a costly lift span or movable bridge section was eliminated.

Maine DOT created contract documents for this project with the intent of aligning risk with control. Contract risks were assigned to the design-build team, along with commensurate controls. The owner's supplemental boring program was an example of the efforts made to align control with risks. The project RFP allowed teams to propose pier locations that did not correspond to the exact positions where soil borings had been extracted in the original geotechnical exploration studies. Each team was allowed to request a maximum of ten additional exploratory borings in the river, along with an opportunity to request tests of their own choosing associated with this supplemental foundation exploration. Maine DOT then combined the requests and performed an additional exploratory program at its own expense. This supplemental geotechnical program was expedited so that the results were provided to the proposing teams in sufficient time to be incorporated into their design concepts and bid pricing. Each design-build team was assigned the risks of the geotechnical site conditions and control of managing those risks by allowing them the opportunity to direct additional exploratory work and testing. Maine DOT retained risks

related to hazardous wastes, archeological finds and buried man-made obstructions.

The level of owner involvement in inspection efforts associated with the warranty items was also reduced in accordance with the relative control and risk assigned to the design-builder. Since the design-build team would be ultimately responsible for warranty item performance, the inspection of those items was also its responsibility.

A financial incentive/penalty program of approximately $1 million was offered through a performance specification that addressed concrete properties of permeability, strength, air entrainment and reinforcing steel clearances. The design-build team was assigned control of designing and monitoring the concrete mix since the team assumed the risks associated with the incentive/disincentive program. In addition, financial incentives of $3,000 per day (to a maximum of $1 million) were offered for early completion of construction. Conversely, a liquidated damages penalty of $4,500 per day was established, should the contractor exceed the scheduled completion date.

The RFP documents were crafted to balance design considerations without hindering the creativity and talents of the proposing design-build teams. Additional needs of the owner that were prescribed in the RFP included:

- granite masonry pier protection for all river piers (Maine DOT had successful experience with this approach in protecting piers at the water level from abrasive ice floes and the significant salt water tides that are common on the Kennebec River);
- trapezoidal box beams of either weathering steel or concrete to address aesthetic and long-term serviceability needs; and,
- epoxy-coated steel and corrosion inhibitor admixtures for long-term durability.

Each design-build team was allowed flexibility in choosing significant design parameters such as pier locations, span lengths, traffic and construction staging, and minor adjustments in roadway geometry, within prescribed limits.

Prescribing these items in the RFP resulted in the owner's specific expectations being met, without making the proposing teams guess owner preferences. It was the intention of the owner to encourage an economical bid price by reducing the uncertainty on the project requirements.

Maine DOT received separate technical and price proposals from each team in July 1997. The first evaluation step was to verify that each technical proposal responded to all required criteria. Of the four proposals submitted, two were found to contain flaws and were categorized as non-responsive and not eligible for further consideration. The corresponding price proposals (bids) were returned to these teams unopened. The flaws included proposed superstructure and river foundation types that were not allowed, along with geometry deviations not in compliance with the alignments established in the RFP. The teams with unresponsive proposals relinquished the $60,000 stipend.

A diverse committee of nineteen members then evaluated the two remaining technical proposals. The committee consisted of:

- Nine Maine DOT members — two bridge design, two construction, one bridge maintenance, one geotechnical, two traffic, one environmental services;
- Four FHWA members — one each from bridge design/construction/maintenance, highway geometrics/safety, traffic/safety and materials/maintenance;
- One peer state member from the Texas DOT (with expertise in bridge design and construction);
- Two local members, one each from the City of Bath and the Town of Woolwich;
- One University of Maine engineering professor;
- One state historic preservation officer; and,
- One private architect consultant.

Scores for the categories were compiled with a maximum possible aggregate technical proposal score of 100 points (see Table 1). Each committee member scored only the items within their expertise.

TABLE 1.
Category Scoring Weight

1. Understanding Scope of Work 10%
2. Quality of Design 17%
3. Durability 20%
4. Maintainability 12%
5. Navigational Vertical Clearance 1%
6. Quality of Schedule 5%
7. Community Impacts 5%
8. Aesthetics 5%
9. Quality of Construction 10%
10. Maintenance of Traffic 15%

The bid openings began by announcing the technical scores from the committee for each bidder. The public opening and reading of the two price proposal bids followed. An overall "best-value" rating was computed as the lump sum bid price divided by the corresponding technical score. From this computation each teams' technical and price proposal resulted in an effective price per technical score point. The project was awarded to the team with the lowest effective price per technical score point.

Winning Team Proposal

The successful team consisted of a general contractor and a lead designer. The expense for developing the winning proposal was carried primarily by the contractor and the lead designer, with contributions from several other team members, which was partially offset by the $60,000 stipend. This design-build team had both the highest technical score (92) and the lowest price ($46.6 million). In contrast, the competing team's unsuccessful proposal had a bid price of $51.3 million for a steel box girder bridge and received a technical score of 76. Table 2 summarizes these resulting selection parameters and overall "best-value" rating.

Examination of these results reveals that the competitor would have needed to decrease their bid by more than $12 million in order to overcome the lower technical score.

The entire design-build selection process required only ten months from September 1996 to July 1997. Maine DOT's decision to use the design-build process allowed the State of Maine to successfully secure federal funding for the project by the October 1997 deadline and to aggressively address the traffic congestion issue.

Quality Assurance/Quality Control

The design-build process was structured so that Maine DOT performed a quality assurance role, and the design-build team provided quality control. These responsibilities were defined in the RFP requirements and the construction contract.

The quality assurance role extended to the design and construction components of the project. The design review involved only concepts and contract compliance, without performing calculations and was performed by a consultant retained by Maine DOT. Maine DOT retained a second consultant to monitor construction scheduling, with a focus on claims avoidance. This monitoring program

TABLE 2.
Design-Build Team Proposal Comparison

Team	Bid Price	Technical Score	Best-Value Rating*
1	$46.6 million	92	$506,500
2	$51.3 million	76	$675,000

Note: * Calculated as bid price divided by technical score to determine $ per technical point.

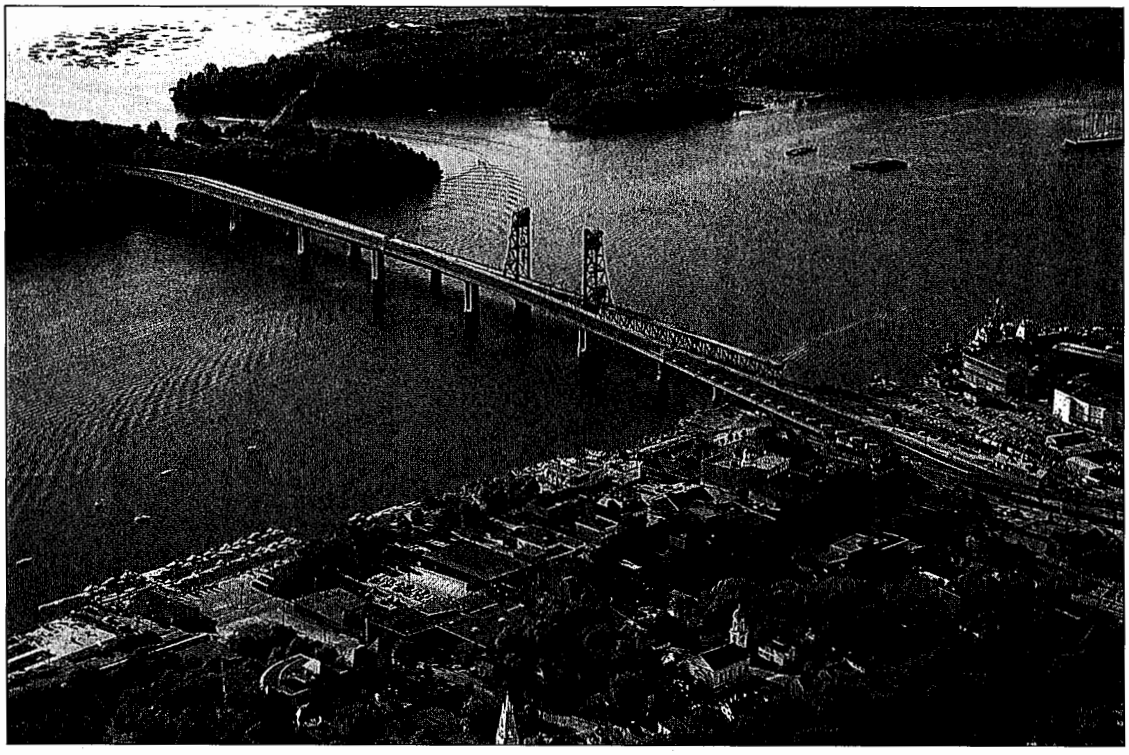

FIGURE 1. An aerial view of the new Sagadahoc Bridge.

was executed through site visits and reports on the general progress and quality of the completed project.

Design quality control was provided by the designer through an independent review performed by another team from the same company, located in a separate design office. Design-build team field representatives managed construction quality control. A detailed quality control manual, prepared by the design-build team and submitted for Maine DOT approval, provided guidance. The manual included descriptions of testing and documentation procedures. Throughout the project, the design-build team performed all material testing, construction inspection and quality control documentation. To close out the project in conformance with project requirements, the design-build team submitted to Maine DOT a record of complete quality control documentation including plans, specifications, calculations, working drawings, test results and certifications.

Maine DOT required a warranty on the specialty features of the bridge, including:

- A five-year warranty for structural bearings, sign and light supports, sign panels, luminaries, pavement and granite pier construction.
- A ten-year warranty for bridge deck expansion joints and the deck waterproofing membrane.

Design Decisions & Influences

The general contractor and lead designer worked closely to develop and evaluate various economical design schemes and alternatives, based on constructibility and schedule considerations. These concepts were also developed in conformance with the specific conditions as described in the owner's RFP documents. The design-build team focused on exceeding Maine DOT expectations. It was understood that any extra value offered by the team would be favorably reflected in the technical evaluation and selection scoring of the team's proposal.

The new Sagadahoc Bridge is a 2,792-foot-long twelve-span structure (see Figure 1). The superstructure consists of:

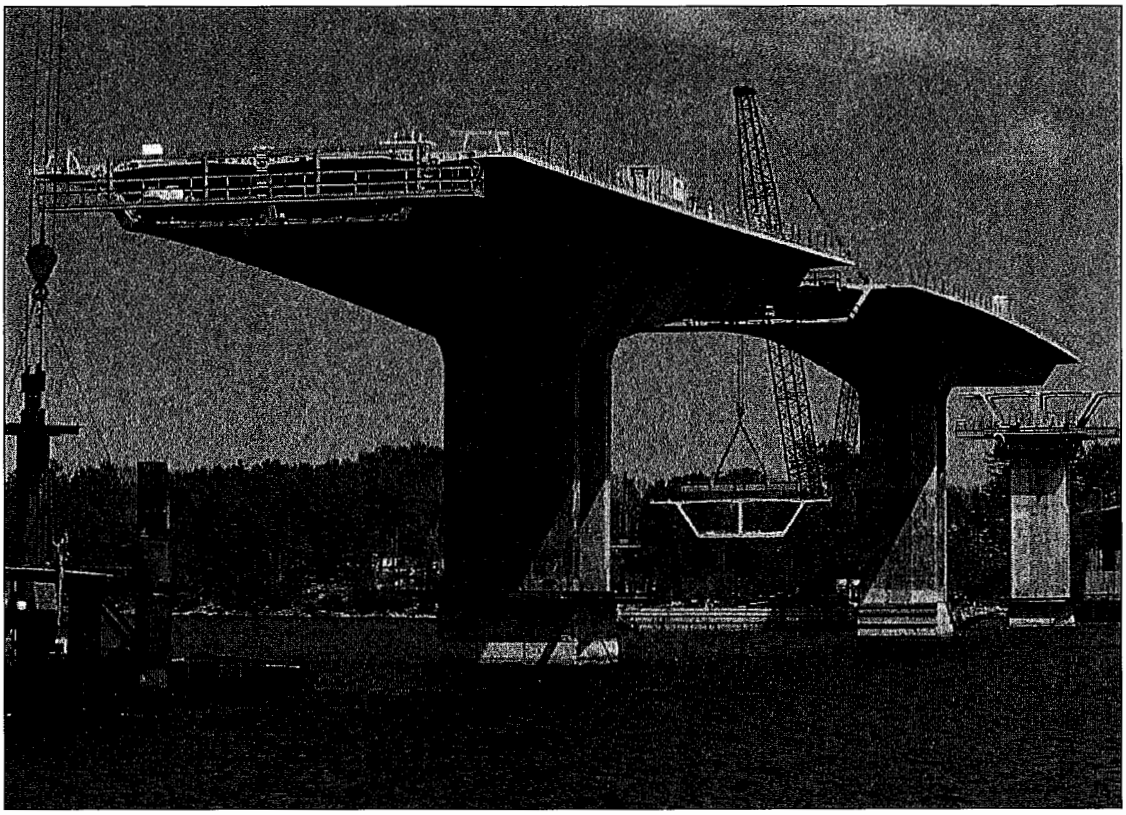

FIGURE 2. Precast balanced cantilever segmental erection.

- *Woolwich approach:* three cast-in-place 184-foot, two-cell trapezoidal box girder spans on a 650-foot horizontal curve, with a constant deck width of 69 feet and a constant superstructure depth of 9 feet.
- *Main span unit:* six precast two-cell trapezoidal box girder spans (west to east span lengths of 262, 420, 380, 331, 331 and 203 feet), a constant deck width of 69 feet and a variable superstructure depth from 20 feet at the main piers to 9 feet at midspan.
- *Bath approach:* three cast-in-place 164-foot four-cell trapezoidal box girder spans, a flared deck width (69 to 120 feet) and a constant superstructure depth of 9 feet.

The use of multiple foundation types is an example of the benefits that evolve from a design-build process where the designer and builder interact to optimize the contractor's resources and scheduling conditions.

The river foundations consisted of four 8-foot-diameter shafts under the main piers and three 8-foot-diameter shafts under the side piers. The shafts were capped with rectangular footings from 31 to 36 feet in width and 12 feet in height, supporting twin wall columns. Both abutments, and three of the four land-based piers, were founded on spread footing widths of approximately 14 to 28 feet and heights from 6 to 8 feet, bearing directly on sound rock. One pier (land-based Pier 12 on the Woolwich approach) was founded on H-piles due to a localized variability in the rock strata. This footing incorporated forty HP360 x 132 piles, each approximately 22 feet long.

Maine DOT identified in the RFP that only driven piles or drilled shaft foundations could be used in the river. Spread footings in the river were not allowed due to scour concerns and maximum impact area agreements with environmental agencies. Long span lengths were explored by the team in order to limit the number of expensive deep-water piers con-

structed in the river. The decision to use a record 420-foot-long precast segmental main span over the navigation channel evolved from the interest in limiting river foundation construction.

The design team chose to use large 8-foot-diameter drilled shafts for piers in the river. The primary reason was that the shafts are secured with rock sockets and provide stability for the long unbraced lengths associated with an extreme design event using potential full scour. In addition, choosing shafts for foundations benefited the schedule, because doing so allowed foundations to be placed quickly before winter. Under Maine DOT directives, spread footings were permissible on land, where the team chose to utilize them. The close proximity of sound rock near the ground line made this option feasible and economical.

In the RFP, Maine DOT required that a concrete or steel trapezoidal box girder section be used for the superstructure. The design-build team determined that a concrete superstructure would be more economical than steel, based on lower delivered material costs and reduced on-site construction time with winter weather constraints. It was further decided to combine a precast main span unit over the river with cast-in-place approach spans on each end as the optimum design.

Use of a precast unit over the river minimized operations over water and allowed precast activities to continue through the winter, benefiting the project schedule. The balanced cantilever method of erecting the precast segments was the most economical technique for constructing the long spans over the river (see Figure 2). Concurrently, cast-in-place spans over land accommodated the flared deck. It also allowed for the convenient staging of construction that would help when shifting traffic from the existing bridge to the new bridge and would eliminate the need for large land-based cranes.

The single 69-foot-wide two-cell box girder was selected for the superstructure to reduce by 50 percent the segment precasting and erection operations needed for separate parallel single-wall box girders. The trade-off was the need for lifting equipment with slightly larger capacity to handle the heavier segments (a maximum weight of 100 tons). Another benefit of using one wide single-box superstructure was a reduction in the required number of piers by 50 percent. Fewer piers make for a less cluttered and more aesthetically pleasing substructure. Additional design decisions were developed through coordination between the designer and builder that improved constructibility. These design considerations included twin-wall piers and a special foundation-forming system described as a "lost" cofferdam.

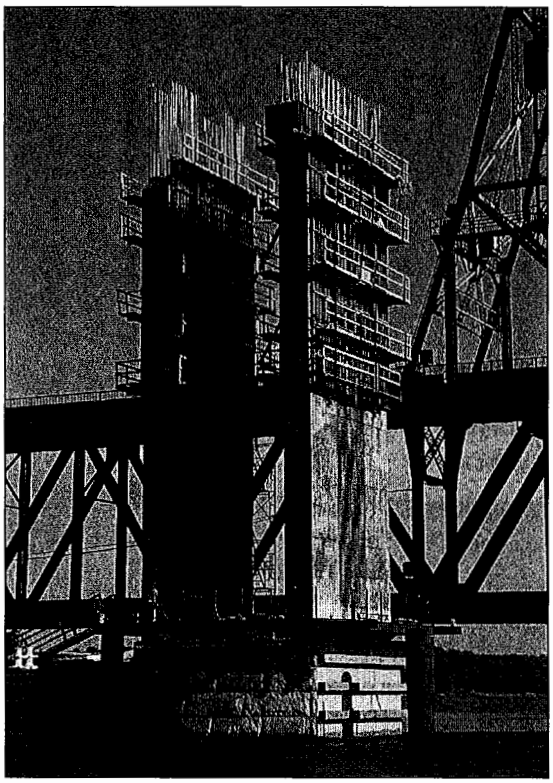

FIGURE 3. Twin wall piers provide structural flexibility.

The twin wall piers deliver the benefits of improved structural flexibility for a long bridge length between expansion joints, stability for the overturning cantilever moment and a wide pier table for beginning balanced cantilever construction (see Figure 3). An innovative construction method using a precast concrete floating "lost" cofferdam was developed to minimize the expense of building a full-depth cofferdam in the 40-foot-deep water

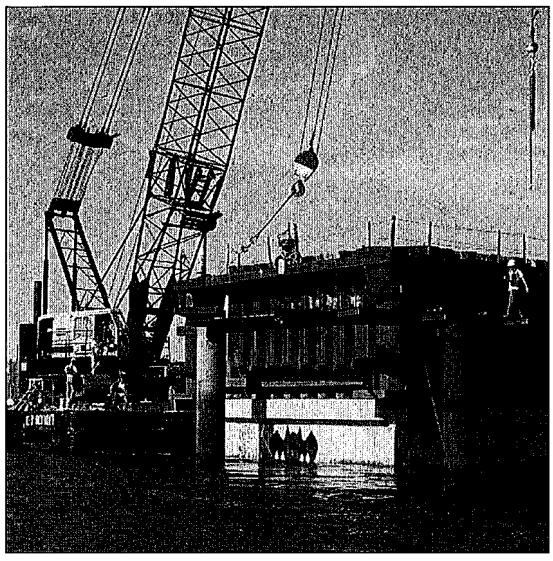

FIGURE 4. Precast concrete "lost" cofferdam with removable steel frame.

around the river piers (see Figure 4). (It was labeled as "lost" because this floating cofferdam did not contribute to the structural capacity of the foundation, but it remains still in place.)

Long-term durability of the bridge was enhanced with several special design features. Maine DOT required that the pier bases be clad with granite for protection from ice floes (a proven technique). In addition, the faces of the pier bases were sloped to encourage the breaking of ice floes, thereby allowing a reduction in the corresponding design loads. The concrete superstructure incorporated 6,000 psi concrete with fly ash to decrease permeability and with calcium nitrite for corrosion resistance. The superstructure is longitudinally post-tensioned along with transverse post-tensioning in the deck. This bi-axial state of pre-compression provided enhanced durability. The deck was further protected with a replaceable asphalt wearing surface over a waterproof membrane.

The citizens of Bath and Woolwich were deeply involved throughout the design-build process. The two communities were interested in both the functional and visual qualities of their new bridge. The design-build team encouraged community involvement by offering a design charette during the fast-tracked design phase. The local communities enthusiastically embraced this offer. Through this process of public involvement, the charette participants selected specialty lighting for the piers, decorative light fixtures on the deck and textured finishes with color-stained concrete for the piers and superstructure. Community pride, fostered through the use of this public involvement process, was evident at the day-long grand opening celebration.

Construction Process

Construction of the Sagadahoc Bridge began in the fall of 1997. Construction was influenced by site-specific challenges, including the severe Maine winter weather, significant tidal flows in the Kennebec River and the need to stage construction to transfer traffic from the old bridge to the new bridge.

Construction began with the Woolwich approach footings and abutments, installation of drilled shafts for Piers 3 through 9 and the concurrent assembly of a casting yard along the Bath waterfront, within view of the bridge site. The 8-foot-diameter drilled shafts were constructed in permanent casings that extended through sediment layers in the riverbed and keyed into the underlying rock strata. Sockets were drilled into the rock and the shafts were poured with tremies in the wet holes. The first drilled shaft was poured for the project in March 1998 and all of the twenty-two required shafts were completed within eight months (see Figures 5 & 6).

Eight steel pipes were installed in each shaft in order to conduct cross-hole sonic log testing of the as-constructed shaft integrity. Two shafts also incorporated an Osterberg-cell (O-cell) for subsequently measuring the load capacity of the drilled shaft. However, the cross-hole sonic log testing was not a contract requirement. This testing was provided by the design-build team under the quality control program and as an extra benefit to the owner to verify the integrity of the shafts. The O-cell load tests were performed in accordance with the contractual requirement specified by the Maine DOT. The contract required that a load test be performed for each different bearing material used in the foundations. The design-build team used the load test program

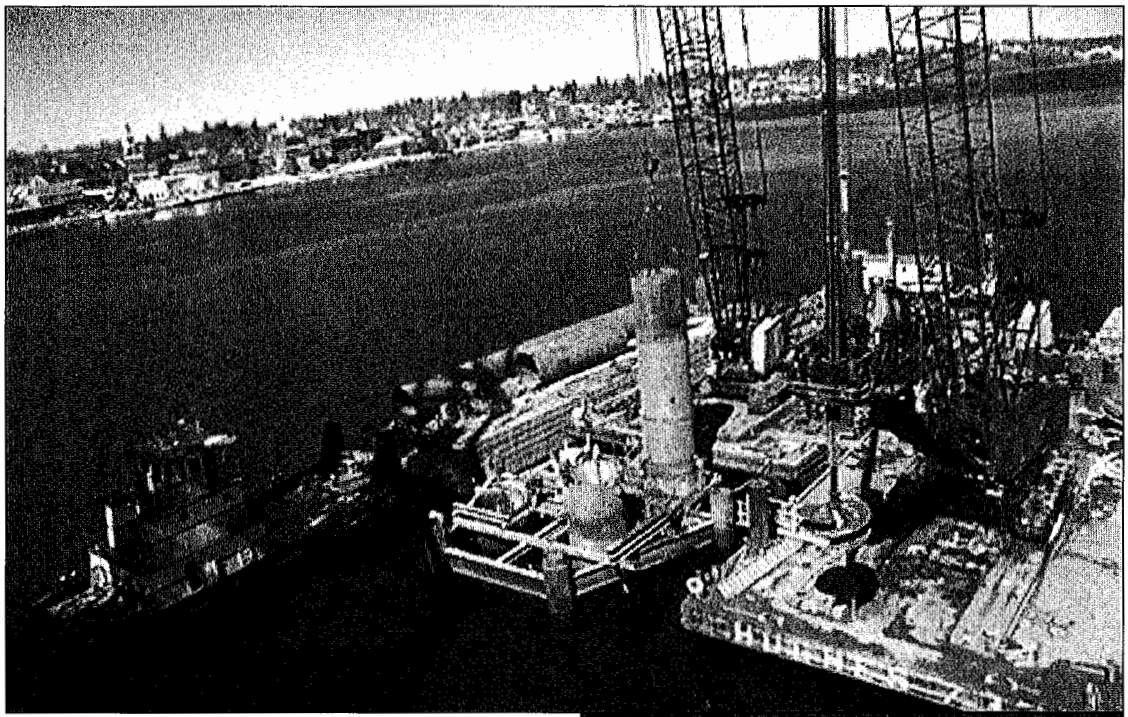

FIGURE 5. Eight-foot-diameter drilled shaft foundations being socketed into bedrock.

to identify increased design capacities and reduce rock socket lengths. The intent was to successfully proof test the 8-foot-diameter production drilled shafts, without actually reaching the ultimate capacity of the shaft.

The first O-cell test location was selected at Pier 6 as representative of the rock strata found at the locations of the shafts for Piers 3 through 6, where the highest shaft capacity was expected. An 8-foot-diameter production drilled shaft was installed at Pier 6 with a 7.5-foot-diameter rock socket of approximately 12 feet in length, based on the AASHTO design method for computing allowable shaft capacities. The O-cell load tests were used to check the design by loading the actual test shaft to approximately three times the design load. Once the loading was completed, the shaft was unloaded and reloaded to approximately two times the design load to ensure the structural integrity of the shaft for use as a production shaft.

Ultimate design values for shaft capacity were computed using an AASHTO method for estimating capacity with design parame-

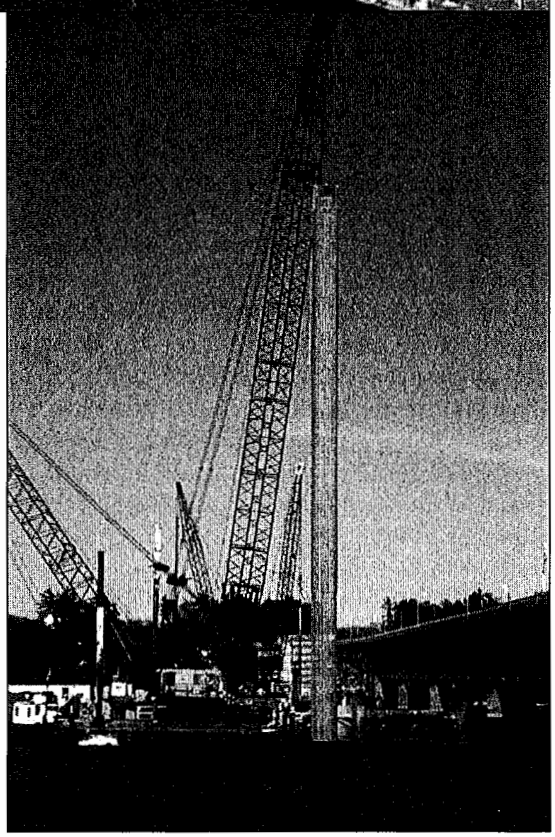

FIGURE 6. First use of large-diameter drilled shafts in Maine (up to 145 feet long).

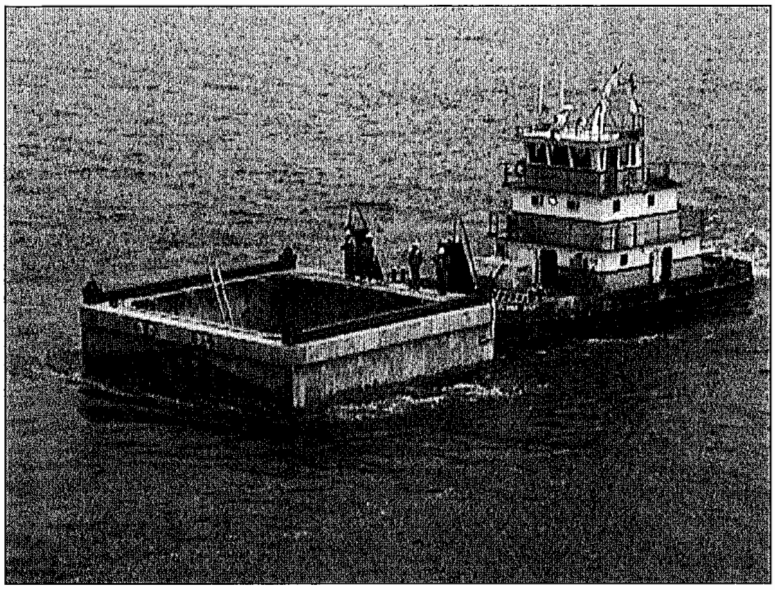

FIGURE 7. Delivery of the unique floating cofferdam.

ters based on an evaluation of subsurface data and laboratory tests from the exploration program. A safety factor of 2.5 was used to determine the rock socket lengths in association with estimated ultimate design values for side shear resistance and end bearing. This AASHTO design method provided an estimate of 17.2 ksf of allowable ultimate unit side resistance capacity and 275 ksf of ultimate end bearing capacity.

Since the O-cell load test applies an equal upward and downward force on the rock socket, the maximum downward load applied during the test did not approach the ultimate end bearing capacity of the shaft. Therefore, the design-build team could not determine an actual ultimate end bearing value from this load test. However, the load test did proof the shaft to three times the design load.

An actual unit side shear of 37.6 ksf was recorded from the O-cell test on the Pier 6 shaft. Examination of the data indicated that this value was close to the estimated ultimate unit side capacity of 41.8 ksf. Using these O-cell load test results indicated that the side resistance capacity of the shaft that was higher than what was estimated under the AASHTO method by a factor of 2.4 for this particular large-diameter drilled shaft and type of bearing material. It was also found that the fractured rock within the upper 8 feet at this shaft location (typically assumed to provide very little capacity under the AASHTO method) in reality contributed significantly to actual side shear resistance. However, this layer of fractured rock was conservatively disregarded in computing the design capacity.

A second load test location was selected by the design-build team to represent the rock strata that were expected to offer the least shaft capacity. A location at Pier 7 was chosen to represent the foundation design parameters for Piers 7 through 9. The 8-foot-diameter production drilled shaft at this Pier 7 test location was installed with a 7.5-foot-diameter rock socket of approximately 19 feet in length, determined using the AASHTO drilled shaft design method. This design method provided an estimated 13 ksf of ultimate unit side resistance capacity and 275 ksf of ultimate end bearing capacity.

Also since this second load test applied equal upward and downward forces on the rock socket, the maximum loads applied during the test did not approach either the ultimate side shear or end bearing capacity of the shaft. Therefore, the design-build team could not determine an actual ultimate side shear or end bearing value from this load test. However, this second O-cell load test was used to successfully proof test the design by loading the actual Pier 7 test shaft to approximately three times the design load.

The second load test results indicated an ultimate unit side resistance in excess of 19 ksf. Using this actual load test result indicated that the side resistance capacity of the shaft that was 1.5 times higher than the conservative estimate developed under the AASHTO method for this particular large-diameter drilled shaft and type of bearing material.

Using load tests was a valuable tool for optimizing and verifying the foundation design. Results from the initial load test were used to increase the estimated design capacity and economically reduce rock socket lengths for the remaining drilled shaft foundations on the project. The second load test verified the design optimization decisions used on Piers 7 through 9 that were based on results from the first load test.

The floating "lost" cofferdams were each precast on a barge anchored in the Kennebec River at the casting yard. To allow the cofferdams to float, each cofferdam was fitted with steel caps bolted over the individual openings for extension of the drilled shafts through the bottom slab of the precast form. The barge was then floated downriver to the Bath Iron Works facility where two permanent shipbuilding cranes worked in tandem to lift each 300-ton precast cofferdam and place them in the river (see Figure 7). A temporary steel extension was attached to the cofferdams, prior to their being guided into position by tugboat. Once positioned over the completed drilled shafts, the cofferdams were submerged to plan elevation and aligned with the shafts using a temporary frame located around the perimeter of the system as a template. A seal was placed in the bottom of the form and the footing was constructed. The top of footing is located just below the historical low-water mark in the river and the bottom of cofferdam matches the top of drilled shaft elevations.

Because the new Sagadahoc Bridge layout overlapped the old Carlton Bridge at the Bath approach, construction of the new abutment and cast-in-place superstructure was staged to accommodate the transfer of traffic (see Figure 8). After completing the north half of the first span on the Bath approach, traffic was transferred from the Carlton Bridge onto the Sagadahoc Bridge. Then the approach spans of the Carlton Bridge were demolished and the remainder of the new abutment and first span were completed in a second phase. This sequencing required designing the post-tensioning layouts and construction details for an overlap of both longitudinal and transverse tendons between the first and second phase of the superstructure. The lower level of the Carlton Bridge remains in place to allow for rail traffic across the old bridge. The lift span remains in a raised position until those times when rail traffic requires it to be lowered.

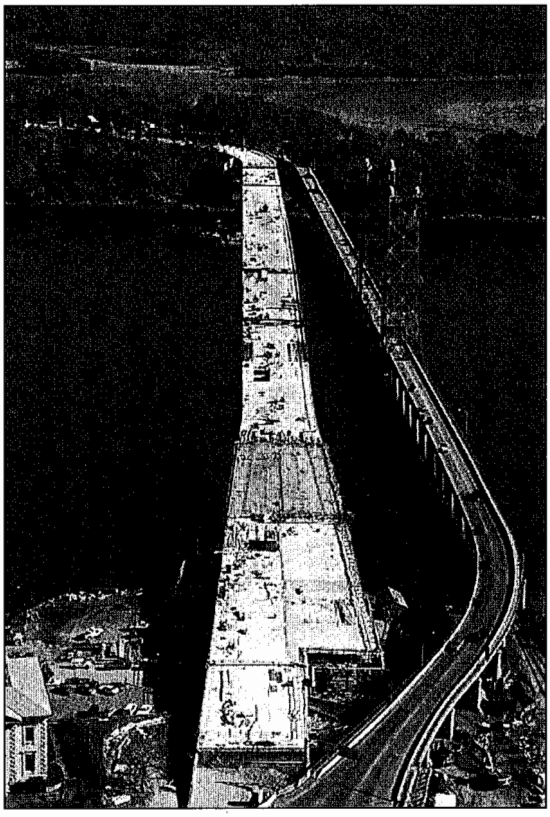

FIGURE 8. The overlapping footprint at the Bath abutment was handled by building the first span in two phases.

The general contractor established the casting yard for the precast main span unit on vacant property along the Bath waterfront, within view of the bridge site (see Figure 9). This property was stabilized to support the casting machine and other heavy equipment. A heated enclosure was constructed around the segment casting bed to allow precasting operations to continue year round. Segments were stored at the casting site prior to erection.

The general contractor cast the first superstructure segment in May 1998 and the last of 202 total main span segments in September 1999 (see Figure 10). The first precast superstructure segment was then erected in September 1998. The final segment was placed in November 1999.

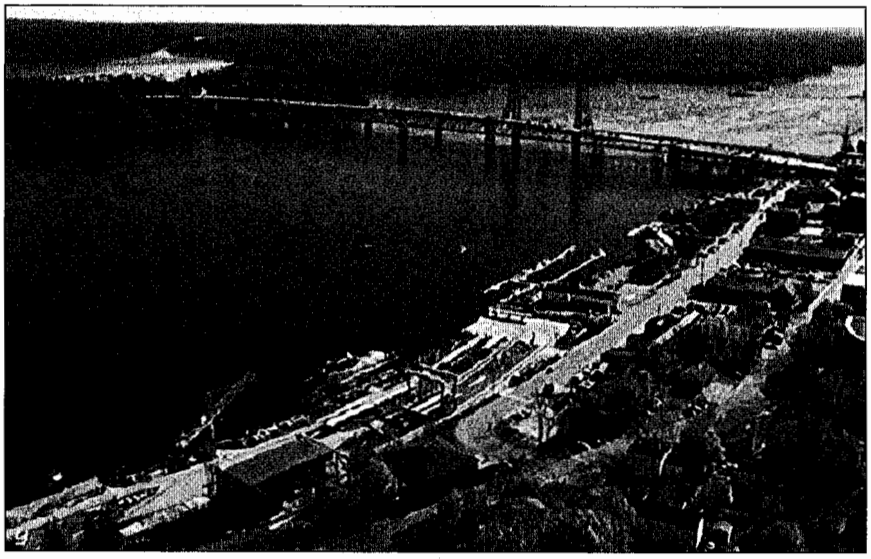

FIGURE 9. The casting yard established adjacent to the bridge site.

The balanced cantilever method for precast segmental erection was used to construct the spans over the river. The segments were delivered by barge from the adjacent casting yard and lifted into place with a barge-mounted crane (see Figure 11). The balanced cantilever process began by constructing a pier table on top of the twin wall piers. A special adaptation of the technique was used to minimize the pier segment weights. Precast segments were erected on top of the twin walls to each side of the twin wall piers (see Figure 12 on page 50). The diaphragm segment was cast in place between the precast elements. The precast segments were then incrementally placed and secured by post-tensioning bars and tendons on alternating sides of the pier table.

Both the Woolwich and Bath approaches are concrete multi-cell box girder superstructures that were cast in place on falsework. The first superstructure placement for the Woolwich approach was performed in August 1998 and casting was complete within five months. The first placement for the three-span, variable-width Bath approach unit occurred in May 1999 and was completed within six months.

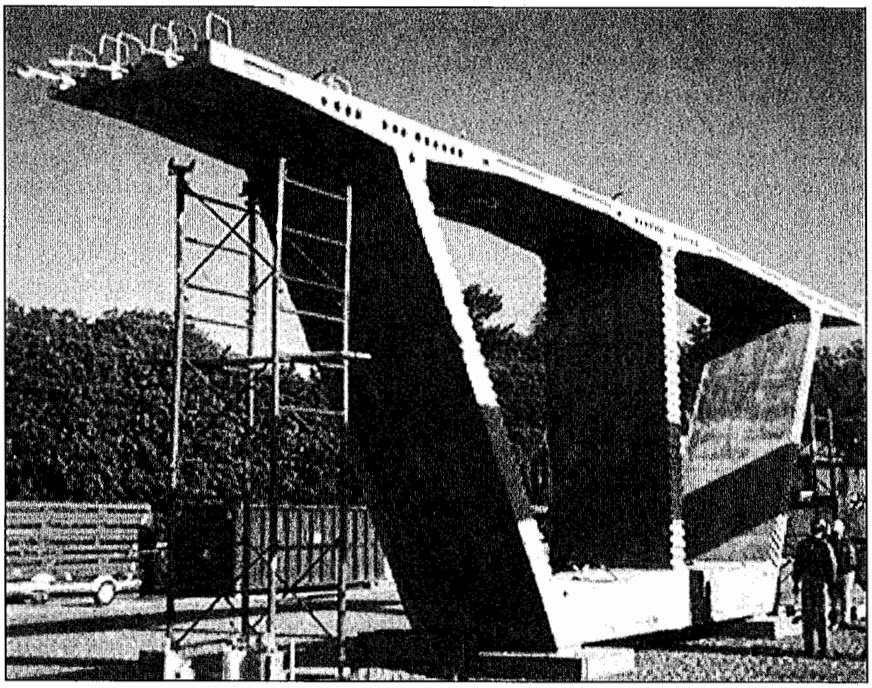

FIGURE 10. Precast segments were 69 feet wide with a variable depth of 9.333 feet to 19 feet 7 inches.

Minimizing the impact of severe Maine winters by precasting the superstructure of the main span unit

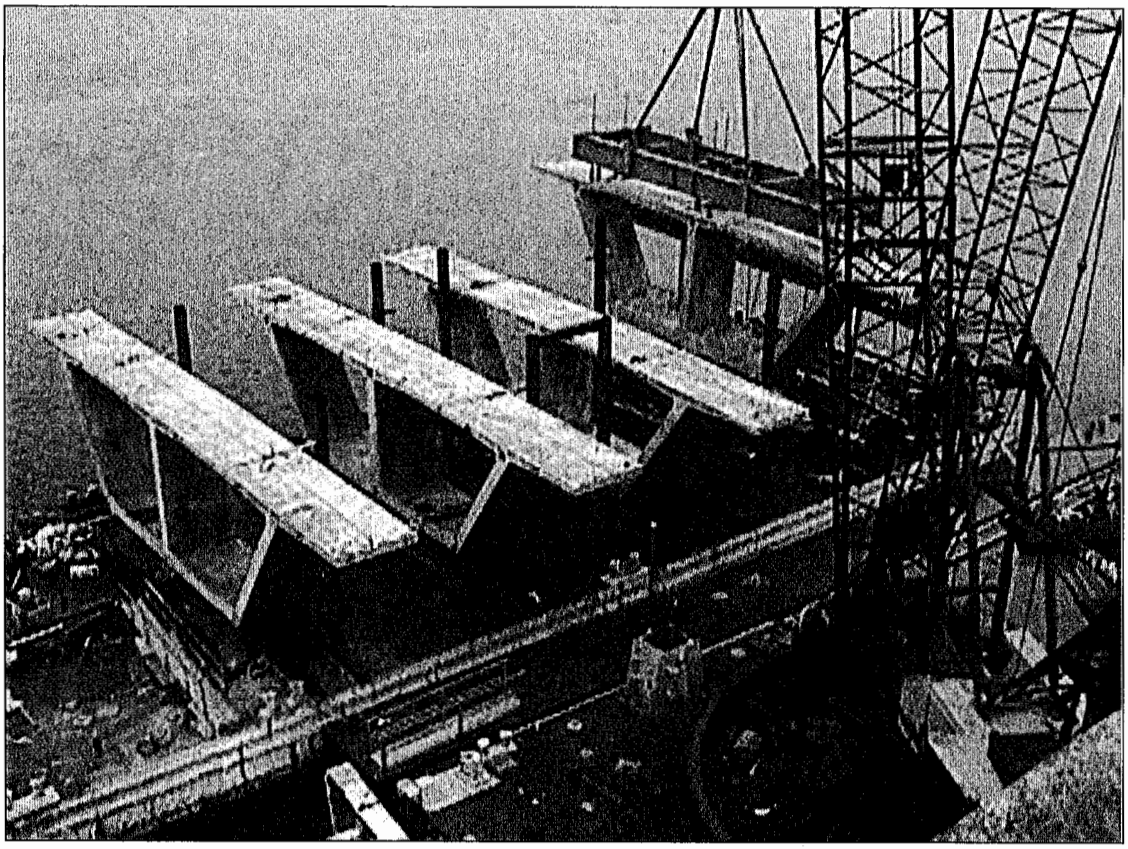

FIGURE 11. Segments were delivered by barge from the precasting site to the site.

contributed to the timely transfer of traffic in May 2000 and completion of the project by August 1, 2000 (see Figure 13 on page 51). The project was completed thirty days early, earning the general contractor a completion bonus of $90,000. In addition, a maximum bonus of approximately $1 million was earned for achieving the concrete quality performance goals. Mixing precast and cast-in-place technologies for the bridge superstructure optimized the overall schedule and provided a quality product.

Appraisal of Finished Product

The State of Maine successfully accomplished its objective of securing federal funding to construct the highest quality new Sagadahoc Bridge for the least price. This project's outcome directly resulted from Maine DOT's use of a customized value-based process for selecting the design-build team. Attention to the following details in the RFP and design-build documents contributed to the successful realization of Maine DOT's objective.

- *Emphasis on a good quality control plan.* The quality control documents set contractual standards and expectations for the design-build team's construction practices. These documents directly influenced product quality and the working relationship between the owner and design-build team field representatives. This project presents a positive example of the importance of attention to quality control. The thoroughness of a responsive design-build team's quality control plan was significantly weighted in the technical scoring.
- *Balance of owner needs with design-build team creativity.* It is preferable for owners to explicitly identify desired items in the RFP, while remaining open to design-build team creativity. This approach involves yielding some control over the

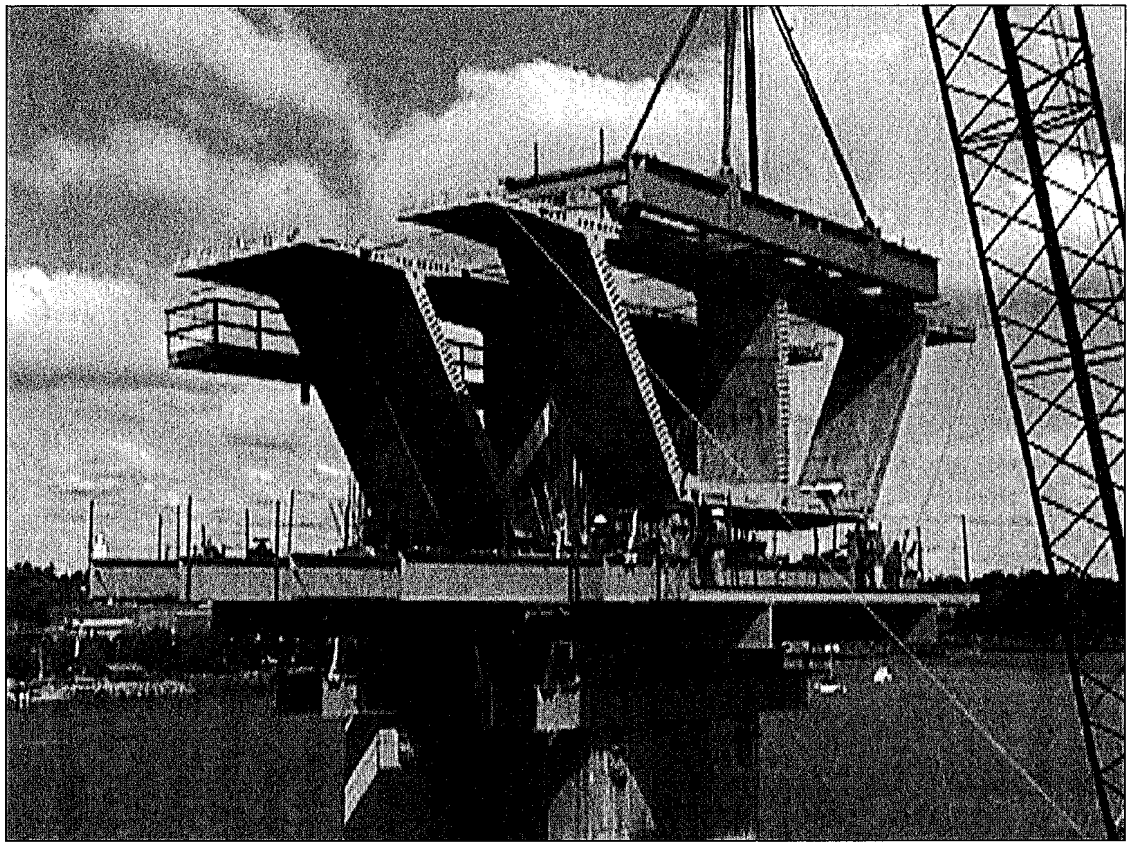

FIGURE 12. Precast segments with cast-in-place diaphragms for pier tables.

design and construction process since the design-build team will be implementing the design and construction at a fast pace. The owner's expectations were met by fully defining the performance specifications as part of the RFP. By releasing control of design details that are not explicitly defined in the RFP, the owner encouraged the possibility for a creative outcome. Given this freedom, the design-builder could introduce an innovative new construction method or design consideration that could then be incorporated by the owner into future projects.

- *Unbiased selection process.* The selection process must be unbiased and comprehensive to fairly and accurately assess and score the proposals. One reason for the success of this project is that the owner defined explicit RFP requirements and then objectively evaluated the proposals based on the submitted proposals' responsiveness to these specific requirements.
- *Explicit monetary incentives/penalties.* This program was successful in motivating the design-build team to provide outstanding materials. The design-build team focused on achieving the defined concrete strength, air entrainment and reinforcing clearances in order to receive financial incentive payments. This program served as a successful method for the owner to obtain quality in the project.
- *Providing a fair contract.* Balancing risk with control is critical to the working relationship between the owner and the design-build team. This balance intrinsically reduces the probability of claims.

As part of the design-build contract, warranty inspections have been performed as prescribed at two and four years after the completion of construction. These warranty

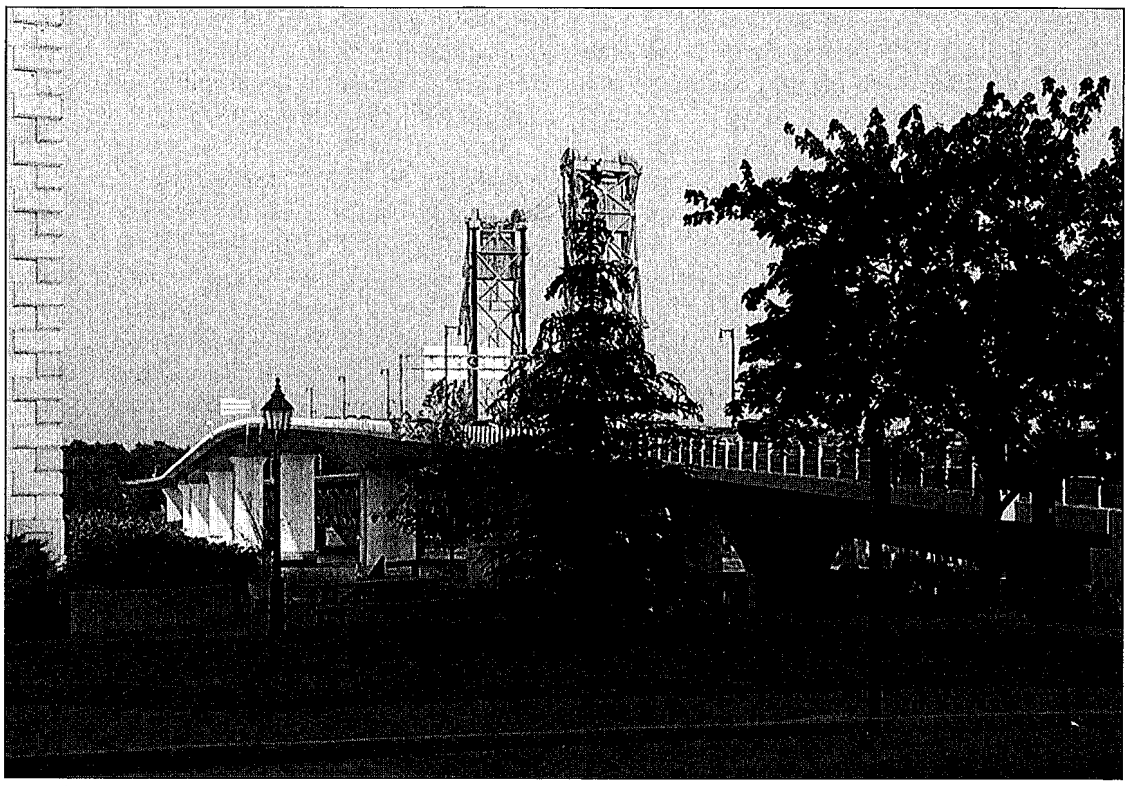

FIGURE 13. A view of the completed Sagadahoc Bridge.

inspections were performed in September 2002 and August 2004 by Maine DOT, along with representatives from the design-build team. After four years of exposure to a wide range of traffic, weather and tidal fluctuations, the inspections confirmed that the structure was in excellent condition.

The design-build team successfully delivered a high-traffic capacity, durable bridge on time and within budget that will provide many years of service. As a bonus, the State of Maine and the local communities are benefiting from the recognition and aesthetic pleasure of this bridge, which has been commended through the following awards:

- 1999 Northern New England Concrete Promotion Association Outstanding Concrete Construction Award;
- 2000 American Council of Engineering Companies, Colorado Chapter Grand Conceptor Award;
- 2001 National Council of Structural Engineering Associations Award of Merit; and,
- 2001 Design Build Institute of America Design Build Excellence Award, Civil (more than $15 million).

ACKNOWLEDGMENTS — *The design-build team for this project consisted of general contractor Flatiron Structures, LLC, with headquarters in Longmont, Colorado, and the lead designer, FIGG Bridge Engineers, Inc., with headquarters in Tallahassee, Florida. The FIGG design effort was led from the firm's Denver, Colorado, office. There were many designers, sub-contractors and material suppliers that contributed to the design-build team. Some of the supporting firms and their responsibilities were: Case Foundations Co. — drilled shaft installation; Libby Steel, Inc. — reinforcing steel placement; S. Williams Concrete, Inc. — concrete supplier; Pike Industries, Inc. — asphalt contractor; A D Electric, Inc. — signing and lighting contractor; A D Rossi, Inc. — traffic rails and waterproof membrane; DSI USA, Inc. — post-tensioning supplier; Ben C. Gerwick, Inc. — cofferdam design; Pine Tree Engineering — roadway and traffic engineering; Law Engineering &*

Environmental Services, Inc. — geotechnical engineering; Bartlett Engineering — electrical engineering; and The Mintz Lighting Group — specialty lighting consultant.

GEORGE R. POIRIER *currently serves as the Federal Highway Administration's (FHWA) Oversight Manager in the Wisconsin Division Office, where he is responsible for statewide field operations and the Marquette Interchange mega-project. His work experience includes eleven years in the FHWA Maine Division where he was responsible for the oversight of the Federal Aid Bridge Program, and twelve years with the California Department of Transportation in bridge design and construction. He attended the University of Vermont were he earned a B.S. in 1980. He is a registered professional engineer in California.*

BRUCE VANNOTE *joined the Maine DOT as an attorney in the department's Legal Division in 1994, where he concentrated on construction, procurement and contracting. Beginning in 2000, he served as director of the department's Office of Policy and Communications, where he worked on a day-to-day basis with the Maine Legislature. He assumed his current position as deputy commissioner in November 2002, where he oversees Maine DOT's operations, capital program delivery, human resources, budget, administration, safety and environmental programs. He holds a B.S. in Engineering from the University of Maine at Orono, and a J.D. from the University of Maine School of Law.*

R. KENT MONTGOMERY *is a Principal Bridge Engineer at FIGG and was the lead technical engineer for the design of the Sagadahoc Bridge and the record-setting 420-foot main span. He has held key roles in the design, construction or inspection of many award-winning long-span concrete and cable-stayed bridges designed by FIGG.*

WILLIAM J. ROHLEDER, JR., *has more than twenty-three years of major bridge design experience and was the project engineer for the Sagadahoc Bridge. He is a regional director of FIGG's Pennsylvania office and leads design teams on complex bridge designs.*

C. ERIC BURKE *is a Senior Project Director in the Geotechnical Division of Wilbur Smith Associates. Previously, he was a Principal Engineer with Law Engineering and Environmental Services and served as the geotechnical engineer of record during design and construction of the Sagadahoc Bridge Project. He is currently based in the Richmond, Virginia, office of Wilbur Smith Associates.*

Lenticular Iron Truss Bridges in Massachusetts

Built from about 1880 to 1900, these bridges represent a unique design during a time when many bridge companies were competing for contracts in a highly competitive market.

ALAN J. LUTENEGGER & AMY B. CERATO

Of the estimated hundred or so lenticular iron truss bridges built by the Berlin Iron Bridge Company between about 1880 and 1900 in the Commonwealth of Massachusetts, only nine are known to still exist. Only three of these bridges are currently open and in use for vehicular traffic. Of the remaining six, two are closed to vehicular traffic, two have been restored and relocated for use as pedestrian/bikeway bridges, and two have been dismantled and their parts are currently in storage, awaiting reuse/restoration.

Lenticular Truss Bridges

During the latter part of the nineteenth century, the Berlin Iron Bridge Company of East Berlin, Connecticut, manufactured and erected something on the order of 400 lenticular truss bridges in the United States.[1,2] These bridges are sometimes referred to as "pumpkin-seed bridges," "fish-belly bridges," "cats-eyes bridges," "elliptical truss bridges," "double bowstring" or "parabolic truss bridges" because of their unique lens shape. Like many other iron truss bridges of the day, these bridges were, in effect, mass produced since the components were built in a factory, sent to the site and then assembled. Many of the components were used repeatedly for different spans or applications.

According to James, lenticular-shaped bridges had previously been used in Europe as early as 1822.[3] It appears that one of the first uses of this type of design was George Stephenson's iron railway bridge designed in 1822 and built between 1823 and 1824 to carry the Stockton & Darlington Railway over the river Gaunless in West Auckland, England. As shown in Figure 1, the bridge consisted of four spans of 12.5 feet (it originally had three spans — the fourth span was added in 1825) with top and bottom chords of wrought iron and the vertical members of cast iron. The members were built by Burrell & Company of Newcastle. The bridge was opened on September 27, 1825, and was in use until about 1856. The bridge stood intact but unused until 1901, when it was dismantled and moved to storage. In 1928, the bridge was re-erected at the York Railway Museum and is currently on display at the British National Railway Museum.

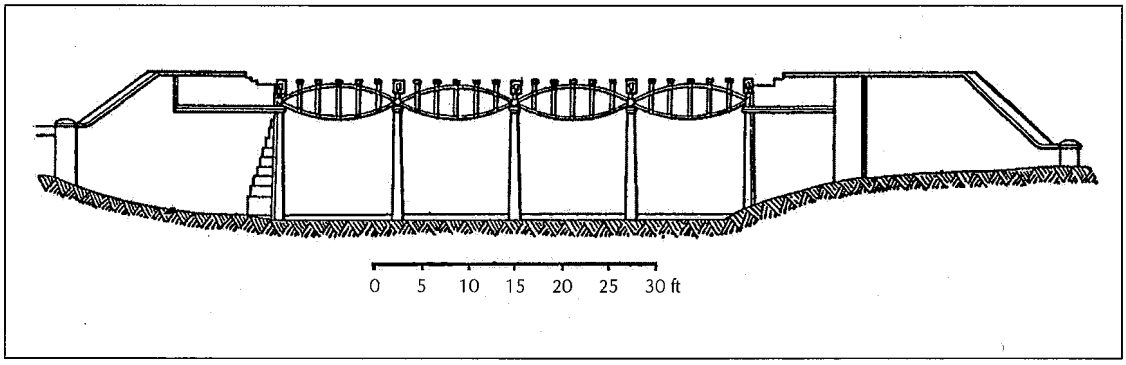

FIGURE 1. Stockton & Darlington Railway Bridge (1823).

One of the most notable bridges of this style was I.K. Brunel's 1855 twin-span lenticular Royal Albert Railway Bridge across the Tamar in Saltash, United Kingdom (see Figure 2). This bridge used tubular upper chords with each span having a span of 445 feet (center to center of the piers). In 1860, the Mainz Bridge was built over the Rhine River in Germany and consisted of at least two large spans and two shorter spans as shown in an early lithograph (see Figure 3). This bridge shows remarkable similarities in form to the lenticular truss bridges built twenty-five years later by the Berlin Iron Bridge Company.

In the United States, Gustav Lindenthal built a lenticular-shaped twin-span bridge across the Monongahela River at Smithfield Street in Pittsburgh in 1883 (see Figure 4). Lindenthal referred to this shape as a "Pauli truss" after the famous German bridge engineer Friedrich August von Pauli (1802–1883).[4] Von Pauli had designed a number of lenticular bridges, referred to as *fischbauchtrager* (fish-bellied) in Germany prior to 1860.[5] Each span

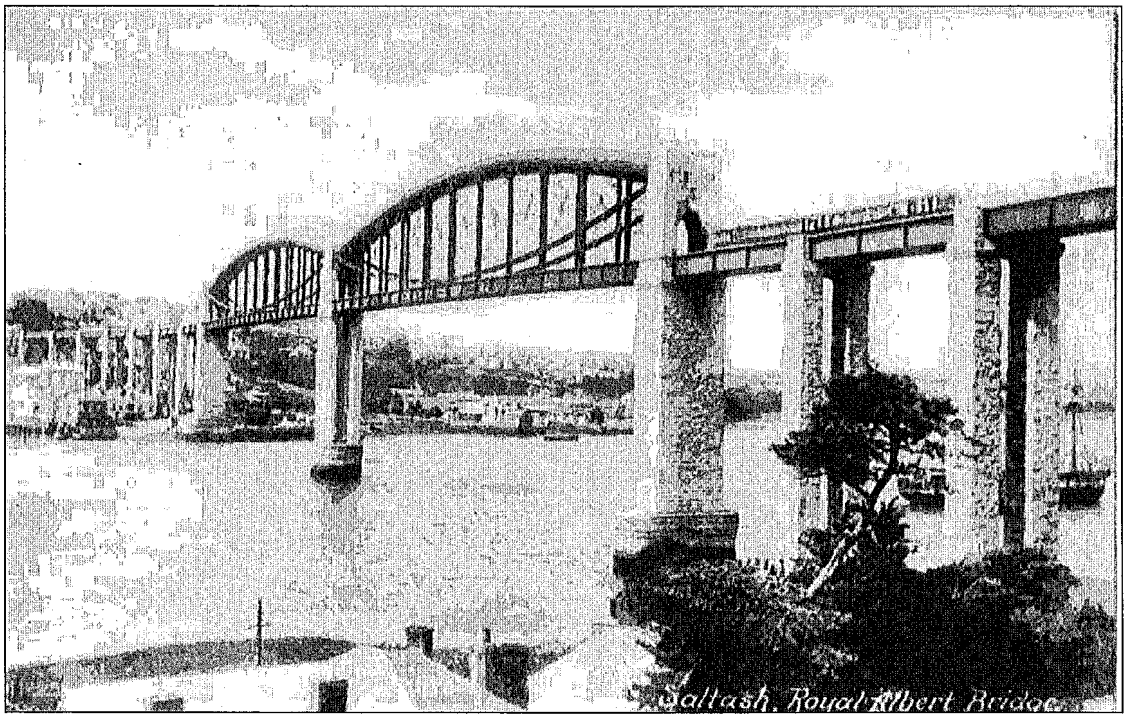

FIGURE 2. I.K. Brunel's Royal Albert Lenticular Bridge (photo circa 1920).

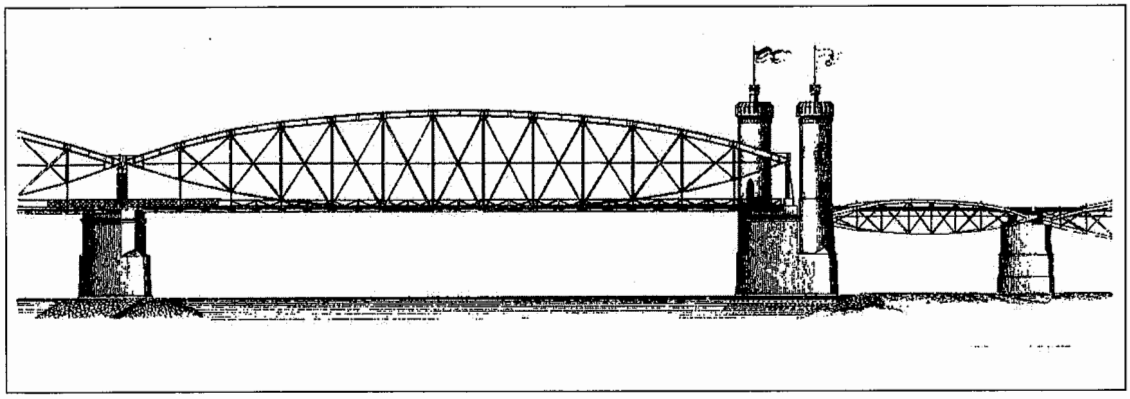

FIGURE 3. Rhine River Bridge in Mainz, Germany.

of the Smithfield Street Bridge originally was constructed using two trusses; a third truss was added to carry additional lanes of traffic in 1891. This bridge replaced an earlier one designed and built by John Roebling, but did not really receive the attention that perhaps Lindenthal had been hoping for. This lack of recognition may have been in part related to the fact that another bridge that opened in 1883, one that may have had considerably more importance at the time — namely, the Brooklyn Bridge. However, the structures of von Pauli, Brunel and Lindenthal were unique, single event, monumental bridges, never to be duplicated in any close form by any other engineer at any other location.

By contrast to these few single large-scale structures, the hundreds of smaller lenticular truss bridges built by the Berlin Iron Bridge Company were catalog bridges and their designs were duplicated many times throughout New England and the mid-Atlantic states. In fact, the Berlin Iron Bridge Company built the only lenticular iron truss bridges known to

FIGURE 4. Gustav Lindenthal's Smithfield Street Bridge in Pittsburgh.

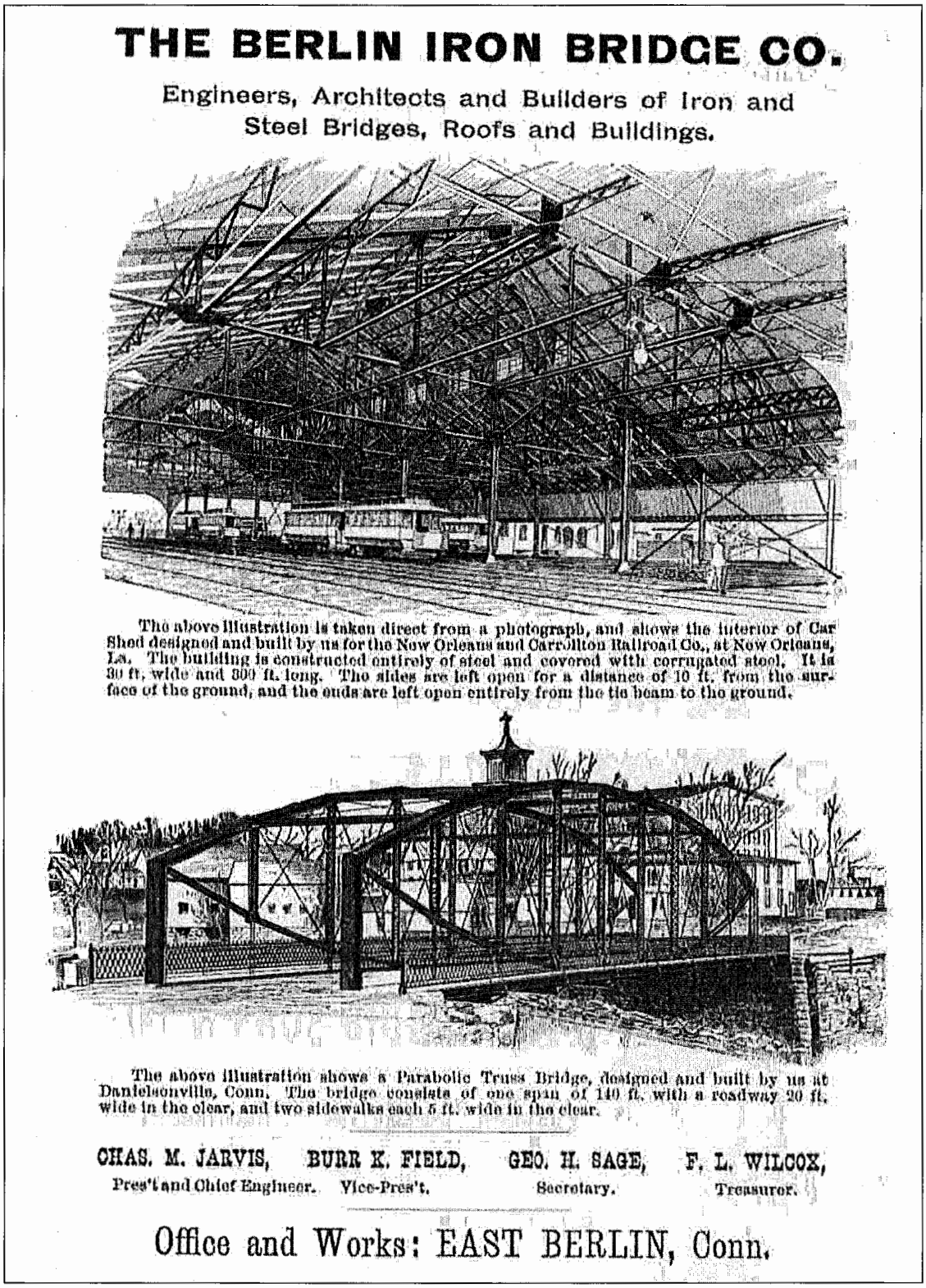

FIGURE 5. Berlin Iron Bridge Company advertisement.

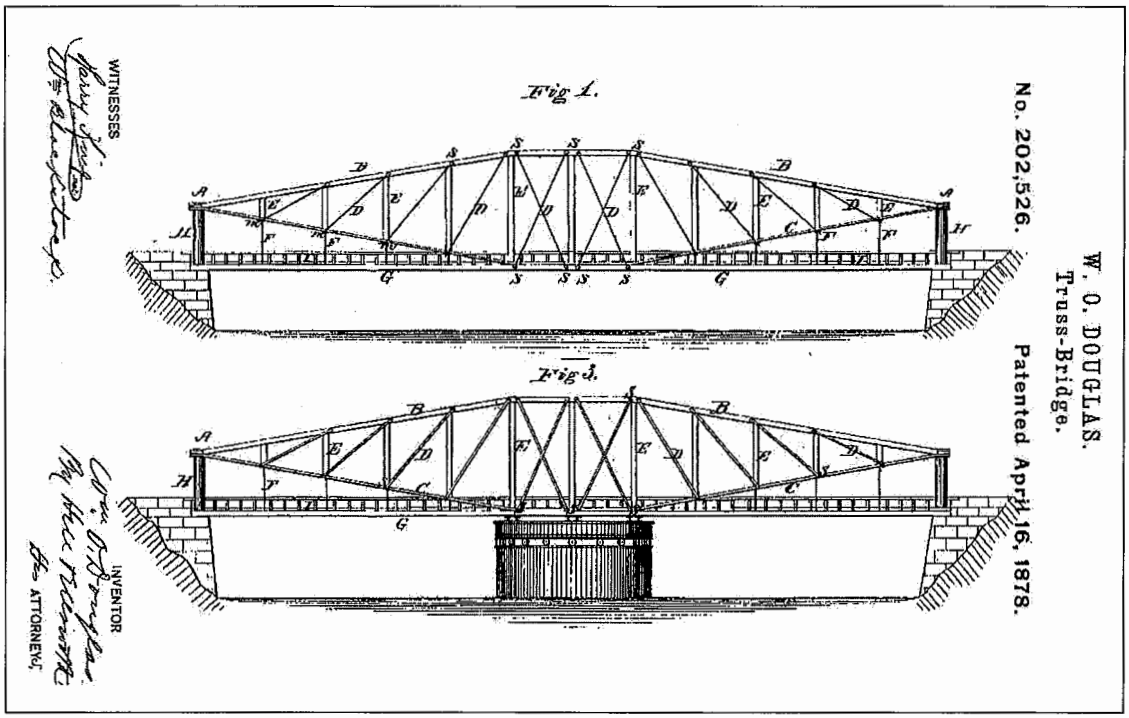

FIGURE 6. Drawings from William O. Douglas's 1878 patent.

have been erected in the United States, aside from Lindenthal's Smithfield Street Bridge. These bridges were only used for vehicular traffic and were generally considered too light to be used for railroad and trolley loads, although it is known that at least one (in Portland, Maine) did also serve as a trolley bridge. Considering that the most common traffic of the era (1880–1900) consisted of horse-drawn carts or wagons, it is amazing that any of the bridges survived through the automobile age and to the present day. Most of the bridges that were lost over the years were not because of failures from overloading; most were swept away during severe floods.

Darnell describes a number of lenticular bridges and gives a detailed account of the history of the Berlin Iron Bridge Company.[1] In addition to the uniquely shaped lenticular truss bridges, the Berlin Iron Bridge Company also built conventional steel truss bridges and even built a few pedestrian suspension bridges (the most notable of which were erected in Keesville, New York, and Milford, New Hampshire). In addition to bridges, the Berlin Iron Bridge Company had a thriving business building roof trusses, water towers and complete steel frames for buildings, as the advertisement shown in Figure 5 illustrates. The company was very persistent in its advertising and routinely placed advertisements in a number of important and influential trade magazines and journals of the day, including the *Transactions of the American Society of Civil Engineers.*

The Patents of William O. Douglas

A patent (No. 202,526) was issued by the U.S. Patent Office on April 16, 1878, to William O. Douglas, of Binghamton, New York, for a truss bridge. This bridge was described in the patent as "a combination of two or more elliptical trusses connected as herein described with the floor and joints and necessary flooring to form a through deck or swing bridge." Two of Douglas's patent drawings showing a suspended deck design (*i.e.*, deck tangent to the lower chord) are shown in Figure 6. A number of these bridges had been built by the predecessor of the Berlin Iron Bridge Company, the Corrugated Metal Company, out of its small manufacturing plant located in

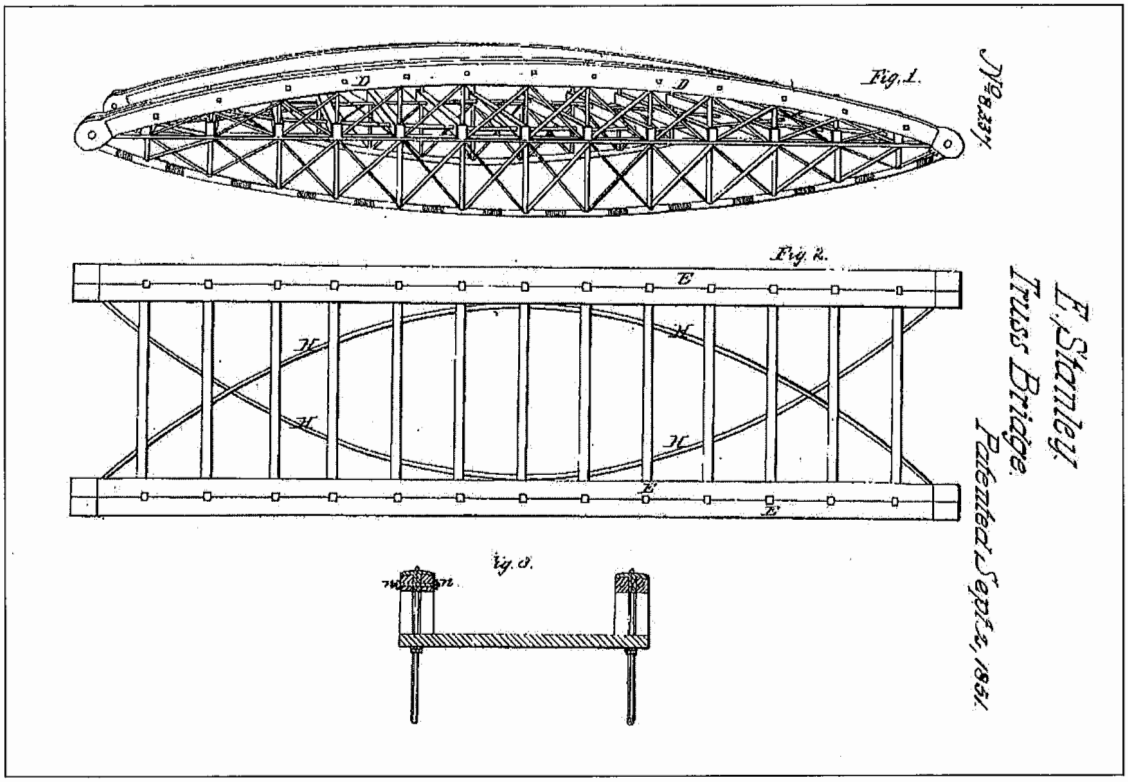

FIGURE 7. Patent drawing of the 1851 Stanley "truss bridge."

Berlin, Connecticut. The smallest of these bridges consisted of three panels, with a span of only about 40 feet. Earlier patents for lenticular-shaped bridges (sometimes referred to as "oval" or "parabolic" or "elliptical" in the patent documents) with similar features to those of Douglas's design had previously been issued by the U.S. Patent Office:

- on March 27, 1849, to James Barnes (No. 6,230); on September 2, 1851, to Edwin Stanley, N.Y. (No. 8,337);
- on August 21, 1855, to Horace L. Hervey and Robert E. Osborn (No. 13,461);
- on March 28, 1871, to Ferdinand Dieckmann (No. 113,030); and,
- on October 22, 1872, to George E. Harding (No. 132,398).

Only Stanley's patent drawing shows any close resemblance to the lenticular shape used by Douglas as shown in Figure 7.

It is interesting to note that Douglas had published his suggestion for his bridge design in an 1877 printing of the *Scientific American Supplement*, showing an illustration of his proposed design (see Figure 8). Douglas referred to his design as "an elliptical truss bridge," noting that:[6]

> "In a bridge as above illustrated in Fig. 2, we have the arch and suspension principles united, forming an elliptical truss. The thrust of the arch equipoises and is equipoised by the pull of the cable."

He further noted that:

> "The roadway is suspended to the two chords so that the arch carries one half of the load and the cable the other, under which circumstances the thrust and the pull at the top of end posts will be equal. The end posts have only to support the dead load of the bridge."

Douglas's public disclosure of his design (on July 14, 1877) predates his patent application

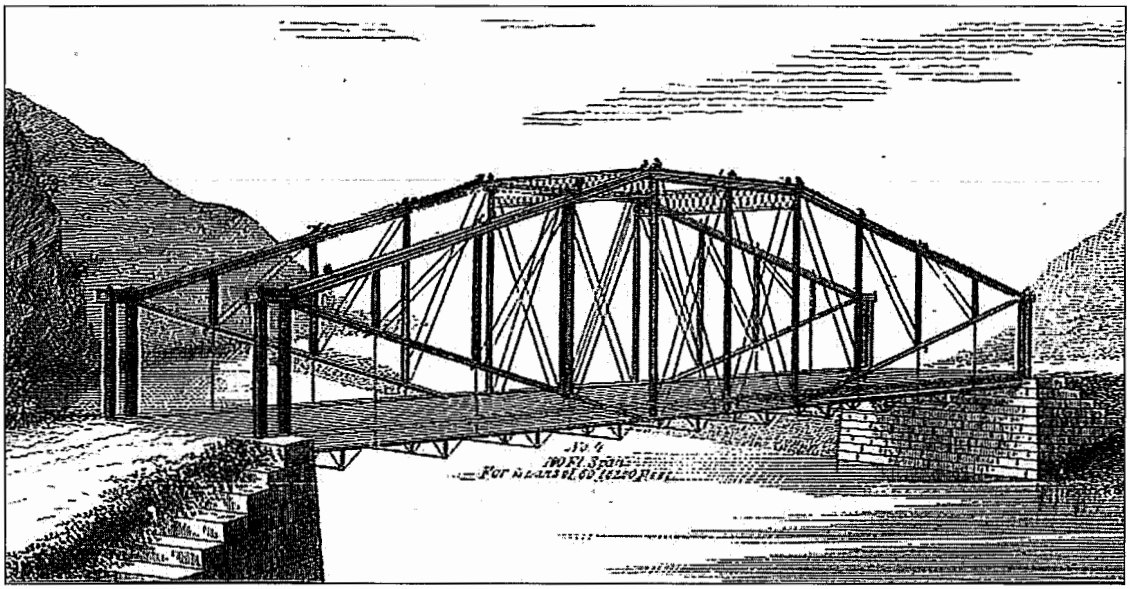

FIGURE 8. Douglas's suggestion for an elliptical truss bridge (1877).

(March 28, 1878) by nearly eight months. This article is the only known publication by Douglas or any other engineer associated with the Corrugated Metal Company or the Berlin Iron Bridge Company related to the lenticular design during this era.

It appears that the first lenticular bridge built by the Berlin Iron Bridge Company was a four-panel pony truss bridge apparently built in 1879 and erected at Waterbury, Connecticut, spanning the Naugatuck River. This bridge is still standing and is still in use as a vehicular bridge. Bridges were built principally throughout the northeast; surviving examples can still be seen in Massachusetts, Vermont, New Hampshire, Rhode Island, Connecticut, and New York, New Jersey and Pennsylvania. It is also known that several bridges were built in Ohio, but it seems that none have survived there. Interestingly enough, there are at least six existing lenticular truss bridges in Texas. These bridges are the only ones known to have been sold and built west of the Mississippi River, and they are thought to have been the work of an extremely enthusiastic freelance salesman in Texas.

The name of the Corrugated Metal Company was changed to the Berlin Iron Bridge Company sometime around 1880 and, according to company literature, the company provided almost 90 percent of the iron bridges roadway bridges throughout New England from 1880 to 1890. Designs for the bridges included both pony truss and through truss configurations. In addition to the suspended deck design shown in Figure 5, Douglas also suggested mid-deck and underdeck designs as illustrated in Figure 9. All of the extant lenticular truss bridges in Massachusetts are of the suspended deck style. A second U.S. patent (No. 315,259) was granted to Douglas for improvements on his design on April 7, 1885. The primary improvements that Douglas incorporated into this patent were the use of floor line tension chords and strut braces. The floor line tension chord was often simply a wrought iron rod running the length of the truss and connected to the end posts on either end. A turnbuckle was used to adjust the tension.

Douglas died around 1890, but the Berlin Iron Bridge Company continued to be very productive under the leadership of several good men. Early advertisements run by the company indicate the following principals: Charles M. Jarvis, President and Chief Engineer; Mace Moulton, Consulting Engineer; Burr K. Field, Vice President; George H. Sage, Secretary; and F. L. Wilcox, Treasurer.

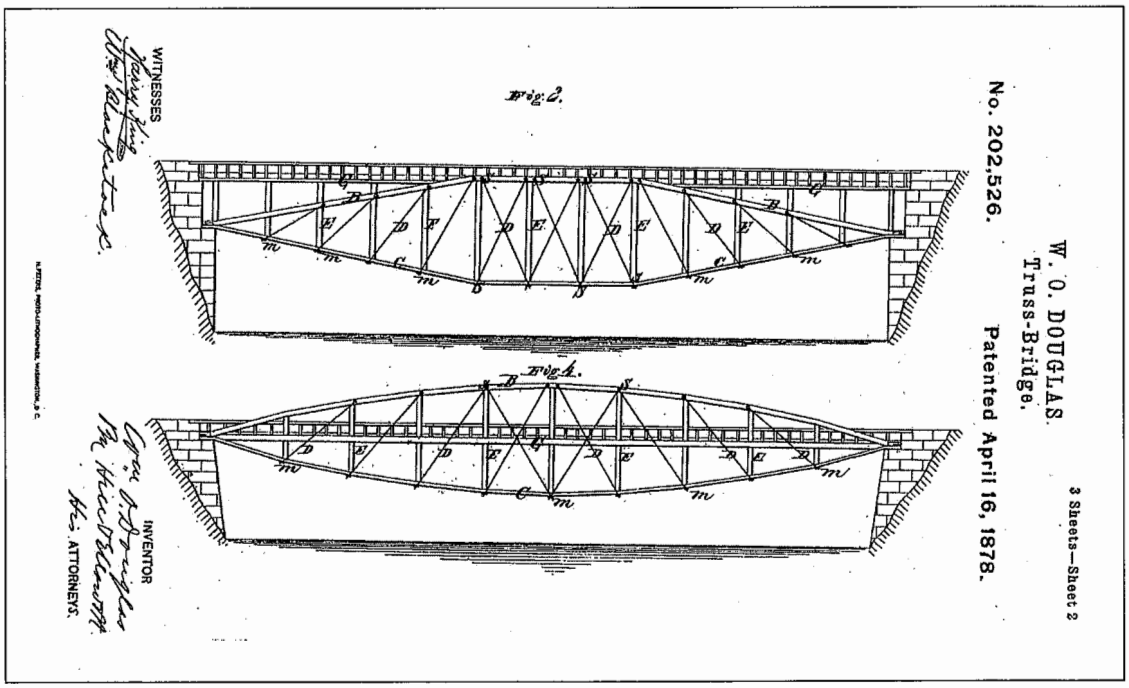

FIGURE 9. Other deck configurations covered by the Douglas 1878 patent.

There was some minor controversy regarding the originality of Douglas's design, especially in so far as the uniqueness of the lenticular shape was concerned.[3] However, it is clear that the Berlin Iron Bridge Company was the only promoter and manufacturer of this style of bridge. The Berlin Iron Bridge Company, under the leadership of Charles M. Jarvis, acquired the rights to Douglas's patent, which accounts for the exclusive promotion of this style of bridge by the company. Additionally, the company must have had excellent salesmen or agents who were most likely paid on commission, many of whom may have had a special affinity to this style of bridge (especially since the company designed and built other more conventional truss bridges). No other lenticular bridges built by any other bridge manufacturer of the era are known to have been designed, built or even advertised. This style of bridge was unique to the Berlin Iron Bridge Company.

Although there is no way to be certain, it is likely that the design of the bridges during this period was by the simple method of graphical analysis (for example, as described by Shreve[7]). This approach would be consistent with design of other, more conventional bridges. In Chapter X of his book, Shreve provides an example of the design of a lenticular truss:[7]

> "The form of this peculiar truss, known also as the Pauli System. . . is not capable of supporting any greater weight than a Bow String Truss of equal depth and length, and practically possesses many disadvantages."

Unfortunately, there are no records available to verify the design approach used by the Berlin Iron Bridge Company. Was Douglas or any of the other engineers associated with these bridges familiar with Shreve's text or other similar structural design books of the era? It is likely that they were familiar with these books and probably did base their designs on the state of the practice for that time period.

Surviving Lenticular Bridges in Massachusetts

Table 1 presents a summary of the surviving lenticular bridges in Massachusetts. Several of the bridges are documented in the *Catalog of*

TABLE 1.
Surviving Lenticular Truss Bridges in Massachsetts

No.	Bridge	Year	Town	Type*	Spanning
1	Golden Hill Rd.	1885	Lee	P	**
2	Pumpkin Hollow Rd.	N/A	Great Barrington	P	***
3	Fort River	ca. 1880	South Amherst	P	Fort River
4	Gilbert Rd.	1888	West Warren	P	Quaboag River
5	Blackstone Bikeway	1887	Millbury	P	Blackstone River§
6	North Canal	N/A	Lawrence	P	North Canal
7	Galvin Rd.	1884	North Adams	T	Hoosac River
8	Bardwell's Ferry Rd.	1882	Shelburne	T	Deerfield River
9	Aiken St.	1883	Lowell	T	Merrimack River

Notes: * P = Pony Truss; T = Through Truss.
** Bridge dismantled and in storage.
*** Bridge dismantled; trusses in open storage.
§ Formerly across the Westfield River in Westfield.

the Historic American Engineering Record. Figure 10 shows the location of the nine remaining lenticular truss bridges in Massachusetts. (The numbers shown in Figure 10 refer to Table 1.) All of the remaining bridges are deck bridges of either the pony truss or through truss configuration and all are single-span bridges (with the exception of

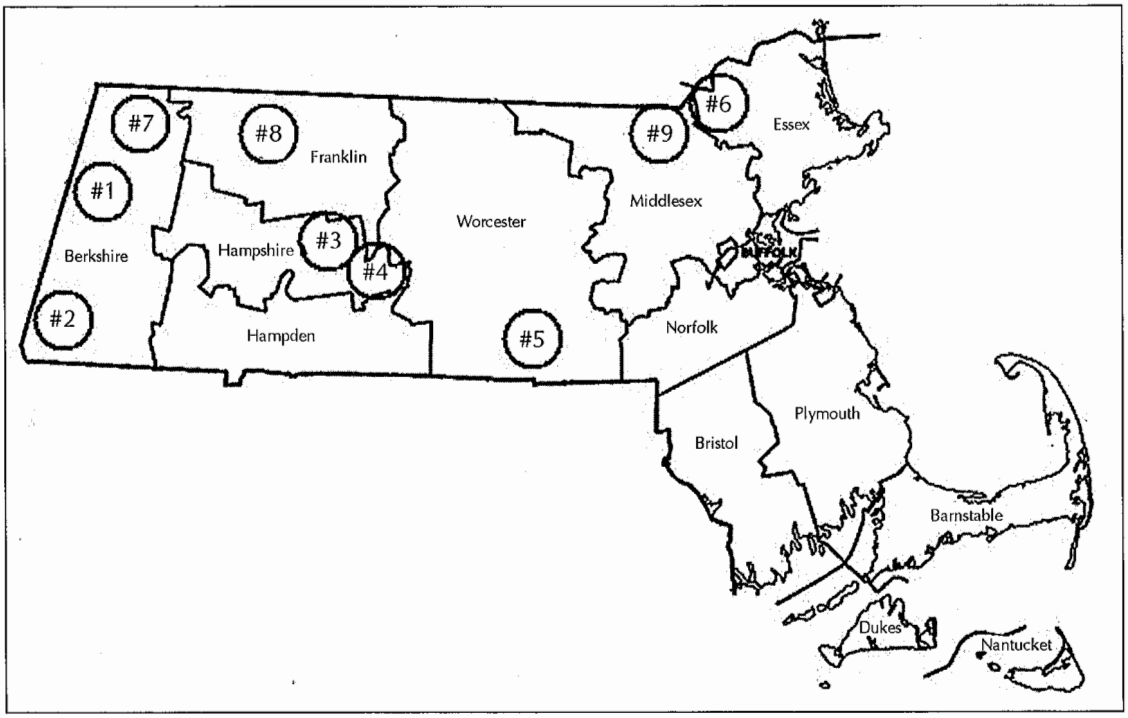

FIGURE 10. The location of the extant lenticular truss bridges in Massachusetts.

TABLE 2.
Characteristics of Surviving Lenticular Truss Bridges in Massachsetts

No.	Bridge	Number of Panels	Span (L) ft.	Mid-Span Height (H) ft.	Aspect Ratio (L/H)
1	Golden Hill Rd.	5	80	8	10.0
2	Pumpkin Hollow Rd.	4	58	8	7.2
3	Fort River	4	60	8	7.5
4	Gilbert Rd.	5	72	8	9.0
5	Blackstone Bikeway	6	74	8	9.2
6	North Canal	5	83	8	10.4
7	Galvin Rd.	7	103	18	5.7
8	Bardwell's Ferry Rd.	13	198	30	6.6
9	Aiken St.	11	153	32.8	4.6

the Aiken Street Bridge in Lowell which has five spans). Table 2 notes the specific characteristics of each bridge.

Surviving Pony Truss Bridges

There are six surviving bridges of the pony truss configuration. Only one of the bridges (the Gilbert Road Bridge in West Warren) is currently being used for vehicular traffic. One of the bridges (the Fort River Bridge in South Amherst) is a pedestrian bridge. One (the Northwest Road crossing in Millbury) is being used as part of a bike trail. One is closed (the North Canal Bridge in Lawrence). Two bridges

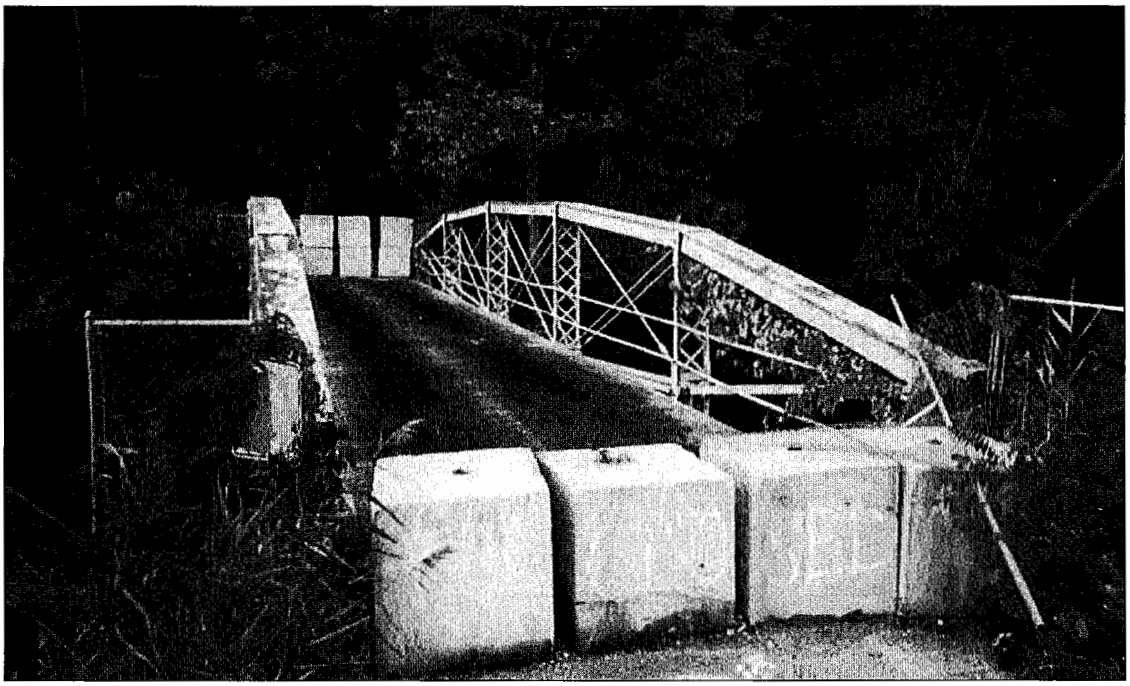

FIGURE 11. The Golden Hill Bridge at its old location in Lee.

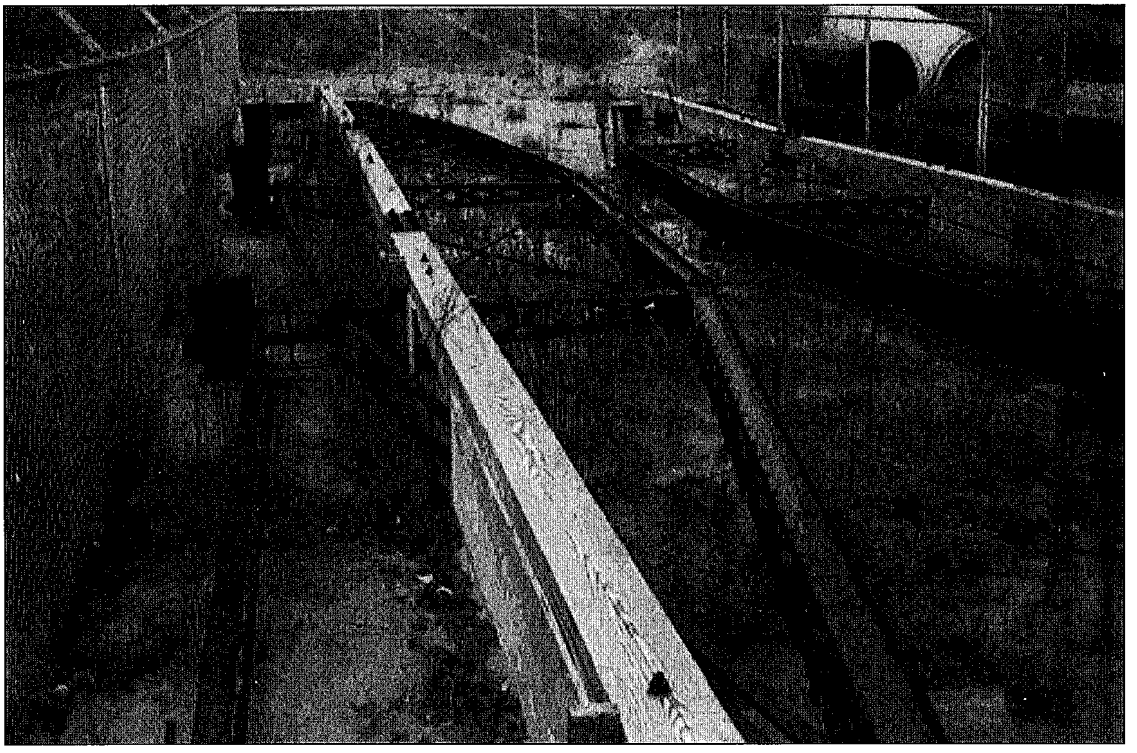

FIGURE 12. The Pumpkin Hollow Road Bridge parts in Great Barrington.

(the Golden Hill Road Bridge in Lee and the Pumpkin Hollow Road Bridge in Great Barrington) have been removed and dismantled. Remaining parts (trusses) of the Pumpkin Hollow Bridge are held in open storage.

Golden Hill Road (Tuttle) — Lee. The Golden Hill Road (Tuttle) Bridge is a five-panel, 80-foot-long pony truss that used to span the Housatonic River in Berkshire County (see Figure 11). The bridge was built in 1885 and is believed to be a virtually unaltered example of the 1885 Douglas patent. The bridge is a typical example of a pony truss and is believed to be one of the longest span pony truss bridges built. The bridge has a mid-span depth of 8 feet and uses all pin connections for the main truss members. Like nearly all lenticular bridges built by the Berlin Iron Bridge Company, the end posts and upper chords are composite open sections constructed of riveted plates and angles and the lower chords are simple eye plates. The bridge has been dismantled and is now in storage. Plans are being developed to reconstruct the bridge as a pedestrian bridge on the campus of the University of Massachusetts at Amherst.

Pumpkin Hollow Road — Great Barrington. The Pumpkin Hollow Road Bridge was originally located in Great Barrington and is believed to have been built around 1885. The bridge was removed several years ago and is currently in storage in Great Barrington. The bridge is a four-panel pony truss configuration with a span of 58 feet and a mid-span depth of 8 feet. Figure 12 shows a photograph of the remaining trusses. The bridge is unique in that it is the only remaining lenticular bridge in Massachusetts in which the end chord connections at the end posts are not pinned; instead, they are bolted through special cast iron end post corner elements (while a number of other lenticular bridges in New Hampshire and New York have similar bolted end post connections, this feature seems to have been fairly rare in Massachusetts).

Only the trusses of the Pumpkin Hollow Bridge were saved when the bridge was dismantled and are apparently all that remain of the bridge. However, they appear to be essentially intact and unaltered from their original construction. The vertical posts are construct-

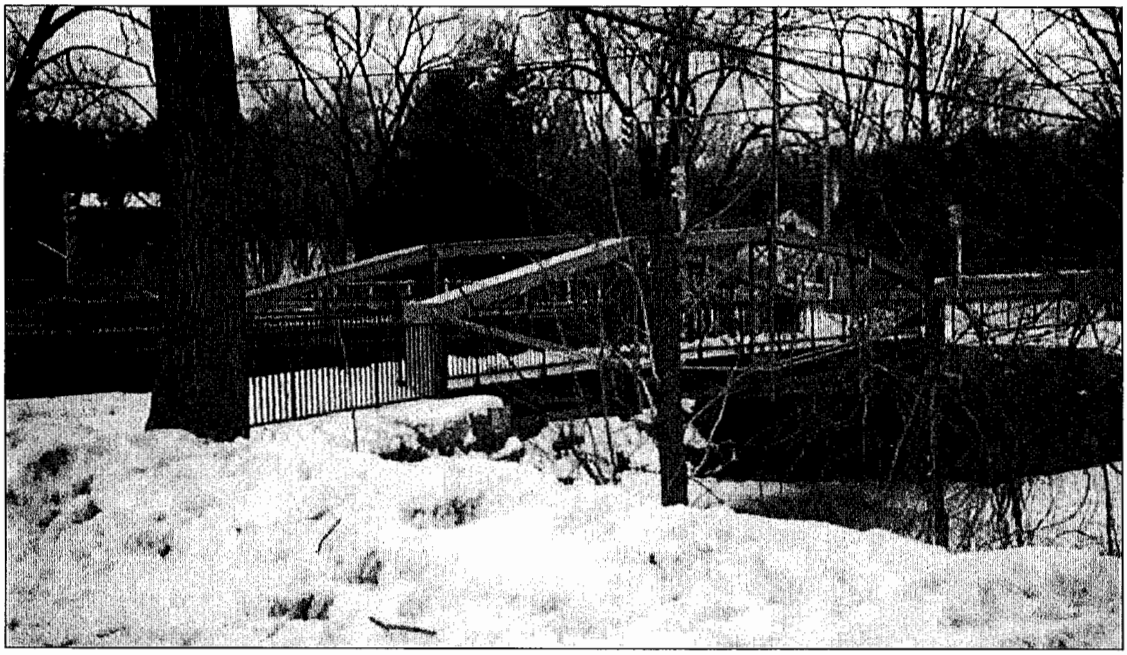

FIGURE 13. The Fort River Bridge in South Amherst.

ed of built-up members and are tapered, much like the Golden Hill Road Bridge.

Fort River — South Amherst. The Fort River Bridge is a 60-foot-long span that crosses the Fort River in South Amherst (see Figure 13). The bridge originally carried vehicular traffic as part of South Pleasant Street (which is now Route 116 between Amherst and South Hadley), but in 1997 the bridge was rebuilt as a pedestrian bridge by the Town of Amherst. The bridge is thought to have been built around 1880 and is the only surviving lenticular bridge in Massachusetts that carries the iron bridge plate indicating that it was built by the Corrugated Metal Company, the predecessor of the Berlin Iron Bridge Company. The Fort River Bridge is a four-panel pony truss with a mid-span depth of 8 feet, giving an aspect ratio of 7.5.

Like all the other pony truss bridges examined for this study, the vertical posts are constructed of riveted angles and plates to create an open lattice form. However, the vertical posts have parallel sides, which is an early configuration used by the manufacturer and which distinguishes it from all other pony truss bridges that were examined. The use of tapered verticals appears to have replaced the parallel verticals so that lighter top chords could be used. The bridge uses pinned connections at the end posts and all other connection points to tie the upper chord and lower chords together. The lower chord is constructed from pairs of 1- by 2-inch eye bars. The Fort River Bridge is very similar in design to the Pumpkin Hollow Bridge in terms of overall aesthetics, but varies significantly in the actual design of both the individual elements and especially in the use of bolted end connections.

Gilbert Road — West Warren. The Gilbert Road Bridge is the only surviving lenticular pony truss bridge that is still in service to carry vehicular traffic. Constructed in 1889, the bridge is a 72-foot span that carries Gilbert Road across the Quaboag River. The bridge is a five-panel pin-connected design with a mid-span depth of 8 feet (see Figure 14). The end posts are topped with an ornate ball-shaped finial and what appears to be the original lattice guardrail. These features give the bridge a slightly more decorative appearance than the other bridges. Like all the other pony truss bridges, the top chords and end posts are composed of a built-up open box section. The upper chord has a width of 14 inches and a

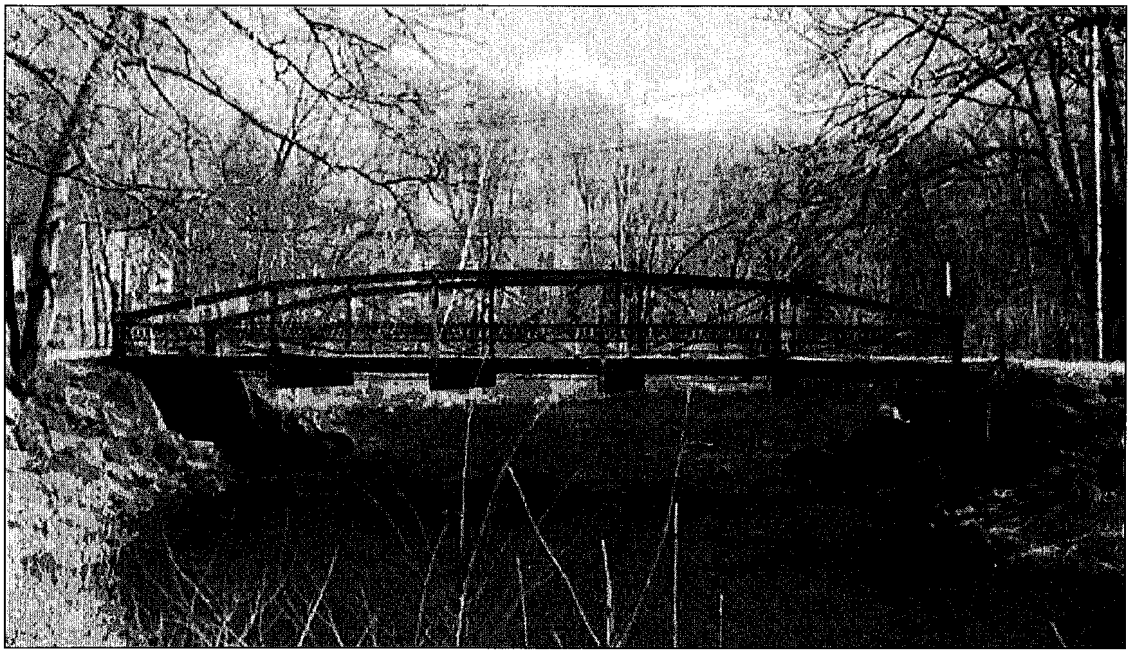

FIGURE 14. The Gilbert Road Bridge in West Warren.

depth of 7.5 inches and is constructed from angles and plates. The lower chords consist of pairs of 1- by 3-inch eye bars. The verticals web posts are tapered built-up lattice members. The bridge is entirely pin connected. This bridge is almost identical in design to the Pumpkin Hollow Bridge, with the exception of the end post top and bottom chord connections.

Northwest Road (Blackstone River Bikeway) — Millbury. The Northwest Road Bridge was originally located in Westfield, Hampden County, and spanned the Little River. In 2001, the bridge was removed and relocated by the Massachusetts Highway Department to Millbury, where it currently is being used as part of the Blackstone Bikeway over the Blackstone River (see Figure 15). The bridge was constructed in 1887 and has a span of 74 feet. The bridge has a pin-connected six-panel pony truss configuration with a mid-span depth of 8 feet. The end posts and upper chords are composed of a built-up open box section consisting of four 1.75-inch angles and a 4-inch plate and two 7-inch plates. The lower chords consist of pairs of 1- by 3-inch eye bars. The vertical members connecting the upper and lower chords are tapered and consist of four 1.75- by 1.75-inch angles with riveted flat bars forming an "X." Diagonal bracing is 1.25-inch-diameter rods. Overall, the bridge is almost identical in construction to the Gilbert Road Bridge.

North Canal — Lawrence. The lenticular bridge spanning the North Canal in Lawrence is a five-panel pony truss. Very little is known about the history and construction of this bridge. The bridge is currently closed and has been extensively modified, with at least one of its top chord members replaced (see Figure 16). On the south end, the original top chord, which consists of riveted plates and angles, has been replaced with welded plates and channel sections. The bridge has a total span of 83 feet and is completely pin connected. The lower chords appear to be original 1- by 4-inch flat eye bars. The mid-span depth is 8 feet. The bridge has tapered vertical members connecting the top and bottom chords and appears very similar in design to both the Gilbert Road and Northwest Road bridges.

Surviving Through Truss Bridges

In addition to the six pony truss bridges described above, there are three surviving lenticular through truss bridges, two of which

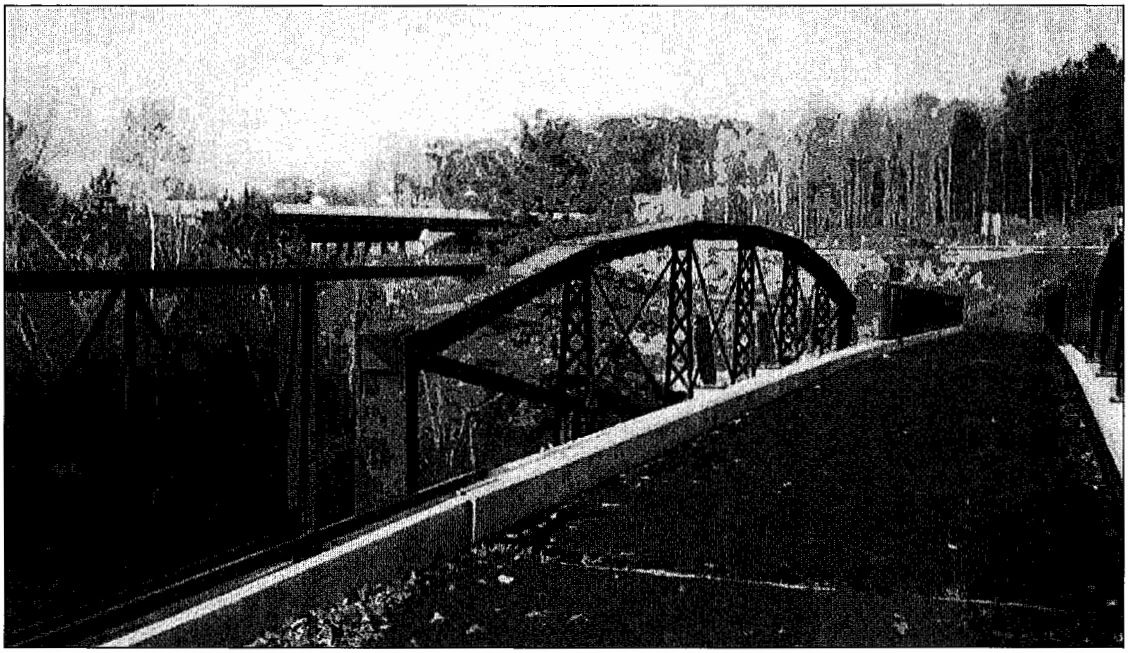

FIGURE 15. The Northwest Road Bridge in Millbury.

(the Bardwell's Ferry and Aiken Street bridges) are still in active use for vehicular traffic. The other one (the Galvin Road Bridge) is currently closed and awaiting removal and restoration. The through truss bridges were generally used for longer spans and are much

FIGURE 16. The North Canal Bridge in Lawrence.

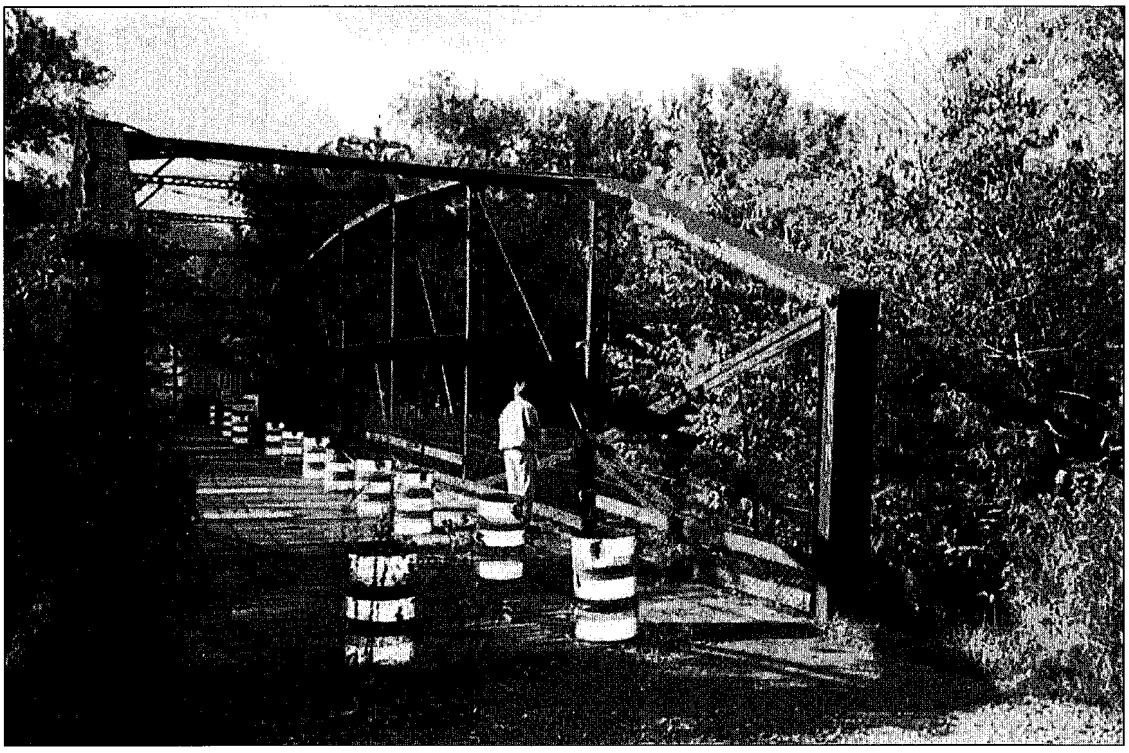

FIGURE 17. The Galvin Road Bridge in North Adams.

larger in every detail than the pony truss bridges.

Galvin Road — North Adams. The Galvin Road (Blankinton) Bridge is located on Galvin Road in Berkshire County. It spans the Hoosac River (see Figure 17). It is an excellent example of Douglas's 1885 patent as applied to a through truss. The bridge was constructed in 1884 and, therefore, likely incorporated the 1885 improvements on Douglas's 1878 patent. The bridge has a span length of 103 feet and consists of seven panels. The mid-span depth of the bridge is 18 feet. The end posts and the upper chords consist of open box sections, built up of three plates and four angles, giving overall dimensions of 16 by 8 inches. The lower chords consist of pairs of 1- by 3-inch eye bars. The entire bridge is pin connected at each chord segment connection point. The vertical members consist of parallel sections of paired angles with flat plate cross members that form an open lattice. Diagonal bracing rods within each pin are 1.5-inch-diameter wrought iron rods. The Galvin Road Bridge has been dismantled and is currently in storage at the University of Massachusetts at Amherst. The bridge will be rebuilt for pedestrian use on the university campus.

Bardwell's Ferry Road — Shelburne. The Bardwell's Ferry Bridge is a 198-foot-long pin-connected through truss bridge, built in 1882. It spans the Deerfield River between the towns of Shelburne and Conway in Franklin County. It is the longest single-span lenticular truss bridge in Massachusetts. The bridge consists of thirteen panels and the design follows closely the Douglas patent of 1878 (see Figure 18). It has a mid-span depth of 29 feet. The end posts and upper chords are built-up open box members, consisting of riveted plates and angles giving dimensions of 18 by 12 inches. The lower chords are constructed from 1- by 3-inch eye bars. The vertical members consist of parallel sections consisting of four channels connected with flat plates to form an "X."

In 1997, the bridge underwent extensive restoration. However, the original form of the bridge appears to have been maintained and the bridge currently exists very close to the configuration as originally constructed. The

FIGURE 18. The Bardwell's Ferry Bridge in Shelburne.

Bardwell's Ferry Bridge is one of the longest single-span through truss bridges built by the Berlin Iron Bridge Company using the traditional lenticular truss panel design. Other, longer-span lenticular trusses were built by the company in Pennsylvania and New York using a "Warren truss" style of bracing within the lenticular shape. In 1988, the Bardwell's Ferry Bridge was designated as a Historic Civil Engineering Landmark by the American Society of Civil Engineers.

Aiken Street — Lowell. The Aiken Street Bridge was built in 1883 (the same year as Lindenthal's Smithfield Street Bridge in Pittsburgh) to span the Merrimack River in Lowell. The bridge consists of five identical spans, each having a length of 152 feet, with a mid-span depth of 32.8 feet (see Figure 19). It is the longest surviving multispan lenticular bridge and is the only remaining bridge with more than three individual spans. The design is similar to the Washington Street Bridge in Binghamton, New York, which consists of three spans of 170 feet each crossing the Susquehanna River. Each span of the Aiken Street Bridge consists of eleven panels. The full span length of the bridge is 675 feet, which presents an impressive sight in Lowell. It is the only multispan lenticular bridge in Massachusetts and the only lenticular bridge in the state that caries a pedestrian walkway on both sides of the bridge outside of the trusses. (This bridge was also described by Bennett and Kaminski.[8])

The end posts are constructed as open box sections from four angles and three plates with overall dimensions of 24 inches wide by 16.5 inches deep. The top chords consist of an open box section constructed similar to the end posts with four angles and three plates with dimensions of 18 by 14 inches. Even though the individual spans are shorter than the Bardwell's Ferry Bridge, the end posts and top chords are heavier (presumably to accommodate the additional weight of the outer walkways). The bottom chords consist of four 1.625- by 4.5-inch flat eye bars. The vertical web posts are constructed from 3- by 5- by 0.438-inch angles with 0.25-inch flat lattice bars. The diagonal bracing consists of pairs of 1.75- or 1.875-inch round wrought iron bars. The bridge was renamed the Joseph R.

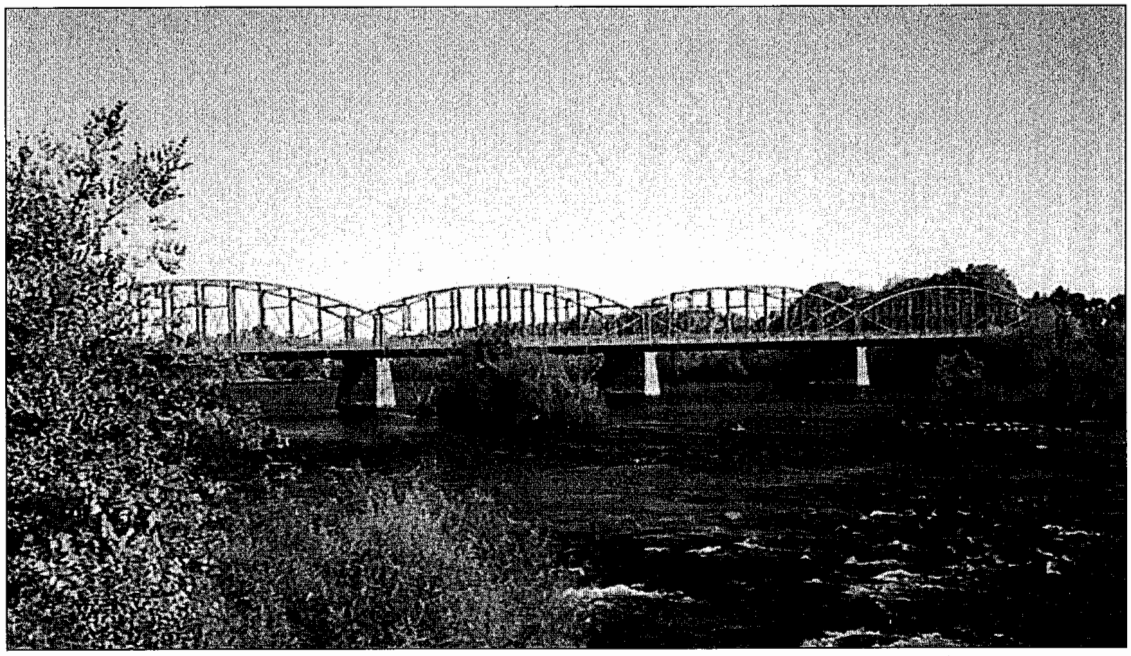

FIGURE 19. The Aiken Street Bridge in Lowell.

Ouelette Bridge in 1954 in honor of a Lowell soldier killed in the Korean War and who was a recipient of the Medal of Honor.

Description of Individual Components

The design of the individual elements of these bridges was exquisitely simple, yet functionally sound and lent itself to economic fabrication at the plant. The use of riveted plates, angles and channel sections to build the end posts, top chords and vertical web posts is characteristic of all these bridges. The construction brings to mind a child's Erector set and one can easily imagine the individual top chord elements, vertical posts, lower chords, bracing bars and other smaller components being transported to the site by horse carts. Assembly at the site would have been relatively easy and fast, even by today's standards. There are relatively few variations in components among the surviving bridges, and the components suggest a simple modular design concept. After all, the market for iron bridges at the end of the nineteenth century was highly competitive and, therefore, any means to reduce fabrication costs would have been exercised.

In comparison with typical bridge sections of today, the components for a lenticular pony truss bridge were relatively light and could have been handled by workers at the site. Using the Fort River Bridge in South Amherst as an example, a 14.75-foot-long upper chord section would weigh about 640 pounds; an 8-foot-long vertical web post would weight about 130 pounds; and a 14.75-foot-long lower eye chord would weight about 105 pounds. Each of these members were constructed of individual components riveted together. The weight of the upper chord was such that it would likely have been handled by a tripod or boom and jib. The other elements could easily be placed by two workers.

Figure 20 shows a side view of a typical bridge and identifies individual components. Table 3 provides a summary of the bridges' key elements. (Descriptions of individual bridge members are given in English units to be consistent with the period of construction.) The segmental upper chords are used as compression members and the lower chords as tension members; the two come together at the end post connection. So, in effect, it is sometimes said that this unique style of bridge combined the attributes of an arch

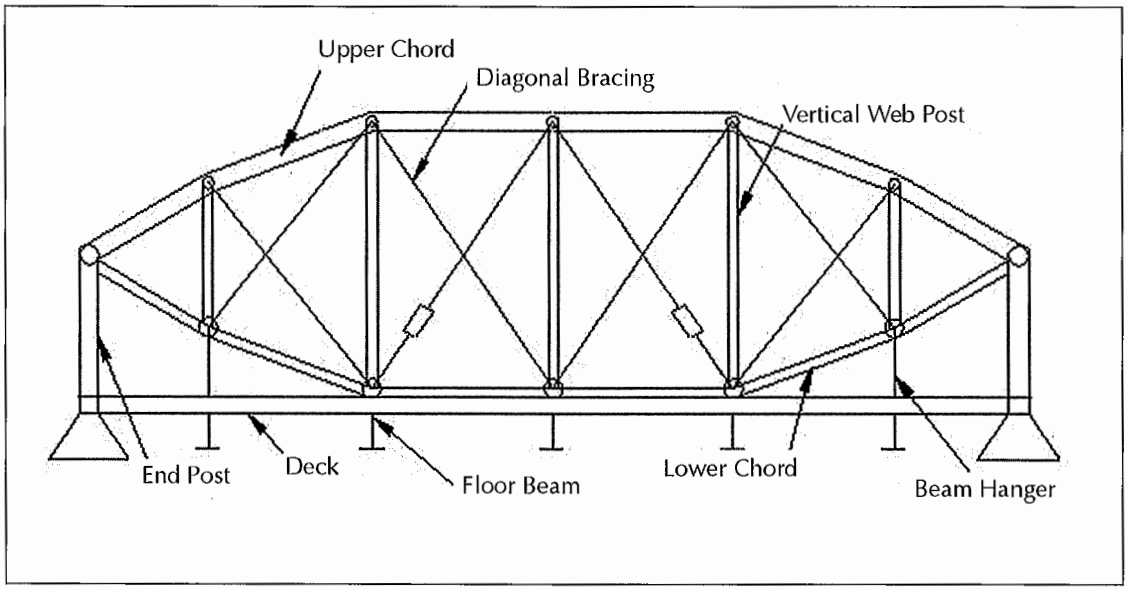

FIGURE 20. Components of a lenticular bridge.

bridge and of a suspension bridge into a single structure. The lower chord member, if allowed to hang freely, would take on the shape of a catenary, much as an early chain bridge and subsequent wire cable suspension bridges.

End Posts & Upper Chords. End posts and upper chords were generally constructed as built-up members, using standard dimension angles and plates riveted into sections with one open side. This type of construction is illustrated in Figure 21. End posts are open box sections with the interior vertical face being the open side. In all cases, flat bar bracing elements are provided along the open face. These bracing elements were either attached as horizontal members or were attached diagonally in a "Z" pattern. These elements were also riveted. End posts of the Bardwell's Ferry Bridge are somewhat unique in that they are actually constructed as open box sections with flat bar "X" bracing. This style of end post design has been seen on very few bridges. There was clearly a cost savings to the Berlin Iron Bridge Company in using this approach and it is surprising that this style of construction is not seen more often. The only thing that keeps these members from being fully built-up tubular sections is that that are open on one side.

The sizes of angles, plates and flat bars used on different bridges were related to the style and span of the bridge. For example, the overall dimensions of the end posts sections of the Bardwell's Ferry Bridge are 18 by 12.375 inches, built up from 2- by 2- by 0.375-inch angles and 18- by 0.75-inch and 12- by 0.25-inch plates. By comparison, the end posts of the Pumpkin Hollow Road Bridge, which is the lightest, are built up from 1.5- by 1.5- by 0.25-inch angles and 10- by 0.125-inch and 5.25- by 0.125-inch plates having overall dimensions of 10 by 5.375 inches.

Top chords were essentially built as extensions of the end posts and were constructed in almost an identical manner as inverted troughs. In most cases, the transition from the end post to the top chord was made by a connection at the top of the end post that consisted of a riveted plate that was placed inside the exterior angles of the end post and top chord on either side. A pin was then positioned to extend through the entire section to serve as a connection for the eye end of the lower chord. On some pony truss bridges, the connection between the end post and the top chord was made using a special cast iron connector with the lower chord extending through and then held with a nut. A comparison of these two styles of end post–top chord connections is

TABLE 3.
Summary of Key Bridge Elements

No.	Bridge	Upper Chord (in.)	Lower Chord (in.)	Central Diagonal Bracing Bars (in.)	Vertical Web Posts
1	Golden Hill Rd.	16 × 8.25	1 × 3	1.25	Parallel
2	Pumpkin Hollow Rd.	10 × 5.25	1 × 3	1.25	Tapered
3	Fort River	16 × 8.5	1 × 2	0.75	Parallel
4	Gilbert Rd.	14 × 7.5	1 × 3	1.5	Tapered
5	Blackstone Bikeway	14.25 × 7.5	1 × 3	1.25	Tapered
6	North Canal	16 × 8.25	1 × 4	1.25	Tapered
7	Galvin Rd.	16 × 8	1 × 3	1.5	Parallel
8	Bardwell's Ferry Rd.	18 × 12	1 × 3	0.75	Parallel
9	Aiken St.	18 × 12	1.625 × 4.5	1.75	Parallel

shown in Figure 22. A third style of end post-top chord connection was occasionally used, as in the case of the Aiken Street Bridge. In that connection, the top chords were actually narrower than the end posts and fit inside at the pinned connection.

Lower Chords. The lower chords in every bridge were constructed using flat stock wrought iron eye bars with the eye ends used to create pin connections at each panel connection point. The size and number of individual elements composing the lower chords was also related to the style and span of the bridge. Sizes and numbers ranged from pairs of 1- by 3-inch sections for the shorter span pony truss bridges, to two pairs of 1.625- by 4.5-inches for the Aiken Street Bridge. The lightest lower chord members are the 1- by 2-inch bars used on the Fort River Bridge.

Vertical Web Posts. Vertical web posts connecting the upper and lower chords are the simplest of all the built-up members and were fabricated from four angle sections with riveted flat bar diagonals. Web posts were either tapered or were constructed with parallel sides as shown in Figure 23. Tapered web posts were connected to the pins at the upper chords on the inside of the chord, while parallel web posts were connected to the upper chords on the outside of the chord. The only remaining pony truss bridges with parallel web posts are the Fort River and the Golden Hill Road bridges. All of the other pony truss bridges use tapered web posts. Parallel web posts are used on all of the surviving through truss bridges.

Diagonal Bracing. Diagonal bracing bars were used in the center of panels and consist-

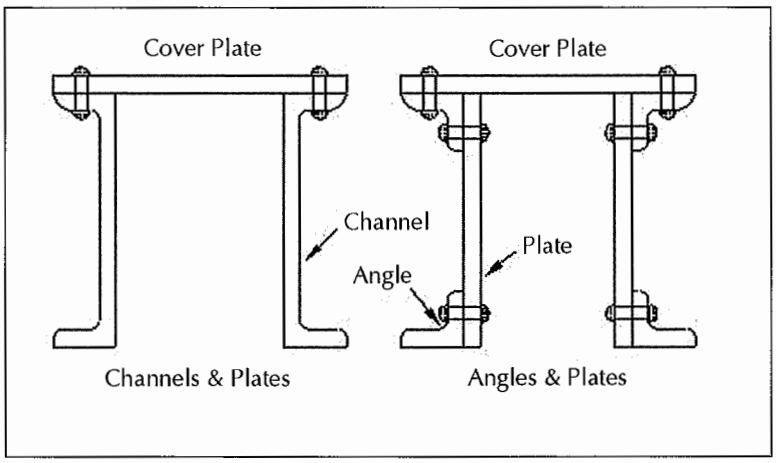

FIGURE 21. End post and top chord construction.

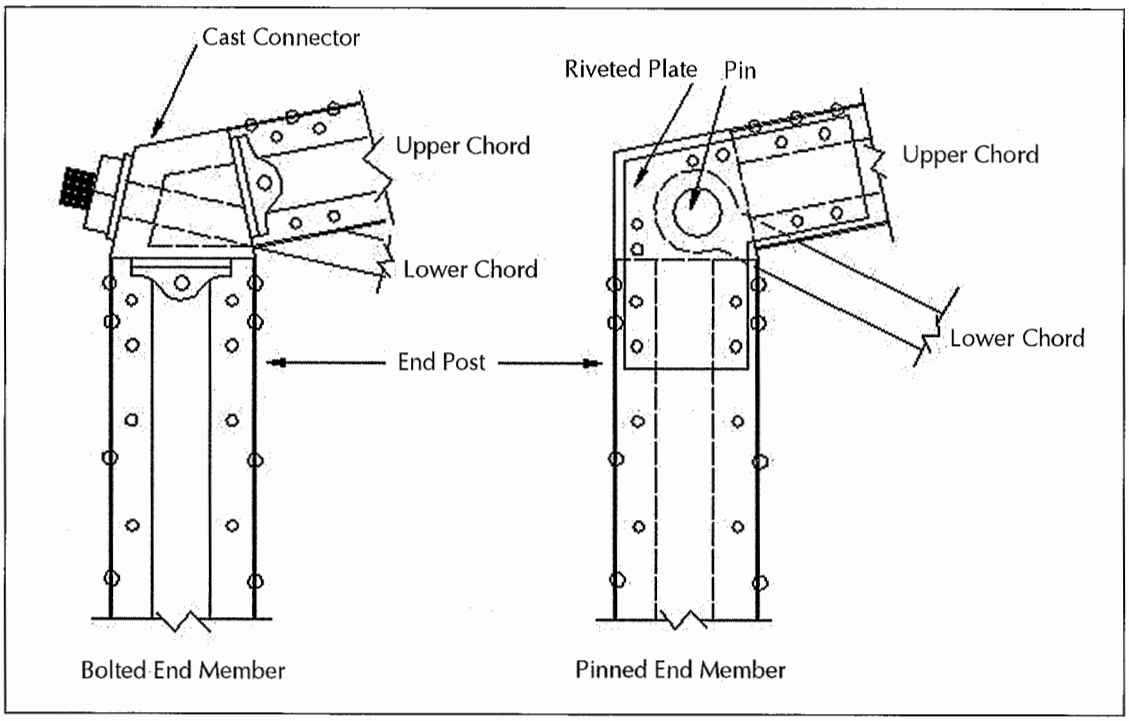

FIGURE 22. A comparison of pinned and bolted end posts.

ed of round iron rods wrapped around pins at connections. The diameters of the bars ranged from 0.75 to 1.5 inches. In many cases, different size bars were used on the same bridge, with the larger diameter bars used in the central panels and progressively smaller bars used on the panels closer to the end of the bridge. Normally, the bars included a turnbuckle to allow for the construction and so that tension could be applied.

Floor Beams. Floor beams on the bridges were again constructed as riveted sections built-up from angles and plates. The top edge of the beams was parallel with the bridge deck, but in many cases the lower edge was tapered, being deeper at the center than at the edge. The central plates appear to all have been fabricated from a single plate. In almost every case of the bridges examined, the individual pieces that were used to fabricate any section were full-length elements.

Beam Hangers. Beam hangers were fabricated from square stock with threaded ends. The hangers were suspended over the lower pins at connection points. These hangers then extended downward through notches cut in the flanges of the floor beams and were connected to a lower plate that supported the beam.

It is clear from the examination of each of these bridges in Massachusetts (and a number of other bridges throughout New England) that many of the individual members were of stock dimensions and that bridge designs were essentially standardized so that various combinations of stock elements could be used to achieve the required span that was needed for a particular site. This method seems especially to be the case for pony truss bridges and accounts for the variation in aspect ratios that have been observed — that is, all six remaining pony truss bridges have mid-span heights of 8 feet, but have span lengths ranging from 58 to 83 feet, yielding aspect ratios ranging from 7.2 to 10.4.

Summary

The nine surviving lenticular truss bridges designed and constructed by the Berlin Iron Bridge Company represent the last structures of their type of perhaps over a hundred such bridges that were built throughout Massa-

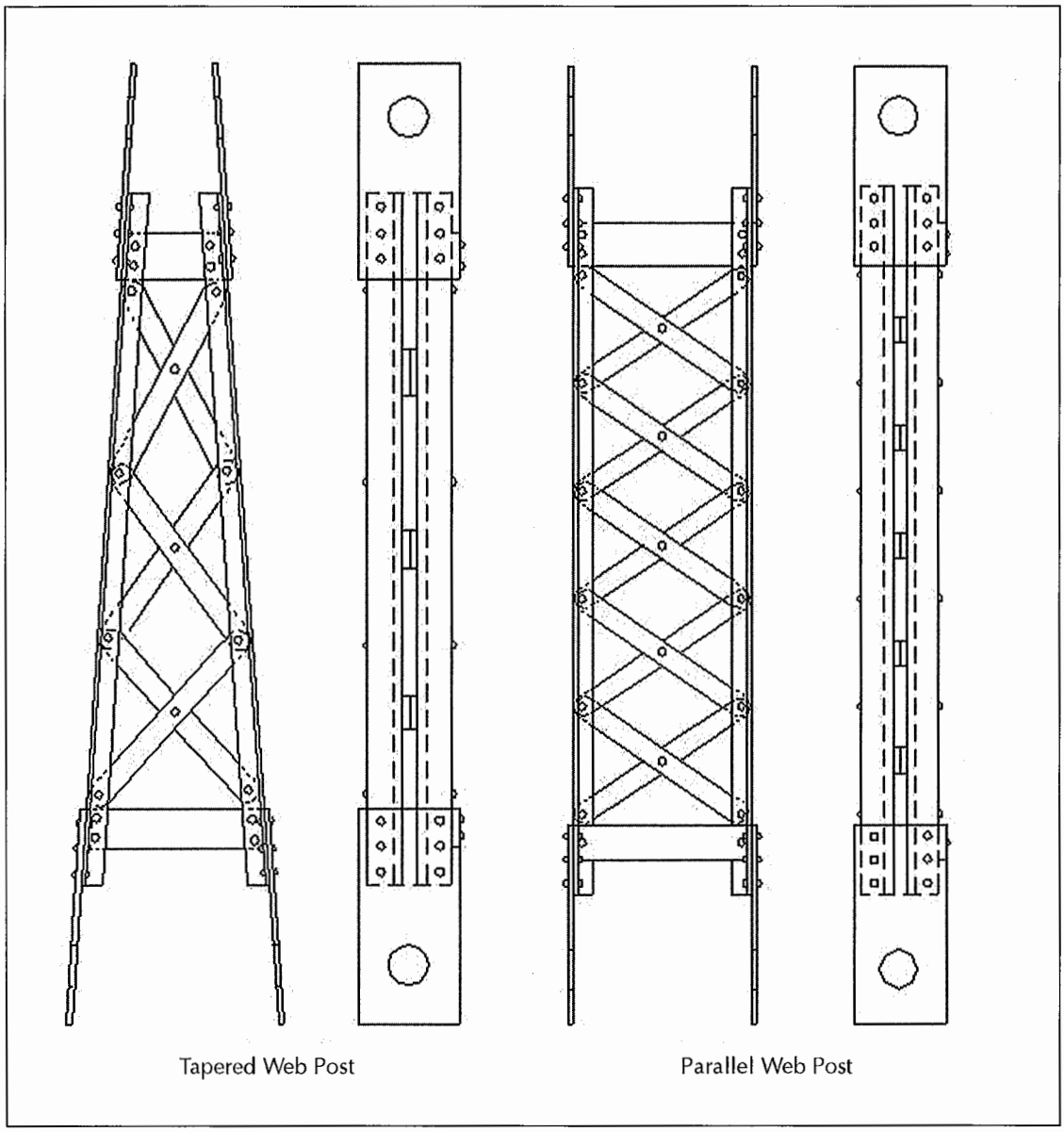

FIGURE 23. Different vertical web post construction methods.

chusetts between about 1880 and 1900. These bridges represent an important era in bridge history at a time when bridge construction was highly competitive and during which the transition was being made from the use of iron to the use of steel. The bridges have a unique shape and were the only lenticular bridges built in the United States by a prominent bridge building company of the late nineteenth century. The bridges provide a look into late nineteenth century bridge design and construction and every effort should be made to preserve them for future generations to study and appreciate.

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Engineering and, in particular, the history of lenticular truss bridges about five years ago. While his research has focused on in-situ testing of soils and foundation design, more recently his interests have included historical aspects of foundation engineering and the preservation of wrought iron bridges.

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Back to School

Sometimes it takes more than borrowing a word to refashion that concept into a new era with different needs.

BRIAN BRENNER

Wrentham Village is a relatively new shopping center constructed off Interstate 495 (I-495) in Wrentham, Massachusetts. Its design incorporates a lot of the latest features for shopping centers. The center is advertised as a collection of factory-direct stores, in which different brands offer their merchandise directly to consumers at discount prices. In comparison to a large department store with everything under one roof, the center has dozens of smaller stores situated in shopping pods. The pods are sited in the middle of a vast parking lagoon. Access to the lagoon is easily provided by an exit off I-495, the Boston metro area's uber-beltway. As you approach the exit a sign for Wrentham Village towers over the trees in the adjacent woods and lights up the sky at night.

Inside the village, shoppers walk along modestly pleasant, nondescript pedestrian lanes lined with stores. As implied by the name, it is a village of sorts, but it is not a real village. It only has stores. But true to the village concept, the pedestrian lanes are all outside, exposed to the weather. Use of open-air public spaces is one of the innovations of this type of discount shopping center (in contrast to an enclosed, environmentally-controlled mall). The costs of maintaining the space must be considerably less, since there are no heating and cooling bills for such common areas as walkways and plazas. The shopping pods are arranged in squares, with the interiors of the squares reserved for trucks bringing in inventory and supplies. Access to the shipping areas is via small paved alleys that connect to the main parking lagoon. The shipping areas with their loading docks and garbage bins, while not directly in the pedestrian paths, are not exactly hidden from view either. The theory behind the architecture seems to be that because it is a discount center, shoppers should expect some degree of less-than-Grade-A accommodations.

The Draw

I needed stuff, so I trekked to the village in September. This is the back-to-school period, when kids are lugged by their parents for the annual clothing resupply. The place was packed. It was impossible to get a spot near the pods in the parking lagoon, although many drivers trolled around for openings. I didn't have the patience, so I parked at the outskirts about a quarter-mile away, and made the five-minute hike across the asphalt to the edge of the shopping pods. As with the rest of the infrastructure, the parking lot design was functional and adequate — good enough to park your car, with a few trees and

some throwaway landscaping, but a sad replacement for the vast expanse of woodlands that had been cut down to provide the space.

Once in the village, I joined the hundreds of shoppers with glazed eyes shuffling from store to store. I thought of a scene from the movie, *Dawn of the Dead,* where the zombies mill around a shopping mall, buying merchandise and chewing on flesh. In keeping with the discount theme, the public spaces of the factory stores had few amenities. There were not many benches or water fountains. At one intersection of pedestrian lanes, the design provided a confusing jumble of pathways and shrub landscaping, as if the designers couldn't decide whether it was more important to make the space look nice or to funnel the shoppers away from the center and closer to the edges. The edges were where the stores were, and the object was to get the patrons to shop and spend. At one pedestrian lane intersection, there was a small overhead trellis that had no discernible function. No grape vines grew on it, and it didn't provide shade or cover from the rain. It was a functionless structure not even in keeping with the discount layout of the rest of the village.

The part of the infrastructure that seemed best developed was the signage. At any location in the village, you could see signs showing you where you were and where the other stores were. There weren't many benches to sit on to read the signs, but at least you could easily make your way from store to store.

It Takes a Village

It's interesting to compare this shopping village to its traditional New England namesake. In the eighteenth and nineteenth centuries, small towns dotted the landscape between urban areas in New England. The traditional farming village layout included a grassy public common with surrounding residential areas. The real Wrentham Village is not too far from the shopping center, and it is representative of this agricultural tradition. However, the real village does not offer the efficiencies provided for mass consumption that the new shopping village does. Its few stores date from the nineteenth century and are not situated, nor do they provide easy access, for resupply via large trucks. Their floor plans are too small by today's big-box standards, and can compete only as boutique shops, small restaurants or niche suppliers. In the real village, space is provided for pedestrians but not for thousands of cars in giant parking lots. The new shopping version of Wrentham Village is representative of the type of shopping infrastructure that is being built all over the United States. These new facilities offer easy access, are designed to get in and out of as efficiently as possible (for both stocking and purchase), provide ample floor space for products and have minimal, cost-efficient public spaces.

What makes the example in Wrentham more jarring than elsewhere is its comparison to the traditional New England farming village just a stone's throw away. In much of the rest of the country, the surrounding sprawl of infrastructure is not that much different from these new shopping villages. But in New England, infrastructure in the traditional towns tends to be human-scaled and of a layout and design that seems to fit the landscape instead of overwhelming it. In the Wrentham Village shopping center, the merchandise is a bargain, but it seems that the design of public space and the center's fit in its surroundings are also offered at a discount.

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The view southbound during the construction of the Zakim Bridge on a cold winter day with the perfect reflection of the bridge deck on the Charles River. Photo by Eric Barber.