WALT WHITMAN BRIDGE SUSPENDED SPANS REDECKING

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Abstract
The Walt Whitman Bridge (WWB) over the Delaware River serves as a vital artery between southern New Jersey and Philadelphia, carrying an average of 140,000 vehicles per day. Since its opening in early 1957; the WWB has become the "workhorse" of the four long-span bridges owned and operated by the Delaware River Port Authority (DRPA), providing primary access to Center City Philadelphia, the Delaware River ports, and the stadium complex in South Philadelphia. The suspended structure features a 2,000 ft main span, two 770 ft back spans and carries seven lanes of traffic. A deck study commissioned by the DRPA in 2005 determined that due to heavy use and deterioration, the original concrete-filled grid deck needed to be replaced. In 2007, AECOM – with Weidlinger Associates (WAI) as a primary sub-consultant – was retained to perform preliminary and final design for the replacement of the WWB's 3500 ft of suspended span deck.

Keywords
jointless, grid deck, floating deck, elastomeric bearings, relief joints, steel orthotropic deck

1. Introduction

The purpose of this paper is to give insight into the decision making and evaluation process used to determine the chosen deck alternative for the suspended spans on the Walt Whitman Bridge (Figure 1). The paper provides background on the deck alternatives studied during the preliminary design phase and the decision process used for evaluating each alternative. The key components of the preliminary design for both an orthotropic and filled grid deck are presented. In addition, the paper presents the numerous traffic schemes that were evaluated to minimize impact on traffic and construction.

Figure 1: Walt Whitman Bridge

The Walt Whitman Bridge was opened to traffic in early 1957, and has become the busiest of the DRPA's four bridges, carrying an average of 140,000 vehicles per day. The suspended structure features a 2,000 ft main span, two 770 ft back spans and a minimum navigational vertical clearance of 150 ft. The bridge carries seven lanes of traffic in seventy-nine feet curb-to-curb width. The seven lanes include a reversible center lane, which can be switched to accommondate peak traffic demands by relocating the moveable median barrier.

The suspended structure is composed of two stiffening trusses connected to the main support cables and suspenders, and transverse floor beam trusses spaced at approximately twenty-feet on centers. The existing deck is supported on stringers. An important characteristic of the existing deck in the suspended spans is the presence of relief joints located approximately every 121 feet (Figure 2). These joints have been the source of corrosion effecting the stringer ends and the top chords of the floor beam trusses (Figure 3). This situation, combined with the design of the stringer to floor beam connection, and the overall lateral-torsional behavior of the suspended structure, has introduced areas of stress concentrations at the relief joints. These factors have combined to create fatigue cracks in the stringer webs. While these cracks are self arresting, they have likely played an important role in the advancement of corrosion damage caused by water infiltration through the relief joints. The elimination of deck joints was an important factor in the deck alternatives evaluation.

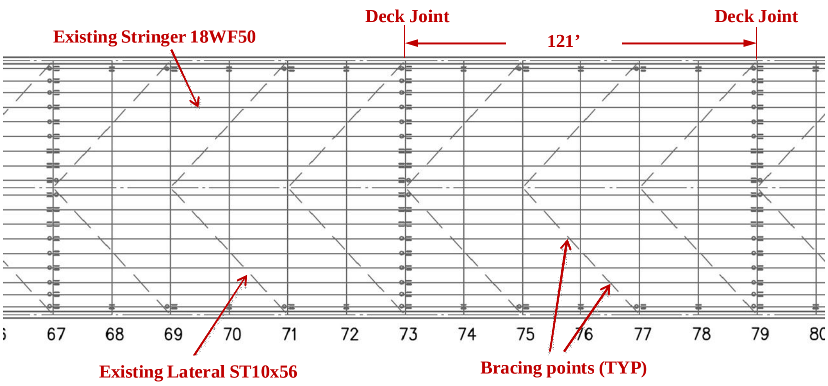


Figure 2: Deck, Stringers and Top Laterals

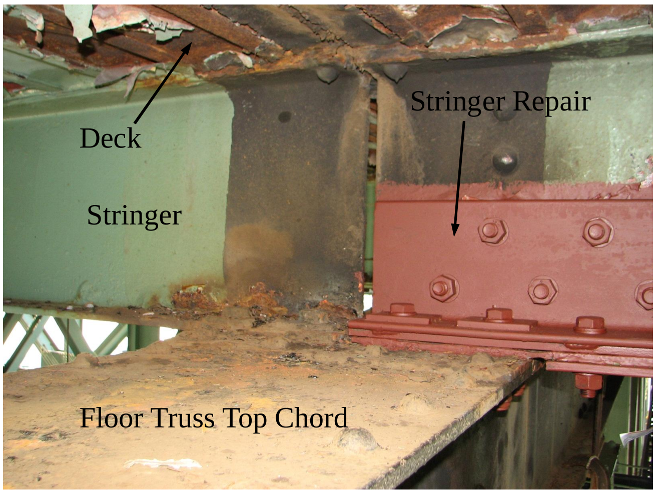


Figure 3: Deterioration in Deck, Stringers and Floor Trusses

Between 2004 and 2005, the DRPA had several studies commissioned to evaluate the condition of the suspended portions of the bridge. These studies included a detailed cable evaluation and a deck condition study. The cable evaluation concluded that the overall cable strength had been reduced by nearly fifteen percent due to wire corrosion. The reduction in cable strength together with the additional weight from the moveable barrier resulted in a reduction in the cable factor of saftey from 2.74 to 2.33. Even though a safety factor of 2.33 was acceptable, the benefit of reducing the overall dead load on the cables was considered an important factor in the evaluation of deck alternatives.

The deck condition study revealed significant corrosion of the steel pan forms, main bars and transverse reinforcing bars, and cracking and deterioration of the concrete fill. The pan forms were severely deteriorated over eighty-percent of the total deck area with nearly ten-percent missing, primarily in the areas of the deck relief joints. Concrete cores revealed expansive corrosion on the reinforcing bars as the cause of the concrete cracking. The concrete cores also revealed a high chloride content, nearly twice the threshold limit for the onset of corrosion. The extent of the deterioration of the stringers at the relief joints was also of particular concern. The results of the study also indicated that significant deterioration of the deck had occured since the previous study was conducted in 1987. Based on the results of these studies, the DRPA determined that a total replacement of the existing deck and stringers was in order.

2. Initial Deck Replacement Alternatives

Two primary deck replacement alternatives, a steel orthotropic deck and a concrete-filled grid, were evaluated. Both alternatives were studied in detail with plans developed to a twenty-five percent level for both deck types. The deck alternatives were evaluated based on initial and life-cycle costs, construction schedule and overall constructability, construction staging and traffic impacts, fabrication and overall weight reduction.

Orthotropic

The use of an orthotropic deck as a replacement alternative offers advantages in weight reduction, design life, maintainability and an increase in the global structural performance of the bridge. The deck plate proposed for the Walt Whitman Bridge was 3/4" thick, with 5/16" rib plates, and a rib spacing of 24 inches. Preliminary design of the orthotropic deck alternative produced a design with a weight of 58 pounds per square-foot (psf), approximately 8 psf less then the original grid deck.

In addition, the use of an orthotropic deck allowed for the removal of the stringers and laterals, further reducing the dead load. This benefit, assuming all retrofitting was completed, reduced the overall deck weight by 15 psf and reduced the cable stress by 5.5 percent relative to the original allowable design stress of 80 kips per square-inch (ksi). Increasing the cable safety factor from 2.33 to 2.47.

Orthotropic decks also have their disadvantages. Initial fabrication costs for orthotropic decks are typically higher due to the increased steel fabrication demands, and the need to apply a high degree of quality control to welded connections and details. To ensure the fabrication is completed without unacceptable weld defects requires an attentive team of inspectors. One important issue with past orthrotropic deck fabrication has been the percentage of penetration of the rib to deck plate weld. Typically, 80% penetration is required, but lack of penetration and/or melt through (Figure 4) have been issues with past generations of orthotropic decks. To solve this problem, the design team evaluated the possibility of using welding processes that are more controllable, such as laser assisted GMAW, that give a consistent 100% penetration with minimal melt-through. Although fatigue testing had been performed by the US Navy using this method, fatigue testing on typical welded joints as recommended by the bridge industry had not been completed at the time of preliminary design. It was estimated that testing and tooling costs associated with this type of welding process would have added an additional $3 million to the cost of fabrication.

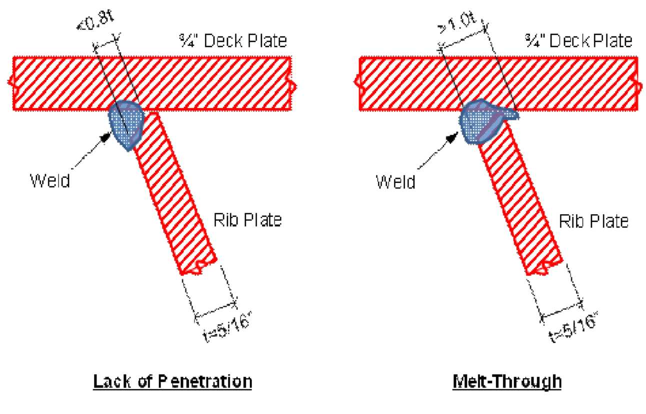


Figure 4: Defects in Rib-to-Deck Partial Joint Penetration (PJP) Welds

In addition, the propability that the orthotropic deck would be fabricated by a foreign fabricator was high due to the cost advantage of foreign fabrication. If the Buy America requirements were to be provisions in the contract, it was estimated the cost of the orthotropic deck would increase by approximately $11.5 million.

Filled Grid

There were two design concepts initially investigated for replacing the existing filled grid deck with another filled grid system. One was to replace it with a system that had deck joints similar to the existing, but with improved stringer to floor beam connections. The other was to construct a continuous grid deck with no deck joints. The relatively low costs of filled grid fabrication and construction came with both options. The latter concept required a more in-depth analysis to verify its ability to perform without failure or damage. Both options had the main grid bars running transversely to the support stringers.

The benefit of a design with deck joints is it allows for the differential temperature and live load displacment of the deck floor system relative to the floor beam trusses and the longitudinal stiffening truss. The deck joints allow for this displacement without introducing large forces into the system.

Preliminary design for the grid deck using normal weight concrete resulted in a design heavier than the existing deck. The new deck including the grid and concrete (flush-filled) weighed about 68.07 psf, as oppose to the 66.02 psf of the existing deck. Grid deck manufacturers consulted during the preliminary design process recommended using a half-filled grid with an overfill design (Figure 5), to provide more concrete cover on the grid bars. This type of design provides better protection against corrosion, better structural performance and provides a better substrate for the wearing surface.

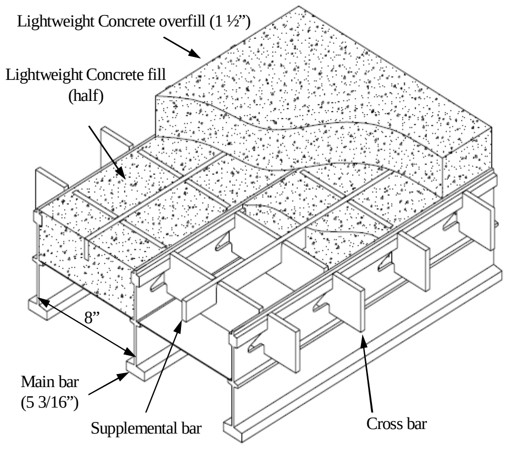


Figure 5: Details of New Grid Deck

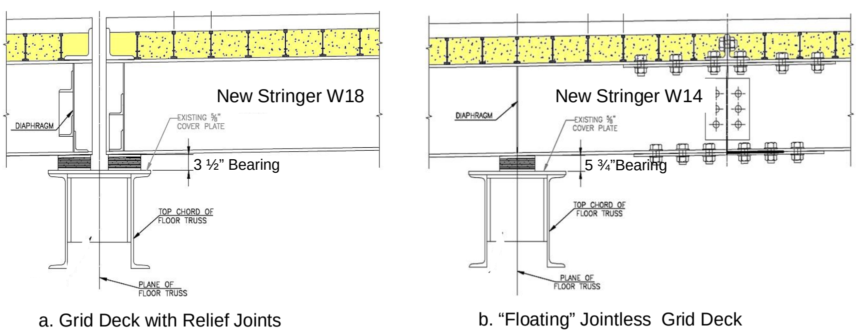
The intial concept for a jointless grid deck was to install steel shear connectors between the deck and the stiffening trusses, so that the relative movements between them would be restrained. After the intial study was completed, it was concluded that in order to transfer the shear forces, special panels of steel orthotropic deck or cast steel grid deck would need to be installed between the shear connectors and the regular grid deck. The additional cost for the shear and the special deck panels was estimated to be around $9 million. In addition, the connection details for the shear connectors and stiffening trusses and between the special deck panels and the regular grid deck panels could cause complications in both the design and construction.

3. Initial Recommendations and More In-Depth Evaluations

The preliminary deck design information was summarized in a draft preliminary design report and was submitted to the DRPA in November 2007. The initial consultant recommendation was to replace the existing deck with an orthotropic steel deck. The decision was based heavily on the advanatages of weight reduction and enhanced global structural performance of the main spans. However, DRPA expressed trepidation with an orthotropic deck based on three primary factors: (1) - their bad experience with maintaining a wearing surface on the Ben Franklin Bridge (BFB) orthotropic deck; (2) the initial cost of orthotropic, particularly given the agency's strong prefernce for domestically fabricated steel ("Buy America"); (3) concerns regarding the ability to control weld quality – in particular they pointed to weld cracking on the recently installed Bronx Whitestone Bridge orthotropic deck as evidence that quality could be an issue. Based on these concerns, DRPA asked the design team to revisit our recommendation in order to develop a grid option that would provide some of the same advantages as an orthotropic deck (weight reduction, low maintenance, improved structural performance).

Additional design and investigation was performed to determine if an economical grid deck option was available that provided some of the same advantages as orthotropic (weight reduction, improved structural performance, joint elimination). In an effort to reduce the weight, a design using lightweight concrete was investigated. Lightweight concrete has been used in bridge decks since the 1930's. In search for a record of performance in which lightweight concrete was used in a grid system on a suspended span bridge, little data was available. There was only one recorded example that had a similar deck and wearing surface condition. The bridge was a 2,300 foot long suspended arch over the Cape Cod Canal. The original lightweight concrete-filled grid was replaced in kind in 1986, with the original deck lasting forty-five years. The reason for the search was to obtain some record of performance that would provide some assurance that the degradation of the lightweight concrete was not faster than that of normal weight concete, so the proper economic comparisons could be made. Since numerous examples of long-term performance of lightweight concrete on grid decks on suspended spans was not found, a reduced service life was estimated for the lightweight concrete grid deck option. In the life-cycle cost analysis performed to evaluate the proposed deck alternatives, sixty-five year service life was assumed for the normal weight concrete grid deck and fifty year service life was assumed for the lightweight concrete grid deck.

Since elastomeric bearings had already been proposed for the grid deck with deck joints (Figure 6a), it was logical to take advantage of their flexibility to eliminate the original deck joints. (Figure 6b). The bearings for the "floating" deck option would be thicker, i.e. more flexible, than those for the deck with deck joints, and would allow relative movement between the deck and the floor beam trusses by deflecting longitudinally. A combination of bonded and sliding bearings were proposed. The bonded bearings would be located in the middle portions of the main and side span, while sliding bearings would be located near the towers and anchorages. Approximately 3,000 elastomeric bearings would be required for the project.



a. Grid Deck with Relief Joints

b. "Floating" Jointless Grid Deck

Figure 6: Both Deck Options and Their Elastomeric Bearings

In this new design with the deck ”floating” and moving relative to the floor trusses and lateral bracing system, the existing top laterals (WT-shape) would loose their bracing supports and hence a significant portion of their capacity. It was therefore decided to replace the existing laterals with heavier W-shape sections.

Although it would have been acceptable to let the entire deck move transversely relative to the floor beam trusses with the help of the elastomeric bearings, we felt it would be beneficial to restrain the deck in the transverse direction, so that total bearing deformation could be minimized. In order to restrain the transverse movement, transverse shear keys would need to be installed near the centerline of the bridge on every other floor beam truss.

The elastomeric bearings proposed for the ”floating deck” concept (Figure 7) , would also provide the benefit of allowing the deck system to function more independently from the floor beam trusses, thereby eliminating the existing hard steel bearings which contributed to the original stringer cracking.

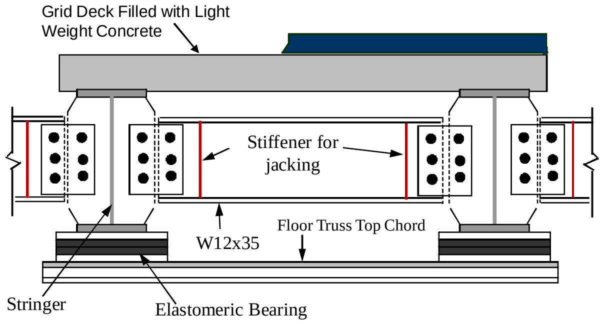


Figure 7: New Grid Deck, Stringers, and Elastomeric Bearings

Due to the new details proposed above, the total weight of the ”floating” deck system was recalculated and compared to the existing deck, as well as the proposed grid deck with joints. The ”floating” deck weight, including the stringers, bearings, laterals and wearing surface was calculated as 96.55 psf, which was slightly higher then the proposed grid deck with joints (92.70 psf), but still less than 105.19 psf of the existing deck.

4. Decision Process and Final Recommendations

The updated evaluations were summarized in a revised preliminary design report that was submitted to the DRPA in January 2008. Recognizing that the final deck type decision would be based largely on a range of DRPA preferences, , a decision matrix tool was developed and provided to assist in the process. The four deck types compared in the matrix were the orthotropic deck – foreign fabricated, orthotropic deck – domestically fabricated, lightweight filled grid with joints and a lightweight filled grid without joints. The goal of the decision matrix was to allow each stakeholder to independedntly apply an importance or weighting factor based on what they felt was most important. For example, one person may feel weight reduction is of primary importance and give that an importance factor of 4 where another stakeholder may consider initial cost as most important and give that category an importance of 4 (See the column shaded in yellow in Figure 8). Based on DRPA input, the evaluation was limited to five key categories: deadload of the deck system, initial cost, life-cycle cost, fabrication and construction. We provided the rating number based on how the systems compared with each other. For example, the lightest deck type would recieve the highest rating number and the heaviest would recieve the lowest rating number. The product of the importance factor and the rating factor produced a total score for each alternative and provided the DRPA a quantitative metod to compare the various deck alternatives with the highest total indicating the preferred deck alternative.

CATEGORY	Importance Factor	Orthotropic Deck (Foreign)		Orthotropic Deck (Domestic)		Light Weight Filled Grid with Relief Joints		Light Weight Filled Grid Without Relief Joints	
		Rating	Total	Rating	Total	Rating	Total	Rating	Total
Dead Load	1	5	**5**	5	**5**	4	**4**	3	**3**
Initial Construction Cost	3	3	**9**	1	**3**	5	**15**	5	**15**
Life Cycle Cost	2	5	**10**	4	**8**	4	**8**	5	**10**
Deck Fabrication	1	1	**1**	1	**1**	1	**1**	1	**1**
Deck Construction (Traffic Impact)	1	1	**1**	1	**1**	1	**1**	1	**1**
TOTAL			26		18		29		**30**

Figure 8: Sample Deck Decision Matrix

Based on aggregate scoring, the lightweight concrete-filled grid deck "jointless" option was selected because it offered a moderate yet meaningful amount of dead load reduction at a significantly reduced construction cost, with no increase in life-cycle cost. The grid deck could also be domestically fabricated and supplied, and is a system familiar to a broad array of contractors. In summary, the DRPA felt that a modern lightweight concrete-filled grid deck, which incorporates a new stringer support system designed to correct the long-standing lateral-torsional stringer cracking inherent in the original bridge, together with a long-term cable protection system program, was the most prudent course for the DRPA to take.

5. Construction Staging and Maintenance and Protection of Traffic

The Walt Whitman's diverse and demanding operational climate makes full use of the bridge's seventy-nine foot cartway. During weekday periods, the bridge functions as a typical commuter facility, with distinct peak periods in the westbound direction in the morning and eastbound direction in the afternoon/evening. Peak demand periods are extended and compounded during the summer months (Memorial Day to Labor Day), with the heaviest travel occuring on Friday evenings in the eastbound direction and on Sunday evenings and Monday mornings in the westbound direction. Demand also increases during heavily attended events at the sports complex in South Philadelphia and the entertainment center in Camden, with peaks occuring prior to and immediately after events.

A capacity analysis of the proposed construction staging was performed using Highway Capacity Manual reduction factors. The analysis assumed a minimum lane width of ten feet for interior lanes, eleven feet for curb lanes, and twelve feet for cattle chute lanes. This analysis determined a per lane capacity of 1740 vph. This capacity was compared to peak volumes calculated for the anticipated construction period by applying a two percent growth factor. This comparison indicated that four lanes of traffic would be required to accommondate peak period volumes, assuming no diversion of traffic existed.

The DRPA's experience during the redecking of approach spans, which occurred between 1995 and 1999, was that three lanes of traffic was sufficient to carry peak traffic in the eastbound direction without causing delays. Using yearly traffic data obtained from the DRPA's annual reports, a broader analysis on the regional traffic patterns during this period was performed to evaluate any changes to WWB traffic or regional traffic patterns that may have helped reduce eastbound peak hour demand at the WWB. The analysis revealed that demand on the WWB was reduced by approximately fifteen percent in the eastbound direction during the peak hour, and that corresponded to an increase of the same amount in the eastbound peak hour traffic at the Benjamin Franklin Bridge (BFB). Westbound diversion to the BFB would not have occured, since this would have been ineffective in reducing travel time due to the bottleneck conditions that exist at the western terminus of the bridge.

Based on this analysis, it was reasonable to expect that a minimum of fifteen percent of peak hour volume would be diverted to the BFB during construction. With this diversion of traffic, an hour-by-hour comparsion of volume/capacity revealed that three lanes of traffic were sufficient to handle peak hour traffic volumes without causing delays in the Eastbound direction. Westbound traffic was evaluated in a similar manner to the eastbound traffic and free flow conditions existed with four lanes but delays would be expereinced if only 3 lanes were provided.

In order to accommondate the anticpated traffic volumes during the construction without causing delays, it was recommended that four lanes be provided in the westbound direction and two lanes in the eastbound direction during the morning peak period, and that the moveable barrier be shifted to provide three lanes in each direction during the evening peak period and at all other times. This maintenance-of-traffic scheme would require that the construction be performed in seven long-term stages. The seven stages was identical to that which was successfully used during the approach reconstruction in the late nineties.

It should be noted that during the fourth stage (Stage 4), the work zone will be in the center of the bridge, making it impossible to shift the moveable barrier to accommodate peak hour traffic. During this stage, three lanes of traffic will be provided in each direction during both morning and evening peak periods, which will result in westbound morning delays. It was recommended that the DRPA engage in an aggressive media and public outreach effort in the weeks leading up to Stage 4, in order to encourage alternative travel patterns during this period. In addition, the DRPA included a monetary incentive for Stage 4, if the contractor could complete the stage work in under 120 calendar days. The contractor completed Stage 4 in 90 days earning the full monetary incentive.

A four stage construction scheme was also evaluated, given the significant cost savings that could be achieved through a reduction in stages. The scheme would utilize a reversible lane to provide three lanes of traffic in the peak direction, and two lanes in the non-peak direction during each stage. As noted in the discussion on westbound traffic operation, without a fifteen percent diversion of westbound traffic away from the WWB, the provisions of three westbound lanes would cause delays and back-ups during the morning peak periods.

In order to achieve the minimum lane widths used in the traffic evaluation, it was recommended that the existing cartway be widened from the existing 79’-0” to 80’-6” (9” on each side). The widening would provide a benefit to the construction staging, but it would also provide a narrow nine inch shoulder, which could be used to store/collect stormwater runoff. The nine inch widening would result in a reduced width of the maintenance walkway behind the roadway barrier (2’-9” to 2’-0”), however this reduction would not impact maintenance access.

It was also recommended to replace the existing two foot wide concrete moveable barrier with a thirteen inch wide steel barrier. The narrower barrier would not only maximize the width available to traffic during construction, but it would also provide better long-term performance, requiring less routine maintenance. The steel barrier also performed better during impact testing, deflecting twenty-eight inches, as opposed to fifty-three inches for the concrete barrier.

6. Status of Construction

The Contractor is currently performing the redecking work using the seven long term construction stages that progress across the bridge one lane at a time, with each stage taking approximately 3 to 4 months. The Contractor is currently working on Stage 6, with all construction expected to be completed by summer 2013.

7. Conclusion

Selecting the preferred deck alterantive for the deck replacement of the Walt Whitman Bridge was a collaborative effort between the design consultants and the owner. The studies conducted by AECOM and Weidlinger provided the engineering and knowledge for the owner to perform their own internal review and objectively evaluate each of the proposed alternatives. The use of the decision matrix provided the owner with a tool to quantitatevly compare the alternatives based on which of the five key categories was most important.

This project has demonstrated that a lightweight concrete filled grid deck without joints (floating deck) is a feasible deck replacement option for owners looking to replace aging decks on suspended type structures. The jointless grid deck has an advantage over orthotropic decks in initial costs and is comparable to orthotropic in life-cycle costs.

Being a toll facility, the Walt Whitman Bridge is a major source of revenue for the DRPA. Long-term construction and traffic delays have a significant effect on their ability to collect this revenue. Reducing construction time and traffic delays are important to any project and the use of monetary incentives can be an effective method of significantly reducing both of these. As this project has shown, when the constuction had the greatest impact on traffic (Stage 4), the monetary incentive helped minimize the impact by increasing the contractor's rate of production.

References

Ye, Qi and Griffith, Daniel, *"Floating Jointless Steel Grid Deck for Walt Whitman Bridge"*, International Bridge Conference, 2010