Applications of Orthotropic Decks in Bridge Rehabilitation

ROMAN WOLCHUK

A bridge deck, subject to direct loading by heavy truck wheels and exposed to weather effects, is the most severely stressed structural element of a bridge. The static and dynamic effects of the moving wheel loads and the high ratio of live to dead load cause a wide range of alternating stresses in the deck. Repeated millions of times during the deck life, these stresses precipitate material fatigue. The decks are designed for standard loads stipulated in the AASHTO specifications. However, the actual wheel and axle loads are frequently much heavier and may cause severe overstress. Another significant contribution to flexural deck stresses is due to differential deflections of the supporting stringers, which is not addressed in the current design specifications for concrete decks.

Exposure of the deck to wide temperature variations, moisture and freeze-thaw cycles further decreases the deck strength by causing concrete volume changes and cracking. However, the most important cause of deck deterioration is the action of deicing salts that penetrate concrete and corrode reinforcing steel. High pressures exerted by the rust cause further cracking, spalling and delamination of concrete, opening the way to more salt penetration and thus accelerating the irreversible process of concrete deck failure (Fig. 1). Under the combined effects of deck overloads, weather and salts, concrete decks on steel, as well as on prestressed concrete bridges, some less than 30 years old, are failing at an alarming rate and must be replaced. Many improvements are being devised for the replacement decks, adding to their cost, such as increased concrete strength and density, thicker deck slabs which offer better reinforcement coverage, epoxy coating of reinforcement, protective membranes and ingenious cathodic protection systems. Yet, because of the nature of concrete, and with ever increasing loads and intensity of truck traffic, useful life of concrete bridge decks will continue to be limited. Such decks may have to be replaced again in the future, before the end of the useful life of bridge superstructures.

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An alternative to bridge decks being treated as disposable elements, to be discarded and replaced when worn out, is offered by steel orthotropic decks that are inherently suited to overloads and are not affected by weather or chemicals. Advantages of orthotropic redecking are illustrated by four examples of recent bridge rehabilitation projects.

FOUR REDECKING CASE HISTORIES

George Washington Bridge[1,2]

This 610–3,500–650–ft span suspension bridge over the Hudson River opened to traffic in 1931. In 1962, the lower deck was added. The average volume of traffic over the 14 lanes of the bridge is now more than 230,000 vehicles per day.

The upper deck is two 44-ft wide, four-lane roadways separated by a 2-ft wide median barrier (Fig. 2). The original deck was a 8½-in. thick concrete slab reinforced with embedded bulb tees spanning in the longitudinal direction of the bridge between steel secondary floorbeams spaced at 5 ft-2 in. The concrete deck was topped with ¾-in. thick

Fig. 1. Deterioration of concrete deck on Ben Franklin Bridge

silica sand asphalt pavement. Deck deterioration (spalling, scaling and potholing) became noticeable in the late 50's, and proceeded at an accelerated rate. By 1970, about 25% of the deck area had been repaired by patching. Subsequent soundings and core tests established extensive delaminations and chloride contamination averaging from two to as high as 14 lbs. of chloride per cu. yd. of concrete. It was estimated that up to 46% of the deck may be in a deteriorated condition. At this stage, after studying various alternatives, the Port Authority decided to redeck the entire 434,000 sq. ft of the upper deck and the New York approach ramp with an orthotropic deck.

The new deck (Fig. 3) consists of a ⅝-in. thick deck plate stiffened with 7-in. deep rolled tee ribs spaced at 15 in. The ribs are continuous over the existing 16-in. deep secondary floorbeams. The 11-ft wide × 60-ft long deck units are bolted to the 16-in. beams through oversized holes in the continuous 8-in. × ½-in. strap plates shop-welded to the flanges of the tees. Longitudinal splices between the adjacent units are made by bolting, back-to-back, the 7-in. deep edge stiffening angles. The deck units are discontinuous at the transverse joints over the main bridge floorbeams spaced at 60 ft. The surfacing is a 1½-in. thick course of bituminous asphalt applied in the shop. The longitudinal and transverse joints in the pavement at the edges of the deck units are filled with preformed neoprene compression seals.

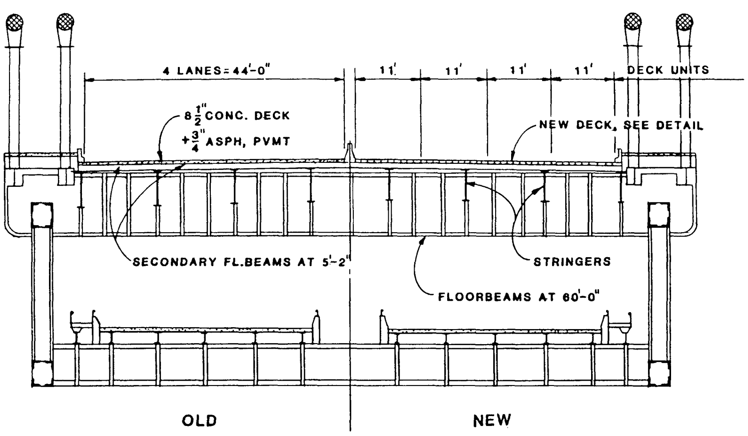

Fig. 2. George Washington Bridge

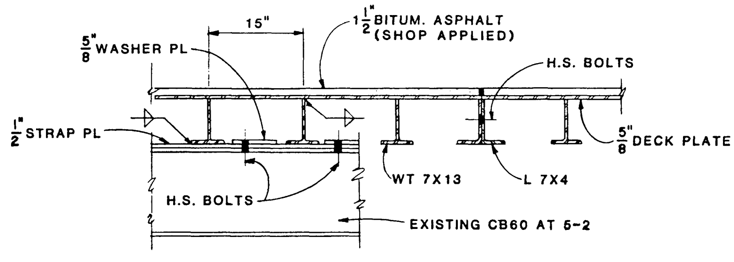

Fig. 3. George Washington Bridge—new deck details

All construction work of pre-cutting and removal of the concrete deck and installation of the new units was done at night, in 10-hr. work shifts, with a full width of roadway available for normal traffic each morning. With a minimum of four deck units placed each night, work was done within a two-year construction period and completed six weeks ahead of schedule in October 1978.

All engineering on the project was done by the Port Authority of New York and New Jersey, the owner of the bridge. Karl Koch Erecting Co., Inc., was general contractor.

Golden Gate Bridge[3]

The Golden Gate Bridge (1,125–4,200–1,125-ft suspended spans) was completed in 1937. Its six traffic lanes carry a traffic volume of about 120,000 vehicles on a typical weekday. The original 7-in. thick concrete roadway deck was 60-ft wide between the curbs. The deck was supported by rolled beam stringers spaced at 4-ft-9-in. (Fig. 4). Because of the salty atmosphere of the San Francisco area, the concrete roadway began to show signs of distress in the 60's. In 1976, it was established that the chloride content in concrete exceeded the threshold limits for rehabilitation. Subsequent engineering studies concluded that an orthotropic deck replacement was more cost effective than any other replacement scheme.

The new 50-ft long deck units are continuous over two 25-ft spans between the existing floorbeams. The ⅝-in. thick deck plate is stiffened by 11-in. deep, ⅜-in. thick closed trapezoidal ribs. The longitudinal splices of the deck plate between the units were welded in the field. The deck units are discontinuous over alternate floorbeams, with open deck joints filled with neoprene seals; 80% partial-penetration groove welds connect the ribs with the deck plate. The deck units are connected to the floor beams by

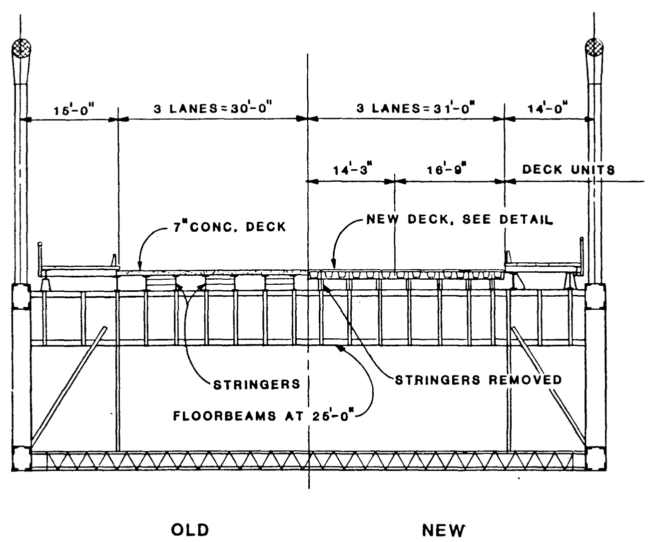

Fig. 4. Golden Gate Bridge

bolting to the support pedestals at the locations of the original stringers that were removed (Fig. 5). The widths of the deck units are 14-ft-3-in. for the two interior and 16-ft-9-in. for the two exterior deck panels, adding up to a widened roadway of 62 ft between the curbs.

The initial surfacing applied on the deck plate prior to installation was a ¼-in. thick layer of crushed rock embedded in epoxy asphalt mastic. This surfacing served the traffic needs for close to two years, until it was topped with the final 2-in. thick course of epoxy asphalt in 1986.

Orthotropic redecking extends over the suspended spans, and the San Francisco and the Marin approach viaducts for a total area of new deck of 567,000 sq. ft. Similarly, as in the case of the George Washington Bridge, all redecking work was done at night, on one-half width of the roadway, with all lanes open to traffic during daytime. Redecking was completed in August 1985, 401 working days after installation of the first deck unit.

Ammann and Whitney were the engineers for the Golden Gate Bridge, Highway and Transportation District. A joint venture of Dillingham Construction Co. and Tokola Offshore were general contractors. The deck panels were fabricated by Chicago Bridge & Iron Co. in Salt Lake City.

Throgs Neck Bridge—Approach Viaduct[4]

The Bronx and the Queens approaches to the Throgs Neck suspension bridge in New York City (1961) are two-girder bridges with simple spans ranging between 140 to 190 ft. The viaduct carries two 38-ft wide roadways separated by a 4-ft wide median divider (Fig. 6). The bridge cross section shows long and relatively shallow floorbeam cantilevers carrying the heavily travelled exterior truck lanes.

In the early 70s, spalling and cracking of the 7½-in. thick concrete deck was observed in the truck lanes, with distress gradually spreading to the remaining parts of the roadway. The reason for deterioration in this case was determined to be overstress of the concrete deck subject to additional effects of differential deflections of stringers elastically supported by flexible floorbeam cantilevers. Such effects are not accounted for in the AASHTO design specifications that stipulate design of concrete decks based on the assumption of rigid supports by the stringers. A contributing factor in this case was the inadequacy of longitudinal distribution reinforcement in the slab that would have alleviated the secondary effects of stringer deflections.

The design of the orthotropic replacement deck (Fig. 7) is characterized by the heavy 1-in. thick deck plate required by the wider than usual spacing of the supporting trapezoidal ribs. The 12-in. deep and 7⁄16-in. thick trapezoidal ribs fit between the existing rolled beam stringers that, unlike in the Golden Gate Bridge, were not removed. The new deck is supported by new W16 support beams between the stringers offset by two feet from the existing floorbeams. The spacing of the supporting beams varies between 20 and 28 ft. The new deck is fully continuous within each simple span of the approach viaduct, with field-bolted rib splices and field-welded longitudinal and transverse deck plate splices. The surfacing consists of a ⅛-in. thick, shop-applied epoxy and grit seal coat topped in the field with a 1½-in. thick course of bituminous asphalt.

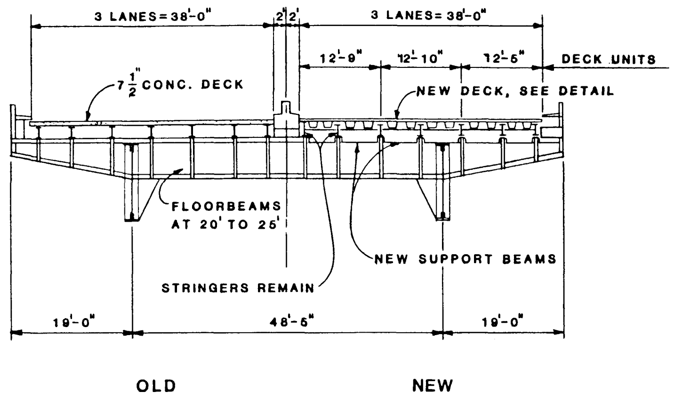

Fig. 6. Approach viaduct to Throgs Neck Bridge

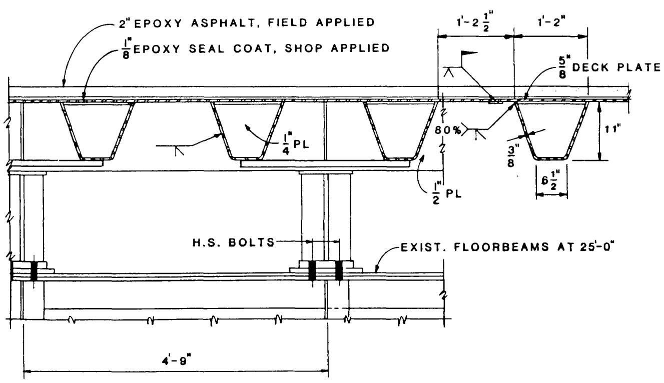

Fig. 5. Golden Gate Bridge—new deck details

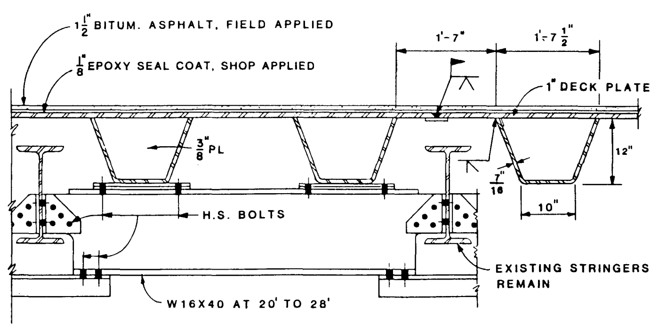

Fig. 7. Throgs Neck viaduct—new deck details

The deck units are from 12-ft-5-in. to 12-ft-10-in. wide and up to 52-ft long. Sections of existing concrete slab were removed and the new deck panels placed during the night hours, with all lanes open in the morning. The total area of redecked roadway in the Bronx and the Queens viaducts was 492,000 sq. ft.

Redecking design was prepared for the Triborough Bridge and Tunnel Authority by Ammann and Whitney. Karl Koch Erecting Co. was general contractor.

Benjamin Franklin Bridge[5]

This suspension bridge with 717–1,750–717-ft spans was opened to traffic in 1927. It carries seven lanes of vehicular traffic, with a daily volume of close to 100,000 vehicles and two rail transit tracks (Fig. 8).

The original 6½-in. thick concrete deck reinforced with trussed bars was progressively deteriorating under the effects of deicing salts and heavy traffic (Fig. 1). Further problems were caused by corrosion of the steel stringers underneath the frequent open joints in the deck, which required closing to traffic of the exterior roadway lanes where the damage was most severe. In 1982, the Delaware River Port Authority decided to redeck the suspended spans and both approaches to the bridge, a total roadway area of 600,000 sq. ft, with an orthotropic deck.

For the new deck an open-rib system was chosen. While somewhat heavier than a closed-rib system, it offered the advantages of simple deck splices and connection details to the existing floorbeams and total accessibility to the underside of the deck. Specially rolled 12½-in. deep bulb sections spaced between 13¾-in. to 15½-in. are used to support the ⅝-in. thick deck plate (Fig. 9). The deck is supported directly by the existing floorbeams spaced between 19–22 ft, which permitted removal of all roadway stringers. The deck units are fully continuous, with field-bolted splices of the deck and ribs. Thus all deck joints in the suspended spans, other than the finger joints at the towers and the anchorages, were eliminated. In the approaches, simple-span girders and trusses, the necessary deck joints are offset by three ft from the existing floorbeams by addition of new support beams at the joints. The purpose was to eliminate the maintenance problems at the joints that plagued the old deck (Fig. 10).

To assure the quality of the 1¼-in. thick epoxy asphalt base surfacing course, it was placed on the deck under controlled conditions in the steel fabricating shop. This required one-sided field splices of the deck plate made on the underside of the deck by means of interference body bolts inserted in the shop prior to paving. The final 1½-in. thick surface course of bituminous asphalt was placed in the field after redecking. The continuous new deck in the suspended spans acts as a fully participating component of the stiffening truss system of the bridge, achieved by shear connectors between the deck and the bottom chords of the stiffening trusses (Fig. 11). The connectors, spaced from 60 to 200 ft, are designed to carry the flexural live-load shears between the deck and the trusses. Structural integration of the deck with the stiffening system considerably increased the flexural and the torsional rigidity of the bridge and improved its aerodynamic characteristics.

Installation of new deck units proceeded in four construction phases, beginning with the east-bound lanes. During each phase, five traffic lanes on the bridge were open in the peak hours and four in off-peak periods. Work was done between fixed traffic barriers during daytime hours.

Engineering on the Benjamin Franklin Bridge rehabilitation project was done jointly by Baker Engineers and Weidlinger Associates, the latter responsible for orthotropic redecking. Cornell & Co., W. F. Hegarty Co. and Karl Koch Erecting Co. (a joint venture) were the general contractors.

For purposes of comparison, the basic engineering data for the four redecking cases discussed are summarized in Table 1.

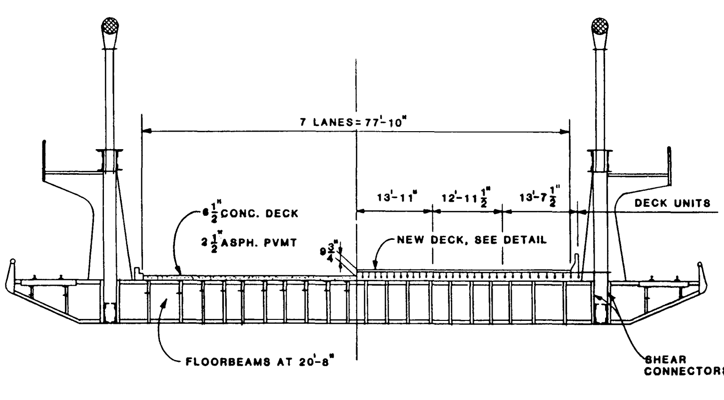

Fig. 8. Benjamin Franklin Bridge

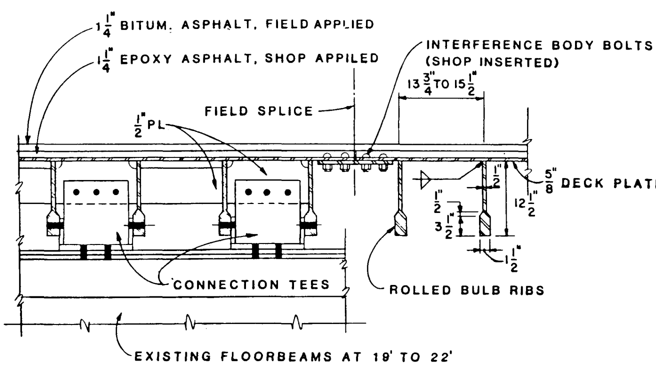

Fig. 9. Ben Franklin—new deck details

Table 1. Redecking of Bridges—Engineering Data

BRIDGE	G. Washington Bridge—NYC	Golden Gate S. Francisco	Throgs Neck Viaduct—NYC	Benj. Franklin Bridge—Phila.
Description of bridge redecked	Suspension bridge	Susp. bridge + truss approach spans	Plate girder viaduct	Susp. bridge + truss and girder approach spans
Redecking completed	1978	1985	1986	1987
Total deck area (sq. ft)	434,000	567,000	493,000	600,000
Type of ribs	Open (Tees) Fig. 3	Closed Fig. 5	Closed Fig. 7	Open (bulb sections) Fig. 9
Rib span	5′-2″	25′	20′ to 28′	19′ to 22′
New deck surfacing	1½″ bitum. asphalt	¼″ epoxy seal coat 2″ epoxy asphalt	⅛″ epoxy seal coat 1½″ bitum. asphalt	1¼″ epoxy asphalt 1¼″ bitum. asphalt
Unit weight of new deck—steel (psf)	42	50	65	60
Unit weight of new deck—surfacing (psf)	18	29	18	29
Total unit weight—new deck (psf)	60	79	83	89
Unit weight—old concrete deck and removed supporting steel (psf)	106[1)]	104	107[2)]	123
Net deck weight saving (psf)	46	25	25	34
New deck integration with the bridge main members	No	No	No	Yes[3)]

Notes:
1) existing stringers and sub-floorbeams remain
2) existing stringers remain
3) suspended spans only

STRUCTURAL ADVANTAGES OF ORTHOTROPIC REDECKING

Reduction of Dead Weight

The weight of a concrete bridge deck, with a 2-in. thick wearing surface may range from 110 to 150 psf., and the supporting stringers between floorbeams may weigh 10–20 psf. The weight of an orthotropic deck may be 45–65 psf, depending on design, and connections and splice details. To this, add the weight of the surfacing of about 30 psf for a 2½-in. thickness, and less for thinner overlays. Thus, redecking by an orthotropic plate may decrease the roadway dead load by as much as one-half. In the cases discussed above, the roadway weight savings ranged from 25 to 46% (Table 1).

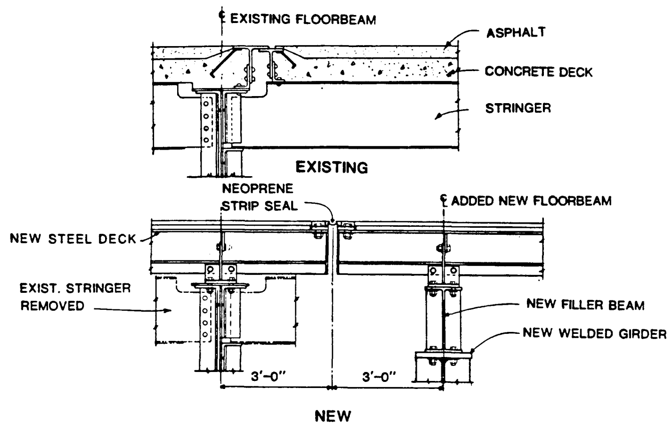

Fig. 10. Ben Franklin. Deck joints in approach girder spans

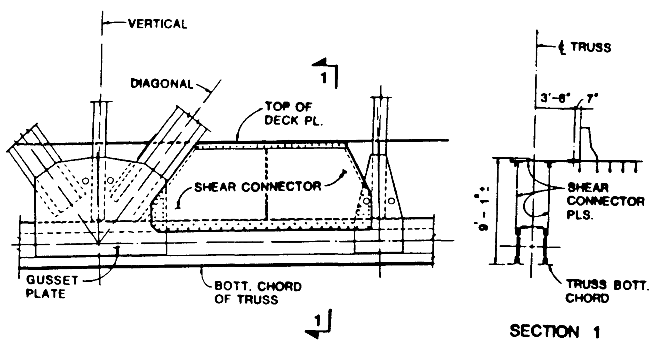

Fig. 11. Ben Franklin. Shear connectors between new deck and stiffening trusses in suspended spans

The roadway weight reduction directly increases the live load carrying capacity of the underlying floorbeams and the main bridge members, which is important in view of the ever heavier actual truck weights and correspondingly increasing live-load design requirements for bridges. A good example of such capacity enhancement by orthotropic redecking are the approach spans of the Benjamin Franklin Bridge. Prior to redecking, the live-load rating of several floorbeams carrying heavy concrete decks topped with brick courses and asphalt overlays was as low as HS13. After removal of the existing deck and the stringers and installation of the new steel deck, the rating of these floorbeams now exceeds HS25, the stipulated design loading specified for redecking.

Dead-weight reduction may be of decisive importance in cases where bridge rehabilitation also includes roadway widening for added new traffic lanes or shoulders. With a new orthotropic deck, the existing steel superstructure and the substructure may be able to carry the added loads, but could be inadequate if heavy concrete decks were used.

Increased Deck Overload Capacity

The remarkable local overload capacity of orthotropic decks, with actual ultimate loads several times greater than the values calculated by the first order flexural theory is a well known property of such decks (Ref. 6, Sect. 1.2.4). Because of second order elastic behavior of orthotropic decks under increasing loading, such decks can easily carry occasional heavy wheel and axle loads of the special "overload" vehicles without suffering structural damage or permanent deformation.

Direct Strengthening of Main Bridge Members with Integration of Deck

In addition to increasing the live-load carrying capacity of the floor beams and the longitudinal main bridge members by decreasing the dead load imposed on them, these members may be strengthened directly by composite action with the orthotropic deck acting as an added top flange of the bridge floorbeams and the longitudinal girders. This was used to strengthen all floor beams on the Benjamin Franklin Bridge where the deck connections to the existing floor beams were designed for shears due to the deck-floor beam composite action.

The much more important enhancement of the stiffening truss system of the Benjamin Franklin Bridge by integration of the new deck with the stiffening trusses was discussed previously.

In a study of another existing suspension bridge with an inadequate capacity for increasing traffic, it was established that orthotropic redecking of the structure and utilization of the new deck as a strengthening element of the existing stiffening trusses may possibly permit installation of a second deck on the bridge. Its stiffening trusses would otherwise be considerably overstressed without such strengthening.

Another benefit is the increased structural redundancy of the integrated bridge superstructure.

Thus, redecking by an orthotropic deck may be viewed not only as replacement of an old concrete deck by a similar deck in steel, but also as a good opportunity to upgrade the bridge overall.

Constructability Under Traffic

One important advantage of redecking in steel rather than concrete is the suitability of modular orthotropic deck units for quick and expedient installation with a minimum of restriction of bridge traffic during construction. In three of the four examples listed in Table 1, deck units installed during the off-peak, night hours were opened to traffic next morning. With poured-in-place concrete, this possibility does not exist. It should also be noted the strength of orthotropic deck units is not impaired by installation under traffic. But doubts exist about the quality of field-poured deck concrete subject to continuous vibrations of the deck supporting members by traffic on adjacent lanes.

Ease of Maintenance

Just as other members of a steel bridge, orthotropic decks require regular maintenance painting. But because the deck undersides are well sheltered, the need to repaint is much reduced. Otherwise, unlike on concrete decks, there is no need for ongoing maintenance inspection and patching of the structural element of the bridge roadway, other than occasional and inexpensive repairs of the nonstructural surfacing. One favorable factor is the suitability of orthotropic decks for continuous construction, which, on long bridge spans, permits eliminating deck joints which always present serious maintenance problems.

Durability and Long Life Expectancy

Orthotropic decks are immune to damage by deicing salts and weather effects. This is well proven by some of the precursors of modern orthotropic decks in New York City that were subject to careful inspection prior to redecking in 1964. In spite of the fact that surfacing on these decks, consisting of 1 ft $\times$ 2 ft asphalt planks set in asphalt cement, was a rather poor protection of the deck steel against water penetration, the deck surface suffered only insignificant corrosion damage under the joints of the planking after 27 years of service.[7] Now these 50-yr. old steel decks, repaved in 1964 with bituminous asphalt, remain in service without noticeable distress or deterioration.

Durability of surfacings on steel decks with sufficient deck plate rigidity is quite satisfactory. One such example is the epoxy asphalt paving on the San Mateo Bridge (1967) which still does not require repairs and remains in a very good condition.

Generally, service life expectancy of orthotropic decks can be assumed to be not less than that of the underlying steel bridge superstructure, which, with proper maintenance painting, is limited by obsolescence only. This contrasts with the reasonable expectations for concrete decks (or thin reinforced concrete slabs placed on gratings). Even if designed and constructed with all currently practiced precautions, they may have to be replaced after 30–40 years of service.

DESIGN FOR REDECKING

Orthotropic decks for highway bridges are usually designed in accordance with the methods and procedures given in the AISC *Design Manual for Orthotropic Steel Plate Deck Bridges*,[6] subject to loadings, material thickness limitations and strength provisions of the current AASHTO Specifications for Highway Bridges.[8]

In redecking of truss and girder bridges with floor systems of longitudinal stringers supported by transverse floor beams, maximum efficiency and dead-weight saving is obtained if the existing stringers are removed and the new deck supported on the floor beams. This requires deck rib lengths between supports well exceeding the usual rib spans of existing orthotropic deck bridges. As seen from redecking examples previously discussed, deck spans exceeding 25 ft are feasible and practical. However, since the standard design methods of orthotropic decks are based on assumptions of much shorter rib spans, certain characteristics of long-span decks must be considered.

In the design of open-rib decks, the usual simplified approach assuming the ribs act as rigid supports of the deck plate is no longer valid. The long-span ribs, deflecting under wheel loads, act as elastic supports of the continuous deck plate, causing a wider transverse wheel load distribution. This results in a reduction of the loads and bending moments acting on the individual ribs, which can be calculated by a method suggested in Sect. 4.2.7 of Ref. 6. In the case of the Benjamin Franklin Bridge, this reduction of the governing rib moments was up to 30%.

The closed-rib decks have a much better capacity for lateral distribution of the individual wheel loads than the open-rib systems. The lateral load distribution increases with the deck span, and the effects of two adjacent wheels on the deck may overlap, thus increasing the loading on an individual rib. Therefore, for spans exceeding 15–20 ft, the loads, bending moments and deflections of closed ribs obtained by the standard formulas and design charts[6] will be unconservative.[9]

Live-load deflections of long-span decks may be considerable and should be limited to avoid undesirable deck springiness. Thus deflections, rather than strength considerations, may be governing the design. The choice between the open and the closed ribs depends on many factors. Designers usually prefer the more structurally efficient closed rib decks requiring less steel. The underside of the deck has less surface area to be painted and is smooth and easily accessible. Another advantage is in the fact that the total length of the rib-to-deck welds of the trapezoidal ribs is one-half of that of the open ribs. Among the disadvantages of closed ribs is the need for permanent air-tightness of the rib interiors requiring seals at the rib ends and at field splices, and the more complex field-splice details. Unlike the open ribs, the webs of trapezoidal ribs are subject to flexure under the wheel loads.

Connection details between the new deck and the existing steelwork must be designed with consideration of fatigue under the effects of alternating stresses in the deck at the supports. Support diaphragms subject to local flexure or twisting must be designed and detailed with care. Orthotropic decks may also be used to redeck multiple girder or stringer-type bridges where the original concrete deck was placed directly on the girder or stringer flanges. On such bridges, the new deck must be supported on suitably spaced transverse members between the girders. Transverse orientation of the deck ribs would be uneconomical and unsuitable because of the very unfavorable stress conditions in the deck plate under the wheel loads that would result in such case. Connection details between the new deck and the longitudinal members must be designed to assure structural integration of the deck with these members, with consideration of the live load shears and temperature effects. In all cases where the new deck acts compositely with the existing structure, proper allowances for the additional axial tensile or compressive stresses must be made in the deck design.

ECONOMIC CONSIDERATIONS

Construction Cost of Orthotropic Redecking

Cost data of the four redecking cases discussed are summarized in Table 2. Recent construction costs of orthotropic redecking of major structures were in the range of \$70–80 per sq. ft of deck. This cost includes removal of the existing concrete deck, fabrication and erection of a new steel deck and the new surfacing.

Note that the fabrication costs of orthotropic decks for the two most recent cases (Throgs Neck and Benjamin Franklin Bridge), 46¢ and 56¢/lb. respectively, could have been reduced further if the contractors were permitted to buy steel without provenance restrictions. The current unit costs compare favorably with some very high prices that were quoted for orthotropic decks in the early 80's. One favorable factor now is the availability of fabricators and contractors with good recent experiences in orthotropic redecking.

The cost of surfacing may vary widely, depending on materials used and the method of application. In the case of Benjamin Franklin Bridge placement of the epoxy asphalt in relatively small quantities (6–10 deck units at a time) under controlled shop conditions added to the cost. On the Golden Gate Bridge, the price reflects the cost of the 2¼-in. thick epoxy asphalt and the fact that the final course had to be placed on the deck at night. On smaller bridge structures with less difficult deck removal and installation problems, redecking costs of \$50–70 psf, or even less, may be expected.

Table 2. Redecking of Bridges—Cost Data

BRIDGE		G. Washington	Golden Gate	Throgs Neck	Benj. Franklin
Year project bid		1976	1982	1984	1984
Total cost of project[1] (million $)		18.5	56.6[2]	35.4[3]	56.4[4]
Cost of re-decking ($/sq. ft)	New steel deck (including old deck removal)	39.20	63.10	68.80	70.50
	New surfacing	.90	6.70	3.00	6.50
	Total cost	40.10	69.80	71.80	77.00
Unit cost of deck steel, erected (not including old deck removal) ($/lb.)		0.77	1.12	0.87	1.02

Notes:

1) Contract bid price. This generally also includes miscellaneous bridge repairs, walkways, utilities and other items not directly related to roadway redecking. Such items are not included in calculating the unit costs of redecking.
2) $52.5M original contract plus $4.1M additional contract for supplemental work and the wearing surface (1986).
3) $32M original bid plus $3.42M additional premium for domestic steel.
4) $53.8M original bid plus $2.6M premium for domestic steel.

Bridge Deck Maintenance and Repair Costs

Maintenance and repair costs of a concrete bridge deck may be formidable and increase rapidly with age. On the George Washington Bridge, the amount of repair work performed between 1970–1975 almost equalled that over the prior 10 years. By 1975, an area equivalent to 48% of the total deck surface had been repaired, at an annual cost averaging $500,000 per year.[1]

The average deck maintenance and repair costs on the Benjamin Franklin Bridge prior to redecking amounted to over $350,000 per year. Studies at that time indicated the average yearly cost would more than double to $760,000 should redecking be deferred 10 years. Thus, the average annual cost of the concrete deck maintenance and repairs on this bridge was estimated roughly at $1.00 per sq. ft of deck.

By contrast, maintenance of an orthotropic deck, consisting mainly of periodic painting of the deck underside and occasional repairs of the wearing surface, is inexpensive. On the Benjamin Franklin Bridge, because of the reliable paint system applied at the time of installation (inorganic zinc primer, topped with epoxy and urethane paint coats) and the fact the deck underside is well sheltered from direct effects of moisture (deck joints having been either totally eliminated, or suitably relocated), the Delaware River Port Authority does not plan to paint the underside again for 20 years. Replacement of the 1¼-in. bituminous surfacing is anticipated every 15 years. Based on these above schedules, the annualized maintenance cost of this deck is estimated conservatively at about $120,000, or $0.20 per sq. ft of deck.

Economic Evaluation of Redecking Alternates

The first-construction cost of redecking a bridge in concrete is generally less than that of redecking in steel. For the Benjamin Franklin Bridge, the cost of deck replacement in concrete was estimated to be lower than an orthotropic alternative by about $10M (on a $56M project).[10] Similarly, in the other cases discussed, the concrete deck solutions were "less expensive."

However, a meaningful economic comparison of alternate engineering solutions can only be made on the annualized life cycle cost basis. This should include, in addition to the first cost, the projected yearly cost of maintenance and repairs, as well as the annualized cost of future replacement (including economic effects of traffic disruption due to replacement) at the end of the useful life of the alternative under consideration. Using the Benjamin Franklin Bridge as an example, and assuming the first-construction cost of redecking in concrete at $60 per sq. ft of deck and a useful life of such deck of 40 years, the annual cost of this solution can be roughly estimated as $60/40 = $1.50 plus $1.00 for maintenance and repairs—a total yearly cost of $2.50 per sq. ft of deck.

For a steel deck alternate, use the service life of 80 years (which is conservative, since the life of a steel bridge superstructure can be prolonged indefinitely with proper maintenance and painting), and the initial cost of $80/sq. ft. An analogous calculation yields a cost of the steel alternate as $80/80 = $1.00 plus the above estimated maintenance and surfacing repair cost of $0.20—for a total yearly cost of $1.20 per sq. ft of deck.

Based on this comparison, it is evident the orthotropic deck solution, in addition to being functionally superior, is clearly more economical.

It is of interest to note the bridges in the four referenced examples are owned by independent user-financed bridge authorities who are legally obligated to take care of the public transportation facilities in their trust on a fiscally sound basis. Yet, for most public supported bridge projects, life cycle considerations including useful life expectancy and the future maintenance, repairs and replacement costs, are not applied to cost comparisons of alternative designs at bidding time. Instead, the choice of the alternate to be constructed is based predominantly on the expedient "lowest first cost" criterion.

It is to be hoped that, in this era of budget constraints, such short-sightedness will be replaced by policies based on prudent economics, instituted through the efforts of engineers and officials engaged in the administration, design and construction of bridges, that permit rehabilitation and preservation of our bridges with the least expenditure of public money in the long run.

SUMMARY AND CONCLUSIONS

The four cases of bridge rehabilitation discussed in this paper show an orthotropic deck is a good, economical and adaptable system excellently suited for redecking bridges. Orthotropic decks save weight, increase deck overload capacity and increase substantially the rigidity and the strength of the bridge.

Modular steel deck units can be erected expeditiously with minimum restrictions of traffic. Since these decks do not deteriorate under the effects of weather or deicing salts, maintenance costs are low. The service life expectancy of an orthotropic deck is as good as that of the underlying steel superstructure; therefore no future redecking need be anticipated, unlike a concrete deck.

The first cost of redecking in steel is likely to be higher than with concrete, but not by a very significant margin. However, keep in mind that "first cost" alone is a poor basis for deciding the method of redecking since a true economic evaluation of any engineering alternative can only be made on the life cycle cost basis. That should include, in addition to the first construction cost, the annual costs of the anticipated maintenance and repairs as well as the annualized cost of future replacement. On this basis, orthotropic decks are clearly superior to other types of bridge decks.

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