6

Strengthening and Rehabilitation

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6.1 Introduction

About one half of the approximately 600,000 highway bridges in the United States were built before 1940, and many have not been adequately maintained. Most of these bridges were designed for lower traffic volumes, smaller vehicles, slower speeds, and lighter loads than are common today. In addition, deterioration caused by environmental factors is a growing problem. According to the Federal Highway Administration (FHWA), almost 40% of the nation's bridges are classified as deficient and in need of rehabilitation or replacement. Many of these bridges are deficient because their load-carrying capacity is inadequate for today's traffic. Strengthening can often be used as a cost-effective alternative to replacement or posting.

The live-load capacity of various types of bridges can be increased by using different methods, such as (1) adding members, (2) adding supports, (3) reducing dead load, (4) providing continuity, (5) providing composite action, (6) applying external post-tensioning, (7) increasing member cross section, (8) modifying load paths, and (9) adding lateral supports or stiffeners. Some methods have been widely used, but others are new and have not been fully developed.

All strengthening procedures presented in this chapter apply to the superstructure of bridges. Although bridge span length is not a limiting factor in the various strengthening procedures presented, the majority of the techniques apply to short-span and medium-span bridges. Several of the strengthening techniques, however, are equally effective for long-span bridges. No information is included on the strengthening of existing foundations because such information is dependent on soil type and conditions, type of foundation, and forces involved.

The techniques used for strengthening, stiffening, and repairing bridges tend to be interrelated so that, for example, the stiffening of a structural member of a bridge will normally result in its being strengthened also. To minimize misinterpretation of the meaning of strengthening, stiffening, and repairing, the authors' definitions of these terms are given below. In addition to these terms, definitions of maintenance and rehabilitation, which are sometimes misused, are also given.

Maintenance: The technical aspect of the upkeep of the bridges; it is preventative in nature. Maintenance is the work required to keep a bridge in its present condition and to control potential future deterioration.

Rehabilitation: The process of restoring the bridge to its original service level.

Repair: The technical aspect of rehabilitation; action taken to correct damage or deterioration on a structure or element to restore it to its original condition.

Stiffening: Any technique that improves the in-service performance of an existing structure and thereby eliminates inadequacies in serviceability (such as excessive deflections, excessive cracking, or unacceptable vibrations).

Strengthening: The increase of the load-carrying capacity of an existing structure by providing the structure with a service level higher than the structure originally had (sometimes referred to as upgrading).

In recent years the FHWA and National Cooperative Highway Research Program (NCHRP) have sponsored several studies on bridge repair, rehabilitation, and retrofitting. Inasmuch as some of these procedures also increase the strength of a given bridge, the final reports on these investigations are excellent references. These references, plus the strengthening guidelines presented in this chapter, will provide information an engineer can use to resolve the majority of bridge strengthening problems. The FHWA and NCHRP final reports related to this investigation are references [1–13].

Four of these references [1, 2, 11, 12] are of specific interest in strengthening work. Although not discussed in this chapter, the live-load capacity of a given bridge can often be evaluated more accurately by using more-refined analysis procedures. If normal analytical methods indicate strengthening is required, frequently more-sophisticated analytical methods (such as finite-element analysis) may result in increased live-load capacities and thus eliminate the need to strengthen or significantly decrease the amount of strengthening required.

By load testing bridges, one frequently determines live-load capacities considerably larger than what one would determine using analytical procedures. Load testing of bridges makes it possible to take into account several contributions (such as end restraint in simple spans, structural contributions of guardrails, etc.) that cannot be included analytically. In the past few years, several states have started using load testing to establish live-load capacities of their bridges. An excellent reference on this procedure is the final report for NCHRP Project 12-28(13)A [14]. Most U.S. states have some type of bridge management system (BMS). To the authors' knowledge, very few states are using their BMS to make bridge strengthening decisions. At the present time, there are not sufficient baseline data (first cost, life cycle costs, cost of various strengthening procedures, etc.) to make strengthening/replacement decisions.

Examination of National Bridge Inventory (NBI) bridge records indicates that the bridge types with greatest potential for strengthening are steel stringer, timber stringer, and steel through-truss. If rehabilitation and strengthening cannot be used to extend their useful lives, many of these bridges will require replacement in the near future. Other bridge types for which there also is potential for strengthening are concrete slab, concrete T, concrete stringer, steel girder floor beam, and concrete deck arch. In this chapter, information is provided on the more commonly used strengthening procedures as well as a few of the new procedures that are currently being researched.

6.2 Lightweight Decks

6.2.1 Introduction

One of the more fundamental approaches to increase the live-load capacity of a bridge is to reduce its dead load. Significant reductions in dead load can be obtained by removing an existing heavier concrete deck and replacing it with a lighter-weight deck. In some cases, further reduction in dead load can be obtained by replacing the existing guardrail system with a lighter-weight guardrail. The concept of strengthening by dead-load reduction has been used primarily on steel structures, including the following types of bridges: steel stringer and multibeam, steel girder and floor beam, steel truss, steel arch, and steel suspension bridges; however, this technique could also be used on bridges constructed of other materials.

Lightweight deck replacement is a feasible strengthening technique for bridges with structurally inadequate, but sound, steel stringers or floor beams. If, however, the existing deck is not in need of replacement or extensive repair, lightweight deck replacement would not be economically feasible.

Lightweight deck replacement can be used conveniently in conjunction with other strengthening techniques. After an existing deck has been removed, structural members can readily be strengthened, added, or replaced. Composite action, which is possible with some lightweight deck types, can further increase the live-load carrying capacity of a deficient bridge.

6.2.2 Types

Steel grid deck is a lightweight flooring system manufactured by several firms. It consists of fabricated, steel grid panels that are field-welded or bolted to the bridge superstructure. The steel grids may be filled with concrete, partially filled with concrete, or left open (Figure 6.1).

Open-Grid Steel Decks

Open-grid steel decks are lightweight, typically weighing 15 to 25 psf (720 to 1200 Pa) for spans up to 5 ft (1.52 m). Heavier decks, capable of spanning up to 9 ft (2.74 m), are also available; the percent increase in live-load capacity is maximized with the use of an open-grid steel deck. Rapid installation is possible with the prefabricated panels of steel grid deck. Open-grid steel decks also have the advantage of allowing snow, water, and dirt to wash through the bridge deck, thus eliminating the need for special drainage systems.

A disadvantage of the open grids is that they leave the superstructure exposed to weather and corrosive chemicals. The deck must be designed so water and debris do not become trapped in the grids that rest on the stringers. Other problems associated with open-steel grid decks include weld failure and poor skid resistance. Weld failures between the primary bearing bars of the deck and the supporting structure have caused maintenance problems with some open-grid decks. The number of weld failures can be minimized if the deck is properly erected.

In an effort to improve skid resistance, most open-grid decks currently on the market have serrated or notched bars at the traffic surface. Small studs welded to the surface of the steel grids have also been used to improve skid resistance. While these features have improved skid resistance, they have not eliminated the problem entirely [12]. Open-grid decks are often not perceived favorably by the general public because of the poor riding quality and increased tire noise.

Concrete-Filled Steel Grid Decks

Concrete-filled steel grid decks weigh substantially more, but have several advantages over the open-grid steel decks, including increased strength, improved skid resistance, and better riding quality. The steel grids can be either half or completely filled with concrete. A 5-in. (130-mm) thick, half-filled steel grid weighs 46 to 51 psf (2.20 to 2.44 kPa), less than half the weight of a reinforced concrete deck of comparable strength. Typical weights for 5-in. (130-mm) thick steel grid decks, filled to full depth with concrete, range from 76 to 81 psf (3.64 to 3.88 kPa). Reduction in the deadweight resulting from concrete-filled

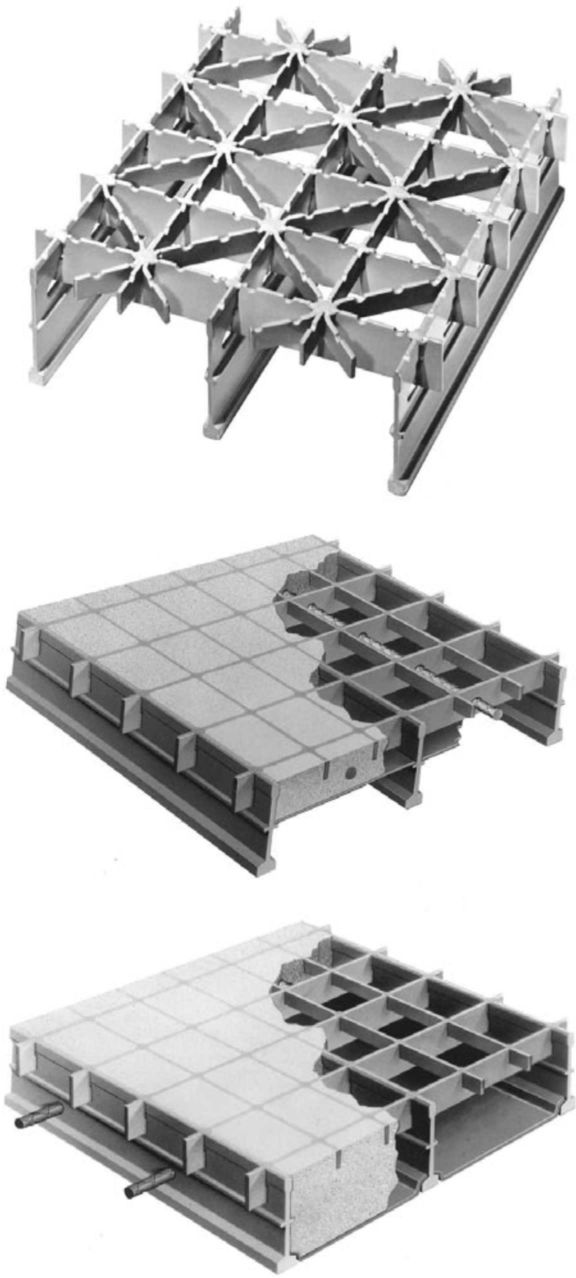


FIGURE 6.1 Steel-grid bridge deck. Top photo shows open steel grid deck; center photo shows half-filled steel grid deck; bottom photo shows filled steel grid deck. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

steel grid deck replacement alone only slightly improves the live-load capacity; however, the capacity can be further improved by providing composite action between the deck and stringers.

Steel grid panels that are filled or half-filled with concrete may either be precast prior to erection or filled with concrete after placement. With the precast system, only the grids that have been left open to allow field welding of the panels must be filled with concrete after installation. The precast system is generally used when erection time must be minimized.

A problem that has been associated with concrete-filled steel grid decks, addressed in a study by Timmer [15], is the phenomenon referred to as deck growth — the increase in length of the filled grid deck caused by the rusting of the steel I-bar webs. The increase in thickness of the webs due to rusting results in comprehensive stresses in the concrete fill. Timmer noted that in the early stages of deck growth, a point is reached when the compression of the concrete fill closes voids and capillaries in the concrete. Because of this action, the amount of moisture that reaches the resting surfaces is reduced and deck growth is often slowed down or even halted. If, however, the deck growth continues beyond this stage, it can lead to breakup of the concrete fill, damage to the steel grid deck, and possibly even damage to the bridge superstructure and substructure. Timmer's findings indicate that the condition of decks that had been covered with some type of wearing surface was superior to those that had been left unsurfaced. A wearing surface is also recommended to prevent wearing and eventual cupping of the concrete between the grids.

Exodermic Deck

Exodermic deck is a recently developed, prefabricated modular deck system that has been marketed by major steel grid deck manufacturers. The first application of Exodermic deck was in 1984 on the Driscoll Bridge located in New Jersey [16]. As shown in Figure 6.2, the bridge deck system consists of a thin upper layer, 3 in. (76 mm) minimum, of prefabricated concrete joined to a lower layer of steel grating. The deck weighs from 40 to 60 psf (1.92 to 2.87 kPa) and is capable of spanning up to 16 ft (4.88 m).

Exodermic decks have not exhibited the fatigue problems associated with open-grid decks or the growth problems associated with concrete-filled grid decks. As can be seen in Figure 6.2, there is no concrete fill and thus no grid corrosion forces. This fact, coupled with the location of the neutral axis, minimizes the stress at the top surface of the grid.

Exodermic deck and half-filled steel grid deck have the highest percent increase in live-load capacity among the lightweight deck types with a concrete surface. As a prefabricated modular deck system, Exodermic deck can be quickly installed. Because the panels are fabricated in a controlled environment, quality control is easier to maintain and panel fabrication is independent of the weather or season.

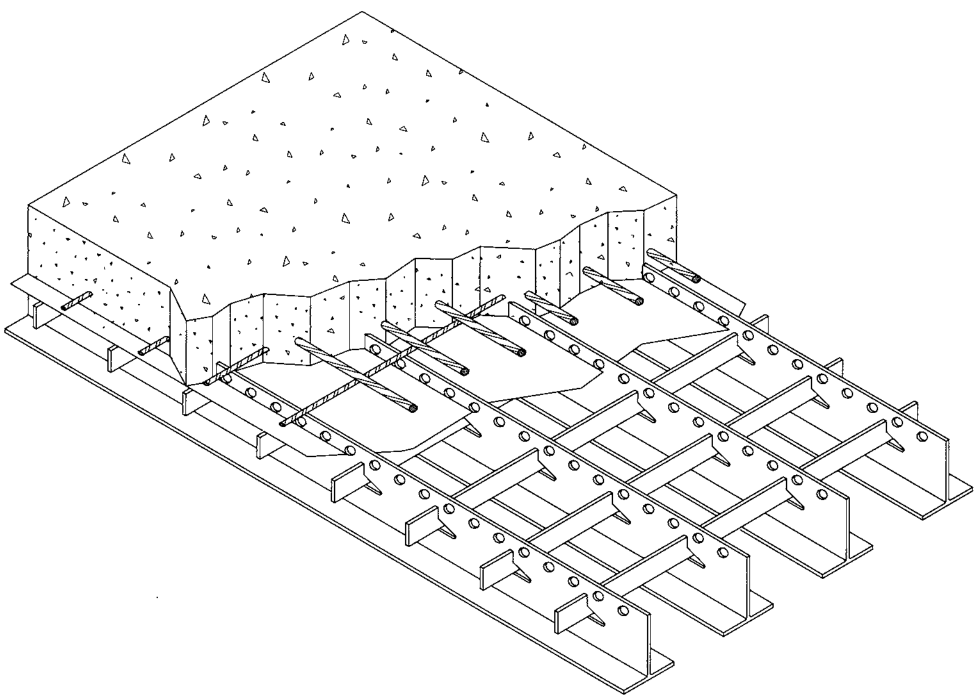


FIGURE 6.2 Exodermic deck system. (*Source:* Exodermic Bridge Deck Inc., Lakeville, CT, 1999. With permission.)

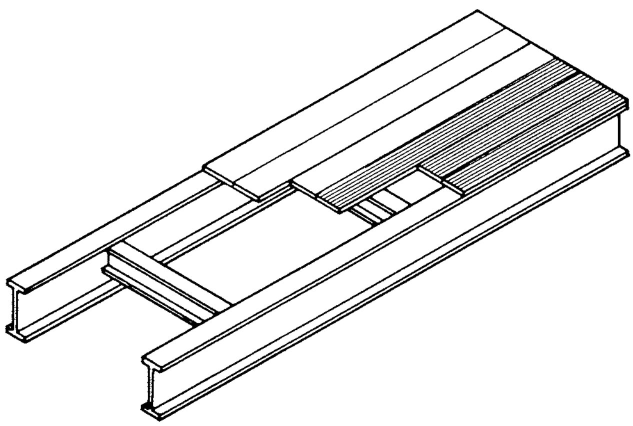
Laminated Timber Deck

Laminated timber decks consist of vertically laminated 2-in. (51-mm) (nominal) dimension lumber. The laminates are bonded together with a structural adhesive to form panels that are approximately 48 in. (1.22 m) wide. The panels are typically oriented transverse to the supporting structure of the bridge (Figure 6.3). In the field, adjacent panels are secured to each other with steel dowels or stiffener beams to allow for load transfer and to provide continuity between the panels.

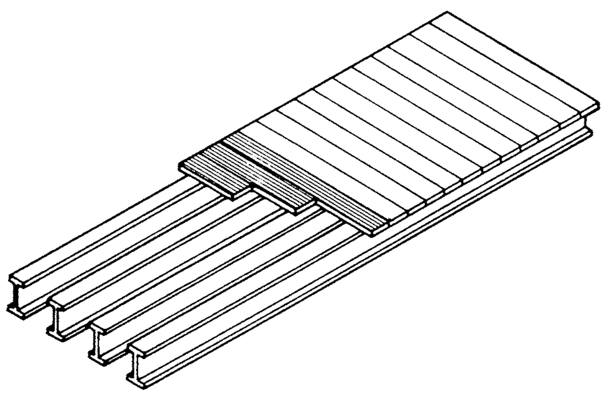
A steel–wood composite deck for longitudinally oriented laminates has been developed by Bakht and Tharmabala [17]. Individual laminates are transversely post-tensioned in the manner developed by Csagoly and Taylor [18]. The use of shear connectors provides partial composite action between the deck and stringers. Because the deck is placed longitudinally, diaphragms mounted flush with the stringers may be required for support. Design of this type of timber deck is presented in References [19–21].

The laminated timber decks used for lightweight deck replacement typically range in depth from 3⅛ to 6¾ in. (79 to 171 mm) and from 10.4 to 22.5 psf (500 to 1075 Pa) in weight. A bituminous wearing surface is recommended.

Wood is a replenishable resource that offers several advantages: ease of fabrication and erection, high strength-to-weight ratio, and immunity to deicing chemicals. With the proper treatment, heavy timber members also have excellent thermal insulation and fire resistance [22]. The most common problem associated with wood as a structural material is its susceptibility to decay caused by living fungi, wood-



(a)



(b)

FIGURE 6.3 Laminated timber deck. (a) Longitudinal orientation; (b) transverse orientation. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

boring insects, and marine organisms. With the use of modern preservative pressure treatments, however, the expected service life of timber decks can be extended to 50 years or more.

Lightweight Concrete Deck

Structural lightweight concrete, concrete with a unit weight of 115 pcf (1840 kg/m^3) or less, can be used to strengthen steel bridges that have normal-weight, noncomposite concrete decks. Special design considerations are necessary for lightweight concrete. Its modulus of elasticity and shear strength are less than that of normal-weight concrete, whereas its creep effects are greater [23]. The durability of lightweight concrete has been a problem in some applications.

Lightweight concrete for deck replacement can be either cast in place or installed in the form of precast panels. A cast-in-place lightweight concrete deck can easily be made to act compositely with the stringers. The main disadvantage of a cast-in-place concrete deck is the length of time required for concrete placement and curing.

Lightweight precast panels, fabricated with either mild steel reinforcement or transverse prestressing, have been used in deck replacement projects to help minimize erection time and resulting interruptions to traffic. Precast panels require careful installation to prevent water leakage and cracking at the panel joints. Composite action can be attained between the deck and the superstructure; however, some designers have chosen not to rely on composite action when designing a precast deck system.

Aluminum Orthotropic Plate Deck

Aluminum orthotropic deck is a structurally strong, lightweight deck weighing from 20 to 25 psf (958 to 1197 Pa). A proprietary aluminum orthotropic deck system that is currently being marketed is shown in Figure 6.4. The deck is fabricated from highly corrosion-resistant aluminum alloy plates and extrusions that are shop-coated with a durable, skid-resistant, polymer wearing surface. Panel attachments between the deck and stringer must not only resist the upward forces on the panels, but also allow for the differing thermal movements of the aluminum and steel superstructure. For design purposes, the manufacturer's recommended connection should not be considered to provide composite action.

The aluminum orthotropic plate is comparable in weight to the open-grid steel deck. The aluminum system, however, eliminates some of the disadvantages associated with open grids: poor ridability and acoustics, weld failures, and corrosion caused by through drainage. A wheel-load distribution factor has not been developed for the aluminum orthotropic plate deck at this time. Finite-element analysis has been used by the manufacturer to design the deck on a project-by-project basis.

Steel Orthotropic Plate Deck

Steel orthotropic plate decks are an alternative for lightweight deck replacements that generally have been designed on a case-by-case basis, without a high degree of standardization. The decks often serve several functions in addition to carrying and distributing vertical live loads and, therefore, a simple reinforced concrete vs. steel orthotropic deck weight comparison could be misleading.

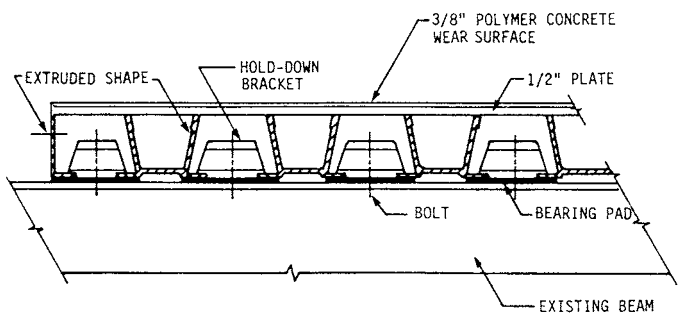


FIGURE 6.4 Aluminum orthotropic deck. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

Originally, steel orthotropic plate decks were developed to minimize steel use in 200- to 300-ft (61- to 91-m) span girder bridges. Then the decks were used in longer-span suspension and cable-stayed bridges where the deck weight is a significant part of the total superstructure design load. Although the steel orthotropic deck is applicable for spans as short as 80 to 120 ft (24.4 to 36.6 m), it is unlikely that there would be sufficient weight savings at those spans to make it economical to replace a reinforced concrete deck with a steel orthotropic plate deck. Orthotropic steel decks are heavier than aluminum orthotropic decks and usually have weights in the 45 to 130 psf (2.15 to 6.22 kPa) range.

6.2.3 Case Studies

Steel Grid Deck

The West Virginia Department of Highways was one of the first to develop a statewide bridge rehabilitation plan using open-grid steel deck [24]. By 1974, 25 bridges had been renovated to meet or exceed AASHTO requirements. Deteriorated concrete decks were replaced with lightweight, honeycombed steel grid decks fabricated from ASTM A588 steel. The new bridge floors are expected to have a 50-year life and to require minimal maintenance.

In 1981, the West Virginia Department of Highways increased the live-load limit on a 1794-ft (546.8-m) long bridge over the Ohio River from 3 tons (26.69 kN) to 13 tons (115.65 kN) by replacing the existing reinforced concrete deck with an open steel grid deck [25, 26]. The existing deck was removed and the new deck installed in sections allowing half of the bridge to be left open for use by workers, construction vehicles, and equipment, and, if needed, emergency vehicles.

The strengthening of the 250-ft (76.2-m) long Old York Road Bridge in New Jersey in the early 1980s combined deck replacement with the replacement of all of the main framing members and the modernization of the piers and abutments [27]. The existing deck was replaced with an ASTM A588 open-grid steel deck. The posted 10-ton (89-kN) load limit was increased to 36 tons (320 kN) and the bridge was widened from 18 ft (5.49 m) to 26 ft (7.92 m).

Exodermic Deck

The first installation of exodermic deck was in 1984 on the 4400-ft (1340-m) long Driscoll Bridge located in New Jersey [16]. The deck, weighing 53 psf (2.54 kPa), consisted of a 3-in. (76-mm) upper layer of prefabricated reinforced concrete joined to a lower layer of steel grating. Approximately 30,000 ft^2 (2790 m^2) of deck was replaced at this site.

Exodermic deck was also specified for the deck replacement on a four-span bridge which overpasses the New York State Thruway [28]. The bridge was closed to traffic during deck removal and replacement. Once the existing deck has been removed, it is estimated that approximately 7500 ft^2 (697 m^2) of exodermic deck will be installed in 3 working days.

Lightweight Concrete Deck

Lightweight concrete was used as early as 1922 for new bridge construction in the United States. Over the years, concrete made with good lightweight aggregate has generally performed satisfactorily; however, some problems related to the durability of the concrete have been experienced. The Louisiana Department of Transportation has experienced several deck failures on bridges built with lightweight concrete in the late 1950s and early 1960s. The deck failures have typically occurred on bridges with high traffic counts and have been characterized by sudden and unexpected collapse of sections of the deck.

Lightweight concrete decks can either be cast in place or factory precast. Examples of the use of lightweight concrete for deck replacement follow.

Cast-in-Place Concrete

New York state authorities used lightweight concrete to replace the deck on the north span of the Newburgh–Beacon Bridge [8, 29]. The existing deck was replaced with 6½ in. (165 mm) of cast-in-place lightweight concrete that was surfaced with a 1½ in. (38 mm) layer of latex modified concrete. Use of the lightweight concrete allowed the bridge to be widened from two to three lanes with minimal modifications to the substructure. A significant reduction in the cost of widening the northbound bridge was attributed to the reduction in dead load.

Precast Concrete Panels

Precast modular-deck construction has been used successfully since 1967 when a joint study, conducted by Purdue University and Indiana State Highway Commission, found precast, prestressed deck elements to be economically and structurally feasible for bridge deck replacement [30, 31].

Precast panels, made of lightweight concrete, 115 pcf (1840 kg/m^3), were used to replace and widen the existing concrete deck on the Woodrow Wilson Bridge, located on Interstate 95 south of Washington, D.C. [32, 33]. The precast panels were transversely prestressed and longitudinally post-tensioned. Special sliding steel-bearing plates were used between the panels and the structural steel to prevent the introduction of unwanted stresses in the superstructure. The Maryland State Highway Commission required that all six lanes of traffic be maintained during the peak traffic hours of the morning and evening. Two-way traffic was maintained at night when the removal and replacement of the deck was accomplished.

Aluminum Orthotropic Plate Deck

The 104-year-old Smithfield Street Bridge in Pittsburgh, Pennsylvania, has undergone two lightweight deck replacements, both involving aluminum deck [34]. The first deck replacement occurred in 1933 when the original heavyweight deck was replaced with an aluminum deck and floor framing system. The aluminum deck was coated with a 1½-in. (38-mm) asphaltic cement wearing surface. The new deck, weighing 30 psf (1.44 kPa), eliminated 751 tons (6680 kN) of deadweight and increased the live-load capacity from 5 tons (44.5 kN) to 20 tons (178 kN).

Excessive corrosion of some of the deck panels and framing members necessitated the replacement of the aluminum deck on the Smithfield Street Bridge in 1967. At that time, a new aluminum orthotropic plate deck with a ⅜-in. (9.5-mm)-thick polymer concrete wearing surface was installed. This new deck weighed 15 psf (718 Pa) and resulted in an additional 108-ton (960-kN) reduction in deadweight. The panels were originally attached to the structure with anodized aluminum bolts, but the bolts were later replaced with galvanized steel bolts after loosening and fracturing of the aluminum bolts became a problem. The aluminum components of the deck have shown no significant corrosion; however, because of excessive wear, the wearing surface had to be replaced in the mid-1970s. The new wearing surface consisted of aluminum-expanded mesh filled with epoxy resin concrete. This wearing surface has also experienced excessive wear, and thus early replacement is anticipated.

Steel Orthotropic Plate Deck

Steel orthotropic plate decks were first conceived in the 1930s for movable bridges and were termed battledecks. Steel orthotropic decks were rapidly developed in the late 1940s in West Germany for replacement of bridges destroyed in World War II during a time when steel was in short supply, and replacement of bridge decks with steel orthotropic plate decks became a means for increasing the live-load capacity of medium- to long-span bridges in West Germany in the 1950s.

In 1956 Woeltinger and Bock [35] reported the rebuilding of a wrought iron, 536-ft (163-m) span bridge near Kiel. The two-hinged, deck arch bridge, which carried both rail and highway traffic, was widened and strengthened through rebuilding essentially all of the bridge except the arches and abutments. The replacement steel orthotropic deck removed approximately 190 tons of dead load from the bridge, improved the deck live-load capacity, and was constructed in such a way as to replace the original lateral wind bracing truss.

The live-load class of a bridge near Darmstadt was raised by means of a replacement steel orthotropic deck also in the mid-1970s [36]. The three-span, steel-through-truss bridge had been repaired and altered twice since World War II, but the deck had finally deteriorated to the point where it required replacement. The existing reinforced concrete deck was then replaced with a steel orthotropic plate deck, and the reduction in weight permitted the bridge to be reclassified for heavier truck loads.

6.3 Composite Action

6.3.1 Introduction

Modification of an existing stringer and deck system to a composite system is a common method of increasing the flexural strength of a bridge. The composite action of the stringer and deck not only reduces the live-load stresses but also reduces undesirable deflections and vibrations as a result of the increase in the flexural stiffness from the stringer and deck acting together. This procedure can also be used on bridges that only have partial composite action, because the shear connectors originally provided are inadequate to support today's live loads.

The composite action is provided through suitable shear connection between the stringers and the roadway deck. Although numerous devices have been used to provide the required horizontal shear resistance, the most common connection used today is the welded stud.

6.3.2 Applicability and Advantages

Inasmuch as the modifications required for providing composite action for continuous spans and simple spans are essentially the same, this section is written for simple spans. Composite action can effectively be developed between steel stringers and various deck materials, such as normal-weight reinforced concrete (precast or cast in place), lightweight reinforced concrete (precast or cast in place), laminated timber, and concrete-filled steel grids. These are the most common materials used in composite decks; however, there are some instances in which steel deck plates have been made composite with steel stringers. In the following paragraphs these four common deck materials will be discussed individually.

Because steel stringers are normally used for support of all the mentioned decks, they are the only type of superstructure reviewed. The condition of the deck determines how one can obtain composite action between the stringers and an existing concrete deck. If the deck is badly deteriorated, composite action is obtained by removing the existing deck, adding appropriate shear connectors to the stringers, and recasting the deck. This was done in Blue Island, Illinois, on the 1500-ft (457-m) long steel plate girder Burr Oak Avenue Viaduct [37].

If it is desired to reduce interruption of traffic, precast concrete panels are one of the better solutions. The panels are made composite by positioning holes formed in the precast concrete directly over the structural steel. Welded studs are then attached through the preformed holes. This procedure was used on an I-80 freeway overpass near Oakland, California [38]. As shown in Figure 6.5, panels 30 ft (9.1 m)

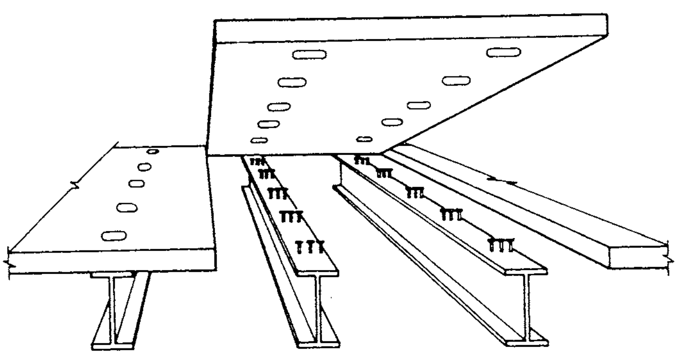


FIGURE 6.5 Precast deck with holes. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

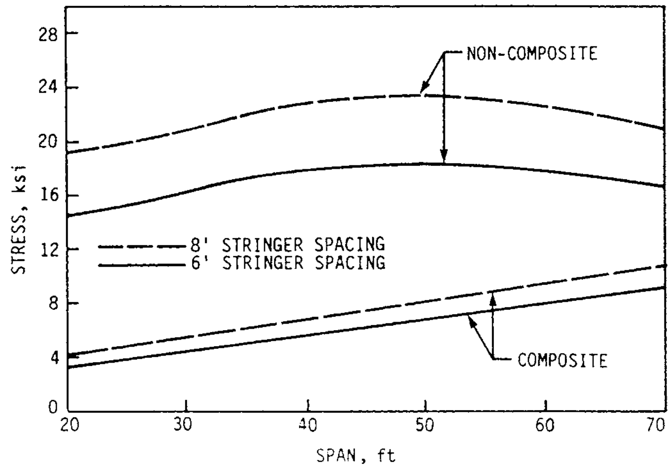


FIGURE 6.6 Stress in top flange of stringer, composite action vs. noncomposite action. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

to 40 ft (12.2 m) long, with oblong holes 12 in. (305 mm) × 4 in. (100 mm) were used to replace the existing deck. Four studs were welded to the girders through each hole. Composite action was obtained by filling the holes, as well as the gaps between the panels and steel stringers, with fast-curing concrete.

If the concrete deck does not need replacing, composite action can be obtained by coring through the existing concrete deck to the steel superstructure. Appropriate shear connectors are placed in the holes; the desired composite action is then obtained by filling the holes with nonshrink grout. This procedure was used in the reconstruction of the Pulaski Skyway near the Holland Tunnel linking New Jersey and New York [38]. After removing an asphalt overlay and some of the old concrete, the previously described procedure with welded studs placed in the holes was used. The holes were then grouted and the bridge resurfaced with latex-modified concrete.

Structural lightweight concrete has been used in both precast panels and in cast-in-place bridge decks. Comments made on normal-weight concrete in the preceding paragraphs essentially apply to lightweight concrete also. However, since the shear strength, fatigue strength, and modulus of elasticity of lightweight concrete are less than that of normal-weight concrete, these lesser values must be taken into account in design.

The advantages of composite action can be seen in Figure 6.6. Shown in this graph is the decrease in the top flange stress as a result of providing composite action on a simply supported single-span bridge with steel stringers and an 8-in. concrete deck. As may be seen in this figure, two stringer spacings, 6 ft (1.8 m) and 8 ft (2.4 m) are held constant, while the span length was varied from 20 ft (6.1 m) to 70 ft (21.3 m). These stresses are based on the maximum moment that results from either the standard truck loading (HS20-44) or the standard lane loading, whichever governs. Concrete stresses were considerably below the allowable stress limit; composite action reduced the stress in the bottom flange 15 to 30% for long and short spans, respectively. As may be seen in Figure 6.6 for a 40-ft (12.2-m) span with 8-ft (2.4-m) stringer spacing, composite action will reduce the stress in the top flange 68%, 22 ksi (152 Mpa) to 7 ksi (48 MPa). Composite action is slightly more beneficial in short spans than in long spans, and the larger the stringer spacings, the more stress reduction when composite action is added. Results for other types of deck are similar but will depend on the type and size of deck, amount of composite action obtained, type of support system, and the like.

6.3.3 Types of Shear Connectors

As previously mentioned, in order to create composite action between the steel stringers and the bridge deck some type of shear connector is required. In the past, several different types of shear connectors were used in the field; these connectors can be seen in Figure 6.7. Of these, because of the advancements

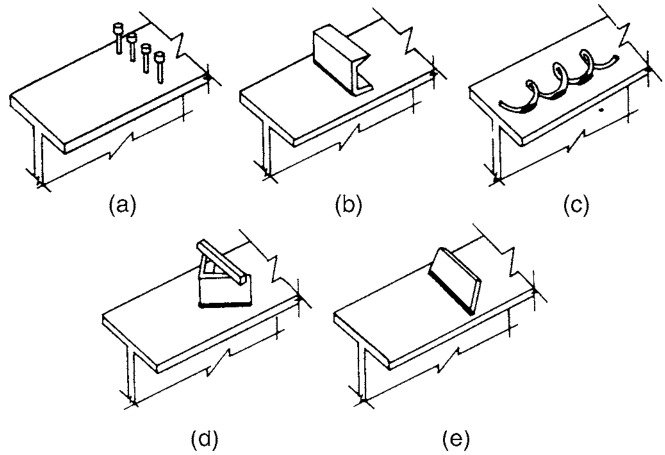


FIGURE 6.7 Common shear connectors. (a) Welded studs; (b) channel; (c) spiral; (d) stiffened angle; (e) inclined flat bar. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

and ease in application, welded studs have become the most commonly used shear connector today. In the strengthening of an existing bridge, frequently one of the older types of shear connectors will be encountered. A strength evaluation must be undertaken to ensure that the shear connectors present are adequate. The following references can be used to obtain the ultimate strength of various types of shear connectors. A method for calculating the strength of a flat bar can be found in Cook [39]; also, work done by Klaiber et al. [40] can be used in evaluating the strength of stiffened angles. Older AASHTO standard specifications can be used to obtain ultimate strength of shear connectors; for example, values for spirals can be found in the AASHTO standard specifications from 1957 to 1968. The current AASHTO specifications only give ultimate-strength equations for welded studs and channels; thus, if shear connectors other than these two are encountered, the previously mentioned references should be consulted.

The procedure employed for using high-strength bolts as shear connectors (Figure 6.8) is very similar to that used for utilizing welded studs in existing concrete, except for the required holes in the steel stringer. To minimize slip, the hole in the steel stringer is made the same size as the diameter of the bolt. Dedic and Klaiber [41] and Dallam [42, 43] have shown that the strength and stiffness of high-strength bolts are essentially the same as those of welded shear studs. Thus, existing AASHTO ultimate-strength formulas for welded stud connectors can be used to estimate the ultimate capacity of high-strength bolts.

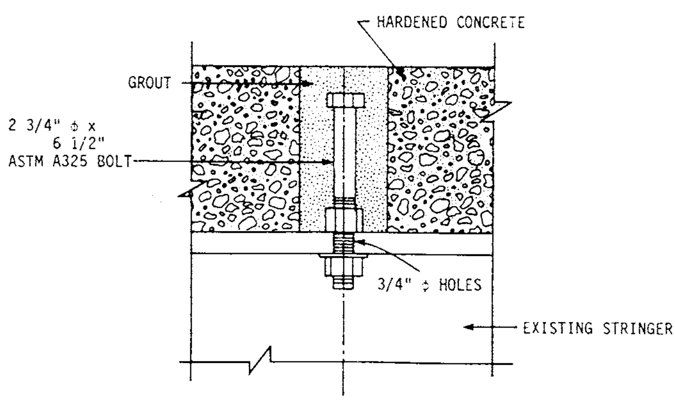


FIGURE 6.8 Details of double-nutted high-strength bolt shear connector. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

6.3.4 Design Considerations

The means of obtaining composite action will depend on the individual bridge deck. If the deck is in poor condition and needs to be replaced, the following variables should be considered: (1) weldability of steel stringers, (2) type of shear connector, and (3) precast vs. cast in place.

To determine the weldability of the shear connector, the type of steel in the stringers must be known. If the type of steel is unknown, coupons may be taken from the stringers to determine their weldability. If it is found that welding is not possible, essentially the only alternative for shear connection is high-strength bolts. Although the procedure is rarely done, bolts could be used to attach channels to the stringers for shear connection. When welding is feasible, either welded studs or channels can be used. Because of the ease of application of the welded studs, channels are rarely used today. In older constructions where steel cover plates were riveted to the beam flanges, an option that may be available is to remove the rivets connecting the top cover plate to the top flange of the beam and replace the rivets with high-strength bolts in a manner similar to that which is shown in Figure 6.8.

According to the current AASHTO manual, *Standard Specifications for Highway Bridges*, in new bridges shear connectors should be designed for fatigue and checked for ultimate strength. However, in older bridges, the remaining fatigue life of the bridge will be considerably less than that of the new shear connectors; thus, one only needs to design the new shear connectors for ultimate strength. If an existing bridge with composite action requires additional shear connectors, the ultimate strength capacity of the original shear connector (connector #1) and new shear connectors (connectors #2) can be simply added even though they are different types of connectors. Variation in the stiffness of the new shear connectors and original shear connectors will have essentially no effect on the elastic behavior of the bridge and nominal effect on the ultimate strength [44].

The most common method of creating composite action when one works with precast concrete decks is to preform slots in the individual panels. These slots are then aligned with the stringers for later placement of shear connectors (see Figure 6.5). Once shear connectors are in place, the holes are filled with nonshrink concrete. A similar procedure can be used with laminated timber except the holes for the shear connectors are drilled after the panels are placed.

When it is necessary to strengthen a continuous span, composite action can still be employed. One common approach is for the positive moment region to be designed using the same procedure as that for simple span bridges. When designing the negative moment region, the engineer has two alternatives. The engineer can continue the shear connectors over the negative moment region, in which case the longitudinal steel can be used in computing section properties in the negative moment region. The other alternative is to discontinue the shear connectors over the negative moment region. As long as the additional anchorage connectors in the region of the point of dead-load contraflexure are provided, as required by the code, continuous shear connectors are not needed. When this second alternative is used, the engineer cannot use the longitudinal steel in computing the section properties in the negative moment region. If shear connectors are continued over the negative moment region, one should check to be sure that the longitudinal steel is not overstressed. Designers should consult the pertinent AASHTO standards to meet current design guidelines.

6.4 Improving the Strength of Various Bridge Members

6.4.1 Addition of Steel Cover Plates

Steel Stringer Bridges

Description

One of the most common procedures used to strengthen existing bridges is the addition of steel cover plates to existing members. Steel cover plates, angles, or other sections may be attached to the beams by means of bolts or welds. The additional steel is normally attached to the flanges of existing sections as a means of increasing the section modulus, thereby increasing the flexural capacity of the member. In most cases the member is jacked up during the strengthening process, relieving dead-load stresses on the existing member. The new cover plate section is then able to accept both live-load and dead-load stresses when the jacks are removed, which ensures that less steel will be required in the cover plates. If the bridge is not jacked up, the cover plate will carry only live-load stresses, and more steel will be required.

Applicability, Advantages, and Disadvantages

The techniques described in this section are widely applicable to steel members whose flexural capacity is inadequate. Members in this category include steel stringers (both composite and noncomposite), floor beams, and girders on simply supported or continuous bridges. Note, however, that cover plating is most effective on composite members.

There are a number of advantages to using steel cover plates as a method of strengthening existing bridges. This method can be quickly installed and requires little special equipment and minimal labor and materials. If bottom flange stresses control the design, cover plating is effective even if the deck is not replaced. In this case, it is more effective when applied to noncomposite construction. In addition, design procedures are straightforward and thus require minimal time to complete.

In certain instances these advantages may be offset by the costly problems of traffic control and jacking of the bridge. As a minimum, the bridge may have to be closed or separate traffic lanes established to relieve any stresses on the bridge during strengthening. In addition, significant problems may develop if part of the slab must be removed in order to add cover plates to the top of the beams. When cover plates are attached to the bottom flange, the plates should be checked for underclearance if the situation requires it. Still another potential problem if welding is used is that the existing members may not be compatible with current welding materials.

The most commonly reported problem encountered with the addition of steel cover plates is fatigue cracking at the top of the welds at the ends of the cover plates. In a study by Wattar et al. [45], it was suggested that bolting be used at the cover plate ends. Tests showed that bolting the ends raises the fatigue category of the member from stress Category E to B and also results in material savings by allowing the plates to be cut off at the theoretical cutoff points.

Another method for strengthening this detail is to grind the transverse weld to a 1:3 taper [46]. This is a practice of the Maryland State Highway Department. Using an air hammer to peen the toe of the weld and introduce compressive residual stresses is also effective in strengthening the connection [46]. The fatigue strength can be improved from stress category E to D by using this technique. Either solution has been shown to reduce significantly the problem of fatigue cracking at the cover plate ends.

Materials other than flange cover plates may be added to stringer flanges for strengthening. For example, the Iowa Department of Transportation prefers to attach angles to the webs of steel I-beam bridges (either simply supported or continuous spans) with high-strength bolts as a means to reduce flexural live-load stresses in the beams. Figure 6.9 shows a project completed by the Iowa Department of Transportation involving the addition of angles to steel I-beams using high-strength bolts. In some instances the angles are attached only near the bottom flange. Normally, the bridge is not jacked up during strengthening, and only the live loads are removed from the particular I-beam being strengthened. Because the angles are bolted on, problems of fatigue cracking that could occur with welding are eliminated. This method does have one potential problem, however: the possibility of having to remove part of a web stiffener should one be crossed by an angle.

Another method of adding material to existing members for strengthening is shown in Figure 6.10 where structural Ts were bolted to the bottom flanges of the existing stringers using structural angles. This idea represented a design alternative recommended by Howard, Needles, Tammen and Bergendoff as one method of strengthening a bridge comprising three 50-ft (15.2-m) simple spans. Each of the four stringers per span was strengthened in a similar manner.

Design Procedure

The basic design steps required in the design of steel cover plates follow:

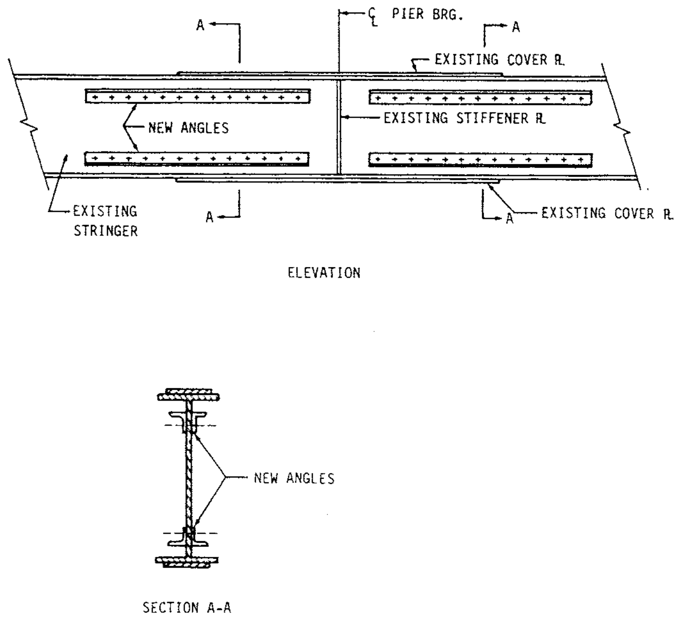


FIGURE 6.9 Iowa DOT method of adding angles to steel I-beams. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

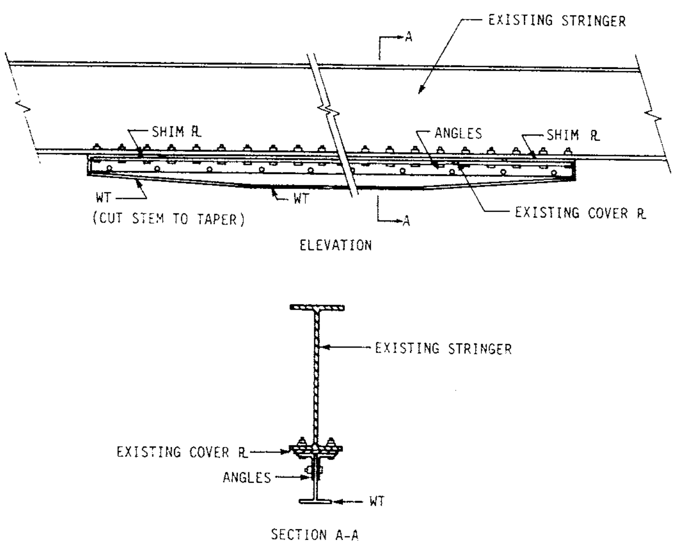


FIGURE 6.10 Strengthening of existing steel stringer by addition of structural T section. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

1. Determine moment and shear envelopes for desired live-load capacity of each beam.
2. Determine the section modulus required for each beam.
3. Determine the optimal amount of steel to achieve desired section modulus–strength requirement, fatigue requirement.
4. Design connection of cover plates to beam strength requirement, fatigue requirement.
5. Determine safe cutoff point for cover plates.

In addition to the foregoing design steps, the following construction considerations may prove helpful:

1. Grinding the transverse weld to a 1:3 taper or bolting the ends of plates rather than welding reduces fatigue cracking at the cover plate ends [46, 47].
2. In most cases a substantial savings in steel can be made if the bridge is jacked to relieve dead-load stresses prior to adding cover plates.
3. The welding of a cover plate should be completed within a working day. This minimizes the possibility of placing a continuous weld at different temperatures and inducing stress concentrations.
4. Shot blasting of existing beams to clean welding surface may be necessary.

Reinforced Concrete Bridges

Description
One method of increasing flexural capacity of a reinforced concrete beam is to attach steel cover plates or other steel shapes to the tension face of the beam. The plates or shapes are normally attached by bolting, keying, or doweling to develop continuity between the old beam and the new material. If the beam is also inadequate in shear, combinations of straps and cover plates may be added to improve both shear and flexural capacity. Because a large percentage of the load in most concrete structures is dead load, for cover plating to be most effective, the structure should be jacked prior to cover plating to reduce the dead-load stresses of the member. The addition of steel cover plates may also require the addition of concrete to the compression face of the member.

Applicability
A successful method of strengthening reinforced concrete beams has involved the attachment of a steel channel to the stem of a beam. This technique is shown in Figure 6.11. Taylor [48] performed tests on a section using steel channels and found it to be an effective method of strengthening. An advantage to this method is that rolled channels are available in a variety of sizes, require little additional preparation prior to attachment, and provide a ready formwork for the addition of grouting. The channels can also be easily reinforced with welded cover plates if additional strength is required. Prefabricated channels

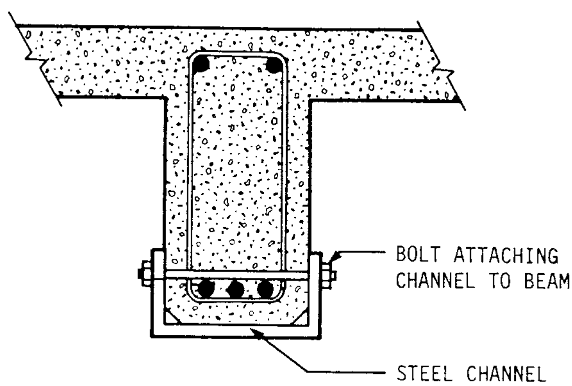


FIGURE 6.11 Addition of a steel channel to an existing reinforced concrete beam. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

are an effective substitute when rolled sections of the required size are not available. It should be noted that the bolts are placed above the longitudinal steel so that the stirrups can carry shear forces transmitted by the channels. If additional sheer capacity is required, external stirrups should also be installed. It is also recommended that an epoxy resin grout be used between the bolts and concrete. The epoxy resin grout provides greater penetration in the bolt holes, thereby reducing slippage and improving the strength of the composite action.

Bolting steel plates to the bottom and sides of beam sections has also been performed successfully, as documented by Warner [49]. Bolting may be an expensive and time-consuming method, because holes usually have to be drilled through the old concrete. Bolting is effective, however, in providing composite action between the old and new material.

The placement of longitudinal reinforcement in combination with a concrete sleeve or concrete cover is another method for increasing the flexural capacity of the member. This method is shown in Figures 6.12a and b as outlined in an article on strengthening by Westerberg [50]. Warner [49] presents a similar method that is shown in Figure 6.12c.

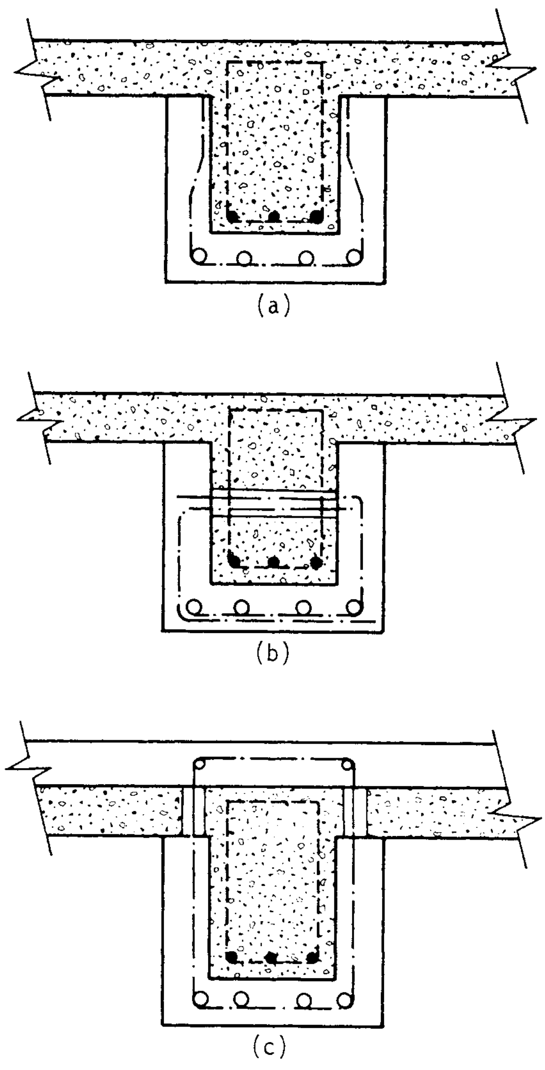


FIGURE 6.12 Techniques for increasing the flexural capacity of reinforced concrete beams with reinforced concrete sleeves. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

Developing a bond between the old and new material is critical to developing full continuity. Careful cleaning and preparation of the old concrete and the application of a suitable epoxy-resin primer prior to adding new concrete should provide adequate bonding. Stirrups should also be added to provide additional shear reinforcement and to support the added longitudinal bars.

Design and Analysis Procedure

The design of steel cover plates for concrete members is dependent on the amount of continuity assumed to exist between the old and new material. If one assumes that full continuity can be achieved and that strains vary linearly throughout the depth of the beam, calculations are basically straightforward. As stated earlier, much of the load in concrete structures is dead load, and jacking of the deck during cover plating will greatly reduce the amount of new steel required. It should also be pointed out that additional steel could lead to an overreinforced section. This could be compensated for by additional concrete or reinforcing steel in the compression zone.

Case Studies

Steel cover plates can be used in a variety of situations. They can be used to increase the section modulus of steel, reinforced concrete, and timber beams. Steel cover plates are also an effective method of strengthening compression members in trusses by providing additional cross-sectional area and by reducing the slenderness ratio of the member.

Mancarti [51] reported the use of steel cover plates to strengthen floor beams on the Pit River Bridge and Overhead in California. The truss structure required strengthening of various other components to accommodate increased dead load. Stringers in this bridge were strengthened by applying prestressing tendons near the top flange to reduce tensile stress in the negative moment region. This prestressing caused increased compressive stresses in the bottom flanges, which in turn required the addition of steel bars to the tops of the stringer bottom flange.

In a report by Rodriguez et al. [52], a number of cases of cover-plating existing members of old railway trusses were cited. These case studies included the inspection of 109 bridges and a determination of their safety. Some strengthening techniques included steel-cover-plating beam members as well as truss members. Cover plates used to reinforce existing floor beams on a deficient through-truss were designed to carry all live-load bending moment. Deficient truss members were strengthened with box sections made up of welded plates. The box was placed around the existing member and connected to it by welding.

6.4.2 Shear Reinforcement

External Shear Reinforcement for Concrete, Steel, and Timber Beams

The shear strength of reinforced concrete beams or prestressed concrete beams can be improved with the addition of external steel straps, plates, or stirrups. Steel straps are normally wrapped around the member and can be post-tensioned. Post-tensioning allows the new material to share both dead and live loads equally with the old material, resulting in more efficient use of the material added. A disadvantage of adding steel straps is that cutting the deck to apply the straps leaves them exposed on the deck surface and thus difficult to protect. By contrast, adding steel plates does not require cutting through the deck. The steel plates are normally attached to the beam with bolts or dowels.

External stirrups may also be applied with different configurations. Figure 6.13a shows a method of attaching vertical stirrups using channels at the top and bottom of the beam. The deck (not shown in either figure) provides protection for the upper steel channel [53]. Adding steel sections at the top of the beam web and attaching stirrups is shown in Figure 6.13b. In this manner, cutting holes through the deck is eliminated. External stirrups can also be post-tensioned in most situations if desired.

Another method of increasing shear strength is shown in Figure 6.14. This method is a combination of post-tensioning and the addition of steel in the form of prestressing tendons. As recommended in a strengthening manual by the OECD [54], tendons may be added in a vertical or inclined orientation and may be placed either within the beam web or inside the box as shown in the figure. Care should be taken to avoid overstressing parts of the structure when prestressing. If any cracks exist in the member, it is a

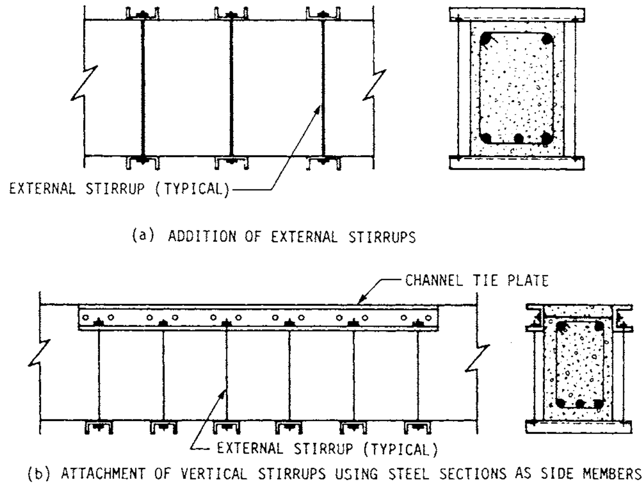


FIGURE 6.13 Methods of adding external shear reinforcement to reinforced concrete beams. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

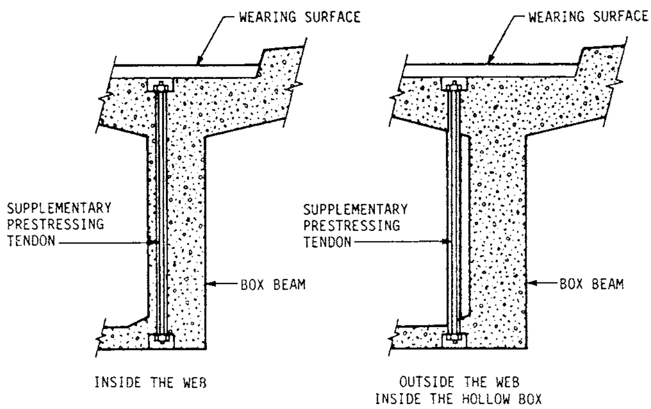


FIGURE 6.14 External shear reinforcement of box beam girders. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

good practice to inject them with an epoxy before applying the prestressing forces. Documentation of this type of reinforcement technique is made also by Suter and Audrey [55] and Dilger and Ghali [56]. Figure 6.15 illustrates the technique used by Dilger and Ghali [56] where web thickening was added to the inside of the box web before adding external reinforcement consisting of stressed steel bars. The thickening was required to reduce calculated tensile stresses at the outside of the web due to prestressing the reinforcement.

West [57] makes reference to a number of methods of attaching steel plates to deficient steel I-beam girder webs as a means of increasing their shear strength. The steel plates are normally of panel size and are attached between stiffeners by bolting or welding. Where shear stresses are high, the plates should fit tightly between the stiffeners and girder flanges. West indicates that one advantage of this method is that it can be applied under traffic conditions.

Timber stringers with inadequate shear capacity can be strengthened by adding steel cover plates. NCHRP Report 222 [11] demonstrates a method of repairing damaged timber stringers with inadequate shear capacity. The procedure involves attaching steel plates to the bottom of the beam in the deficient region and attaching it with draw-up bolts placed on both sides of the beam. Holes are drilled through the top of the deck, and a steel strap is placed at the deck surface and at the connection to the bolts.

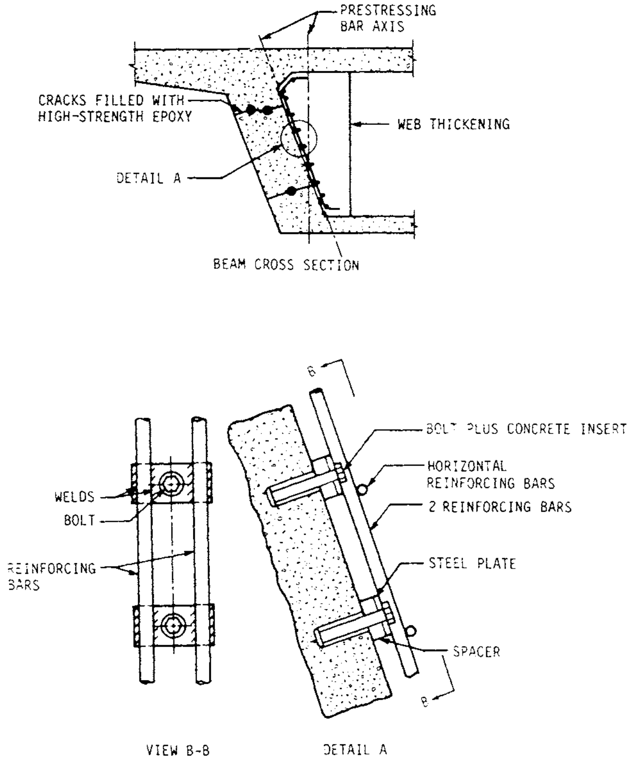


FIGURE 6.15 Details of web reinforcement to strengthen box beam in shear. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

Epoxy Injection and Rebar Insertion

The Kansas Department of Transportation has developed and successfully used a method for repairing reinforced concrete girder bridges. The bridges had developed shear cracks in the main longitudinal girders [58]. The procedure used by the Kansas Department of Transportation not only prevented further shear cracking but also significantly increased the shear strength of the repaired girders.

The method involves locating and sealing all of the girder cracks with silicone rubber, marking the girder centerline on the deck, locating the transverse deck reinforcement, vacuum drilling 45° holes that avoid the deck reinforcement, pumping the holes and cracks full of epoxy, and inserting reinforcing bars into the epoxy-filled holes. A typical detail is shown in Figure 6.16.

An advantage of using the epoxy repair and rebar insertion method is its wide application to a variety of bridges. Although the Kansas Department of Transportation reported using this strengthening method on two-girder, continuous, reinforced concrete bridges, this method can be a practical solution on most types of prestressed concrete beam and reinforced concrete girder bridges that require additional shear strength. The essential equipment requirements needed for this strengthening method may limit its usefulness, however. Prior to drilling, the transverse deck steel must be located. The drilling unit and vacuum pump required must be able to quickly drill straight holes to a controlled depth and keep the holes clean and free of dust.

Addition of External Shear Reinforcement

Strengthening a concrete bridge member that has a deficient shear capacity can be performed by adding external shear reinforcement. The shear reinforcement may consist of steel side plates or steel stirrup reinforcement. This method has been applied on numerous concrete bridge systems.

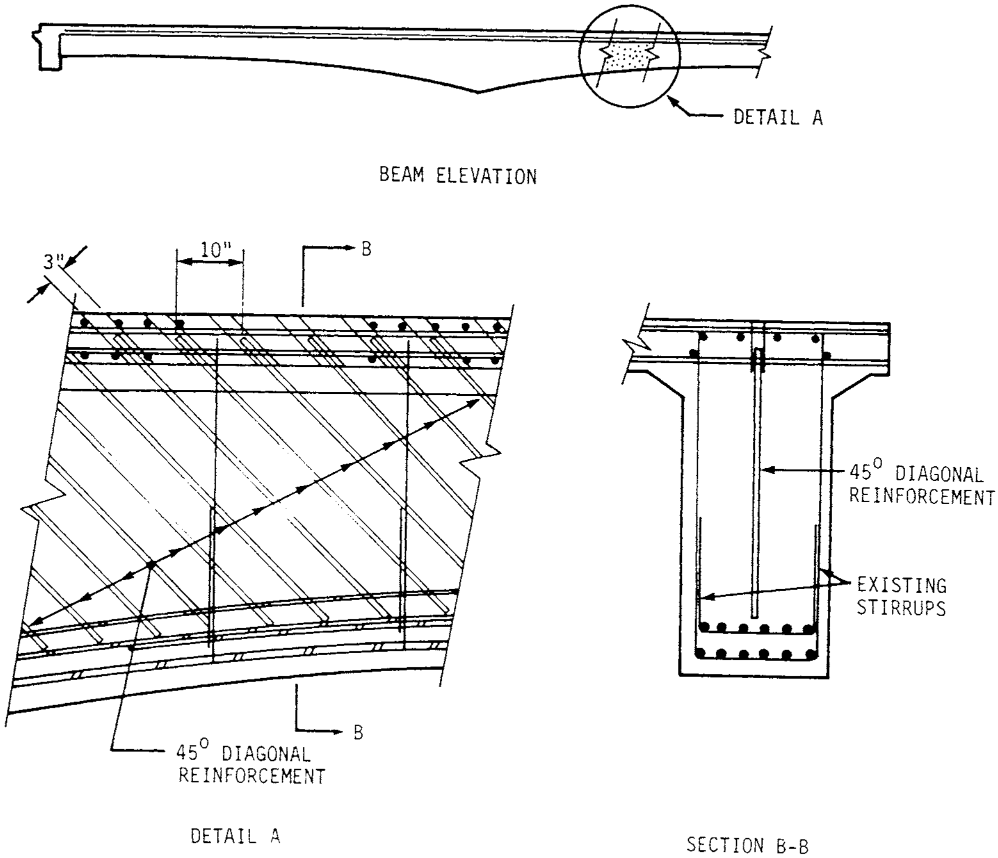


FIGURE 6.16 Kansas DOT shear strengthening procedure. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

A method proposed by Warner [49] involves adding external stirrups. The stirrups consist of steel rods placed on both sides of the beam section and attached to plates at the top and bottom of the section. In some applications, channels are mounted on both sides at the top of the section to attach the stirrups. This eliminates drilling through the deck to make the connection to a plate.

In a study by Dilger and Ghali [56], external shear reinforcement was used to repair webs of prestressed concrete bridges. Although the measures used were intended to bring the deficient members to their original flexural capacity, the techniques applied could be used for increasing the shear strength of existing members. Continuous box girders in the 827-ft (252-m) long bridges had become severely cracked when prestressed. The interior box beam webs were strengthened by the addition of 1-in. (25-mm) diameter steel rods placed on both sides of the web. Holes were drilled in the upper and lower slabs as close as possible to the web to minimize local bending stresses in the slabs. Post-tensioning tendons were placed through the holes, stressed, and then anchored.

The slanted outside webs were strengthened with reinforcing steel. Before the bars were added, the inside of the web was "thickened" and the reinforcement was attached with anchor bolts placed through steel plates that were welded to the reinforcement. The web thickening was necessary because the prestressing would have produced substantial tensile stresses at the outside face of the web.

6.4.3 Jacketing of Timber or Concrete Piles and Pier Columns

Improving the strength of timber or concrete piles and pier columns can be achieved by encasing the column in concrete or steel jackets. The jacketing may be applied to the full length of the column or

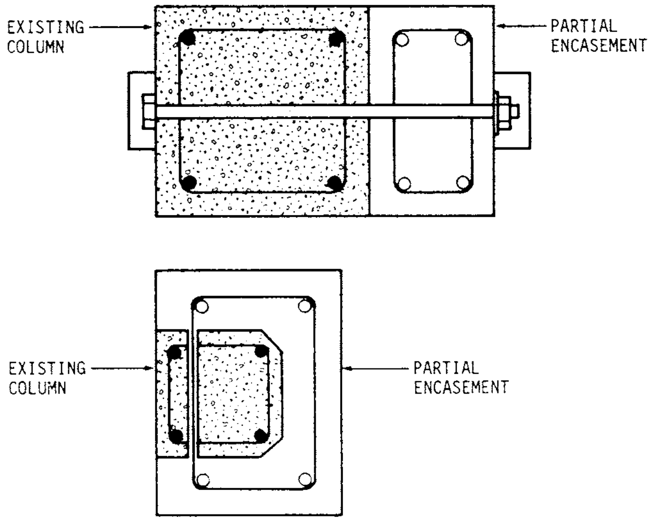


FIGURE 6.17 Partial jacketing of an existing column. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

only to severely deteriorated sections. The jacketing increases the cross-sectional area of the column and reduces the slenderness ratio of the column. Partial encasement of a column can also be particularly effective when an unbalanced moment acts on the column. Figure 6.17 illustrates two such concepts for member addition that were noted from work on strengthening reinforced concrete structures in Europe [50].

Completely encasing the existing column in a concrete jacket has been a frequently used method of strengthening concrete pier columns. Normally, the reinforcement is placed around the existing column perimeter inside the jacket and "ramset" to the existing member [50]. The difficulty most often observed with this technique is developing continuity between the old and new material. This is critical if part of the load is to be transferred to the new material. Work by Soliman [59] on repair of reinforced concrete columns by jacketing has included an experimental investigation of the bond stresses between the column and jacket. The first step is normally surface preparation of the existing concrete column. Consideration should also be given at this time to jacking of the superstructure and placing temporary supports on either side of the column. Soliman [59] concludes that this is an important step, since the shrinkage phenomenon causes compressive stresses on the column that will be reduced if the existing column is unloaded. In addition, supports will be necessary if the column shows significant signs of deterioration. This procedure will also allow the new material to share equally both dead and live loads after the supports are removed. Additional longitudinal reinforcing bars and stirrups are then placed around the column. Spiral stirrup reinforcement should be used because it will provide greater strength and ductility than normal stirrups [59]. An epoxy resin is then applied to the old concrete to increase the bonding action between the old concrete and the concrete to be added. Formwork is then erected to form the jacket, and concrete is placed and compacted.

Jacketing techniques have been used extensively for seismic retrofitting of existing pier columns. A recent report by Wipf et al. [60] provides an extensive list and discussion of various retrofit methods for reinforced concrete bridge columns, including the use of steel jackets and fiber-reinforced polymer wraps.

Modification Jacketing

Increasing the load-carrying capacity of bridge pier columns or timber piles supporting bent caps is normally achieved through the addition of material to the existing cross section. Jacketing or adding a sleeve around the column perimeter can be performed a number of ways.

In a paper by Karamchandani [61], various concepts for jacketing existing members are illustrated. These include addition of reinforcement and concrete around three sides of rectangular beams as well as placement only at the bottom of the beam web. Additional schemes are also illustrated for column members. The effectiveness of this method depends on the degree of adhesion between new and existing concrete, which can vary between 30 and 80% of the total strength of the *in situ* concrete. The author suggests welding new reinforcing to the existing reinforcement and using concrete with a slump of 3 to 4 in. (75 to 100 mm). The use of rapid hardening cements is not recommended, since it results in a lower strength of concrete on the contact surface because of high contraction stresses.

The addition of concrete collars on reinforced concrete columns is performed most efficiently by using circular reinforcement rather than dowels or shear keys according to Klein and Gouwens [62]. While the other methods may require costly and time-consuming drilling and/or cutting, circular reinforcement does not. When this method is used, shear-friction is the primary load-transfer mechanism between the collar and the existing column. Klein and Gouwens have outlined a design procedure for this strengthening method.

In a paper by Syrmakezis and Voyatzis [63], an analytical method for calculating the stiffness coefficients of columns strengthened by jacketing is presented. The procedure uses compatibility conditions for the deformations of the strengthened system and the analysis can consider rigid connections between the jacket and column on a condition where relative slip is allowed.

6.5 Post-Tensioning Various Bridge Components

6.5.1 Introduction

Since the 19th century, timber structures have been strengthened by means of king post and queen post-tendon arrangements; these forms of strengthening by post-tensioning are still used today. Since the 1950s, post-tensioning has been applied as a strengthening method in many more configurations to almost all common bridge types. The impetus for the recent surge in post-tensioning strengthening is undoubtedly a result of its successful history of more than 40 years and the current need for strengthening of bridges in many countries.

Post-tensioning can be applied to an existing bridge to meet a variety of objectives. It can be used to relieve tension overstresses with respect to service load and fatigue-allowable stresses. These overstresses may be axial tension in truss members or tension associated with flexure, shear, or torsion in bridge stringers, beams, or girders.

Post-tensioning also can reduce or reverse undesirable displacements. These displacements may be local, as in the case of cracking, or global, as in the case of excessive bridge deflections. Although post-tensioning is generally not as effective with respect to ultimate strength as with respect to service-load-allowable stresses, it can be used to add ultimate strength to an existing bridge. It is possible to use post-tensioning to change the basic behavior of a bridge from a series of simple spans to continuous spans. All of these objectives have been fulfilled by post-tensioning existing bridges, as documented in the engineering literature.

Most often, post-tensioning has been applied with the objective of controlling longitudinal tension stresses in bridge members under service-loading conditions. Figure 6.18 illustrates the axial forces, shear forces, and bending moments that can be achieved with several simple tendon configurations. The concentric tendon in Figure 6.18a will induce an axial compression force that, depending on magnitude, can eliminate part or all of an existing tension force in a member or even place a residual compression force sufficient to counteract a tension force under other loading conditions. The amount of post-tensioning force that can safely be applied, of course, is limited by the residual-tension dead-load force in the member.

The tendon configuration in Figure 6.18a is generally used only for tension members in trusses, whereas the remaining tendon configurations in Figure 6.18 would be used for stringers, beams, and girders. The eccentric tendon in Figure 6.18b induces both axial compression and negative bending. The eccentricity

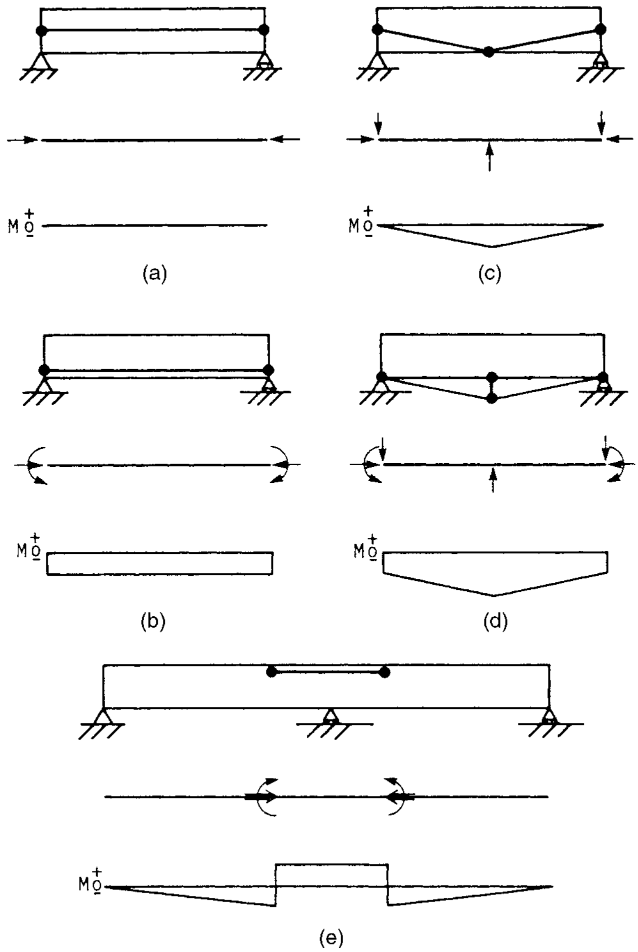


FIGURE 6.18 Forces and moment induced by longitudinal post-tensioning. (a) Concentric tendon; (b) eccentric tendon; (c) polygonal tendon; (d) king post; (e) eccentric tendon, two-span member. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

of the tendon may be varied to control the proportions of axial compression vs. bending applied to the member. Length of the tendon also may be varied to apply post-tensioning only to the most highly stressed portion of the member. The polygonal tendon profile in Figure 6.18c also induces axial compression and negative bending, but the negative bending is nonuniform within the post-tensioned region. Locations of bends on the tendon and eccentricities of the attachments at the bends can be set to control the moments caused by the post-tensioning. The polygonal tendon also induces shear forces that are opposite to those applied by live and dead loads.

The king post tendon configuration in Figure 6.18d is a combination of the eccentric and polygonal tendon configurations. Because the post is beyond the profile of the original member, the proportion of moment to axial force induced in the member to be strengthened will be large.

The tendon configuration in Figure 6.18e is an eccentric tendon attached over the central support of a two-span member. In this configuration, the amount of positive moment applied in the central support region depends not only on the force in the tendon and its eccentricity, but also on the locations of the anchorages on the two spans. If the anchorages are moved toward the central support, the amount of positive moment applied will be greater than if the anchorages are moved away from the central support. This fact and the fact that there is some distribution of moment and force among parallel post-tensioned members have not always been correctly recognized, and there are published errors in the literature.

The axial force, shear force, and bending moment effects of post-tensioning described above have enough versatility in application so as to meet a wide variety of strengthening requirements. Probably this is the only strengthening method that can actually reverse undesirable behavior in an existing bridge rather than provide a simple patching effect. For both these reasons, post-tensioning has become a very commonly used repair and strengthening method.

6.5.2 Applicability and Advantages

Post-tensioning has many capabilities: to relieve tension, shear, bending, and torsion-overstress conditions; to reverse undesirable displacements; to add ultimate strength; to change simple span to continuous span behavior. In addition, post-tensioning has some very practical advantages. Traffic interruption is minimal; in some cases, post-tensioning can be applied to a bridge with no traffic interruption. Few site preparations, such as scaffolding, are required. Tendons and anchorages can be prefabricated. Post-tensioning is an efficient use of high strength steel. If tendons are removed at some future date, the bridge will generally be in no worse condition than before strengthening.

To date, post-tensioning has been used to repair or strengthen most common bridge types. Most often, post-tensioning has been applied to steel stringers, floor beams, girders, and trusses, and case histories for strengthening of steel bridges date back to the 1950s. Since the 1960s, external post-tensioning has been applied to reinforced concrete stringer and T bridges. In the past 20 years, external post-tensioning has been added to a variety of prestressed, concrete stringer and box beam bridges. Many West German prestressed concrete bridges have required strengthening by post-tensioning due to construction joint distress. Post-tensioning even has been applied to a reinforced concrete slab bridge by coring the full length of the span for placement of tendons [63].

Known applications of post-tensioning will be idealized and summarized as Schemes A through L in Figures 6.19 through 6.22. Typical schemes for stringers, beams, and girders are contained in Figure 6.19. The simplest and, with the exception of the king post, the oldest scheme is Scheme A: a straight, eccentric tendon shown in Figure 6.19a. Lee [64] reported use of the eccentric tendon for strengthening of British cast iron and steel highway and railway bridges in the early 1950s. Since then, Scheme A has been applied to many bridges in Europe, North America, and other parts of the world. Scheme A is most efficient if the tendon has a length less than that of the member, so that the full post-tensioning negative moment is not applied to regions with small dead-load moments. The variation on Scheme A for continuous spans, Scheme AA in Figure 6.19e, has been reported in use for deflection control or strengthening in Germany [65] and the United States [66] since the late 1970s.

The polygonal tendon, Scheme B in Figure 6.19b and its extension to continuous spans, Scheme BB in Figure 6.19f, has been in use since at least the late 1960s. Vernigora et al. [67] reported the use of Scheme BB for a five-span, reinforced-concrete T-beam bridge in 1969. The bridge over the Welland Canal in Ontario, Canada, was converted from simple-span to continuous-span behavior by means of external post-tensioning cables.

Scheme C in Figure 6.19c provided the necessary strengthening for a steel plate, girder railway bridge in Czechoslovakia in 1964 [68]. The tendons and compression struts for the bridge were fabricated from steel T sections, and the tendons were stressed by deflection at bends rather than by elongation as is the usual case. The tendons for the plate girder bridge were given a three-segment profile to apply upward forces at approximately the third points of the span, so that the existing dead-load moments could be counteracted efficiently. In the late 1970s in the United States, Kandall [69] recommended use of Scheme C for strengthening because it does not place additional axial compression in the existing structure. For other schemes, the additional axial compression induced by post-tensioning will add compressive stress to regions that may be already overstressed in compression.

Scheme D in Figure 6.19d was used in Minnesota in 1975 to strengthen temporarily a steel stringer bridge [70]. It was possible to strengthen that bridge economically with scrap timber and cable for the last few years of its life before it was replaced.

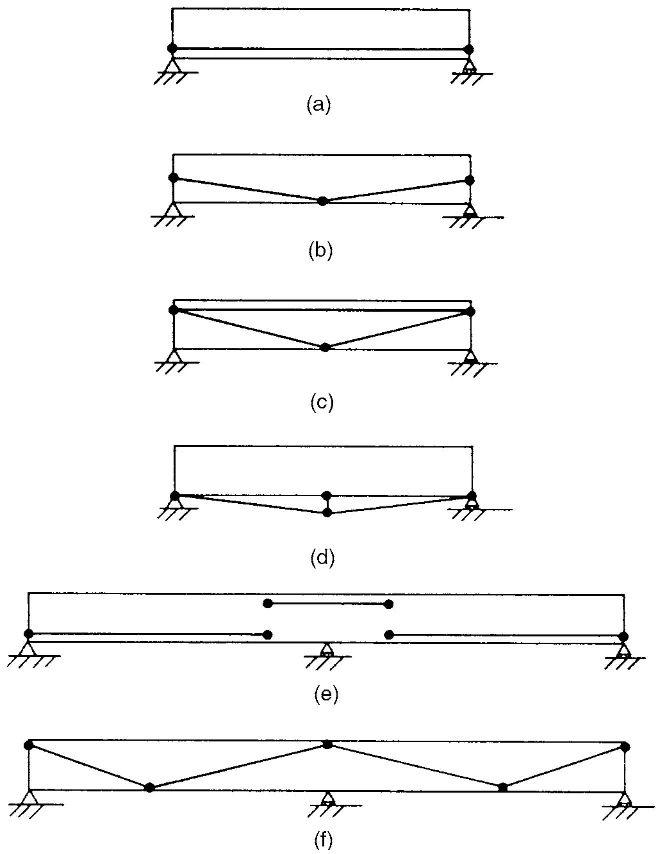


FIGURE 6.19 Tendon configurations for flexural post-tensioning of beams. (a) Scheme A, eccentric tendon; (b) Scheme B, polygonal tenton; (c) Scheme C, polygonal tendon with compression strut; (d) Scheme D, king post; (e) Scheme AA, eccentric tendons; (f) Scheme BB, polygonal tendons. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

The tendon schemes in Figure 6.19, in general, appear to be very similar to reinforcing bar patterns for concrete beams. Thus, it is not surprising that post-tensioning also has been used for shear strengthening, in patterns very much like those for stirrups in reinforced concrete beams. Scheme E in Figure 6.20a illustrates a pattern of external stirrups for a beam in need of shear strengthening. Types of post-tensioned external stirrups have been used or proposed for timber beams [11], reinforced concrete beams and, as illustrated in Figure 6.20b, for prestressed concrete box-girder bridges [71].

Post-tensioning was first applied to steel trusses for purposes of strengthening in the early 1950s [64], at about the same time that it was first applied to steel stringer and steel girder, floor beam bridges. Typical strengthening schemes for trusses are presented in Figure 6.21. Scheme F, concentric tendons on individual members, shown in Figure 6.21a, was first reported for the proposed strengthening of a cambered truss bridge in Czechoslovakia in 1964 [68]. For that bridge it was proposed to strengthen the most highly stressed tension diagonals by post-tensioning. Scheme F tends to be uneconomical because it requires a large number of anchorages, and very few truss members benefit from the post-tensioning.

Scheme G in Figure 6.21b, a concentric tendon on a series of members, has been the most widely used form of post-tensioning for trusses. Lee [64] describes the use of this scheme for British railway bridges in the early 1950s, and there have been a considerable number of bridges strengthened with this scheme in Europe.

The polygonal tendon in Scheme H, Figure 6.21c, has not been reported for strengthening purposes, but it has been used in the continuous-span version of Scheme I in Figure 6.21d for a two-span truss

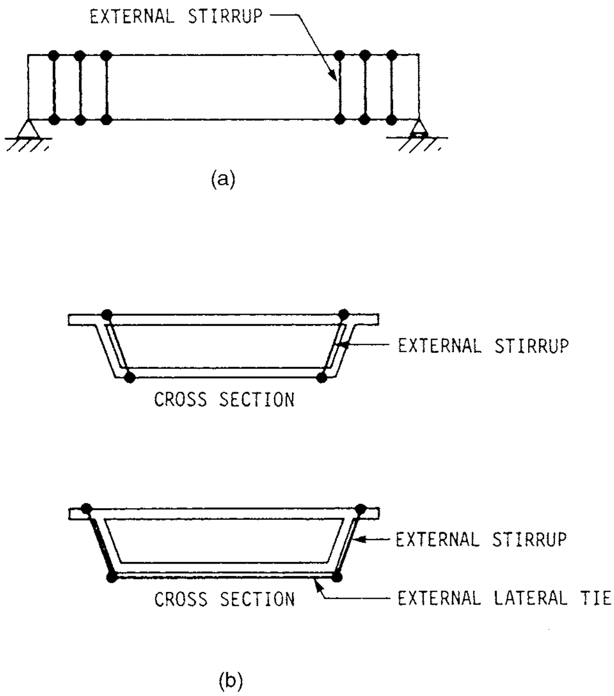


FIGURE 6.20 Tendon configurations for shear post-tensioning. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

bridge in Switzerland [72]. In the late 1960s, a truss highway bridge in Aarwangen, Switzerland, was strengthened by means of four-segment tendons on each of the two spans. The upper chord of each truss was unable to carry the additional compression force induced by the post-tensioning, and, therefore, a free-sliding compression strut was added to each top chord to take the axial post-tensioning force.

Scheme J, the king post in Figure 6.21e, has been suggested for new as well as existing trusses [7]; however, cases of its actual use for strengthening have not been reported in the literature. Because most trusses are placed on spans greater than 100 ft (30.5 m), the posts below the bridge could extend down quite far and severely reduce clearance under the bridge. The king post or queen post would thus be in a very vulnerable position and would not be appropriate in many situations.

Most uses of post-tensioning for strengthening have been on the longitudinal members in bridges; however, post-tensioning has also been used for strengthening in the transverse direction. After the deterioration of the lateral load distribution characteristics of laminated timber decks was noted in Canada in the mid-1970s [73], Scheme K in Figure 6.22a was used to strengthen the deck. A continuous-steel channel waler at each edge of the deck spreads the post-tensioning forces from threadbar tendons above and below the deck, thereby preventing local overstress in the timber. A similar tendon arrangement, Scheme L in Figure 6.22b, was used in an Illinois bridge [74] to tie together spreading, prestressed concrete box beams.

The overview of uses of post-tensioning for bridge strengthening given above identifies the most important concepts that have been used in the past and indicates the versatility of post-tensioning as a strengthening method.

6.5.3 Limitations and Disadvantages

When post-tensioning is used as a strengthening method, it increases the allowable stress range by the magnitude of the applied post-tensioning stress. If maximum advantage is taken of the increased allowable-stress range, the factor of safety against ultimate load will be reduced. The ultimate-load capacity thus will not increase at the same rate as the allowable-stress capacity. For short-term strengthening

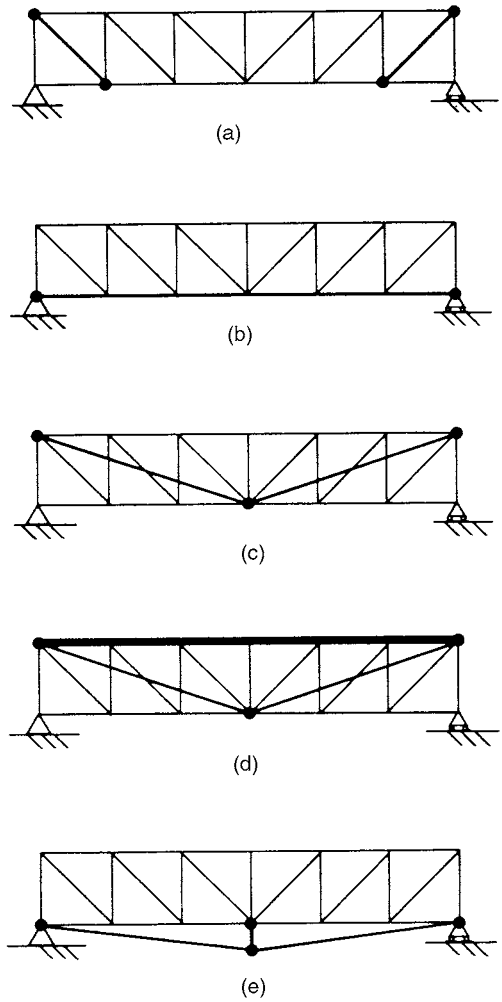


FIGURE 6.21 Tendon configurations for post-tensioning trusses. (a) Scheme F, concentric tendons on individual members; (b) Scheme G, concentric tendon on a series of members; (c) Scheme H, polygonal tendon; (d) Scheme I, polygonal tendon with compression strut; (e) Scheme J, king post. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

applications, the reduced factor of safety should not be a limitation, especially in view of the recent trend toward smaller factors of safety in design standards. For long-term strengthening applications, however, the reduced factor of safety may be a limitation.

At anchorages and brackets where tendons are attached to the bridge structure, there are high local stresses that require consideration. Any cracks initiated by holes or expansion anchors in the structure will spread with live-load dynamic cycling.

Because post-tensioning of an existing bridge affects the entire bridge (beyond the members that are post-tensioned), consideration must be given to the distribution of the induced forces and moments within the structure. If all parallel members are not post-tensioned, if all parallel members are not post-tensioned equally, or if all parallel members do not have the same stiffness, induced forces and moments will be distributed in some manner different from what is assumed in a simple analysis.

Post-tensioning does require relatively accurate fabrication and construction and relatively careful monitoring of forces locked into the tendons. Either too much or too little tendon force can cause overstress in the members of the bridge being strengthened.

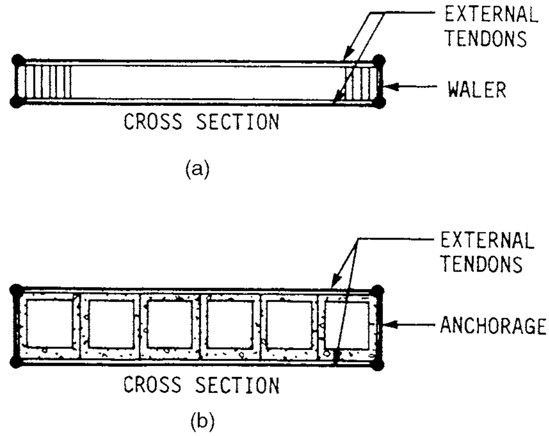


FIGURE 6.22 Tendon configuration for transverse post-tensioning of decks. (a) Scheme K, concentric tendons and walers, laminated timber deck; (b) Scheme L, concentric tendons, box beams. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

Tendons, anchorages, and brackets require corrosion protection because they are generally in locations that can be subjected to saltwater runoff or salt spray. If tendons are placed beyond the bridge profile, they are vulnerable to damage from overheight vehicles passing under the bridge or vulnerable to damage from traffic accidents. Exposed tendons also are vulnerable to damage from fires associated with traffic accidents.

6.5.4 Design Procedures

In general, strengthening of bridges by post-tensioning can follow established structural analysis and design principles. The engineer must be cautious, however, in applying empirical design procedures as they are established only for the conditions of a particular strengthening problem.

Every strengthening problem requires careful examination of the existing structure. Materials in an existing bridge were produced to some previous set of standards and may have deteriorated due to exposure over many years. The existing steel in steel members may not be weldable with ordinary procedures, and steel shapes are not likely to be dimensioned to current standards. Shear connectors and other parts may have unknown capacities due to unusual configurations.

Strengthening an existing bridge involves more than strengthening individual members. Even a simple-span bridge is indeterminate, and post-tensioning and other strengthening will affect the behavior of the entire bridge. If the indeterminate nature of the bridge is not recognized during analysis, the post-tensioning applied for strengthening purposes may not have the desired stress-relieving effects and may actually cause overstress.

Post-tensioning involves application of relatively large forces to regions of a structure that were not designed for such large forces. There is more likelihood of local overstress at tendon anchorages and brackets than at conventional member connections. Brackets need to be designed to distribute the concentrated post-tensioning forces over sufficiently large portions of the existing structure.

Members and bridges subjected to longitudinal post-tensioning will shorten axially and, depending on the tendon configuration, also will shorten and elongate with flexural stresses. These shortening and elongation effects must be considered, so that the post-tensioning has its desired effect. Frozen bridge bearings require repair and lubrication, and support details should be checked for restraints.

External tendons, whether cable or threadbar, are relatively vulnerable to corrosion, damage from overheight vehicles, traffic accidents, or fires associated with accidents. Corrosion protection and placement of the tendons are thus very important with respect to the life of the post-tensioning. Safety is also a consideration because a tendon that ruptures suddenly can pose a hazard.

For the past few years, the authors and other Iowa State University colleagues have been investigating the use of external post-tensioning (Scheme A and AA in Figure 6.19) for strengthening existing single-span and continuous-span steel stringer bridges. The research, which has been recently completed, involved laboratory testing, field implementation, and the development of design procedures. The strengthening procedures that were developed are briefly described in the following sections.

6.5.5 Longitudinal Post-Tensioning of Stringers

Simple Spans

Essentially all single-span composite steel stringer bridges constructed in Iowa between 1940 and 1960 have smaller exterior stringers. These stringers are significantly overstressed for today's legal loads; interior stringers are also overstressed to a lesser degree. Thus, the post-tension system developed is only applied to the exterior stringers; through lateral load distribution a stress reduction is also obtained in the interior stringers.

By analyzing an undercapacity bridge, an engineer can determine the overstress in the interior and exterior stringers. This overstress is based on the procedure of isolating each bridge stringer from the total structure. The amount of post-tensioning required to reduce the stress in the stringers can then be determined if the amount of post-tensioning force remaining on the exterior stringers is known; this force can be quantified with force and moment fractions. A force fraction, FF, is the ratio of the axial force that remains on a post-tensioned stringer at midspan to the sum of the axial forces for all bridge stringers at midspan, while a moment fraction, MF, is the moment remaining on the post-tensioned stringer divided by the sum of midspan moments for all bridge stringers. Knowing these fractions, the required post-tensioning force may be determined by utilizing the following relationship:

$$f = FF\left[\frac{P}{A}\right] + MF\left[\frac{Pec}{I}\right] \tag{6.1}$$

where

- f = desired stress reduction in stringer lower flange
- P = post-tensioning force required on each exterior stringer
- A = cross-sectional area of exterior stringers
- e = eccentricity of post-tensioning force measured from the neutral axis of the bridge
- c = distance from neutral axis of stringer to lower flange
- I = moment of inertia of exterior stringer at section being analyzed

Force fractions and moment fractions as well as other details on the procedure may be found in reference [75].

Span length and relative beam stiffness were determined to be the most significant variables in the moment fractions. As span length increases, exterior beams retain less moment; exterior beams that are smaller than the interior beams retain less post-tensioning moment than if the beams were all the same size.

The strengthening procedure and design methodology just described have been used on several bridges in the states of Iowa, Florida, and South Dakota. In all instances, the procedure was employed by local contractors without any significant difficulties. Application of this strengthening procedure to a 72-ft (34.0-m) long 45° skewed bridge in Iowa is shown in Figure 6.23.

Continuous Spans

Similar to the single-span bridges, Iowa has a large number of continuous-span composite steel stringer bridges that also have excessive flexural stresses. Through laboratory tests, it was determined that the desired stress reduction could be obtained by post-tensioning the positive moment regions of the various stringers in most situations. In the cases in which there are excessive overstresses in the negative moment regions, it may be necessary to use superimposed trusses (see Figure 6.24) on the exterior stringers in addition to post-tensioning the positive moment regions. Similar to single-span bridges, it was decided to use force fractions and moment fractions to determine the distribution of strengthening forces in a

FIGURE 6.23 Single-span bridge strengthened by post-tensioning.

given bridge. As one would expect, the design procedure is considerably more involved for continuous-span bridges as one has to consider transverse and longitudinal distribution of forces.

The required strengthening forces and final stringer envelopes should be calculated. The various strengthening schemes that can be used are shown in Figure 6.25. A designer selects the schemes required for obtaining the desired stress reduction. For additional details on the strengthening procedure the reader is referred to reference [76]. Shown in Figure 6.26 is a three-span continuous bridge near Mason City, Iowa, that has been strengthened using the schemes shown in Figure 6.25.

6.6 Developing Additional Bridge Continuity

6.6.1 Addition of Supplemental Supports

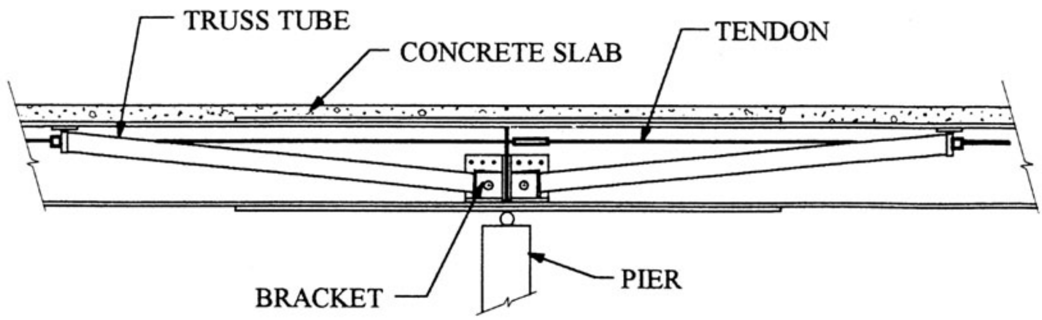
Description

Supplemental supports can be added to reduce span length and thereby reduce the maximum positive moment in a given bridge. By changing a single-span bridge to a continuous, multiple-span bridge, stresses in the bridge can be altered dramatically, thereby improving the maximum live-load capacity of the bridge. Even though this method may be quite expensive because of the cost of adding an additional pier(s), it may still be desirable in certain situations.

Applicability and Advantages

This method is applicable to most types of stringer bridges, such as steel, concrete, and timber, and has also been used on truss bridges [7]. Each of these types of bridges has distinct differences.

If a supplemental center support is added to the center of an 80-ft (24.4-m) long steel stringer bridge that has been designed for HS20-44 loading, the maximum positive live-load moment is reduced from 1164.9 ft-kips (1579.4 kN·m) to 358.2 ft-kips (485.7 kN·m), which is a reduction of over 69%. At the



(a)

(b)

FIGURE 6.24 Superimposed truss system. (a) Superimposed truss; (b) photograph of superimposed truss.

same time, however, a negative moment of 266.6 ft-kips (361.5 kN·m) is created which must be taken into account. In situations where the added support cannot be placed at the center, reductions in positive moments are slightly less.

Limitations and Disadvantages

Depending on the type of bridge, there are various limitations in this method of strengthening. First, because of conditions directly below the existing bridge, there may not be a suitable location for the pier, as, for example, when the bridge requiring strengthening passes over a roadway or railroad tracks. Other constraints, such as soil conditions, the presence of a deep gorge, or stream velocity, could greatly increase the length of the required piles, making the cost prohibitive.

This method is most cost-effective with medium- to long-span bridges. This eliminates most timber stringer bridges because of their short lengths. In truss bridges, the trusses must be analyzed to determine the effect of adding an additional support. All members would have to be examined to determine if they could carry the change in force caused by the new support. Of particular concern would be members originally designed to carry tension, but which because of the added support must now carry compressive stresses. Because of these problems, the emphasis in this section will be on steel and concrete stringer bridges.

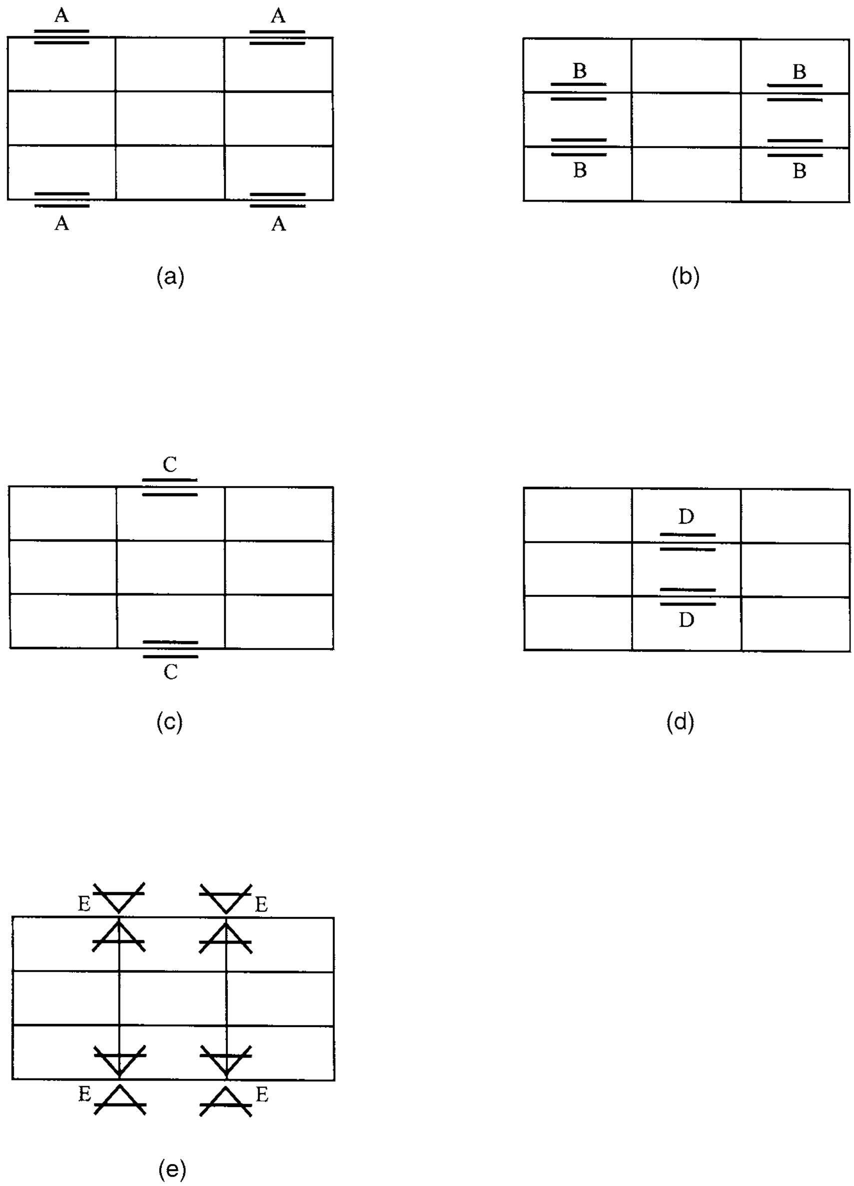


FIGURE 6.25 Strengthening schemes for continuous-span bridge. (a) Strengthening Scheme A: post-tensioning end spans of the exterior stringers; (b) strengthening Scheme B: post-tensioning end spans of the interior stringers; (c) strengthening Scheme C: post-tensioning center spans of the exterior stringers; (d) strengthening Scheme D: post-tensioning center spans of the interior strangers; (e) strengthening Scheme E: superimposed trusses at the piers of the exterior stringers.

Design Considerations

Because the design of each intermediate pier system is highly dependent on many variables such as the load on pier, width and height of bridge, and soil conditions, it is not feasible to include a generalized design procedure for piers. The engineer should use standard pier design procedures. A brief discussion of several of the more important considerations (condition of the bridge, location of pier along bridge, soil condition, type of pier, and negative moment reinforcement) is given in the following paragraphs.

FIGURE 6.26 Photograph of three-span continuous bridge strengthened with post-tensioning and superimposed trusses.

Providing supplemental support is quite expensive; therefore, the condition of the bridge is very important. If the bridge is in good to excellent condition and the only major problem is that the bridge lacks sufficient capacity for present-day loading, this method of strengthening should be considered. On the other hand, if the bridge has other deficiencies, such as a badly deteriorated deck or insufficient roadway width, a less expensive strengthening method with a shorter life should be considered.

The type of pier system employed greatly depends on the loading and also the soil conditions. The most common type of pier system used in this method is either steel H piles or timber piles with a steel or timber beam used as a pier cap. A method employed by the Florida Department of Transportation [77] can be used to install the piles under the bridge with limited modification to the existing bridge. This method consists of cutting holes through the deck above the point of application of the piles. Piles are then driven into position through the deck. The piles are then cut off so that a pier cap and rollers can be placed under the stringers. Other types of piers, such as concrete pile bents, solid piers, or hammerhead piers, can also be used; however, cost may restrict their use.

Another major concern with this method is how to provide reinforcement in the deck when the region in the vicinity of the support becomes a negative moment region. With steel stringers the bridge may either be composite or noncomposite. If noncomposite, the concrete deck is not required to carry any of the negative moment and therefore needs no alteration. On the other hand, if composite action exists, the deck in the negative moment region should be removed and replaced with a properly reinforced deck. For concrete stringer bridges the deck in the negative moment region should be removed. Reinforcement to ensure shear connection between the stringers and deck must be installed and the deck replaced with a properly reinforced deck. This method, although expensive and highly dependent on the surroundings, may be quite effective in the right situation.

6.6.2 Modification of Simple Spans

Description

In this method of strengthening, simply supported adjacent spans are connected together with a moment and shear-type connection. Once this connection is in place, the simple spans become one continuous span, which alters the stress distribution. The desired decrease in the maximum positive moment, however, is accompanied by the development of a negative moment over the interior supports.

Applicability and Advantages

This method can be used primarily with steel and timber bridges. Although it could also be used on concrete stringer bridges, the difficulties in structural connecting to adjacent reinforced concrete beams result in the method being impractical. The stringer material and the type of deck used will obviously dictate construction details. Thus, the main advantage of this procedure is that it is possible to reduce positive moments (obviously the only moments present in simple spans) by working over the piers and not near the midspan of the stringers. This method also reduces future maintenance requirements because it eliminates a roadway joint and one set of bearings at each pier where continuity is provided [12].

Limitations and Disadvantages

The main disadvantage of modifying simple spans is the negative moment developed over the piers. To provide continuity, regardless of the type of stringers or deck material, one must design for and provide reinforcement for the new negative moments and shears. Providing continuity also increases the vertical reactions at the interior piers; thus, one must check the adequacy of the piers to support the increase in axial load.

Design Considerations

The main design consideration for both types of stringers (steel and timber) concerns how to ensure full connection (shear and moment) over the piers. The following sections will give some insight into how this may be accomplished.

Steel Stringers

Berger [12] has provided information, some of which is summarized here, on how to provide continuity in a steel stringer concrete deck system. If the concrete deck is in sound condition, a portion of it must be removed over the piers. A splice, which is capable of resisting moment as well as shear, is then installed between adjacent stringers. Existing bearings are removed and a new bearing assembly is installed. In most instances, it will be necessary to add new stiffener plates and diaphragms at each interior pier. After the splice plates and bearing are in place, the reinforcement required in the deck over the piers is added and a deck replaced. Such a splice is shown in Figure 6.27.

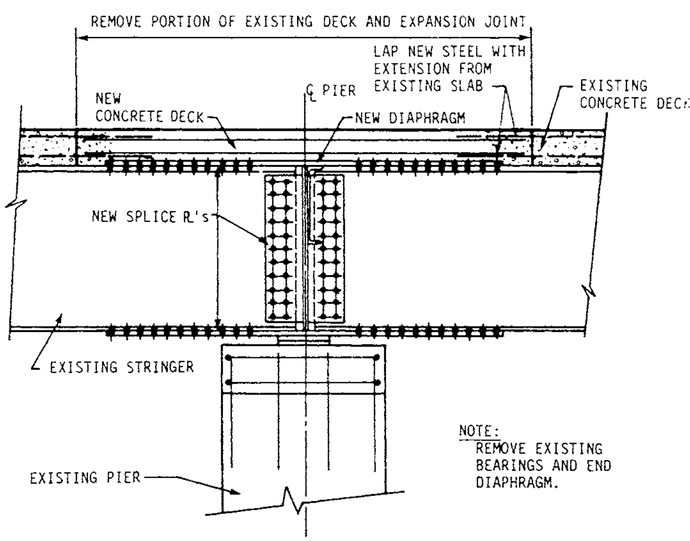


FIGURE 6.27 Conceptual details of a moment- and shear-type connection. (*Source:* Klaiber, F.W. et al., NCHRP 293, Transportation Research Board, 1987. With permission.)

Recently, the Robert Moses Parkway Bridge in Buffalo, New York [78] which originally consisted of 25 simply supported spans ranging from 63 ft (19.2 m) to 77 ft (23.5 m) in length was seismically retrofitted. Moment and shear splices were added to convert the bridge to continuous spans: one two-span element, one three-span element, and five four-span elements. This modification not only strengthened the bridge, but also provided redundancy and improved its earthquake resistance.

Timber Stringers

When providing continuity in timber stringers, steel plates can be placed on both sides and on the top and bottom of the connection and then secured in place with either bolts or lag screws. When adequate plates are used, this provides the necessary moment and shear transfer required. Additional strength can be obtained at the joint by injecting epoxy into the timber cracks as is suggested by Avent et al. [79]. Although adding steel plates requires the design and construction of a detailed connection, significant stress reduction can be obtained through its use.

6.7 Recent Developments

6.7.1 Epoxy Bonded Steel Plates

Epoxy-bonded steel plates have been used to strengthen or repair buildings and bridges in many countries around the world including Australia, South Africa, Switzerland, the United Kingdom, and Japan, to mention a few.

The principle of this strengthening technique is rather simple: an epoxy adhesive is used to bond steel plates to overstressed regions of reinforced concrete members. The steel plates are typically located in the tension zone of a beam; however, plates located in the compression and shear zones have also been utilized. The adhesive provides a shear connection between the reinforced concrete beam and the steel plate, resulting in a composite structural member. The addition of plates in the tension zone not only increases the area of tension steel, but also lowers the neutral axis, resulting in a reduction of live-load stresses in the existing reinforcement. The tension plates effectively increase the flexural stiffness, thereby reducing cracking and deflection of the member.

Although this procedure has been used on dozens of bridges in other countries, to the authors' knowledge, it has not been used on any bridges in the United States due to concerns with the method. Some of these concerns are plate corrosion, long-term durability of the bond connection, plate peeling, and difficulties in handling and installing heavy plates.

In recent years, the steel plates used in this strengthening procedure have been replaced with fiber-reinforced plastic sheets; the most interest has been in carbon fiber-reinforced polymer (CFRP) strips. Although CFRP strips have been used to strengthen various types of structures in Europe and Japan for several years, in the United States there have only been laboratory investigations and some field demonstrations. Discussion in the following section is limited to the use of CFRP in plate strengthening. For information on the use of FRP for increasing the shear strength and ductility of reinforced concrete columns in seismic area, the reader is referred to Reference [91]. This reference is a comprehensive literature review of the various methods of seismic strengthening of reinforced columns.

6.7.2 CFRP Plate Strengthening

CFRP strips have essentially replaced steel plates as CFRP has none of the previously noted disadvantages of steel plates. Although CFRP strips are expensive, the procedure has many advantages: less weight, strengthening can be added to the exact location where increased strength is required, strengthening system takes minimal space, material has high tensile strength, no corrosion problems, easy to handle and install, and excellent fatigue properties. As research is still in progress in Europe, Japan, Canada, and the United States on this strengthening procedure, and since the application of CFRP strips obviously varies from structure to structure, rather than providing details on this procedure, several examples of its application will be described in the following paragraphs.

In 1994, legal truck loads in Japan were increased by 25% to 25 tons. After a review of several concrete slab bridges, it was determined that they were inadequate for this increased load. Approximately 50 of these bridges were strengthened using CFRP sheets bonded to the tension face. The additional material not only reduced the stress in the reinforcing bars, it also reduced the deflections in the slabs due to the high modulus of elasticity of the CFRP sheets.

Recently, a prestressed concrete (P/C) beam in West Palm Beach, Florida, which had been damaged by being struck by an overheight vehicle, was repaired using CFRP. This repair was accomplished in 15 hours by working three consecutive nights with minimal disruption of traffic. The alternative to this repair technique was to replace the damaged P/C with a new P/C beam. This procedure would have taken close to 1 month, and would have required some road closures.

The Oberriet–Meiningen three-span continuous bridge was completed in 1963. This bridge over the Rhine River connects Switzerland and Austria. Due to increased traffic loading, it was determined that the bridge needed strengthening. Strengthening was accomplished in 1996 by increasing the deck thickness 3.1 in. (8 cm) and adding 160 CFRP strips 13.1 ft (4 m) long on 29.5-in. (75-cm) intervals to the underside of the deck. The combination of these two remedies increased the capacity of the bridge so that it is in full compliance with today's safety and load requirements.

Three severely deteriorated 70-year-old reinforced concrete frame bridges near Dreselou, Germany, have recently been strengthened (increased flexure and shear capacity) using CFRP plates. Prior to strengthening, the bridges were restricted to 2-ton vehicles. With strengthening, 16-ton vehicles are now permitted to use the bridges. Prior to implementing the CFRP strengthening procedure, laboratory tests were completed on this strengthening technique at the Technical University in Brauwschweigs, Germany.

6.8 Summary

The purpose of this chapter is to identify and evaluate the various methods of strengthening existing highway bridges and to a lesser extent railroad bridges. Although very few references have been made to railroad bridges, the majority of the strengthening procedures presented could in most situations be applied to railroad bridges.

In this chapter, information on five strengthening procedures (lightweight deck replacement, composite action, strengthening of various bridge members, post-tensioning, and development of bridge continuity) have been presented. A brief introduction to using CFRP strips in strengthening has also been included.

In numerous situations, strengthening a given bridge, rather than replacing it or posting it, is a viable economical alternative which should be given serious consideration.

For additional information on bridge strengthening/rehabilitation, the reader is referred to References [1, 2, 60] which have 208, 379, and 199 references, respectively, on the subject.

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