Perspectives on Structural Behavior and Redundancy:

Concrete Bridge Behavior and New Materials

by Dr. Oguzhan Bayrak, University of Texas at Austin

My first article on structural, load path, and internal redundancies, published in the Summer 2020 issue of *ASPIRE*®, started a series of discussions. To further these discussions, my article in the Winter 2021 issue focused on first principles and fundamentals that are of primary importance in the context of redundancy, ductility, and resilience. This article focuses on the behavior of concrete bridges and considerations for incorporation of new materials into design specifications.

Behavior of Concrete Bridge Beams

The American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] has been calibrated on the basis of element-level behavior. To explore the fundamental aspects of this calibration, my Winter 2021 article focused on flexural response. Let us now focus on an example where we look at the behavior of reinforced concrete beams failing in flexure and shear. Envision classical laboratory tests where simply supported beams are subjected to two-point loading. **Figure 1** illustrates a series of four comparable tests resulting in four different failure modes. As shown in the figure, the underreinforced beam experiencing flexural failure displays a substantial amount of ductility, which is demonstrated by increasing deflection at a relatively constant load. Such ductile behavior is desirable so it has some of the highest strength-reduction factors (ϕ-factors) in the AASHTO LRFD specifications.

In cases where the behavior can be classified as tension controlled, prestressed concrete bridge elements (both pre- and post-tensioned) have a ϕ-factor of 1.0, whereas comparable reinforced concrete beams have a ϕ-factor of 0.9. The difference between these two ϕ-factors is rooted in higher quality control and quality assurance associated with prestressed concrete elements, minimal levels of statistical variation in prestressing strand properties, better control of component dimensions or bridge geometry associated with prestressed concrete bridges, and a greater level of variability and construction tolerances associated with cast-in-place, reinforced concrete bridges, as well as a higher variability of material properties in ordinary reinforcing bars. In the context of calibrating ϕ-factors,

EDITOR'S NOTE ON STRUCTURAL BEHAVIORS SERIES

Robustness, redundancy, resiliency, and ductility of concrete bridges are being discussed by Dr. Bayrak in a series of Perspective articles that began in the Summer 2020 issue of ASPIRE®. *This series is seen by* ASPIRE *as an important discussion for the concrete bridge community as it begins to consider new materials. These new materials have properties that differ significantly from conventional materials, which may lead to different element behavior. For example, some of the new materials exhibit ductile behavior, whereas others do not. These differences require new approaches to design, but the framework necessary for establishing these new design approaches is not clearly defined by current design specifications.*

It should be noted that not all potential failure modes considered in the American Association of State Highway and Transportation Officials' AASHTO LRFD Bridge Design Specifications *are ductile failure modes, so ductility should not be considered as the only criterion for acceptable bridge designs. For example, concrete breakout failure for embedded anchors is quite brittle, which is recognized in calibration of the applicable equations in the AASHTO LRFD specifications. Furthermore, a more complete understanding of the actual capacity of a bridge is provided when the system-level robustness is considered. Different types of redundancies are inherent in concrete bridges, such as load transfer between girders; these contribute to the overall robustness of the bridge by providing multilayer protection against sudden failure. However, quantifying the contribution of these redundancies is not easy. Current AASHTO LRFD specifications have simplified bridge design by considering only element behavior. Therefore, they do not consider overall system behavior and the redundancies that contribute to it.*

The article in this issue focuses on prestressed concrete bridge components reinforced with emerging materials. Some background information is provided on the toughness and ductility of those materials, and the impact these properties have on element behavior. Approaches to including consideration of the behavior of concrete bridge systems will be a topic discussed in a future article in this series.

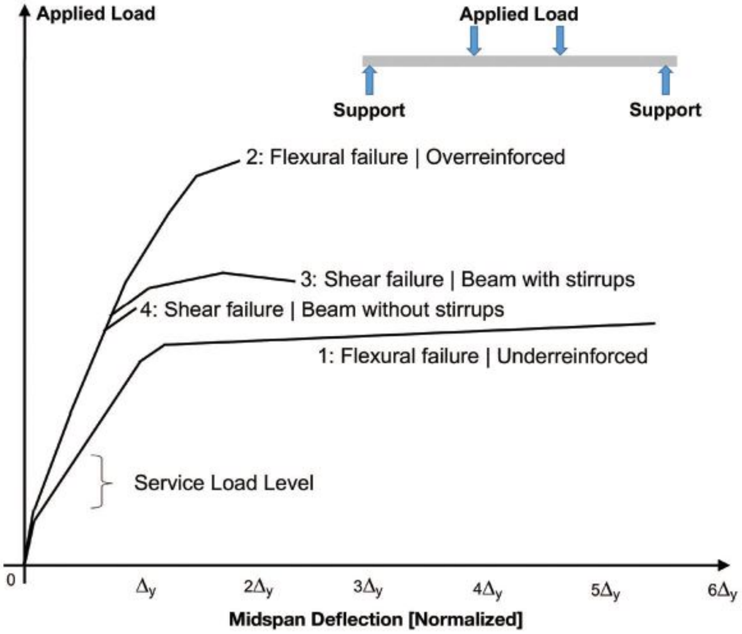


Figure 1. Load-deflection behavior of concrete beams showing different types of performance. Figure: Dr. Oguzhan Bayrak.

In the context of calibrating ϕ-factors, reduced variability in prestressed concrete bridge components (due to all of the aforementioned factors) results in an increase in the ϕ-factor. Use of steel forms instead of wood forms allowing tight dimensional tolerances, verification of the prestressing force by measuring both force and elongation, and the high emphasis placed on controlling bridge geometry (for example, in segmental bridge construction) all contribute to a higher ϕ-factor.

The behavior of the overreinforced beam shown in Fig. 1 is quite different from that of the underreinforced beam. Whereas the underreinforced beam displays substantial levels of ductility, the overreinforced beam's behavior is governed by crushing of concrete before yielding of the flexural tension reinforcement. The AASHTO LRFD specifications make accommodations for this type of behavior (see AASHTO LRFD specifications Fig. C5.5.4.2-1) and explain the behavioral difference and its impact on the ϕ-factor in Article C5.5.4.2, as follows:

> *A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.*

With this behavioral difference, the specifications require that a ϕ-factor of 0.75 be used for compression-controlled sections, rather than the 0.9 to 1.0 values for tension-controlled sections. The lower value is used to build in a greater safety margin for compression-controlled flexural failures. In short, if failure is controlled by yielding of reinforcement with minimal variability in material properties and the associated ductile behavior, a higher ϕ-factor is used. If the behavior is governed by crushing of concrete, which has a higher variability in material response and is coupled with a brittle failure mode, a lower ϕ-factor is used.

Focusing on shear, we can see that the behavior of a beam failing in shear can be quite brittle if the beam does not have any web reinforcement or contains insufficient shear reinforcement (case 3 in Fig. 1). By providing shear reinforcement, the nature of the brittle failure shown in Fig. 1 (compare case 3 with case 4) is altered. At least two factors contribute to the ϕ-factor of 0.9 for shear in the AASHTO LRFD specifications: the more ductile shear behavior and a shear strength estimate methodology (the modified compression field theory) that was recognized to be an accurate predictor of shear strength when it was incorporated into the specifications in 1994.

Starting with the eighth edition of the AASHTO LRFD specifications,[2] additional emphasis has been placed in differentiating B-regions (beam or Bernoulli regions) from D-regions (disturbed or discontinuity regions) and using strut-and-tie modeling (STM) provisions for D-regions. That is to say, a great majority of the substructure components and portions of segmental bridges and cable-stayed bridges are designed to comply with the STM provisions of the AASHTO LRFD specifications. Similarly, STM of end regions of pretensioned concrete girders is necessary, especially in cases where shear stresses greater than 0.18 f'_c are to be justified. For STM, the AASHTO LRFD specifications require ϕ-factors for tension ties that are consistent with the ϕ-factors of the previously discussed tensioned-controlled cases—that is, ϕ = 0.9 for reinforcing bars serving as tension ties, and ϕ = 1.0 for prestressing strands serving as tension ties. For compression members analyzed with STM, the AASHTO LRFD specifications require ϕ = 0.7. Once again, these ϕ-factors reflect behavioral differences as well as variability seen in test results.

Conventional and Emerging Materials in Concrete Bridges

Throughout the history of civilization, construction materials and means and methods have continually evolved. For example, breathtaking ancient stone masonry arches, which continue to serve communities around the world, would be prohibitively expensive to construct today.

Bridge engineers try to balance aesthetics, bridge owners' requirements, and budgets with the needs of the communities that will be served by the bridges they design. Concrete bridges continue to be the preferred solution in most cases due to their resilience, cost-effectiveness, and safety record.

In the early part of the 20th century, cast-in-place reinforced concrete bridges were more common. With the introduction of standardized precast, pretensioned concrete bridge beams, prestressed concrete bridges quickly became the dominant bridge type. Over the years, pretensioned concrete girder sections have been optimized. Segmentally constructed concrete bridges (both cast-in-place and precast concrete) became popular for long spans and have been used by some communities as signature bridges. The

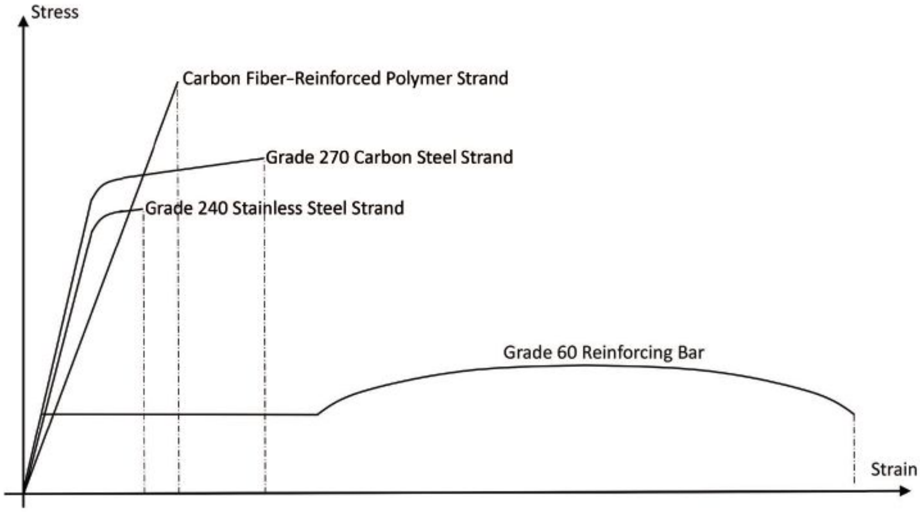


Figure 2. Comparison of stress-strain curves for several strand materials and Grade 60 reinforcing bar. Figure: Dr. Oguzhan Bayrak.

reinforcement have improved for both pretensioned and post-tensioned concrete bridges. Additional sizes and grades of prestressing strands were introduced into the marketplace. More recently, to improve durability of concrete bridges, new materials have been introduced to the marketplace. Stainless steel strands and carbon fiber–reinforced polymer (CFRP) strands are now commercially available.

Figure 2 compares typical stress-strain curves for conventional and emerging types of reinforcement. It shows that Grade 60 reinforcing bars have the highest deformation capacity (rupture strain) of the four materials included in this graphic. Conversely, stainless steel strands and CFRP strands display lower deformation capacities. On the basis of this observation, let us focus on the behavior of Grade 60 reinforcing bars and the Grade 270 seven-wire strands, both of which have been routinely used in reinforced and prestressed concrete bridges. Examining the ultimate strain experienced by these materials, there is a substantial difference (a factor of three to four) between these two materials with respect to ultimate deformation capacity.

Let us now consider the areas under the stress-strain curves for these two materials. The areas are a measure of toughness of the materials. Engineering toughness is the ability of a material to absorb energy before failure. Toughness is typically quantified by integrating the stress-strain curve or looking at the area under the curve. The assumed yield and ultimate strengths of Grade 270 low-relaxation strands are about three to four times as large as corresponding values for Grade 60 reinforcing bars. Without getting into detailed calculations, the areas under the curves, or toughness, for these two materials, which have been successfully and routinely used in practice, are somewhat comparable.

Let us now introduce new materials into this discussion. CFRP and stainless steel strands have comparable levels of toughness with each other, with stainless steel strands enclosing less area under its stress-strain curve.

This observation leads us to a natural question: What is the acceptable level of material toughness to provide acceptable structural response? This question may be followed with another: Should other deformation-based measures be considered at the element, or even structure, level to provide an acceptable structural response? This relates to the ability of a material, element, or structure to deform enough to provide warning prior to any type of brittle failure, as discussed in my article in the Winter 2021 issue. As long as adequate levels of cracking and structural deformation can be observed before collapse, we can conclude that sufficient material toughness, or satisfactory behavior at the element or structure level, exists.

Figure 3 compares the moment-deflection behavior of simply supported prestressed concrete girders reinforced with conventional carbon steel strands and those reinforced with CFRP strands. The ultimate moment and deflection experienced by both types of prestressed concrete beams shown in this figure are nearly the same for this particular case. However, the area under the curve is less for the beam with the CFRP strands than for the beam with conventional strands. This difference can be accommodated in the context of structural redundancy and associated resiliency. If an equivalent area under the curve is desired, the design for the beam with CFRP can be modified to include more strands, compressive strength of concrete can be adjusted, or both can be modified. Therefore, by appropriate design and detailing, we can achieve a similar level of toughness in both cases. The same approach can be taken for designs using stainless steel strands.

The preceding discussion excludes the primary benefit of using CFRP or stainless steel strands. The corrosion resistance of these two materials, when needed, offers tremendous advantages over conventional carbon steel. Where environmental exposure conditions are

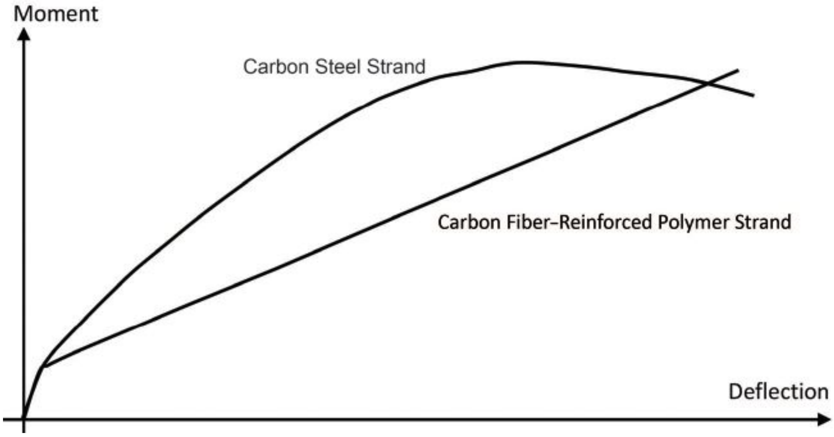
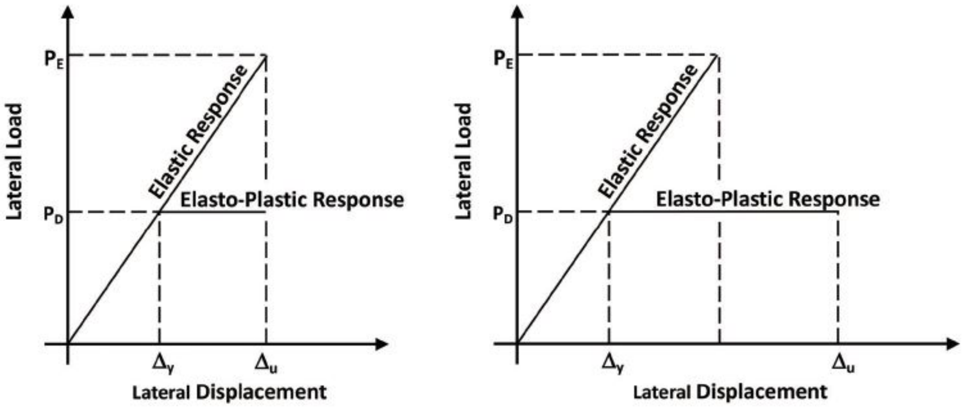


Figure 3. Comparison of moment-deflection response of prestressed concrete beams with carbon steel and carbon fiber–reinforced polymer strands. Figure: Dr. Oguzhan Bayrak, adapted from ACI 440.4R-04, *Prestressing Concrete Structures with FRP Tendons*, (Farmington Hills, MI: American Concrete Institute, 2004).



a. Principle of Maximum Displacement **b. Principle of Maximum Potential Energy**

Figure 4. Equivalent structural systems can be established on the basis of principle of maximum displacement or principle of maximum potential energy. Figure: Dr. Oguzhan Bayrak, adapted from Paulay, T., and M. J. N. Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings* (New York: Wiley, 1992).

severe, and where a long service life for a bridge is desired, these materials have their place. With appropriate ϕ-factors, both of these materials can be used in bridges. Like conventional materials we use in our concrete bridges, we can welcome the use of new materials, put their performance in proper context with expected levels of system performance, and continue on the path of constructing concrete bridges with new materials. System performance is an important factor related to performance and is a topic that will be discussed in a future article.

In the context of equivalency, there are some parallels with seismic design principles. **Figure 4** shows that equivalency between a yielding system and an elastic system can be established on the basis of principle of maximum displacement or principle of maximum potential energy. For stiff structural systems, the use of equivalent maximum potential energy is more appropriate. For more flexible systems, equivalent displacement leads to more realistic approximations of actual response. Although these are seismic behavior and design principles, the parallels between these principles and the preceding discussion are clear. That is to say, the need to look for equivalency between yielding systems and systems that remain linear-elastic has been a challenge both in seismic applications and at the strength limit state for other loading scenarios such as gravity loading.

New Technologies

As we advance concrete bridge construction, we continually seek to find new materials that offer advantages over conventional materials to improve structural efficiency or address a problem. Similarly, the concrete bridge community considers the implementation of new systems to address observed field problems or to advance the state of practice. In this context, and to address tendon corrosion issues related to grouting practices, the post-tensioning industry is considering the use of flexible fillers.

This technology has previously been used in Europe and the Far East but is new to the U.S. market. The goal of using flexible fillers is to improve the in-service performance and durability of tendons. Furthermore, using flexible fillers allows tendons to be replaced, more post-tensioning to be added, or the post-tensioning force to be modified to accommodate new design loads.

An unintended consequence of the use of flexible fillers in bridges is the loss of internal redundancy that we have in grouted systems. In a grouted tendon, if a wire breaks due to an unforeseen reason, the tendon has a chance to redevelop the broken wire over a certain distance from the wire break point. In effect, grout facilitates internal redistribution so that the impact of a wire break at a section can be contained within the immediate vicinity. With flexible fillers, this type of internal redundancy is eliminated. This does not mean we cannot or should not use tendons protected with flexible fillers; their use with external tendons can be appropriate. The use of grouted tendons within girder webs has advantages that relate to the shear strength of the webs and the system redundancy. In short, while new technologies have their place in our tried-and-true concrete bridge solutions, their advantages and potential negative impacts on structural behavior should be thoroughly considered before their adoption.

Final Remarks

Overall, concrete bridges have been performing exceedingly well. We have examples of prestressed concrete bridges built in the 1950s that continue to serve their intended design functions. Many of the early segmental bridges in the United States are still in service and in good shape. From the simplest to the most complex, concrete bridges have gone through a wide variety of laboratory and field testing. It is not a coincidence that our cost-effective concrete bridges continue to serve our communities as originally intended. Many of these systems have been optimized in many ways over the years.

As the bridge engineering community, we must exercise caution to adopt and bring in new materials and technologies. This is not intended to mean that we should not improve our state of practice. It is quite the opposite—we should continuously improve our practice. However, in doing so, we should be careful about intended and unintended consequences of our decisions. In bridge engineering, we do not get something for nothing. For example, implementation of corrosion-resistant materials may be coupled with the use of more brittle materials. The science and engineering that back the concrete bridge industry are quite mature, and we have the necessary tools to scrutinize new materials and technologies during their adoption stage. Once a bridge is in service, field inspection personnel need to be trained for the performance indicators that relate to new materials, systems, and technologies. Frankly, this logic is a part of the simple evolution (as opposed to revolution) we have witnessed in concrete bridges since their worldwide adoption for their cost-effectiveness, maintainability, and overall versatility.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*, 9th ed. Washington, DC: AASHTO.
2. AASHTO. 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.