Design Optimization Study of a Three-Span Continuous Bridge Using HPS70W

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Since its introduction to the bridge market in 1996, High Performance Steel (HPS) has been successfully implemented in a wide range of bridge projects by several state transportation departments. Examples of which include the Snyder South Bridge in Nebraska that was initially designed using AASHTO M270 Grade 50 weathering steel. The Nebraska Department of Roads opted to use HPS70W in place of the Grade 50 weathering steel without performing any redesign. This was done to provide information on welding and fabrication issues with HPS70W (Wasserman, Azizinamini, Pate and Greer, 1998). Also, in 1996, the Tennessee Department of Transportation (TDOT) partnered with FHWA through the Innovative Bridge Research and Construction Program to use HPS70W in a bridge project. The TDOT was in the process of designing a two-span continuous bridge, with span lengths of 236 ft 6 in., located on Tennessee State Route 53 over Martin Creek (FHWA, 1998). The bridge has a roadway width of 28 ft on three I-girders spaced at 10-ft 6-in. The superstructure was redesigned utilizing HPS70W for the girders and Grade 50W steel for the stiffeners, cross-frames, and other miscellaneous steel. The designs were performed using the 1994 AASHTO LRFD Bridge Design Specifications. These bridges are reported to have cost savings on the order of 10 percent through the inclusion of HPS70W in their superstructures.

While HPS does offer improved material performance characteristics relative to other conventional bridge steels, it does have higher material costs. It is therefore important to develop an understanding of how this material may most economically be incorporated in the design of composite I-girder bridges. Therefore, this study focuses on developing extensive optimized parametric design studies for a typical three-span continuous plate girder bridge. Major design parameters are varied (which include span length, girder spacing, span-to-depth ratio, and girder material configurations) and cost estimates are generated for each design variation. Finally, resulting trends regarding the influence of the use of HPS70W on the superstructure economy are summarized.

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BACKGROUND

High strength steels with yield strengths up to 100 ksi have been available for many years; however, they are not widely used in bridge applications. The existing specification for the conventional high strength steels, ASTM A709 (prior to the adoption of current HPS specifications), permitted carbon contents as high as 0.19 percent for 70-ksi and 0.21 percent for 100-ksi steels. These high levels of carbon content allow the desired high strength but lead to problems with weldability. Other problems with conventional high strength steel are low fracture toughness, high ductility ratio (i.e. F_y / F_u), risk of brittle fracture, and potential for stress-corrosion cracking (Fisher and Dexter, 1994).

In 1992, a High Performance Steel Steering Committee, composed of the American Iron and Steel Institute (AISI), the Federal Highway Administration (FHWA), and the U.S. Navy, began an effort to develop new High Performance Steels (HPS) that had higher strength, improved weldability, high toughness, and enhanced weathering capabilities. The Steering Committee decided that the grades to be developed were 70 ksi for plates up to 4-in.-thick and 100 ksi for plates up to 2-in.-thick (Manganello, 1998). These steels would be designated under ATSM A709 as HPS70W and HPS100W. Further requirements of new high strength steels include a minimum weathering index of 6.5, 0.006 percent maximum sulfur level for improved toughness levels, and a tighter chemistry composition so the steels could be readily reproducible from mill to mill (Krouse, 1999).

The two manufacturing processes that have been developed for HPS are quench and tempering (Q&T) and thermomechanical processing (TMCP). The Q&T process may be used for plates up to 4-in.-thick but has a practical length restriction of 50 ft, which may influence the total number of shop butt welds along the length of the bridge. The TMCP process does not have the plate length restriction, however, presently it may only be used for plates up to 2-in.-thick (Manganello, 1998).

The new high strength steels that were developed from the collaboration of the Steering Committee have carbon contents less that 0.11 percent. This low carbon content creates steels that have high strength, excellent fracture toughness, and good weldability (Sause and Pense, 1999). The low carbon content of HPS also produces steels that are not susceptible to hydrogen cracking. This is a considerable improvement over conventional high strength steels in the ease of welding.

In addition to improved welding characteristics, HPS has increased fracture toughness that satisfies the requirements for fracture critical members and the most severe Charpy V-notch test requirement of 35 ft-lb at -10 °F (Krouse, 1999). Since HPS70W has improved welding characteristics and increased fracture toughness, the removal of stringent requirements on non-redundant structures and fracture critical details would benefit the use of HPS70W in bridge design.

Another benefit of HPS steel is the improved weathering characteristics. High performance steels, targeted to have a weathering index of 6.5, exceed the minimum index of 6.0 that is felt to provide sufficient weathering characteristics (Krouse, 1999).

Previous Design Studies

There have essentially been three main design studies that investigated performance and cost issues regarding the use of HPS70W in bridge design. The cost issue was an important aspect of the previous studies because HPS70W material costs are higher than traditional steels. These studies examined the benefits realized by weight savings and reduced fabrication costs, which may offset the increased material costs.

Homma and Sause (1995)

Homma and Sause (1995) investigated the use of high performance steel in bridge design. These studies were performed using the 1st Edition of AASHTO LRFD Bridge Design (AASHTO, 1998). The study is based on the original designs of two bridges on Interstate Highway I-78. The first bridge is a 110 ft simple span steel I-girder structure over Lehigh Street and the second bridge is a 7-span continuous steel I-girder structure over the Delaware River. Portions of the original cross-sections that were designed with AASHTO M270 Grade 50 steel according to the AASHTO Standard Specifications were redesigned for minimum weight cross sections in both conventional grade steel and High Performance Steel. The sections were redesigned using the following yield strengths: 36 ksi, 50 ksi, 70 ksi, 85 ksi, 100 ksi, and 120 ksi.

The midspan section of the Lehigh Street Bridge was redesigned for three cases:

- Weight minimization without considering fatigue,
- Weight minimization considering fatigue, and
- Weight minimization considering fatigue and allowing the compact sections to reach the plastic moment.

They found that designs comprised of higher yield strengths resulted in lighter weight per unit length when fatigue was not considered. An exception was found to occur between 70 and 85 ksi; the 1st Edition of LRFD Design Specifications by AASHTO only permitted development of the plastic moment for sections with yield strengths, F_y less than or equal to 65 ksi. For girders with F_y greater than 70 ksi, the maximum bending resistance is limited to the yield moment, M_y (Sause and Fisher, 1995). When the AASHTO code limitation on yield strengths above 65 ksi was ignored, their studies demonstrated continual weight savings in designs with F_y up to 120 ksi.

When the fatigue requirement was considered, the potential for reduced weight cross-sections as yield stress increases is affected. The girder weight could not be reduced for yield strengths above 76-ksi because of the limit on the stress range of Category C fatigue details. Homma and Sause (1995) found the most critical section concerning fatigue to be located 20 ft. from the abutment where a plate transition occurs. They found that a weight decrease was not possible for yield strengths above 70 ksi when Category C fatigue details are considered in design at this location.

Homma and Sause also considered the impact of current restrictions on live load deflections. For their designs at optimum depths, no section violated the $L/800$ limit. It should be noted that the minimum span-to-depth (L/D) ratios that are defined in AASHTO for composite steel I-sections of $L/D = 25$ for simple spans, where L is the span length and D is the overall depth, were met in this study.

The Delaware River Bridge focused on the redesign of two sections: the maximum positive bending section and the maximum negative bending section. For the positive bending section, when the fatigue limit state of the Category C detail was neglected, higher yield strength resulted in smaller weight per unit length, which was also seen with the Lehigh Street Bridge. However, an exception in weight savings occurred between 70 and 85 ksi due to the restrictions on yield strengths greater than 65 ksi. When Category C fatigue limits were considered, there were no weight savings for F_y greater than or equal to 74 ksi. This occurs when the strength of all compact sections are controlled by the plastic moment, neglecting the limit on yield strength for the compact section criteria in the 1st Edition of the LRFD Bridge code.

For the Delaware Bridge, the maximum negative bending section occurred at the third pier. The web slenderness requirements were not satisfied for all sections; consequently, all the girder cross-sections were designed as noncompact. Homma and Sause (1995) found they could achieve weight savings up to yield strengths of 120 ksi with and without the consideration of the Category C fatigue detail.

Homma and Sause (1995) concluded that HPS70W could be used effectively in design under the 1st Edition of the AASHTO LRFD Bridge Specifications. They suggested that effective and economical use of steels with yield strengths above 70 ksi may be possible through further research of the upper limit of yield strength for the use of compact sections and developing new fatigue resistant connection details for attachment of transverse stiffeners and diaphragm connection plates.

Barker and Schrage (2000)

Barker and Schrage (2000) developed six alternative designs for an existing bridge site in Missouri for comparison of steel weight and costs. The bridge selected was a symmetrical 153-ft two-span continuous bridge with a 24° skew and a roadway width of 68 ft. Three girder spacings were examined: 9 girders at 8-ft spacing, 8 girders at 9-ft 3-in. spacing, and 7 girders at 10-ft 8-in. spacing. The designs include three homogeneous HPS70W girders, two homogeneous 50W girders, and one hybrid 50W / HPS70W girder. The designs were developed using the AASHTO Standard Specifications (16th Ed.) Load Factor Design method.

They performed a comparison of all designs for weight and cost and found that as a girder line is removed there were significant weight savings for the total system. Also, the HPS70W girders were lighter than the conventional Grade 50W steels for all configurations. They found the HPS70W webs required significantly fewer intermediate transverse stiffeners than the 50W designs. Thus, from a weight savings standpoint, HPS70W steel bridges were found to offer advantages over traditional grade steels.

For the cost comparisons, designs were sent to a fabricator who estimated the material, transportation, and erection costs for the Grade 50W steel to be \$842/ton and the HPS70W steel to be \$1142/ton. What this represents is an increase of 35.6 percent between the two grades of steel. They speculated that an as-rolled, i.e. a steel that need not be processed using Q&T, HPS steel will cost 15 percent more (\$968/ton) than Grade 50W steel.

They found that the 7-girder hybrid design offered the lowest cost, compared to the 9-girder and 8-girder designs, for both current costs and projected costs. The homogeneous HPS70W designs proved to be more expensive for the current costs, with the exception of the 7-girder HPS design, which only showed a 0.4 percent benefit as compared to the 9-girder system.

On the basis of the above results, Barker and Schrage (2000) concluded that greater girder spacing leads to improved bridge economy through reduced fabrication, erection, fewer diaphragms and stiffeners, and lighter steel weight. They also suggest that designs with minimal stiffeners and flange transitions offer reduced fabrication costs. Homogeneous HPS70W designs were found to allow for lighter sections but the lighter weight came with an increased cost. Hybrid designs were found to be the most economical and efficient. Barker and Schrage (2000) state additional benefits will be realized if the premium cost of HPS decreases.

Horton, Power, Van Ooyen, and Azizinamini (2000)

Horton et al. (2000) performed a parametric design study focused on investigating the economy of incorporating HPS70W in traditional girder designs. Designs were performed for a two-span continuous I-girder bridge with the following variable parameters:

- Two cross-sections: 5 girders at 9-ft and 4 girders at 12-ft spacing
- Span lengths: 150 ft, 200 ft, and 250 ft
- Yield strength: F_y = 50 and 70 ksi
- L/D ratios: 25 - 40 (where *D* was defined as the web depth)

The designs incorporated all Grade 50W, all HPS70W, and five different hybrid configurations. Designs were performed using the AASHTO LRFD Bridge Design Specifications (AASHTO, 1998), 2nd Edition with interims through 2000, and were optimized for least weight. Since deflection limitations are considered optional in the LRFD Design Specifications, they were not considered in the initial girder optimization. Final girder designs were submitted to fabricators for cost comparisons.

Optimal girder depths in these studies were found to vary for Grade 50W and HPS70W designs. The HPS70W designs for optimal depth were typically shallower than the Grade 50W steels with total steel weight savings of approximately 17 percent. As the web depths become shallower than the optimal depth, HPS70W offered increased weight savings. Horton et al. (2000) also found eliminating one girder line showed increased weight savings from 3 to 10 percent and was consistent for both HPS70W and Grade 50W steel.

Horton et al. (2000) reviewed the final designs for the deflection limitation of *L*/800. None of the 50W designs exceeded the deflection limit, but the HPS70W designs failed the limitation at some web depths below optimal depth and one design failed at optimal depth. They suggest that if the deflection limitation was considered for HPS70W, it may control designs for shallow web depths.

The final designs for the homogeneous and hybrid designs were also investigated for cost with an average price of 0.61 $/lb for 50W steel and 0.75 $/lb. for HPS70W used for the comparison. For this study, the hybrid design that proved to be most economical was the combination of Grade 50W for all webs, Grade 50W positive moment top flanges, and HPS70W negative moment top flanges and all bottom flanges. At optimal depth the 50W designs were found to be more economical than the HPS70W designs; however, at shallower web depths the HPS70W designs were the least expensive.

To determine if results would be similar using AASHTO LFD, Horton et al. (2000) generated several designs for one of the bridge configurations using Grade 50W and HPS70W steels. The relative cost differences between the two grades of steel were similar to trends seen in the LRFD designs.

Horton et al. (2000) concluded that the most economical choice for all spans and girder spacings was a hybrid design. HPS70W steel was determined to have a 20 percent greater cost than 50W steel at optimal depth, thus cost savings could only be realized at shallower depths. They also found that girder efficiency was increased with the allowance of 70-ksi steel to reach the plastic moment in the 1999/2000 AASHTO LRFD interims.

SCOPE OF CURRENT STUDY

The objective of this study is to develop cost comparisons for HPS70W and Grade 50 girder configurations. This study considered the design of a fixed three-span bridge with span lengths of 175 ft, 220 ft, and 175 ft., shown in Figure 1 (Clingenpeel, 2001). This is a five-lane bridge with a median barrier and sidewalks on both sides. The three configurations examined in this study were 7 girders spaced at 11-ft 3-in., 8 girders spaced at 9 ft 9 in., and 9 girders spaced at 8 ft 6 in. Figure 2 shows a framing plan for each of these geometries. Additionally, for each of the girder spacings, there are 5 alternative configurations of plate material within the girder: homogeneous HPS70W, homogeneous Grade 50W, and three hybrid configurations, shown in Figure 3. The three hybrid configurations were:

1. 50-ksi flanges in the positive moment region and web and 70-ksi flanges in the negative moment region,
2. All 70-ksi flanges and 50-ksi web, and
3. 70-ksi top and bottom flanges in the negative bending region and in the bottom flange of the positive bending regions, 50-ksi steel in the top flange of the positive bending region and in the web.

DESIGN ASSUMPTIONS

These studies were conducted using a fixed span-to-depth ratio, *L/D*, of 27.5. Preliminary efforts, discussed later in the paper, showed that this *L/D* ratio corresponded to the least weight solution for both the Grade 50 and the HPS 70W designs. Also, to fully examine the cost benefits of using hybrid designs, flange plate width transitions were used only between bolted field splices. Parameters that were held constant throughout the study include:

- HL-93 live loading,
- Stay in place metal forms = 15 psf,
- Future wearing surface = 25 psf,
- Parapet plus railing weight = 330 lb/ft,
- Median = 300 lb/ft,
- Sidewalk loading = 640 lb/ft,
- Cross frame spacing: 7 spaces at 25 ft in span 1 and 3, and 10 spaces at 22 ft in span 2,
- 5 percent increase in dead weight for miscellaneous steel,
- Interior girder design, and
- Class I roadway for fatigue

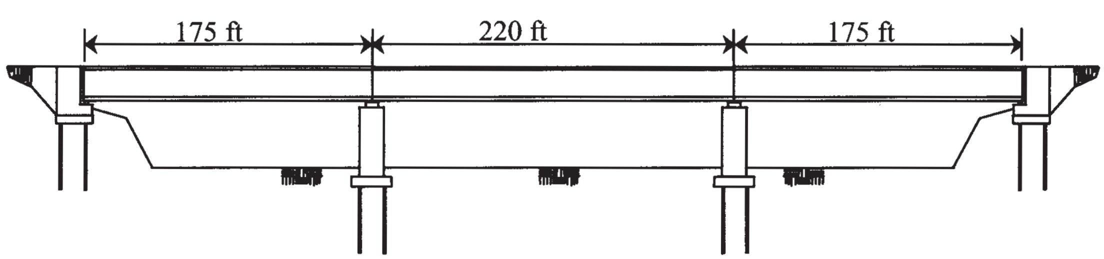
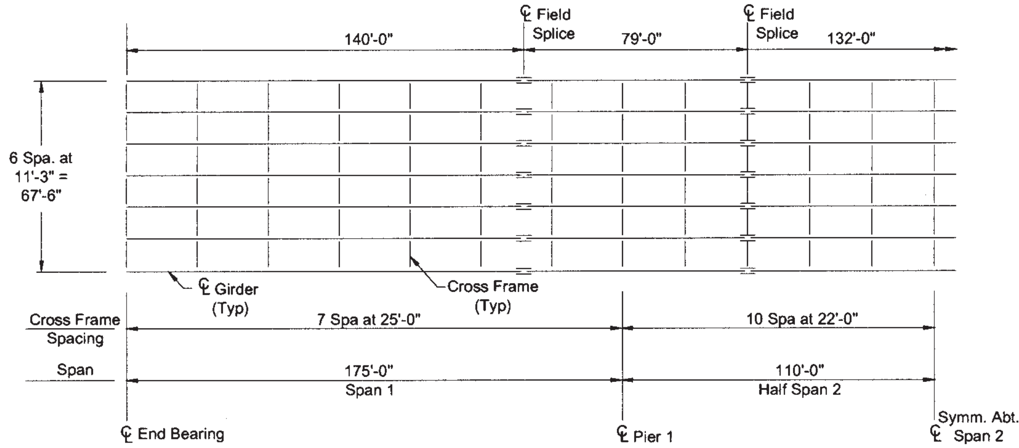
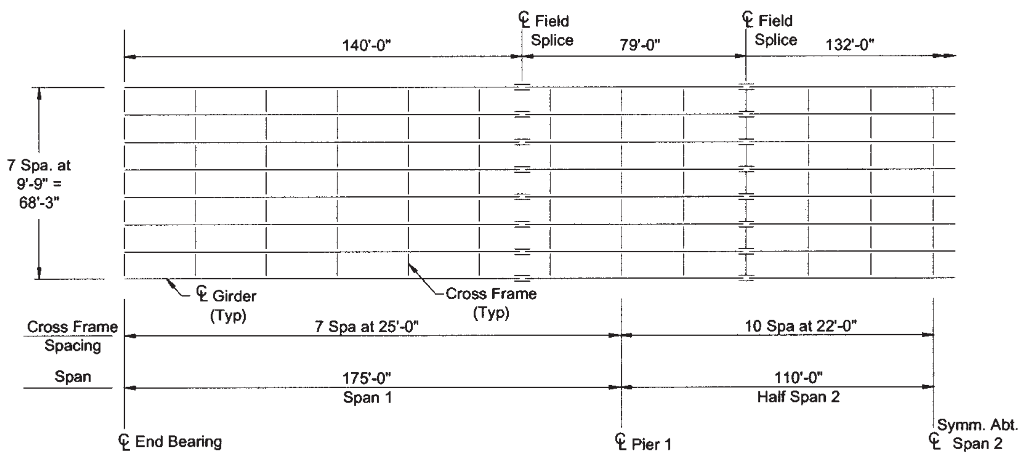


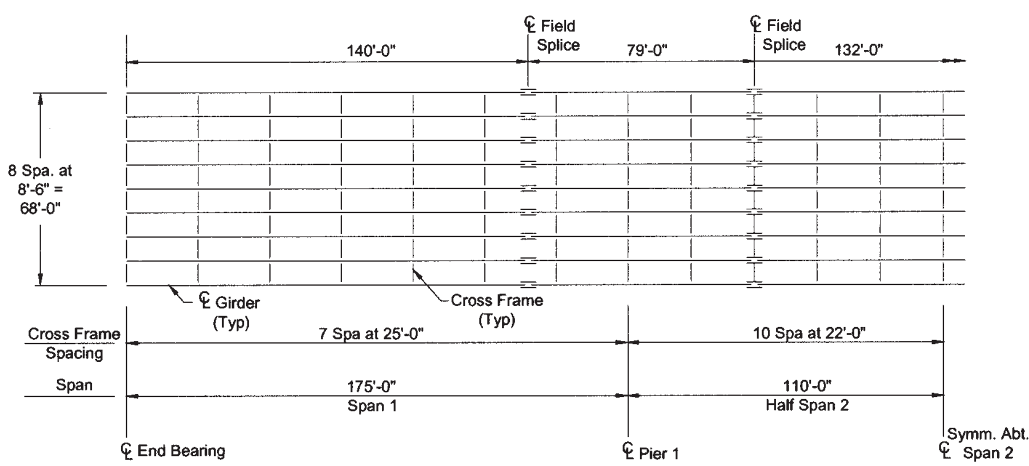
Fig. 1. Bridge profile.



(a) 7-girder



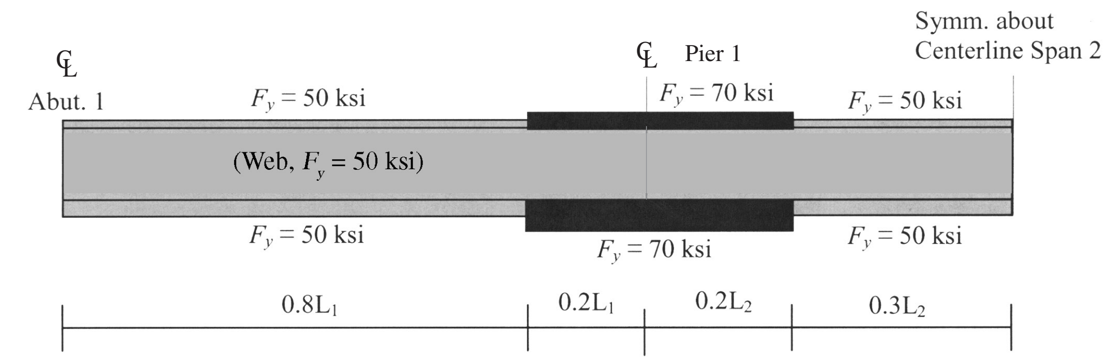
(b) 8-girder



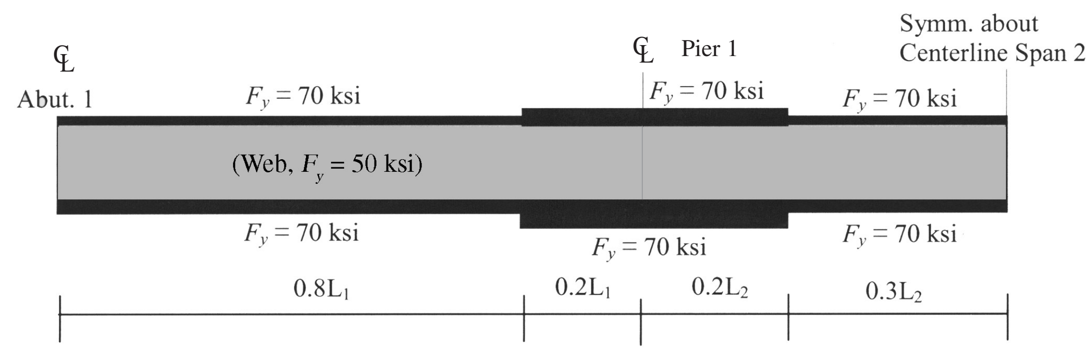
(a) 9-girder

Fig. 2. Girder Framing Plans: (a) 7-girder, (b) 8-girder, (c) 9-girder.

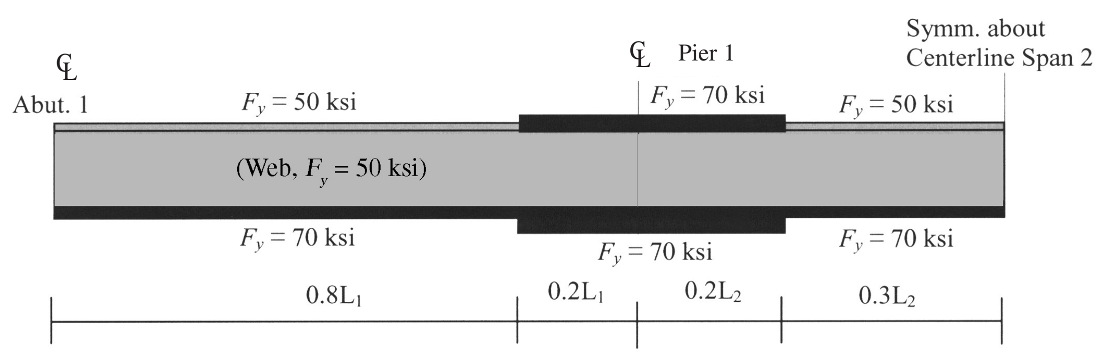
The total deck thickness was taken to be 8½ in. for the 8- and 9-girder cross section and 10 in. for the 7-girder cross section. These deck thicknesses were selected as conservative upper bounds on those that might be used for the respective girder spacings. However, some state transportation departments may choose to use decks with thicknesses less than those incorporated in these studies. Therefore, to better understand the influence of the deck thicknesses used on the final optimized girder weight, a refined set of studies was focused on using varying deck thicknesses for the 7-girder configuration and for the 9-girder configurations (see Figure 2) using hybrid configuration 3 (see Figure 3). For the 7-girder configurations, 8-in., 9-in., and 10-in. thicknesses were analyzed and for the 9-girder configuration, 7.5-in. (the minimum permitted by AASHTO) and 8-in. configurations were analyzed. Results of these efforts showed negligible influence on final design economy. The largest difference in final superstructure steel weight was approximately 1 percent between the 7-girder 8-in. and 10-in. configurations. It should be noted that these results can not be specifically extrapolated to other span configurations



(a) Hybrid Girder 1



(b) Hybrid Girder 2



(c) Hybrid Girder 3

Fig. 3. Hybrid Configurations: (a) Hybrid Girder 1, (b) Hybrid Girder 2, (c) Hybrid Girder 3.

Table 1. Total System Weight of Design Alternatives, Tons

Configuration	7 girders	8 girders	9 girders
Homogenous 50 ksi	684	701	752
Homogeneous 70 ksi	595	631	687
Hybrid 1	641	656	703
Hybrid 2	622	637	693
Hybrid 3	622	637	687

as they are sensitive to girder spacing and span length as these parameters influence the dead-to-live load ratio.

The haunch (which includes the top flange thickness) was taken as 2 in., unless section requirements cause the top flange to be thicker than 2 in. In these cases, the haunch was increased to the thickness of the top flange.

Relative costs were computed for each of the resulting designs. Cost data for plate material were taken from Horton et al. (2000), which stated that Grade 50W is 0.61 $/lb. and Grade HPS70W is 0.75 $/lb. As reported in the Horton et al. (2000) study, these values were obtained from averaging unit costs supplied by three anonymous fabricators. The prices were reported to include mill shipment costs, shop drawing preparation, and waste material and were reported not to include:

- Non-destructive testing costs (NDT),
- Cost of shear connectors,
- Cost of cross frames,
- Cost of field splices,
- Concrete deck,
- Shipment costs to job site, and
- Any associated painting costs

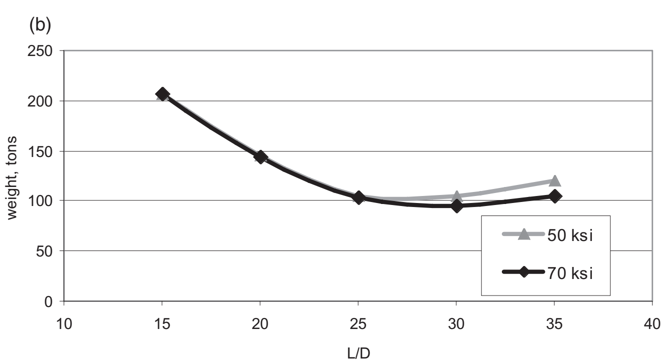


Fig. 4. Optimal L/D.

DEVELOPMENT OF OPTIMIZED *L/D* RATIO

A preliminary study was performed, using only the homogeneous 50- and 70-ksi configurations, to determine an approximate "optimal depth ratio" that was then used for all designs generated in this investigation (Clingenpeel, 2001). Five initial *L/D* ratios were used for the preliminary study: 15, 20, 25, 30, and 35. The resulting optimal *L/D* ratio was 27.5, taking *L* as 220-ft and *D* as the total superstructure depth, see Figure 4. This corresponds to a web depth of 84 in., which was used in all designs included in this work. While changes in *L/D* may result in different girder weights, resulting trends will not be significantly affected.

DESIGN APPROACH

To begin a design, an initial flange width was selected such that d/b_f fell in the range of 3.00 to 4.5 where d is the web depth and b_f is the flange width. These target d/b_f ratios resulted from previous research by Barth, White, and Bobb (2000). After a preliminary girder was chosen, the appropriate information was input into MDX Curved & Straight Bridge Design & Rating Software® to obtain an optimized section (MDX, 2001). Various limits were checked by hand calculations to ensure validity of the software package.

For all designs, in the negative moment regions, a flange thickness transition was included 15 ft away from the pier if a weight savings of more than 900 lbs was achieved. As mentioned previously, flange plate width transitions were allowed only between bolted field splice locations.

The web thickness was selected by incorporating a partially-stiffened approach. That is, for a given girder, a web thickness, t_w, that would require no transverse stiffeners was determined. This thickness was then reduced by 1/16-in. to 1/8-in., depending upon the resulting stiffener layout and weight savings. The web thickness was held constant for a given girder.

DESIGN COMPARISONS

After developing the designs, a comparison of weights was made and is presented in Table 1. The weight given is for bare steel of all girders in a given cross section for all material configurations considered.

Table 1 illustrates several trends that were consistent throughout this effort. Removing a girder line consistently reduced total system weight. For example, the 9-girder Hybrid 3 system weighs 687 tons and the 7-girder Hybrid 3 system weighs 622 tons, a savings of 10 percent. As girder spacing decreases, there is a significant increase in bridge costs due to erecting an additional girder, increased substructure costs, cost of an entire line of cross-frames, and other painting, fabrication, and maintenance costs of an additional girder line.

As expected, the all HPS70W alternative produced the lightest designs for all given cross sections compared to the all Grade 50W girders. For all cross sections the 50-ksi bridge resulted in the heaviest design. The hybrid configurations exhibited weight savings compared to the 50-ksi designs, with Hybrid 2 and Hybrid 3 weighing approximately the same for all cross sections. For example, the Hybrid 2 and 3 alternatives weigh 622 tons for the 7-girder cross section and 684 tons for the all 50-ksi alternative. This is a difference of 10 percent.

COST ANALYSIS

Figures 5 through 7 show a comparison of the cost data for the three girder configurations. Several trends can be observed from these plots. In all cases, the 70-ksi bridge resulted in the lightest but most expensive design. The cost for the 7-girder bridge using the homogeneous 70-ksi alternative was $902,903 compared to $786,484 for the Hybrid 3 alternative, see Figure 5. This is a savings in cost of 15 percent, which is the average cost difference between these two material configurations for all cross sections. The use of HSP70W for all plate steel in a girder results in inefficient use of the material in low stress regions.

Although all hybrid configurations exhibited weight savings compared to the 50-ksi designs, Hybrid 2 was always the most expensive hybrid configuration. This hybrid configuration consists of all 70-ksi flanges and 50-ksi webs. For most cases, Hybrid 3, which consists of all 70-ksi bottom flanges and top flange in the negative moment region and 50-ksi in the top flange positive moment region and web, resulted in a design that weighed the same as Hybrid 2, see Table 1. However, as a result of the difference in grades of steel in the top flange, the Hybrid 2 always resulted in a more expensive design. For example, Hybrid 2 cost 5 percent more than Hybrid 3 for the 7-girder cross section as shown in Figure 5.

Removing a girder line not only decreases the weight of the system but also the cost. For example, the cost savings between the Hybrid 3 alternatives for the 7 and 9-girder configurations is 13 percent; see Figures 5 and 7, respectively. Therefore, the most economical and beneficial cross section was the 7-girder Hybrid 3 configuration.

CONCLUDING REMARKS

This paper presents a cost analysis of a fixed three-span bridge incorporating various material and girder configurations. An optimized parametric design study was conducted by varying key superstructure design variables. Girders are optimized in this study based on least weight for a fixed span-to-depth ratio. Total weights and associated estimated girder costs were compared for resulting designs. Predominant trends are summarized below

- The most efficient material combination was found to be the Hybrid 3 configuration, which consists of 70-ksi bottom flanges and top flange of the negative moment

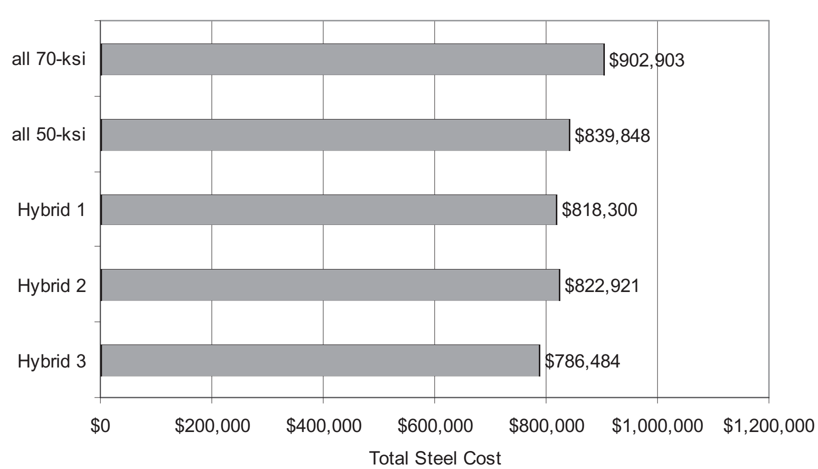


Fig. 5. Total system cost for 7-girder bridge.

region and 50-ksi top flange in the positive moment region and in the entire web.

- Designs incorporating hybrid girder configurations produced girders with weights between similar homogeneous 50-ksi and 70-ksi designs, respectively. In some cases, the hybrid designs weighed approximately the same as the all 70-ksi design. However, cost considerations showed hybrid configurations to be the most economical.

- Removal of a girder line yielded significant weight savings for all yield strength configurations. The wider girder spacings also resulted in cost savings.

- The homogeneous 70-ksi alternative always resulted in the most expensive design.

- Hybrid 2, which incorporated all 70-ksi flanges and 50-ksi webs, resulted in the most expensive hybrid configuration.

- The most economical design was always a hybrid configuration for all girder spacings considered.

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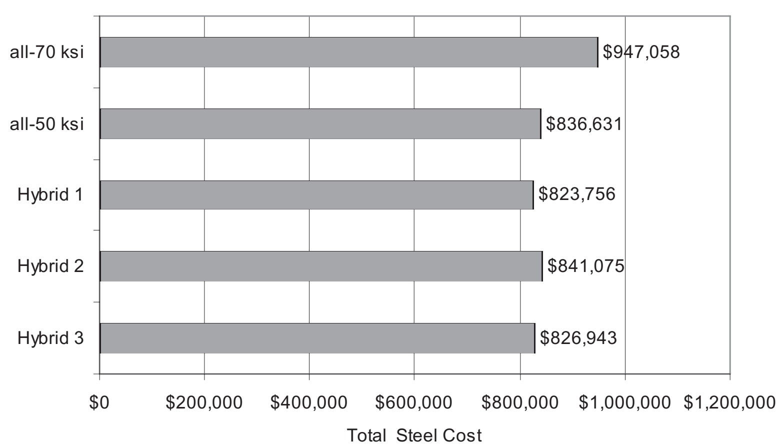

Fig. 6. Total steel cost for 8-girder bridge.

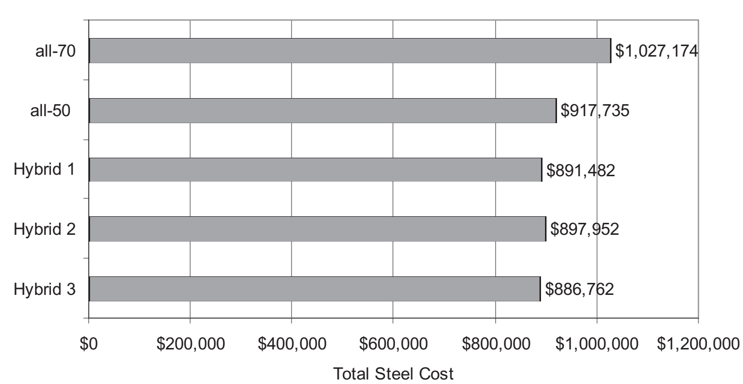

Fig. 7. Total steel cost for 9-girder bridge.

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