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Potential for high performance steel in plate girder bridge designs under the LRFD code

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POTENTIAL FOR HIGH PERFORMANCE STEEL IN PLATE GIRDER BRIDGE DESIGNS UNDER THE LRFD CODE

by

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Abstract

This research is a part of the project "Innovative Bridge Designs Using Enhanced Performance Steels" sponsored by Federal Highway Administration. The objective of the project is to determine the feasibility of using high performance steel in highway bridges.

The thesis focuses on the potential for high performance steel in plate girder bridges designed for the new LRFD bridge design code. The potential for high performance steel is studied by comparing plate girder bridge designs using high performance steel with those using conventional steel.

Assumed properties of high performance steel are established in the thesis so that the potential for high performance steel in plate girder bridge designs can be studied. To compare designs using high performance steel with designs using conventional steel, optimum designs are developed. To establish optimum designs, a computer program, based on the LRFD code, was developed and used. Base case designs using conventional steel are established for two existing bridges: a simple and a continuous composite I-section plate girder bridge. These two bridges are then re-designed using high performance steel.

The comparison of the base case designs with the redesigns reveals the potential for high performance steel in plate girder bridges under the LRFD code. The study shows that weight reduction with increasing the yield strength can be obtained up to a yield strength of 70 ksi. However, 85 ksi steel is not effectively used because of a code-specified limit on yield strength for compact sections. For 100 ksi steel, weight reduction is obtained in some

cases relative to 70 ksi steel. However, for yield strength above 100 ksi, the fatigue limit state is the most critical requirement, and it usually prevents a weight reduction with increasing the yield strength.

In conclusion, 70 ksi steel is the most promising one under the current system. The study also shows that the possibility of effective use of higher yield strength steel (85 ksi or more), if the limitation ~~on~~ the yield strength for compact sections is eliminated. To make more effective use of high performance steel, new fatigue resistant bracing and stiffener concepts are proposed.

1. Introduction

This research is a part of the project " Innovative Bridge Designs Using Enhanced Performance Steels" sponsored by Federal Highway Administration (FHWA), and conducted at the Engineering Research Center for Advanced Technology for Large Structural System (ATLSS) at Lehigh University. The project is being conducted with Modjeski and Masters Inc., a well known bridge engineering firm, and the University of Michigan. The objective of the project is to determine the feasibility of using high performance steel in highway bridges.

This project can be divided into two steps. The first step of the project is to focus on the potential for high performance steel in current bridge designs. The second step is to study the development of new, innovative bridge design concepts which make more effective use of high performance steel.

The project has the following tasks:

(1) Review domestic bridge designs

(2) Review international bridge designs

(3) Study the potential for high performance steel in current bridge designs

(4) Study the effect of material properties on structural reliability

(5) Develop innovative design concepts

(6) Develop recommendations for high performance steel research

(7) Develop code recommendations

This thesis is part of the work on task (3) "Study the potential for high performance steel in current bridge designs". The thesis focuses on the design of high performance steel plate girder bridges under the recently adopted LRFD code.

1.1 Objectives

The objectives of the research described in this thesis are:

(1) To study the potential for high performance steel in plate girder highway bridges.

(2) To develop tentative properties of high performance steel to be used in work toward objective (1).

1.2 Scope

The research reported here is based on designs for two highway bridges: (1) a simple composite steel plate I-section girder bridge, and (2) a continuous composite steel plate I-section girder bridge. These bridges were originally designed by Modjeski and Masters, Inc. in the 1980's, and currently are part of Interstate Highway I-78 in Pennsylvania. The bridges are redesigned according to the LRFD code to provide base case designs in currently available conventional steel. The redesigns in high performance steel are then developed, and comparisons are made. The new LRFD bridge design code [AASHTO 1993], adopted in 1993 is used because it is the standard for future highway bridge design specifications in the United States.

1.3 Approach

The research presented in this thesis consists of 5 steps:

(1) Establish expected properties of high performance steel for highway bridges

(2) Establish base case designs using conventional steel for two existing bridges:

- A simple composite steel plate I-section girder bridge
- A continuous composite steel plate I-section girder bridge

(3) Redesign the base case designs using high performance steel of different strength levels

(4) Compare designs using high performance steel of different strength levels with those using conventional steel

(5) Determine limiting code requirements for these designs

The purpose of this thesis is to discuss advantages and disadvantages of high performance steel in plate girder bridges designed for the new LRFD bridge design code. First, the expected properties of high performance steel are discussed so that the potential for high performance steel can be studied in the following chapters.

1.4 Outline of the thesis

The thesis is organized as follows. In Chapter 2, the expected properties of high performance steel are discussed to allow appropriate assumptions to be made for the reminder of the study. Before addressing the potential of high performance steel, a brief overview of new LRFD code is described in Chapter 3, and the analysis approach is explained in Chapter 4. In Chapter 5, the potential for high performance steel is investigated.

Discussion, conclusions, and recommendations are presented in Chapter 6 and Chapter 7.

2. Expected Properties of High Performance Steel

In this chapter, expected properties of "high performance steel" are discussed. This discussion includes high strength steels that are currently available in the U.S., as well as steels that are used in Japan. Mechanical properties such as strength and toughness, and other properties such as weldability and corrosion resistance are considered.

In the U.S., there were significant developments in high strength steels for bridges in 1950's. Quenched and tempered steels such as T-1 steel developed by U.S. Steel were the result of this effort. The current ASTM A-514 specification [ASTM 1991] covers high strength quenched and tempered alloy steel suitable for welding. Its specified minimum yield strength is 100 ksi (690MPa) for thicknesses less than 2.5 inches (65mm). Other high strength steels for bridges are shown in Table 2.1. Most bridges in the U.S. employ ASTM A36 and A572 Grade 50 [ASTM 1991], whose yield strengths are 36 ksi (250MPa) and 50 ksi (345MPa) respectively.

"High performance steel" requires important properties other than high strength. Past experience with the application of high strength steels such as T-1 steel indicates the potential for problems such as cracking in welded connections . One of the reasons for these welding problems is the high carbon or carbon equivalent of the chemical composition of the steel. Conventional high strength steels contained relatively high carbon levels, because carbon is the principal element used to increase the hardness and tensile strength of the steel. Other properties which are thought to be important for structural steels in bridges are: fracture toughness, ductility, fatigue resistance, and corrosion resistance.

Although all these properties of high performance steel will influence the safety and service performance of steel highway bridges, the analysis in this study is based mainly on the strength of the steel, because this is the primary property considered in the design code. However, high strength steels cannot be used without considering fabricability and serviceability. The purpose of this chapter is to establish the expected properties of high performance steel. Each property and its related processing or chemical composition are reviewed to propose expected properties of high performance steel. Strength, weldability, toughness, fatigue, and ductility are considered. These are discussed in the following sections. Corrosion resistance is also important, but is not considered in the present discussion.

2.1 Strength

As described before, the greatest problem with conventional high strength steels is the high levels of carbon and carbon equivalent required to achieve high strength. Another effective method to achieve high strength, hardness, and toughness is heat treatment. This method is broadly employed by many steel producers.

Quenching and tempering is a typical example of heat treatment. This process consists of heating the steel to the austenitizing temperature, holding it in the transformation temperature range, and quenching it in water [Krouse 1989]. After quenching, the steel is tempered at an appropriate temperature to relieve internal stress and improve ductility and toughness.

ASTM A-514, "High yield strength, quenched and tempered alloy steel plate, suitable for welding" [ASTM 1991], is a steel processed by this treatment. The most advanced method for obtaining high strength is TMCP (Thermo Mechanical Controlling Processing) which is a continuous on-line process of controlled rolling and controlled cooling. TMCP has been successfully employed by Japanese steel producers.

Fig. 2.1 shows the yield strength and tensile strength of steels for bridge construction specified by ASTM. In this study, the yield strength of high performance steel is assumed to be between 50ksi (345MPa) and 100ksi (690MPa) based on the fact that the steels in this yield strength range are both desirable for current bridge designs and feasible for manufacture with current steel making technology without sacrificing weldabilty. Steel with a yield strength of 120 ksi (827MPa) is also considered in the study reported in Chapter 5, to determine if benefits could be gained by using steel with strength above 100 ksi (690MPa).

2.2 Weldability

Weldability is an important property for high performance steel that considers both ease of welding and performance of welded joints during service life.

In general, the ease of welding is highly dependent on the chemical composition of the steel, especially the carbon content and the carbon equivalent. The higher the carbon or carbon equivalent, the more difficult welding becomes. Therefore it is important to achieve high strength without significantly increasing carbon or carbon equivalent. Thus, quenched and tempered and TMCP steels are the most promising high performance steels.

Fig. 2.2 provides an example in which TMCP steels show an advantage compared with conventional as-rolled steels [Nippon Steel 1992]. The TMCP steels shown in the figure are processed with controlled rolling and accelerated cooling. As shown, the carbon equivalent of TMCP steels is much lower than that of conventional as-rolled steel at same strength level.

Fig. 2.3 shows the upper limit of the carbon and the carbon equivalent specified by ASTM for bridge construction steels. Most of the conventional high strength steels, such as ASTM A709-100 or A709-70, are allowed to contain relatively high levels of carbon and carbon equivalent. Recent steel making developments permit the carbon content of high strength steel to be drastically lower. For example, HSLA-80 and HSLA-100 are newly developed steels which contain less than 0.10% carbon and show excellent weldability [Dexter 1992].

Diagrams such as the one shown in Fig. 2.4, proposed by Gravile [1976], help to define the ease-of-welding criteria for high performance steel. This figure indicates the influence on susceptibility to HAZ (heat affected zone) cracking of welded steel plate. Susceptibility to HAZ cracking is categorized in 3 zones based on the carbon and carbon equivalent: (1) susceptibility is high under all conditions, (2) susceptibility depends on welding conditions, and (3) welded plate is not susceptible to HAZ cracking. The performance of welded joints during fabrication and service life is related to this feature.

As shown in Fig. 2.4, TMCP steels [e.g.,. Nippon Steel 1992], HSLA-80, and HSLA-100 fall in zone (3) which is the most desirable zone. It is reasonable to assume in this study that the steel within the zone(3) is defined as "high performance steel" (Fig. 2.8).

2.3 Fracture Toughness

Fracture toughness is closely related to the micro-structure of the steel. This property is often measured by Charpy V-notch (CVN) tests specified in ASTM A 673. High CVN absorbed energy and low transition temperature are indicators of excellent fracture toughness.

Fig. 2.5 shows the specified CVN absorbed energy for each operating temperature in the ASTM specification. Higher absorbed energy at lower testing temperature indicates better toughness. However, it is not clear how much CVN absorbed energy is required for high performance steel.

To discuss this in detail is beyond the scope of this thesis. However it can be said at least that newly developed TMCP steels often show extraordinary high CVN absorbed energy. Fig. 2.6 is a typical example [Nippon Steel 1992] of the results of impact tests for TMCP steel. The CVN absorbed energy shows more than 100J (73ft lbf) at -40°C. This is extremely high, considering the highest CVN absorbed energy specified in ASTM A709 is 48J (35ft lbf) at -34°C for bridge members that are defined to be fracture critical (i.e., that are determined to require special protection against fracture).

2.4 Ductility

Ductility is also an important property for structural steel. Ductility depends on the ultimate elongation and the ratio of yield strength to tensile strength of the steel. In general, as the yield strength of steel increases, the ratio of yield strength to tensile strength (the yield ratio) increases. A higher yield ratio leads to lower ductility of structural members. It is important to maintain a low yield ratio even for high strength steels. In Fig. 2.1, broken lines represent yield ratios of 0.9, 0.85 and 0.8. Although the current ASTM specification does not specify the upper limit of yield ratio, it is necessary to guarantee some level of ductility especially for high strength steels. In this thesis, a yield ratio of 0.85 is assumed for all high performance steels (Fig. 2.7).

2.5 Fatigue Resistance

Generally fatigue resistance is not dependent on the material itself. Fatigue resistance is categorized in two parts: the resistance against crack initiation and the resistance against crack propagation. Neither of the resistances is much affected by material properties. Therefore, the material property of fatigue resistance is not defined specially for high performance steel. The fatigue resistance of high performance steel is assumed to be similar to that of currently available steels (Fig. 2.7).

2.6 Summary

In summary, the term "high performance steel" is used in this thesis to refer to the steel whose properties are as follows:

(1) Yield strength between 50ksi (345MPa) and 100ksi (690MPa) (Fig. 2.7).

(2) Carbon and carbon equivalent within zone 3 in Fig. 2.8.

(3) High CVN absorbed energy.

(4) Fatigue resistance similar to currently available structural steels.

(5) Yield ratio no more than 0.85 (Fig. 2.7).

3. Overview of the LRFD Code Developed by NCHRP in 1993

This study is based on the new LRFD (Load and Resistance Factor Design) bridge design specification which was approved in 1993. Highway bridges of the future will be governed by this new bridge design code. Therefore, it is thought to be appropriate to study the potential for high performance steel using this code. Before addressing the potential for high performance steel, a brief overview of the new bridge design code is provided in this chapter.

3.1 The Basis of LRFD Methodology

The basis of the LRFD methodology is expressed by the following equation.

$$\eta \sum \gamma i \; Qi \leq \phi \; Rn \qquad (3.1)$$

where

η: $\eta = \eta_R \, \eta_D \, \eta_I$

η_R: Coefficient of redundancy

η_D: Coefficient of ductility

η_I: Coefficient of operational importance

γi: Load factor

Qi : Force effect

ϕ: Resistance factor

Rn: Nominal resistance

Equation 3.1 states that the factored resistance shall be greater than or equal to the factored load (force effect). Coefficient η accounts for ductility, redundancy, and operational importance. These are significant aspects affecting the margin of safety of bridges.

Equation 3.1 shall be satisfied for all limit states which are considered of equal importance. There are four types of limit states considered by the code: service limit states, fatigue and fracture limit states, strength limit states, and extreme event limit states. Details about the limit states considered in this thesis are discussed in Chapter 4.

(1) Service Limit States

Service limit states are intended to allow the bridge to perform acceptably throughout its service life. These limit states provide restrictions on stress, deformation, and crack width under regular service conditions. In this thesis, one service limit state is considered; the control of permanent deflection, which is intended to prevent objectionable permanent deflections due to severe level traffic loading. This limit state is different from the elastic deflection criteria which is intended to avoid undesirable effects of excessive elastic deflection or vibration. The elastic deflection criteria is considered optional in the new LRFD code.

(2) Fatigue and Fracture Limit States

Fatigue limit states are intended to limit crack growth under repetitive loads to prevent fracture during the design life of bridges. Fatigue limit states are a restriction on the live load stress range under regular service conditions which reflects the expected number of significant live load stress cycles. Fracture is controlled by the material toughness requirements of the AASHTO material specification, and by special fabrication procedures for fracture critical members.

(3) Strength Limit States

Strength limit states are intended to ensure that strength and stability are provided to safely resist the loads that a bridge will experience in its design life. Strength limit states for composite steel I-section bending members depend on whether the member is a compact section or a non-compact section according to the properties of the member, such as web slenderness, compression flange slenderness, and compression flange bracing.

(4) Extreme Event Limit States

Extreme event limit states are intended to ensure the survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle or ice flow.

3.2 Load Combinations and Load Factors

Bridges are designed for the applicable combinations of factored loads specified for each limit state. Table 3.1 shows several load combinations and load factors which are used for the calculation of force effects. The LRFD code includes other limit states. Only these shown in Table 3.1 were considered in the study. Strength-I is the limit state for a basic load combination relating to the normal vehicular use of the bridge without wind.

Service-II is the limit state for a load combination whose objective is to prevent excessive yielding of steel members under vehicular live load. Fatigue is the limit state to control fatigue and fracture under vehicular live load including dynamic response.

3.3 Design Vehicular Live Load

The design vehicular live load for highway bridges in the LRFD code consists of a combination of the design truck, design tandem, and design lane load.

(1) Design truck

The design truck consists of a pair of 32.0 kip axles and an 8.0 kip axle. As shown in Figure 3.1, the spacing between two 32.0 kip axles is varied from 14.0 ft to 30.0 ft to produce the extreme force effect.

(2) Design tandem

The design tandem consists of a pair of 25.0 kip axles spaced 4.0 ft apart. The transverse spacing of wheels is taken as 6.0 ft.

(3) Design lane load

The design lane load is 0.64 kip/ft which is uniformly distributed in the longitudinal direction. This load is assumed to occupy 10.0 ft transversely.

3.4 Extreme Live Load Force Effect

The extreme live load force effect is the largest value of force in a component determined from the following load combinations.

(1) Effect of the design tandem combined with the effect of the design lane load.

(2) Effect of one design truck with the variable axle spacing, combined with the effect of the design lane load.

(3) For negative moment and reaction at interior piers only, 90% of the effect of two design trucks spaced a minimum of 50.0 ft apart, combined with 90% of the effect of the design lane load.

The extreme live load force effect is included in strength I and service I and II limit states load combinations. The fatigue limit state uses the design truck with a fixed spacing between the 32 kip axles of 30.0 ft to generate the extreme force effect.

4. Analysis Approach

In this chapter, the analytical approach of the study is described. The objective of the analysis is to compare optimal composite steel plate girder designs using high performance steel with optimal designs using conventional steel, so that the potential for high performance steel can be defined. To produce optimum designs under the limitations of the new LRFD code, computer programs were developed. The computer programs are divided into two parts: (1) code conformance analysis and (2) optimization analysis.

4.1 Code Conformance Analysis

A computer program for code conformance analysis was developed to test if a cross section of a composite steel plate I-section girder satisfies the requirements of the new LRFD code. This section outlines the code requirements addressed by the computer program. Table 4.1 shows examples of LRFD code requirements for composite steel I-section bending members which are classified into five main groups: (1) general requirements, (2) strength limit states under service conditions, (3) strength limit states during construction, (4) service limit state, and (5) fatigue limit state.

(1) General requirements

General requirements govern member proportions, depth-to-span ratios, and minimum thicknesses of steel elements. Member proportions are regulated by the moment of inertia of the steel section about the vertical axis, that is:

$$0.1 \leq I_{yc} / I_y \leq 0.9 \tag{4.1}$$

where

I_{yc} : moment of inertia of the compression flange about the vertical axis in the plane of web

I_y : moment of inertia of the steel section about the vertical axis in the plane of web

The LRFD code says that, in the absence of other criteria, an owner may choose to consider traditional minimum depth-to-span ratio criteria, such as the criteria described in the previous edition of the Standard Specification for Highway Bridges by AASHTO [AASHTO 1989]. For steel structures, minimum overall depths for composite steel I-section bending members are 0.040L for simple spans and 0.032L for continuous spans, where L is the span length. Minimum depths for the steel portion of the composite steel I-section members are 0.033L for simple spans and 0.027L for continuous spans. The minimum steel plate thickness is limited to not less than 5/16 in..

In the code conformance analysis, these criteria are investigated for a proposed section. A section must satisfy these criteria before strength limit state requirements are investigated.

(2) Strength limit states under service conditions

Strength limit states for composite steel I-section bending members are classified into three categories: compact composite steel I-sections, non-compact composite steel I-sections, and composite steel sections governed by elastic lateral torsional buckling. Sections that satisfy the web slenderness, compression flange slenderness, and compression flange bracing requirements shown in Table 4.2 are termed compact sections. Compact sections are designed for a nominal bending resistance equal to the plastic moment, Mp, unless otherwise specified. Mp is calculated by the first moment of plastic forces about the plastic neutral axis Sections that satisfy the requirements shown in Table 4.3 are termed non-compact sections. Non-compact sections are governed by the yield stress at the extreme fiber of the steel I-section unless otherwise specified. The yield moment, My, of a composite steel I-section is defined as the bending moment that causes yielding in either steel flange. My is given by the sum of: (1) the moment applied separately to the steel section alone, such as moment due to load applied before composite action develops; (2) the moment applied to the long-term composite section, such as moment due to the dead load of the wearing surface; and (3) the moment applied to the short-term composite section such as moment due to live load. The investigation for the shear strength limit state is also carried out, based on the factored shear resistance which is given by the nominal shear resistance multiplied by the resistance factor (1.0). The nominal shear resistance is given in Table 4.4 for girders with an unstiffened web or a stiffened web.

A section must satisfy the strength limit states under service conditions, before strength during construction is investigated.

(3) Strength limit states during construction

The LRFD code requires composite steel I-section bending members to be investigated for strength and stability in positive and negative bending during sequential deck placement. In this analysis, the moment induced during construction is calculated from the dead load of steel I-section members and the wet concrete used to construct the deck. To simplify the analysis, it is assumed that the entire bridge deck is placed at once. The moments induced in the girders during deck placement are considered to be carried by the steel I-section alone which is regarded as a non-composite section. The section should satisfy the requirements for web slenderness, compression flange slenderness, and compression flange bracing specified for the non-composite steel I-section bending members, as shown in Table 4.5. The nominal bending resistance for the strength limit state during construction is the yield moment with flange stress reduction factors shown in Table 4.6. The yield moment, My, is defined as the moment required to cause the first yielding in either flange in the steel I-section member. The investigation for the shear strength limit state is also carried out using a shear resistance based on the nominal resistance given in Table 4.4.

(4) Service limit state

As described in the previous chapter, the service limit state included in this analysis is a control of permanent deflection. The objective of this criteria is to prevent objectionable permanent deflections due to live loads. The flange stress caused by the factored loading (see Chapter 3.2 for load factors) is limited to:

$$f_f \leq 0.95\, R_h\, F_{yf} \qquad (4.2)$$

where

f_f : elastic flange stress caused by the factored loading

R_h : flange stress reduction factor

F_{yf}: yield stress of the flange

(5) Fatigue limit state

The fatigue limit state is also considered in this analysis. For load induced fatigue, each detail shall satisfy:

$$0.75\,(\Delta f) \leq (\Delta F)_n \qquad (4.3)$$

where

(Δf): live load stress range due to the passage of the fatigue load

$(\Delta F)_n$: the nominal fatigue resistance

The fatigue load consists of the design truck described in the Chapter 3, but with a constant spacing of 30.0 FT between the 32.0 KIP axles. An impact factor of 15% is applied to the fatigue load. The nominal fatigue resistance is taken as:

$$(\Delta F)_n = (A\,/\,N)^{1/3} \geq 1/2\,(\Delta F)_{th} \qquad (4.4)$$

where

A : constant in accordance with the detail category given in the code

N : $N=(365)(75)\, n\, (ADTT)_{SL}$

n : number of stress range cycles per truck passage

$(ADTT)_{SL}$: single lane ADTT(the average number of trucks per lane per day in one direction)

$(\Delta F)_{th}$: constant amplitude fatigue threshold

In the code conformance analysis, all requirements described herein are investigated and a judgement is made whether the section satisfies the code or not. The flowchart of this analysis is shown in Fig. 4.1. The program is useful for identifying the limiting code requirement for a given section and is also used as part of the optimization analysis of composite steel plate I-section girders.

4.2 Optimization Analysis

To compare designs using high performance steel with designs using conventional steel, it is necessary to establish an optimum design for each case. In this study, the lightest cross-section is considered optimum. For this purpose, an optimization by "brute force" iteration is carried out. The section to be optimized is a composite steel plate I-section girder, as shown in Fig. 4.2. Assuming the spacing between girders is constant, there are eight variables to be optimized. If the longitudinal distances such as the distance between compression flange bracing (Lb), and the spacing between transverse stiffeners (d) are also constants, there are only six variables to be considered, those governing the dimensions of the I-section itself: web height (Dg), web thickness (tw), bottom flange width (bfb), bottom flange thickness (tfb), top flange width (bft), and top flange thickness (tft).

The flowchart of the optimization is shown in Fig. 4.3. The iteration concept is simple. First, assume a section arbitrarily. Then there are six variables to be optimized and each variable changes step by step. If the number of values considered for each variable (the number of iterations for each variable) is N, the total number of iterations is N to the sixth power. The plate thickness increments considered in the optimization analysis are 1/16 in. for web plate and 1/8 in. for flanges. The web height and flange width increments are 2 in..

The code conformance analysis described in the previous section is employed in the optimization analysis. The optimization produces a section which satisfies the LRFD code requirements and which is also the lightest weight.

5. Potential for High Performance Steel

In this chapter, the potential for high performance steel in composite steel plate I-section girder bridges is investigated using the optimization analysis discussed in Chapter 4. First, two base case designs of existing bridges are discussed. Then, the procedure for optimizing the bridge girder cross-sections is discussed. Finally, the results of optimization are shown for the two bridges. As described in Chapter 4, several requirements from the new LRFD code are examined in the code conformance analysis including (1) general requirements, (2) strength limit states under service conditions, (3) strength limit states during construction, (4) service limit state, and (5) fatigue limit state.

The discussion of the results is divided into two cases: (1) all requirements are checked except the fatigue limit state, and (2) all requirements including the fatigue limit state are checked. The discussion is divided in this way, because the fatigue limit state is regarded as a critical requirement for steels with higher yield strength. To take advantage of high performance steel, the fatigue limit state is one of the critical problems to be solved, and the result of this study illustrate how critical this limit state is.

5.1 Base Case Bridges

The two base case bridges are taken from Interstate Highway I-78 in Pennsylvania. Table 5.1 outlines the two bridges that are investigated. More information on these bridges is given in Appendix A. The first bridge is I-78 over Lehigh Street, which is 1-span simple composite steel plate I-section girder bridge. The second bridge is I-78 over the Delaware River, which is 7-span continuous composite steel plate I-section girder bridge. Both bridges are constructed of ASTM grade 50 steel which has a 50 ksi yield strength.

The two bridges were originally designed by Modjeski and Masters Inc. in 1980's based on a previous edition of the AASHTO Standard Specification for Highway Bridges that was not based on the LRFD methodology [see, for example, AASHTO 1989]. For the purpose of comparison, the maximum positive moment and maximum negative moment girder cross-sections of these bridges are redesigned in 36 ksi and 50 ksi yield strength steel using the optimization program described in Chapter 4. Then these sections are redesigned using high performance steels with yield strengths of 70 ksi, 85 ksi, 100 ksi and 120 ksi. A section near the flange plate transition in the Lehigh Street bridge was also studied to illustrate the problem of fatigue.

5.2 Optimization of a Section

To illustrate the procedure of optimization, an example is taken from the I-78 bridge over Lehigh Street. The optimization program identifies the lightest section within the limitation of the LRFD code using the procedure described in Chapter 4. The section analyzed is the point of maximum positive bending moment at the center of the span. Fig 5.1 shows how the optimization proceeds as the web varies. The initial section is arbitrarily chosen. Fig. 5.2 (a) shows the section initially chosen for 36 ksi steel and Fig. 5.2 (b) shows the optimized section which is obtained through the analysis. The web height of the initial section is 60 in., and the weight per foot of the initial steel section is 295 lbs/ft. The web height of the optimized section is 90 in., and the weight per foot of the optimized steel section is 250 lbs/ft.

It is important to know which of the code requirements are critical for high performance steel, so the barriers to the use of new steels can be identified. Fig. 5.3 shows an example of the optimized 70 ksi section for the center of the span of the Lehigh Street bridge, and the code requirements that control the section. For example, if the web height is reduced from 74 in. to 72 in., the section violates the service limit state which controls permanent deflection. If the thickness of web is reduced from 3/8 in. to 5/16 in., the section violates the strength limit state during construction in addition to the service limit state. Similarly, the controlling code requirements are identified for other changes in the design variables.

5.3. Case 1: All Requirements Except Fatigue Limit State

As described in Chapter 4, the code conformance analysis includes all code requirements including the fatigue limit state. However, the fatigue limit state was initially excluded to enable its importance to be assessed. This section discusses results when fatigue is not considered.

5.3.1 I-78 over Lehigh Street

Fig. 5.4 and Table 5.2 show the optimized sections for the location of maximum moment at the center of the span of the Lehigh Street bridge. Steels with yield strengths varying from 36 ksi to 120 ksi were considered. Each section is optimized by the procedure described in Chapter 4. The weight per length of the 50 ksi case is taken as 100%, and the results for the other yield strengths are shown relative to this case (e.g., the 36 ksi case is 119%). As seen in Fig. 5.4, a higher steel yield strength usually results in a smaller weight per length. The exception to this occurs between the 70 ksi case and the 85 ksi case. The weight per length of the 70 ksi case is 84%, and that of the 85 ksi case is 86%. This is because the LRFD code specifies an upper limit of 70 ksi for the yield strength of steel for I-sections designed as compact sections. That is, the requirements for compact sections are applicable only when the yield strength is no more than 70 ksi. As a result, the 70 ksi steel section can be designed as a compact section, whereas the 85 ksi steel section must be designed as a non-compact section, and its nominal bending resistance is controlled by the yield stress at the extreme fiber.

The weight per length of the optimized sections is plotted versus the yield strength in Fig. 5.5. As described above, there is a sudden change between 70 ksi and 85 ksi. The broken line represents the hypothetical case when the section is designed as a compact section by ignoring the restriction on steels with yield strength greater than 70 ksi. High performance steel, as defined in Chapter 2, may not require this type of restriction in the code, although further review of this issue is needed.

5.3.2 I-78 over Delaware River

Using the procedure discussed in Section 5.2, sections at the points of maximum positive and negative moment of the I-78 bridge over the Delaware River are investigated. Fig. 5.6 and Table 5.2 show the optimized sections for the location of the maximum positive bending moment which is between the third pier and fourth pier in the 7-span continuous composite steel plate I-section girder. Fig 5.7, which plots weight per length versus yield strength, shows the same tendency as Fig. 5.5 (center of the span of the Lehigh Street bridge). Again, the broken line represents a hypothetical case when the section is designed as a compact section. For the 120 ksi steel, the broken line coincides with the solid line, because the cross-section is governed by the bending strength limit state during construction (deck placement) which is controlled by lateral-torsional buckling.

Fig. 5.8 and Table 5.2 show the optimized sections for the location of the maximum negative bending moment which is at the third pier. The code conformance analysis indicates that every section in this figure is a non-compact section because of the web slenderness requirement. That is, these sections are governed by non-compact section requirements. Therefore, there is no sudden change between 70 ksi and 85 ksi in Fig. 5.9 which shows weight per length versus yield strength.

Both Fig. 5.7 and Fig. 5.9 are the results of optimization considering all requirements except the fatigue limit state. In this case, the potential weight savings from high strength steel are clearly visible, especially for yield strengths between 36 ksi and 70 ksi.

5.4. Case 2: All Requirements Including Fatigue Limit State

5.4.1 I-78 over Lehigh Street

In the results presented in Section 5.3, the fatigue limit state is ignored. However, the fatigue limit state is one of the most critical requirements governing steel plate girder designs. Fig. 5.10 and Table 5.3 show the results of optimization of the girder cross-section at the center of the span of the Lehigh Street bridge including the fatigue limit state. Fatigue of the Category C detail, where the transverse stiffeners (or diaphragm connection plates) are welded to the web plate, is considered in the analysis.

The weight per length of the optimized girder cross-sections is plotted versus the yield strength in Fig. 5.11. There are three lines in the figure which are: (1) the result of optimization including fatigue, (2) the result of optimization without fatigue, and (3) the result of optimization without fatigue and without the upper limit on yield strength of 70 ksi for compact sections. As seen in Fig. 5.11, the weight reduction with increasing yield strength ends at a yield strength of 100 ksi, when the fatigue limit state for the Category C detail is considered. For steels with a yield strength of 100 ksi or more, no weight reduction is possible because of the limit on stress range required for the Category C detail. If the Category C detail is eliminated and only the Category B detail (welds between the web and flanges) is considered, the results of optimization including the fatigue limit state for the Category B detail are identical to the results where fatigue is not considered. If the Category C detail can be eliminated, weight reduction with increasing yield strength will occur up to a yield strength of 120 ksi, although the Category B detail will control eventually.

In Fig. 5.11, the limit imposed by including fatigue forms a horizontal line between 100 ksi and 120 ksi. If the upper limit on yield strength of 70 ksi for compact sections is neglected, the broken line for the section designed as a compact section would indicate the possible weight reduction with increasing yield strength. In that case, the fatigue limit state of the Category C detail is a more critical problem. By extrapolating the horizontal line for the fatigue limit, it can be seen that weight reduction with increasing yield strength will end at a yield strength of approximately 80 ksi because of the fatigue limit state, when the upper limit on yield strength for compact sections is neglected.

The results of optimization, including the fatigue limit state, for a cross-section near the flange plate transition in the Lehigh Street bridge is obtained (Fig. 5.12). This cross-section is located 20 ft from the abutment, which is 18% of the bridge span from the abutment. This is a more critical section for fatigue than the center of the span because of the higher ratio of live load to dead load. As seen in Fig. 5.12, the weight reduction with increasing yield strength ends at a yield strength of 70 ksi, if the fatigue limit state of the Category C detail is taken into account. Thus, 70 ksi is the maximum yield strength which can be effectively used.

5.4.2 I-78 over Delaware River

Fig. 5.13 shows the results of optimizing girder cross-sections at the point of maximum positive bending moment of Delaware River bridge. Again, the results are similar to those of the Lehigh Street bridge. The weight reduction with increasing yield strength ends at a yield strength of 100 ksi, when the fatigue limit state of the Category C detail is considered. For steels with a yield strength of 100 ksi or more, no weight reduction is permitted. By extrapolating the horizontal line that represents the fatigue limit state, it can be seen that weight reduction with increasing yield strength will end at a yield strength of approximately 74 ksi, when the upper limit on yield strength of 70 ksi for compact section is neglected.

Fig. 5.14 shows the results of optimizing girder cross-sections at the point of maximum negative bending moment of the Delaware River bridge. The weight reduction with increasing yield strength is possible up to a yield strength of 120 ksi, even if fatigue limit state of the Category C detail is taken into account.

5.5 Summary

It is observed from the results presented above that there are two obstacles to the effective use of high performance steels with the yield stress of 85 ksi or more. The obstacles are: (1) the upper limit on yield strength for compact sections, and (2) the fatigue limit state of the Category C detail.

If the upper limit on yield strength for compact sections can be neglected, weight reduction is possible with increasing yield strength up to the limit imposed by the fatigue limit state of the Category C detail. For the Lehigh Street bridge, the fatigue limit state controls at a yield strength of 80 ksi for the section at the center of the span, and 70 ksi for the section at the flange plate transition point. For the Delaware River bridge, the fatigue limit state controls at a yield strength of 74 ksi for the point of maximum bending moment.

Thus, fatigue of the Category C detail limits the potential advantages of higher yield strength steels. If the Category C detail is eliminated, weight reduction is possible at least up to a yield strength of 120 ksi. The current plate girder system includes the Category C detail where transverse stiffeners and diaphragm connection plates are welded to the web. To eliminate the Category C detail, it is necessary to assume a hypothetical detail where transverse stiffeners and diaphragm connection plates are not welded to the web. Instead, these attachments must be provided without degrading fatigue performance.

In summary, 70 ksi steel appears to be the most promising one under the current system. However, it is possible to take advantage of 85 ksi steel or more, if a new system is developed. The development of a new diaphragm system which eliminates the Category C detail is necessary.

6. Discussion

The results of Chapter 5 lead to the following observations regarding the use of high performance steel for composite steel plate I-section girders:

(1) For steels with a yield strength of 70 ksi or less, the weight of the cross section can be reduced by increasing the yield strength.

(2) For steels with a yield strength of 85 ksi, the cross section weight will not be less than a 70 ksi steel cross-section because of the limit on yield strength for compact sections. However, even if this upper limit is abolished, the weight of the cross section will not be significantly reduced, because the section will be controlled by fatigue of the Category C detail.

(3) For steels with a yield strength of 100 ksi, weight reduction is obtained in some cases relative to 70 ksi steel. However, for yield strength above 100 ksi, the fatigue limit state usually prevents a weight reduction with increasing yield strength.

6.1 Steels with Yield Strength of 70 ksi or Less

In the case of steels with yield strength of 70 ksi or less, the higher the yield strength, the smaller the weight of the cross-section. In the examples that were studied, the weight reduction with an increase of yield strength from 50 ksi to 70 ksi is 14 to 16%. The 36 ksi cross sections are 17 to 19%

heavier than the 50 ksi cross-sections, although neither grade of steel fits the definition of high performance steel provided in Chapter 2.

6.2 Steels with Yield Strength of 85 ksi

The LRFD code does not permit the design criteria for compact sections to be used for steels with yield strength greater than 70 ksi. Thus a 70 ksi steel section can be designed for the plastic moment, whereas an 85 ksi steel section must be designed for the yield moment. Thus, if the 70 ksi section is a compact section, it is not possible to find a lighter 85 ksi cross-section for the girder cross-sections investigated in this study. This upper limit on yield strength is based on the plastic deformation capability of existing conventional high strength steels. Therefore, there is a possibility of changing this restriction, if high performance steel, as defined in Chapter 2, is applied. Further investigation of this possibility is needed to advance the use of high performance steel.

Another restriction which prevents the effective use of high performance steel with yield strength of 85 ksi is the fatigue limit state of the Category C detail. This is the main problem for steels of 100 ksi or more, and thus, this problem is discussed below.

6.3 Steels with Yield Strength of 100 ksi or More

Weight reduction by using 100 ksi yield strength steel can be obtained in some cases relative to 70 ksi steel. However, for yield strength above 100 ksi, weight reduction cannot be achieved in many cases because of problems with fatigue, unless more fatigue-resistant details are developed.

For the bridges considered in this study, the Category C detail limits the use of high strength steel. With current plate girder designs, it is unrealistic to consider plate girder bridges without the Category C detail, which is used to weld transverse stiffeners to the girder webs and flanges, and to weld diaphragm connection plates to the webs and flanges as seen in Fig. 4.2. To design a plate girder economically, compression flange bracing must be provided to the girders to avoid lateral torsional buckling. In current designs, compression flange bracing is usually provided by the diaphragms. That is, during construction, the compression flange in the positive moment region is braced by the diaphragms, and during construction and under the service conditions, the compression flange in the negative moment region is braced by the diaphragms.

If compression flange bracing and web stiffeners are provided without welding, it is possible to avoid the Category C detail. An illustration of this idea is shown in Fig. 6.1. To accomplish this, a new bracing system without welding must be developed. The material for the bracing system is not necessarily steel. The bracing should have enough compressive and shear stiffness and strength. The development of a connection between the plate girder and bracing system is also required. The connection should transfer compressive stress between the girder and bracing, but not bond the bracing and web together. To reduce the potential for fatigue, it is preferable to have slip longitudinally between the bracing and the web. The main point here is the development of "fatigue-resistant details". Although the development of fatigue-resistant details needs further study, there is a potential for using high performance steel in plate girder bridge designs.

6.4 Deflection Limit

The use of a deflection limit, which is optional in the LRFD code, was not considered in this thesis. However, most designers consider this criteria to be a mandatory requirement. The effect of deflection and vibration on the use of high performance steel should be studied.

7. Conclusions and Recommendations

7.1 Limitations of This Study

This study is limited under the following conditions:

(1) The conclusions are based on the study of only two bridges.

(2) Only a few cross-sections of these bridges are investigated.

(3) Deflection limits were not considered in the analysis.

7.2 Summary of Conclusions

The main results and conclusions of this study are as follows:

(1) It is important to define high performance steel not only by the yield strength, but also by the carbon or carbon equivalent, the fracture toughness, and the ductility. Tentative properties for high performance steel were proposed to enable the potential for high performance steel in plate girder bridge designs to be studied.

(2) Programs for code conformance analysis and optimization were developed, based on the new LRFD bridge design code, to allow the limiting code requirements and optimized sections to be studied.

(3) The weight of composite steel plate I-section girder cross-sections can be reduced by increasing the steel yield strength up to a yield strength of 70 ksi.

(4) Due to a limit on the use of design criteria for compact sections, the use of steel with yield strength of 85 ksi or more appears to be ineffective, with the exception of the negative moment sections of the I-78 over Delaware River bridge. An investigation of the validity of this criteria for high performance steel is needed.

(5) The fatigue limit state is often critical for steels with yield strength of 100 ksi or more, and weight reduction cannot always be obtained using steels in this strength range with current plate girder designs.

(6) The development of "fatigue-resistant details" is necessary to take advantage of steel with 100 ksi yield strength or more.

7.3 Recommendations

These above results indicate that it may be possible to make more effective use of high performance steel by conducting the following future work:

(1) Determine the validity of the limitation on the yield strength of compact sections for high performance steels as defined in Chapter 2.

(2) Develop new bracing and stiffener concepts which do not use welding of elements to the web and flange of the girder.

(3) Study the effect of deflection and vibration.

By overcoming these problems, reduced weight, and ultimately reduced cost, plate girders may be obtained using high strength, high performance steel. If it is possible to overcome these problems, other benefits of high performance steel, such as excellent weldability and remarkable toughness can be utilized. It should be noted that these other benefits can also be obtained from steels with lower strengths (up to 70 ksi) in current bridge designs. Thus, it is recommended that the development of high performance steel at lower strength level be undertaken. The benefits of high performance steel have a potential to lead ultimately to more effective plate girder bridge designs.

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Table 2.1 Specifications for Structural Steels

AASHTO Designation	ASTM Designation	
M183	:A36	Structural Steel
M223	:A572	High Strength Low Alloy Columbium-Vanadium Steels of Structural Quality
M222	:A588	High Strength Low Alloy Steel with 50 ksi [345MPa] Minimum Yield Point to 4 in Thick
M244	:A514	High Yield Stress Quenched and Tempered Alloy Steel Suitable for Welding
M270	:A709	Structural Steel for Bridges

Table 3.1 Load Combinations and Load Factors

Limit State	DC (Dead Load of Component)	DW (Dead Load of Wearing Surface and Utility)	LL+IM (Live Load+ Impact)
Strength-I	Max: 1.25 Min: 0.90	Max: 1.50 Min: 0.85	1.75
Service-I	1.00	1.00	1.00
Service-II	1.00	1.30	1.00
Fatigue	-	-	0.75

Table 4.1 Examples of LRFD Code Requirements
(I-Section Bending Member)

General Requirement Member Proportions Depth / Span Ratio Minimum Thickness of Steel
Strength Limit States Compact Section Web Slenderness Compression Flange Slenderness Compression Flange Bracing → L-T Buckling Non-Compact Section Web Slenderness Compression Flange Slenderness Compression Flange Bracing → L-T Buckling
Constructibility (Strength Limit States)
Service Limit State Control of Permanent Deflection
Fatigue Limit State

Table 4.2 Requirements for Compact Section

	Positive Bending	Negative Bending
Nominal Bending Resistance	for simple-spans: $M_n = M_p$ for continuous spans: $M_N = M_p \leq 1.3\, R_h\, M_y$	$M_n = M_p$
Web Slenderness	$\dfrac{2D_{cp}}{t_w} \leq 3.76\sqrt{\dfrac{E}{F_{yc}}}$ If $2D_{cp}/t_w$ is not met, go to Article 6.10.5.3.2, see also Article 6.10.10.2.2	$\dfrac{2D_{cp}}{t_w} \leq 3.76\sqrt{\dfrac{E}{F_{yc}}}$ If $2D_{cp}/t_w$ is not met, go to Article 6.10.5.3.3 or Article 6.10.5.6
Compression Flange Slenderness	Article 6.10.10.2.3 applies	$\dfrac{b_f}{2t_f} \leq 0.382\sqrt{\dfrac{E}{F_{yc}}}$ If $b_f/2t_f$ is not met, go to Article 6.10.5.3.3 or 6.10.5.6
Compression Flange Bracing	Article 6.10.10.2.4 applies	$L_b \leq \left[0.124 - 0.0759\left(\dfrac{M_l}{M_p}\right)\right]\left[\dfrac{r_y E}{F_{yc}}\right]$ If L_b is not met, go to Article 6.10.5.3.3

Table 4.3 Requirements for Non-Compact Section

	Positive Bending	Negative Bending
	for each flange:	for each flange:
Nominal Bending Resistance	$F_n = R_b R_h F_{yf}$	$F_n = R_b R_h F_{yf}$
Web Slenderness • without longitudinal stiffeners • with longitudinal stiffeners	$\frac{2D_c}{t_w} \leq 6.77\sqrt{\frac{E}{f_c}}$ $\frac{2D_c}{t_w} \leq 11.63\sqrt{\frac{E}{f_c}}$	$\frac{2D_c}{t_w} \leq 6.77\sqrt{\frac{E}{f_c}}$ $\frac{2D_c}{t_w} \leq 11.63\sqrt{\frac{E}{f_c}}$
Compression Flange Slenderness	Article 6.10.10.2.3 applies	$\frac{b_f}{2t_f} \leq 1.38\sqrt{\frac{E}{f_c\sqrt{\frac{2D_c}{t_w}}}}$ If $b_f/2t_f$ not satisfied, go to Article 6.10.5.6
Compression Flange Bracing	Article 6.10.10.2.4 applies	For a uniform moment: $L_b \leq 1.76\, r_t\sqrt{\frac{E}{F_{yc}}}$ If L_b not satisfied, go to Article 6.10.5.5

Table 4.4 Nominal Shear Resistance

Unstiffened Web			If $\frac{D}{tw} \leq 2.46\sqrt{\frac{E}{Fyw}}$ Then :	$Vn = Vp = 0.58\ Fyw\ D\ tw$
			If $2.46\sqrt{\frac{E}{Fyw}} < \frac{D}{tw} \leq 3.07\sqrt{\frac{E}{Fyw}}$ Then :	$Vn = 1.48\ tw^2\sqrt{E\ Fyw}$
			If $\frac{D}{tw} > 3.07\sqrt{\frac{E}{Fyw}}$ Then :	$Vn = 4.55\ tw^2\ E / D$
Stiffened Web	Interior Panel	Compact	If $Mu \leq 0.5\ \phi_f\ Mp$, Then :	$Vn = Vp\ [C + \frac{0.87\ (1-C)}{\sqrt{1 + (\frac{do}{D})^2}}]$
			If $Mu > 0.5\ \phi_f\ Mp$, Then :	$Vn = R\ Vp\ [C + \frac{0.87\ (1-C)}{\sqrt{1 + (\frac{do}{D})^2}}]$
		Non-Compact	If $fu \leq 0.75\ \phi_f\ Fy$, Then :	$Vn = Vp\ [C + \frac{0.87\ (1-C)}{\sqrt{1 + (\frac{do}{D})^2}}]$
			If $fu > 0.75\ \phi_f\ Fy$, Then :	$Vn = B\ Vp\ [C + \frac{0.87\ (1-C)}{\sqrt{1 + (\frac{do}{D})^2}}]$
	End Panel			$Vn = C\ Vp = 0.58\ C\ Fyw\ D\ tw$

Table 4.5 Requirements for Non-Composite Section

(Non-Compact Section)

	Positive or Negative Bending
	for each flange:
Nominal Bending Resistance	$F_n = R_b R_h F_{yf}$
Web Slenderness • without longitudinal stiffeners • with longitudinal stiffeners	$\frac{2D_c}{t_w} \le 6.77\sqrt{\frac{E}{f_c}}$ $\frac{2D_c}{t_w} \le 11.63\sqrt{\frac{E}{f_c}}$
Compression Flange Slenderness If $b_f/2t_f$ is not satisfied, go to Article 6.10.5.6	$\frac{b_f}{2t_f} \le 1.38\sqrt{\frac{E}{f_c\sqrt{\frac{2D_c}{t_w}}}}$
Compression Flange Bracing If L_b is not satisfied, go to Article 6.10.6.5	For a uniform moment: $L_b \le 1.76\, r' \sqrt{\frac{E}{F_{yc}}}$

Table 4.6 Flange Stress Reduction Factors

Nominal Bending Resistance	Flange Stress Reduction Factor	Description
$M_n = R_b R_h M_y$	R_h	This factor accounts for non-linear variation of stresses caused by yielding of the lower strength steel in the web.
	R_b	This factor accounts for non-linear variation of stresses caused by local buckling of slender webs subjected to bending stresses.

Table 5.1 Base Case Bridges

Bridge	Type	Span Length	Y.S. of Steel	Girder Spacing	Stiffener Spacing	Diaphragm Spacing
I-78 Over Lehigh Street	1-Span Simple	110 '	50 ksi	12'10"	9' 8"	19' 4"
I-78 Over Delaware River	7-Span Continuous	228 ' X 3 169 ' X 2 100 ' X 2	50 ksi	14' 3"	8'	25' (Other than Support) 14' (at support)

Table 5.2 Optimized Cross Sections without Fatigue Limit State

Bridge	Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
I-78 over Lehigh Street	Dg (in)	90.00	86.00	74.00	70.00	66.00	66.00
	Tw	0.4375	0.4375	0.375	0.4375	0.375	0.375
	bft	16.00	15.00	15.00	13.00	15.00	13.00
	tft	0.8125	0.6875	0.875	0.6875	0.8125	0.875
	bfb	28.00	22.00	18.00	18.00	24.00	14.00
	tfb	0.75	0.625	0.625	0.75	0.5	0.625
	LBS/FT	250	210	177	181	167	153
I-78 over Delaware River (Positive)	Dg	108.00	108.00	90.00	90.00	84.00	80.00
	Tw	0.497	0.5	0.5	0.5	0.4375	0.4375
	bft	14.00	14.00	14.00	14.00	14.00	14.00
	tft	0.9375	0.8125	0.8125	0.8125	0.9375	0.9375
	bfb	16.00	16.00	12.00	12.00	12.00	12.00
	tfb	1.6875	0.9375	1.0625	1.1875	1.1875	1.1875
	LBS/FT	319	273	235	240	218	212
I-78 over Delaware River (Negative)	Dg	136.00	114.00	98.00	86.00	74.00	74.00
	Tw	0.625	0.625	0.625	0.625	0.625	0.625
	bft	30.00	25.00	15.00	21.00	21.00	13.00
	tft	1.175	1.25	1.25	1.125	1.125	1.25
	bfb	32.00	31.00	21.00	21.00	21.00	17.00
	tfb	1.75	1.5625	1.8125	1.6875	1.5625	1.8125
	LBS/FT	600	514	419	384	349	318

Table 5.3 Optimized Cross Sections with Fatigue Limit State

Bridge	Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
	Dg(in)	90.00	86.00	74.00	70.00	82.00	82.00
	Tw	0.4375	0.4375	0.375	0.4375	0.375	0.375
I-78 over	bft	16.00	15.00	15.00	13.00	16.00	16.00
Lehigh	tft	0.8125	0.6875	0.875	0.6875	0.75	0.75
Street	bfb	28.00	22.00	18.00	18.00	8.00	8.00
	tfb	0.75	0.625	0.625	0.75	0.875	0.875
	LBS/FT	250	210	177	181	169	169
	Dg	108.00	108.00	90.00	90.00	90.00	90.00
	Tw	0.497	0.5	0.5	0.5	0.5	0.5
I-78 over	bft	14.00	14.00	14.00	14.00	14.00	14.00
Delaware	tft	0.9375	0.8125	0.8125	0.8125	0.8125	0.8125
River	bfb	16.00	16.00	12.00	12.00	12.00	12.00
(Positive)	tfb	1.6875	0.9375	1.0625	1.1875	0.9375	0.9375
	LBS/FT	319	273	235	240	230	230
	Dg	136.00	114.00	98.00	86.00	74.00	74.00
	Tw	0.625	0.625	0.625	0.625	0.625	0.625
I-78 over	bft	30.00	25.00	15.00	21.00	21.00	13.00
Delaware	tft	1.175	1.25	1.25	1.125	1.125	1.25
River	bfb	32.00	31.00	21.00	21.00	21.00	17.00
(Negative)	tfb	1.75	1.5625	1.8125	1.6875	1.5625	1.8125
	LBS/FT	600	514	419	384	349	318

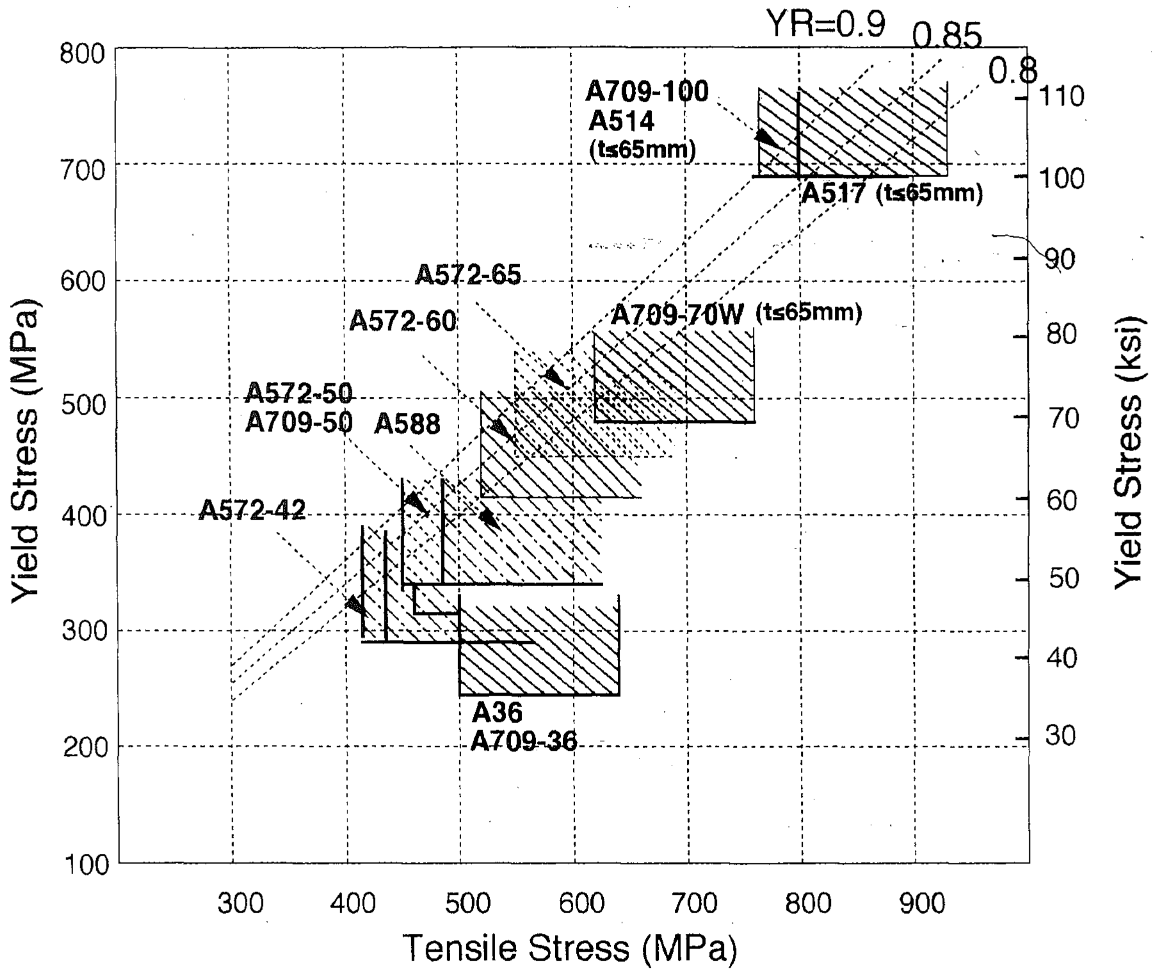


Fig. 2.1 Yield Strength and Tensile Strength in ASTM Specification

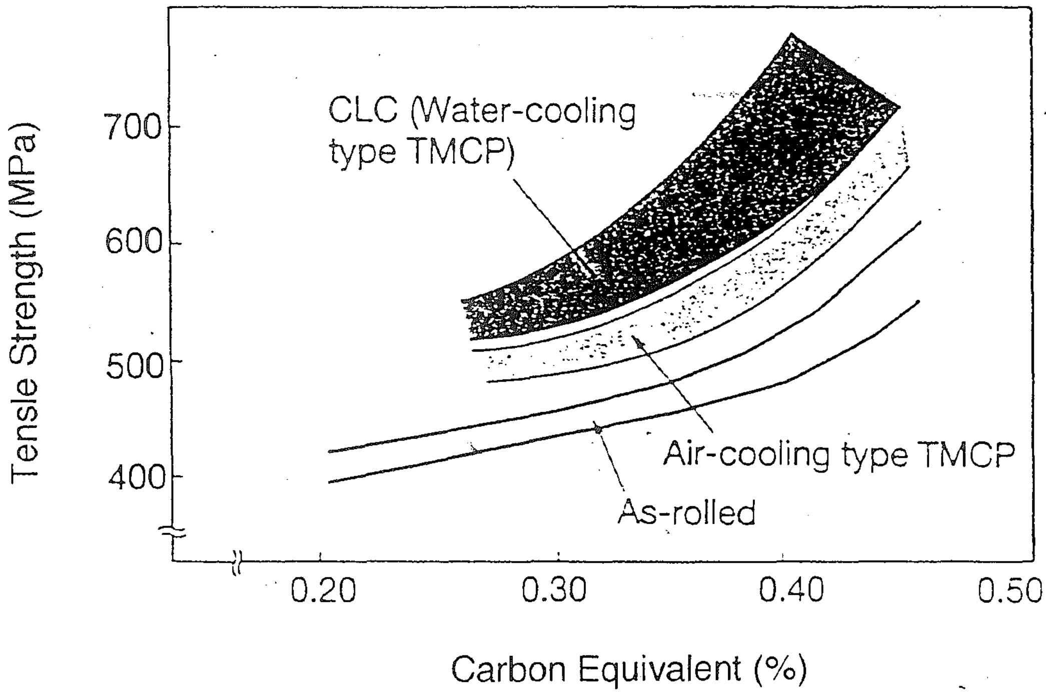


Fig. 2.2 Carbon Equivalent and Tensile Strength of TMCP Steel

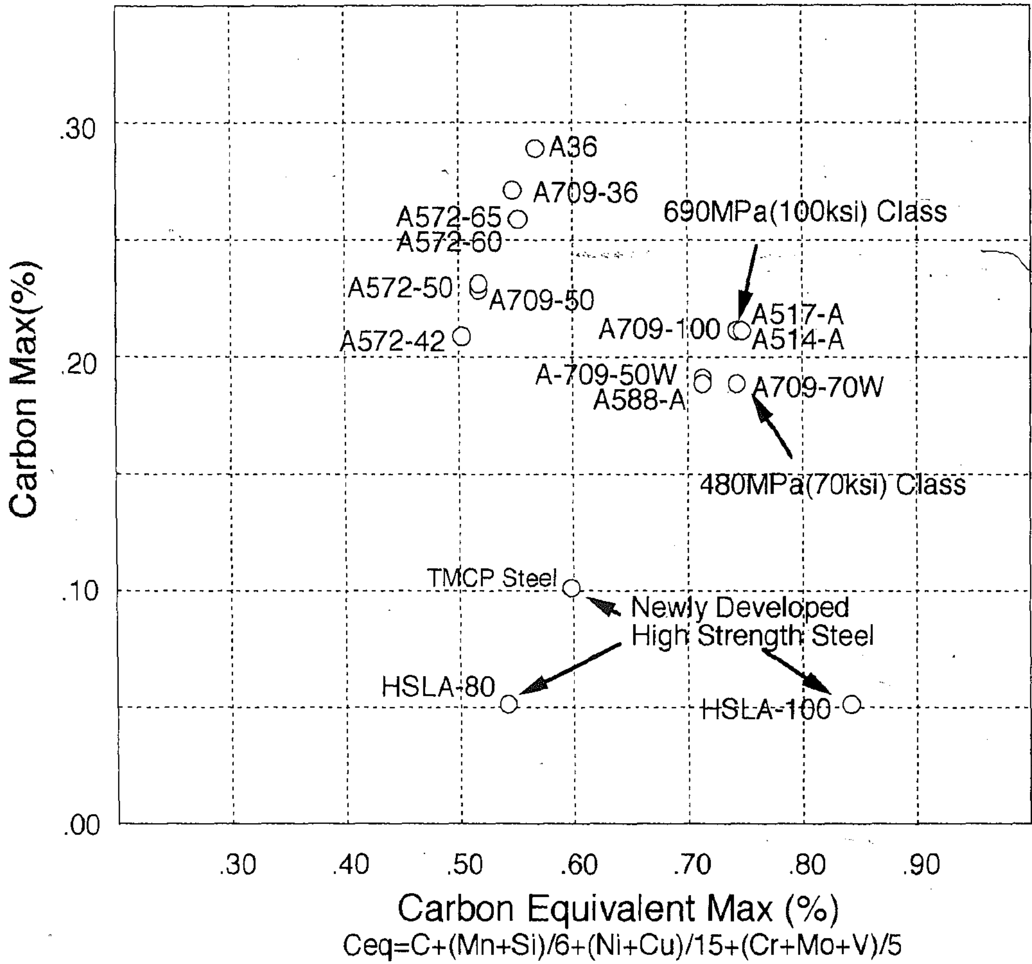


Fig. 2.3 Carbon Equivalent and Carbon in Steels in ASTM Specification

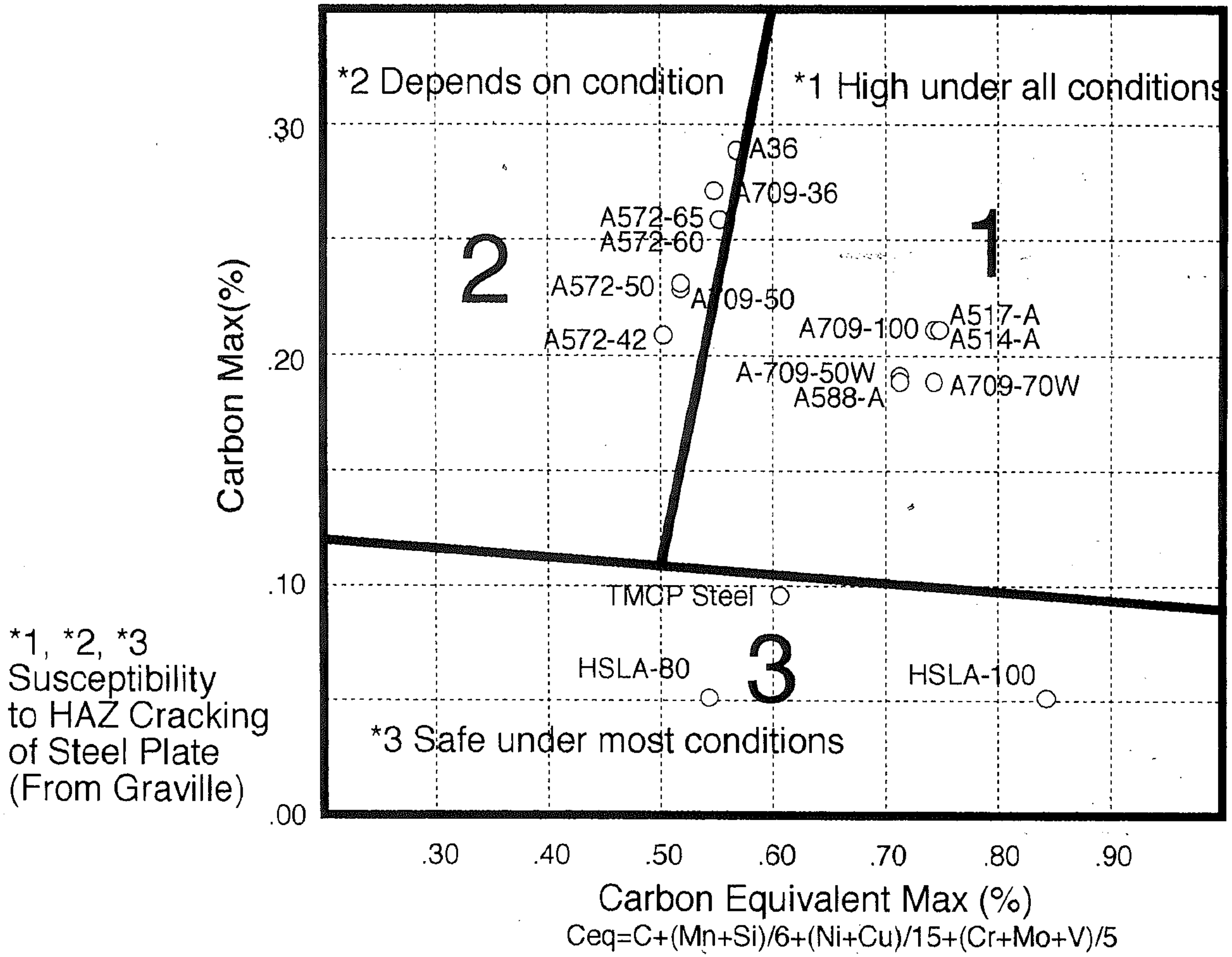


Fig. 2.4 Influence on Susceptibility to HAZ Cracking of Steel Plate

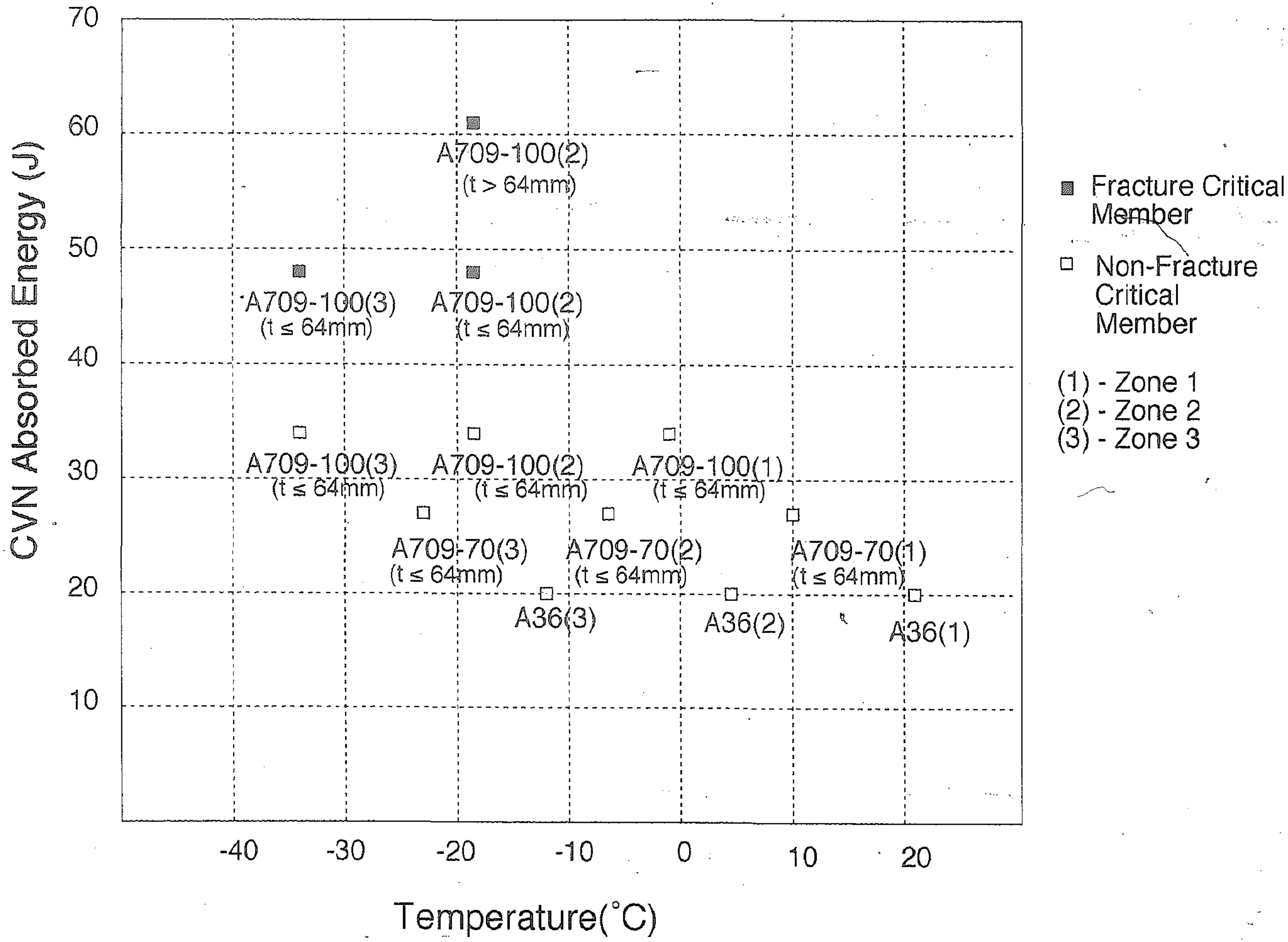


Fig. 2.5 CVN Absorbed Energy Specified in ASTM Specification

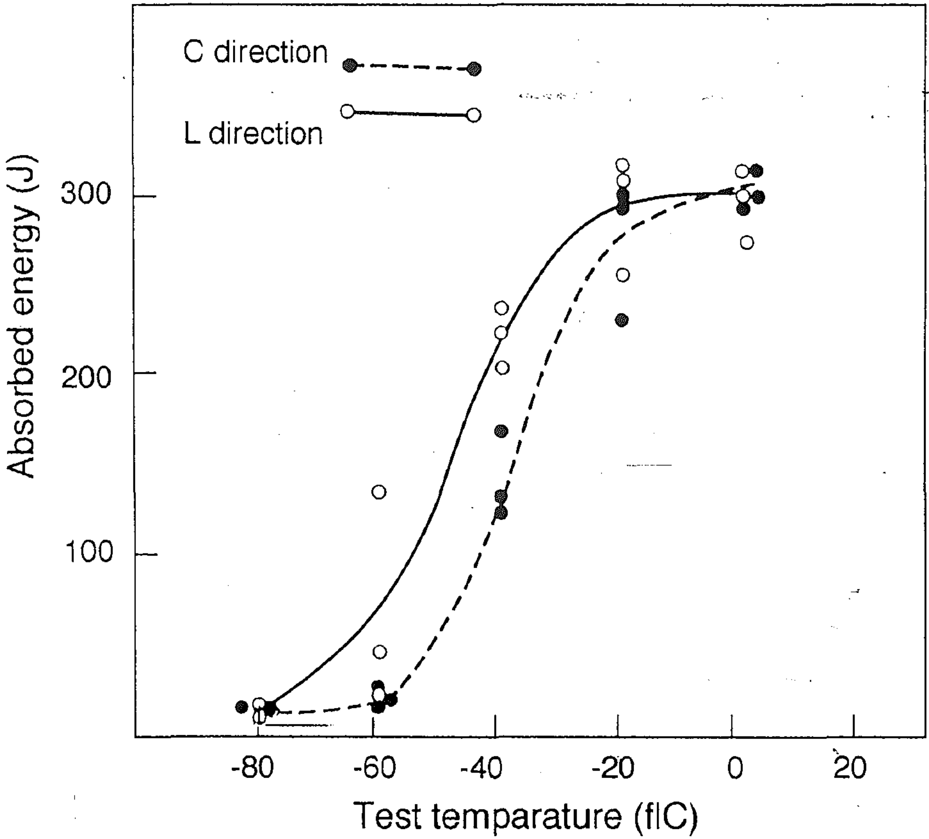


Fig. 2.6 Result from Impact Test for TMCP Steel

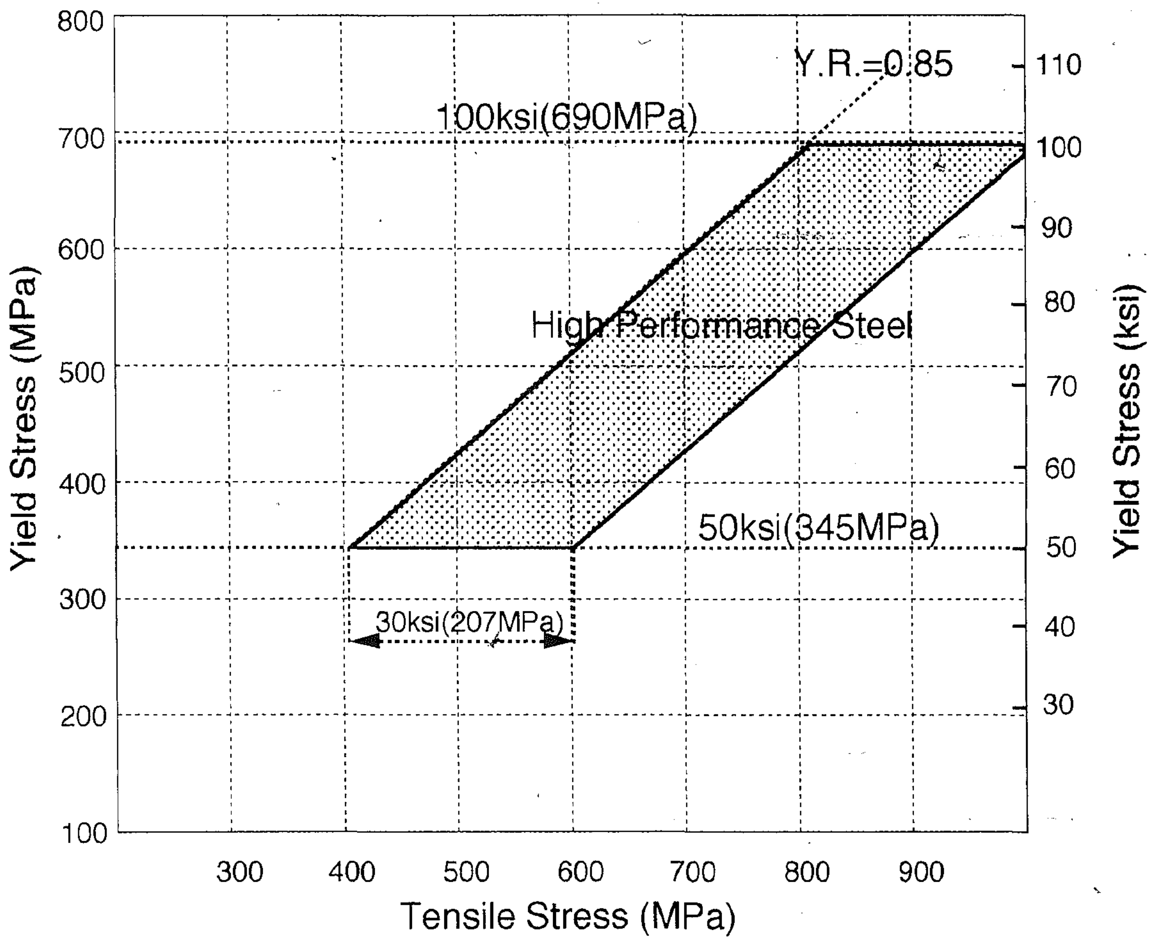


Fig. 2.7 Assumed Yield Strength and Tensile Strength of High Performance Steel

*1, *2, *3 :
Susceptibility
to HAZ Cracking
of Steel Plate
(From Graville)

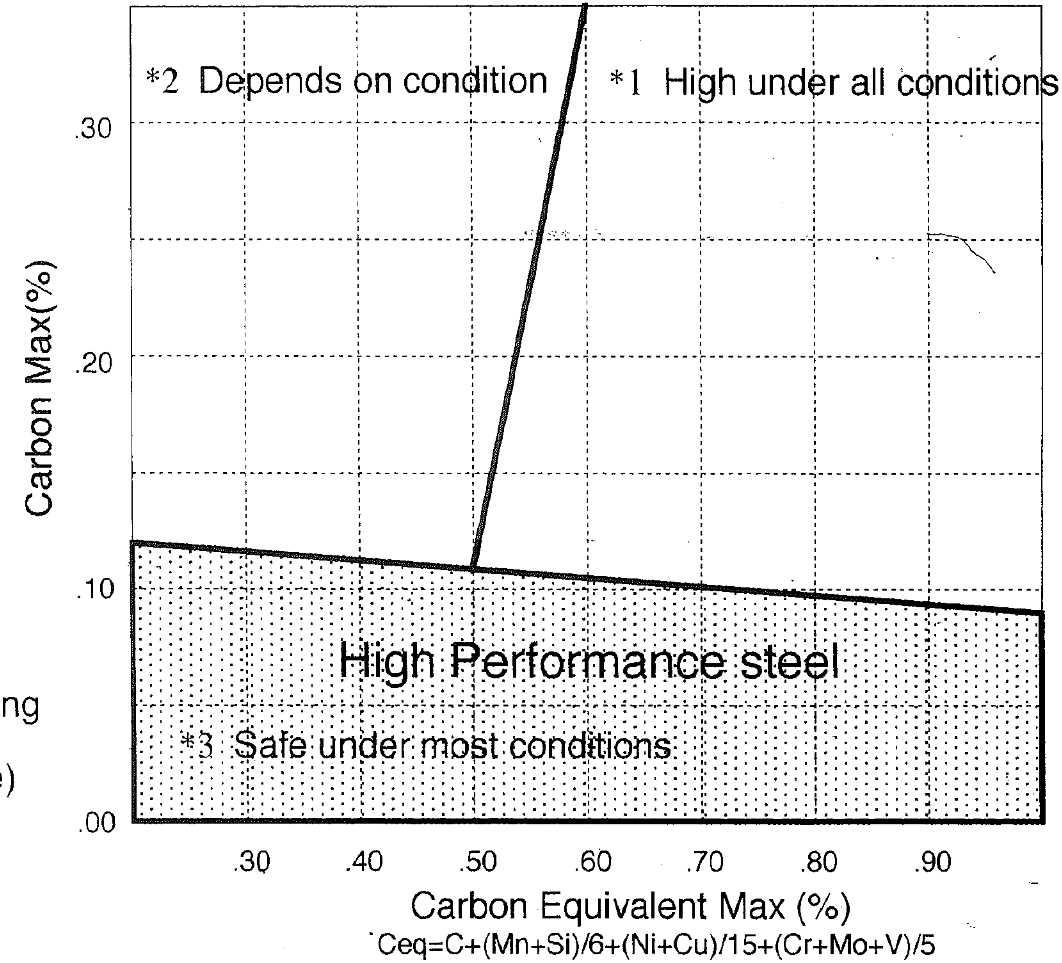


Fig. 2.8 Assumed Carbon Equivalent and Carbon of High Performance Steel

Design Truck = Two 32.0 kip and one 8.0 kip Axles

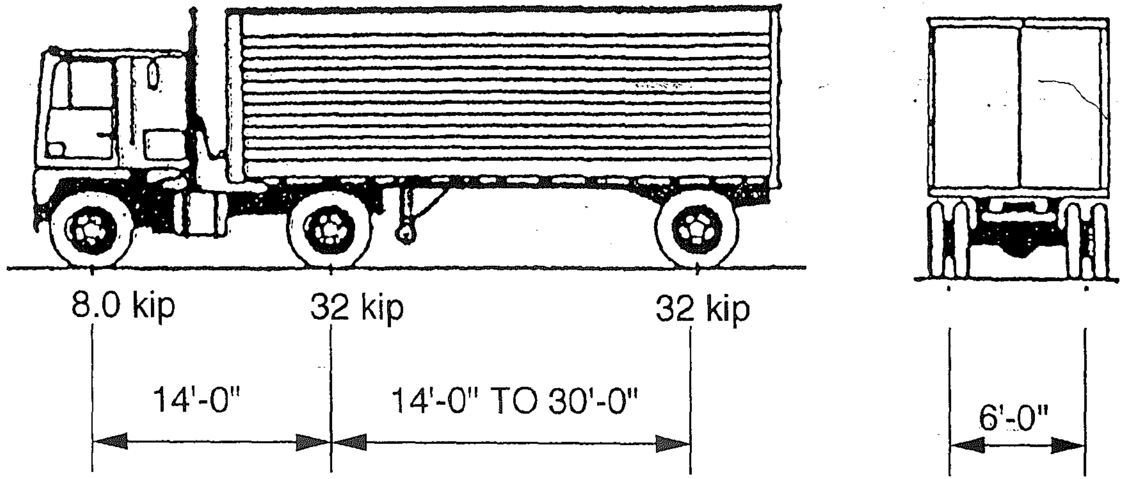


Fig. 3.1 Design Truck Specified in the LRFD Code

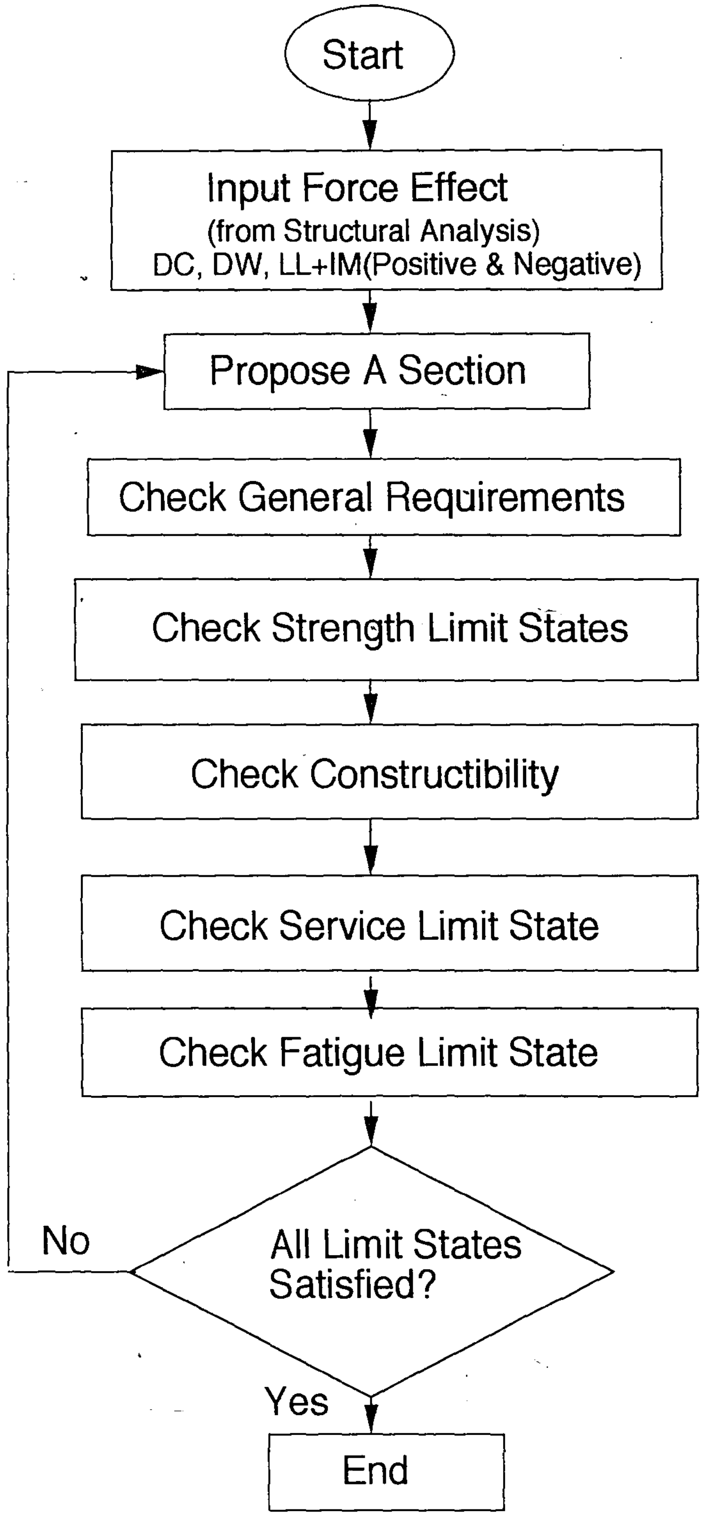


Fig. 4.1 Flowchart of Code Conformance Analysis

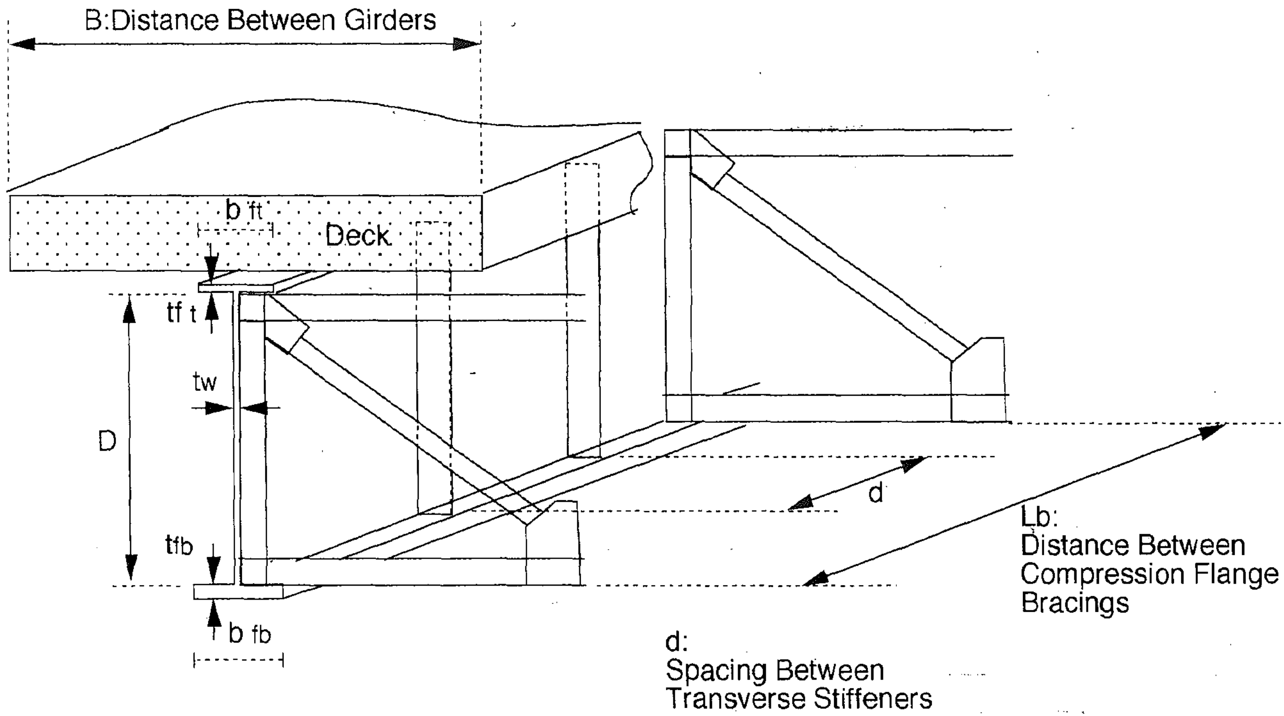


Fig. 4.2 I-Section Composite Plate Girder

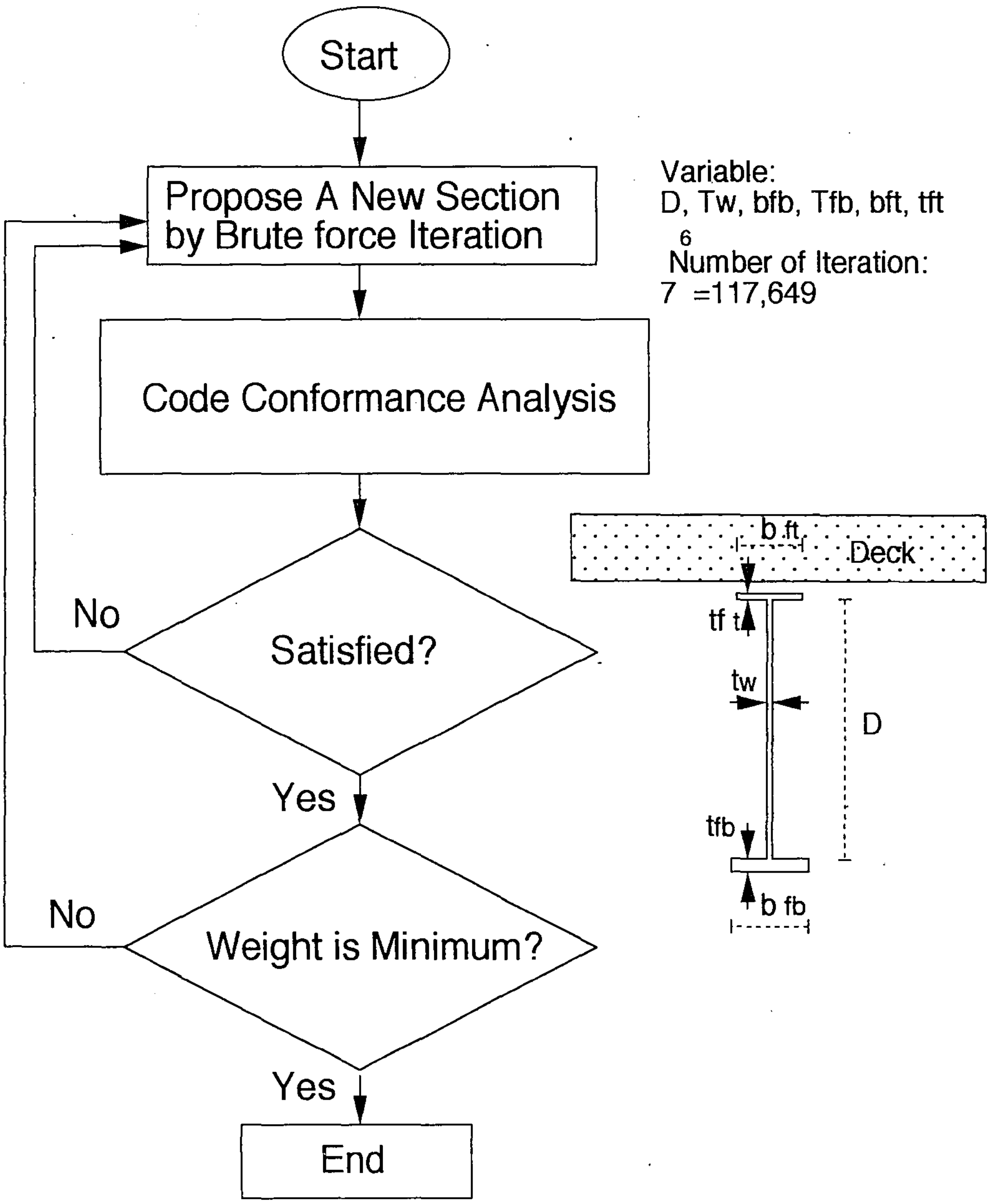


Fig. 4.3 Flowchart of Optimization Analysis

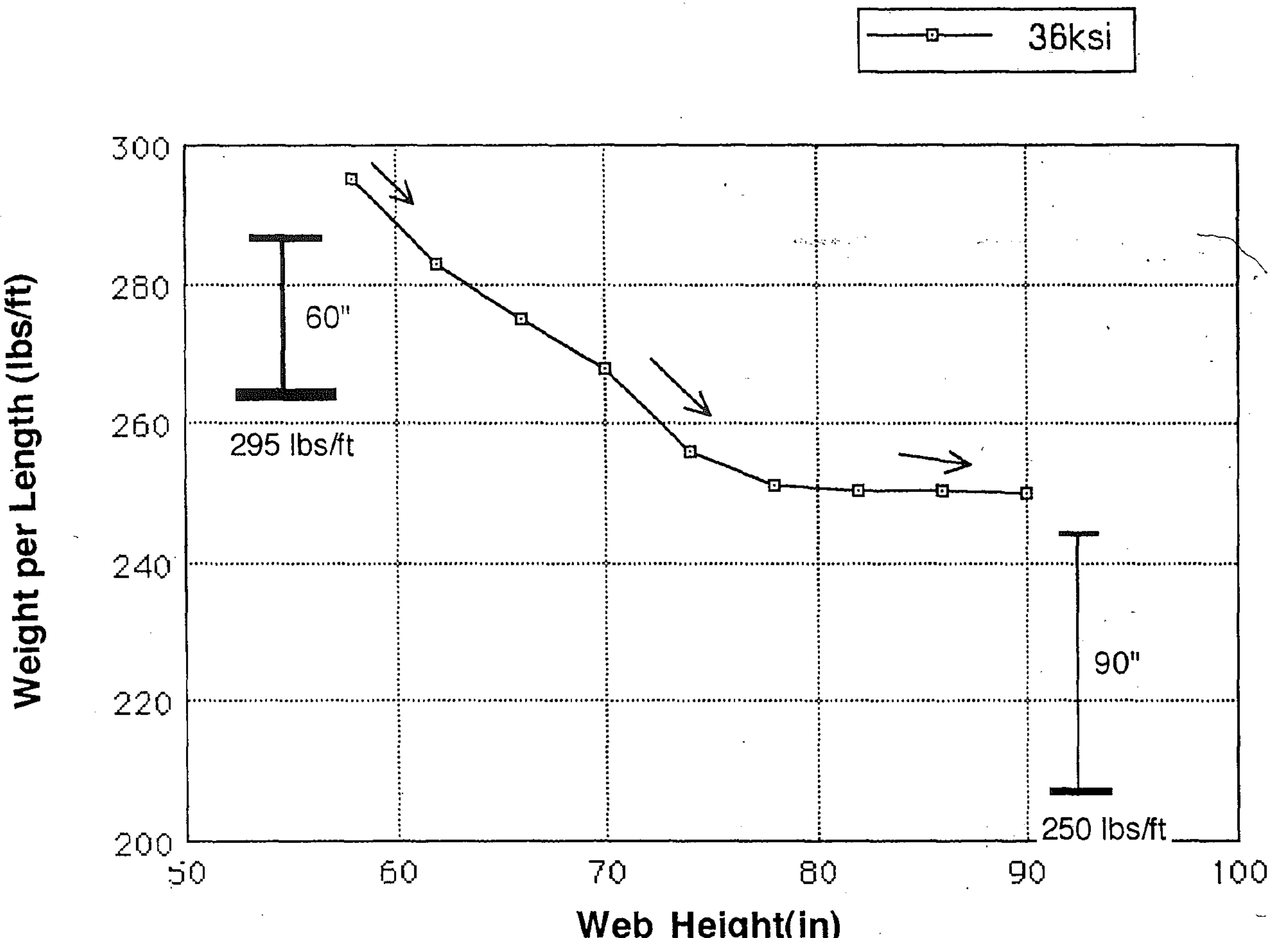


Fig. 5.1 Optimization of I-78 over Lehigh Street with Respect to Web Height

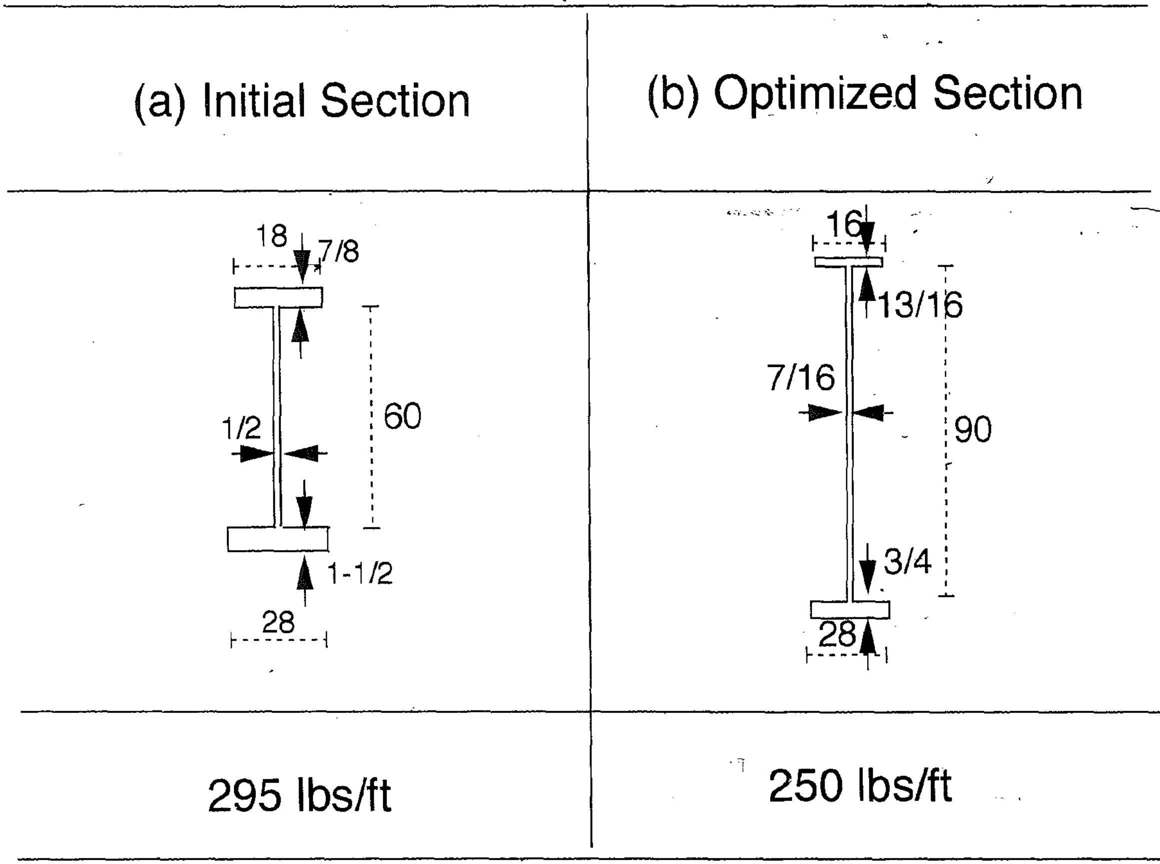


Fig. 5.2 Initial and Optimized Section for the 36 ksi Steel

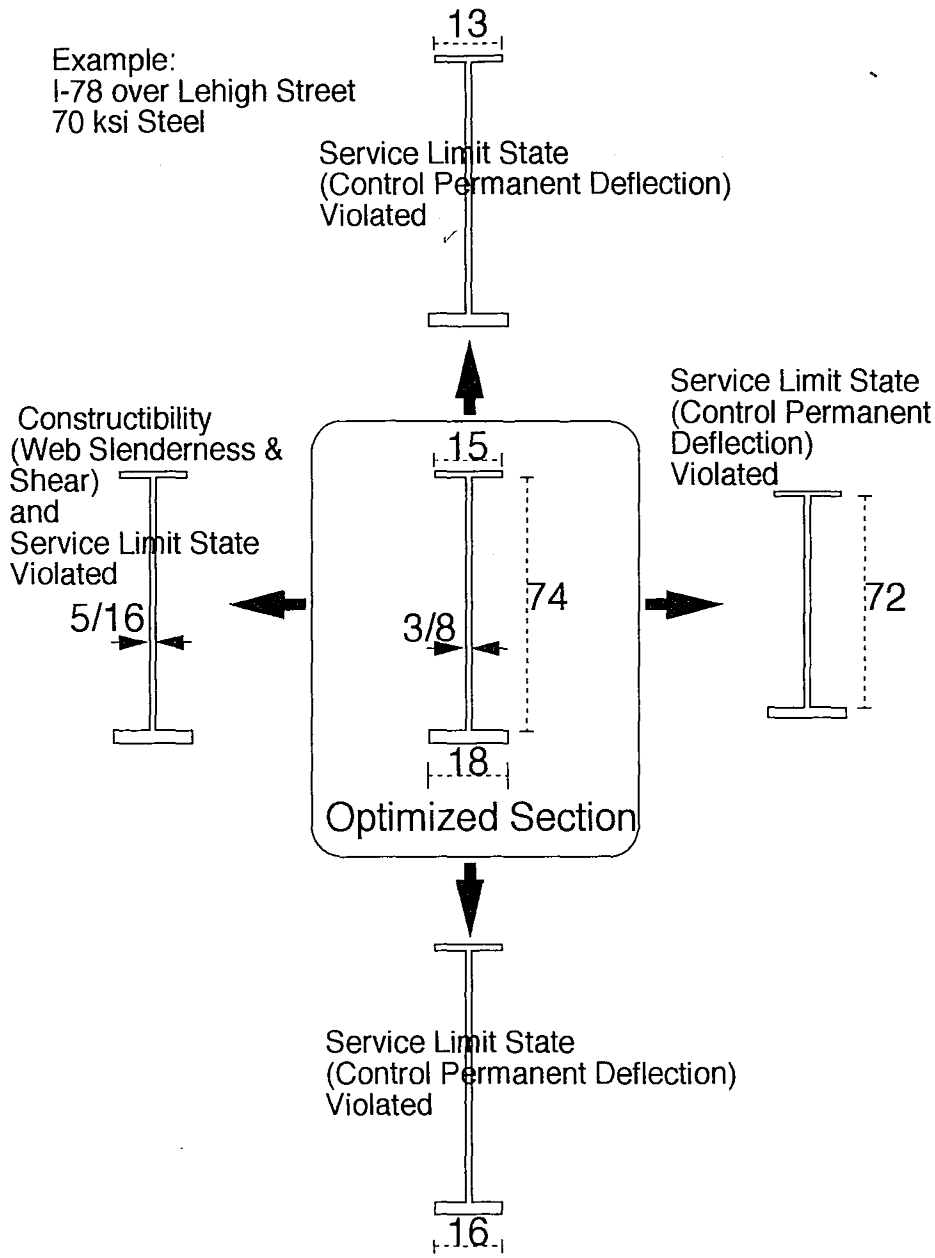
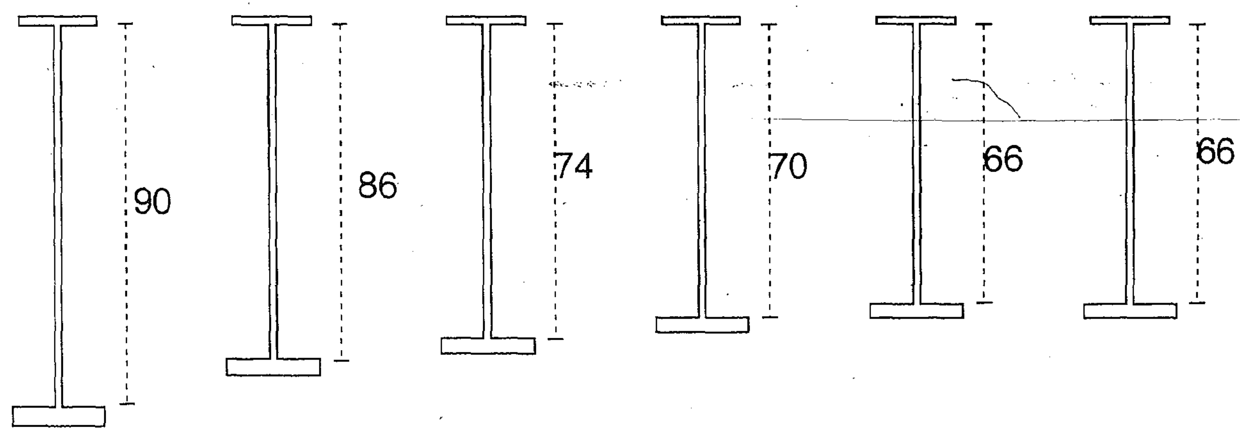


Fig. 5.3 Optimized Cross-Section and Controlling Code Requirements



Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
Section	90	86	74	70	66	66
Weight	119%	100%	84%	86%	80%	73%
Compact Section?	Compact	Compact	Compact	* Non-compact	Non-compact	Non-compact

* $F_y \leq 70$ ksi : Compact Section is applicable

Fig.5.4 Optimized Sections for I-78 over Lehigh Street

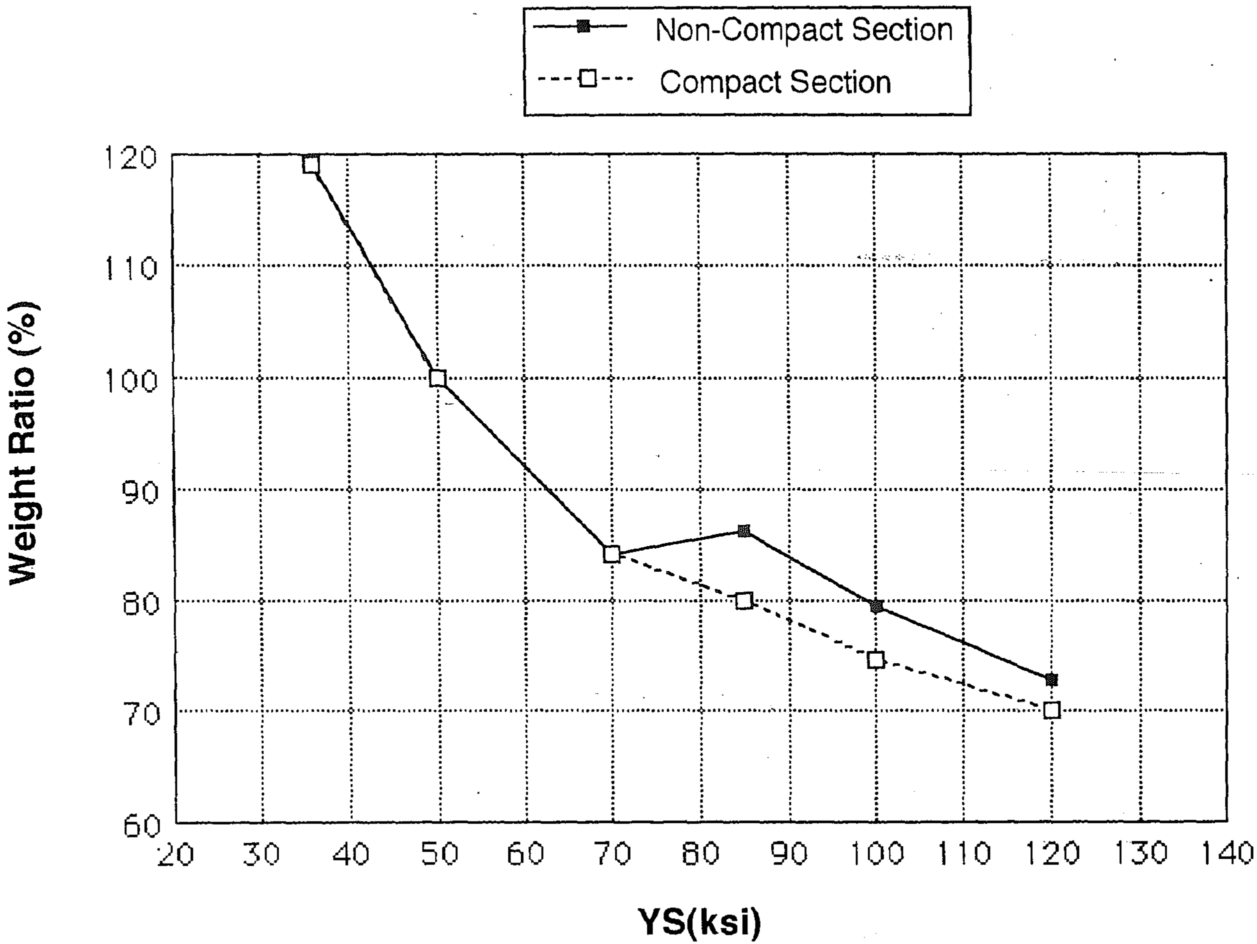
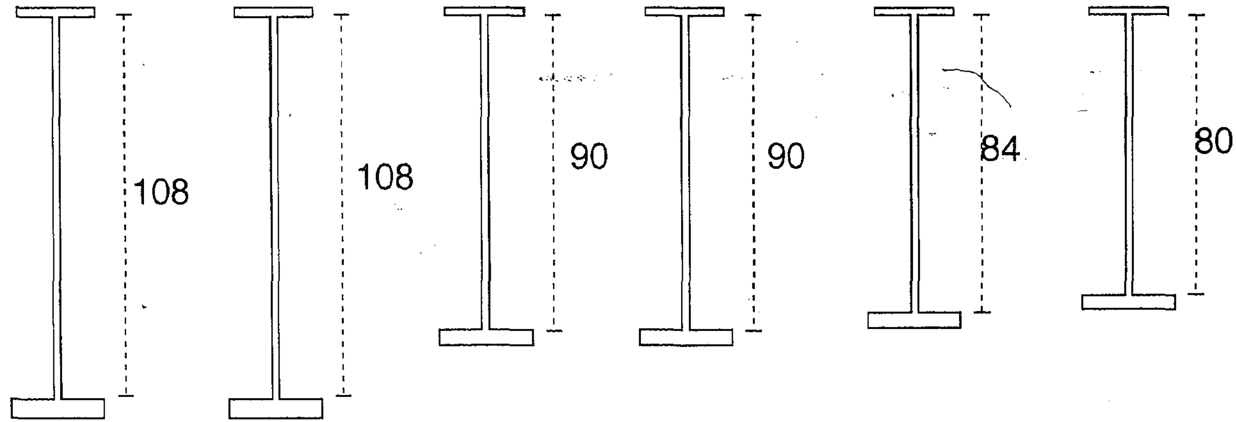


Fig. 5.5 Weight versus Yield Strength (Maximum Positive Bending Section of I-78 over Lehigh Street)



Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
Section	108	108	90	90	84	80
Weight	117%	100%	86%	88%	80%	78%
Compact Section?	Compact	Compact	Compact	* Non-compact	Non-compact	Non-compact

* Fy ≤ 70 ksi : Compact Section is applicable

Fig.5.6 Optimized Sections for I-78 over Delaware River (Maximum Positive Bending Section)

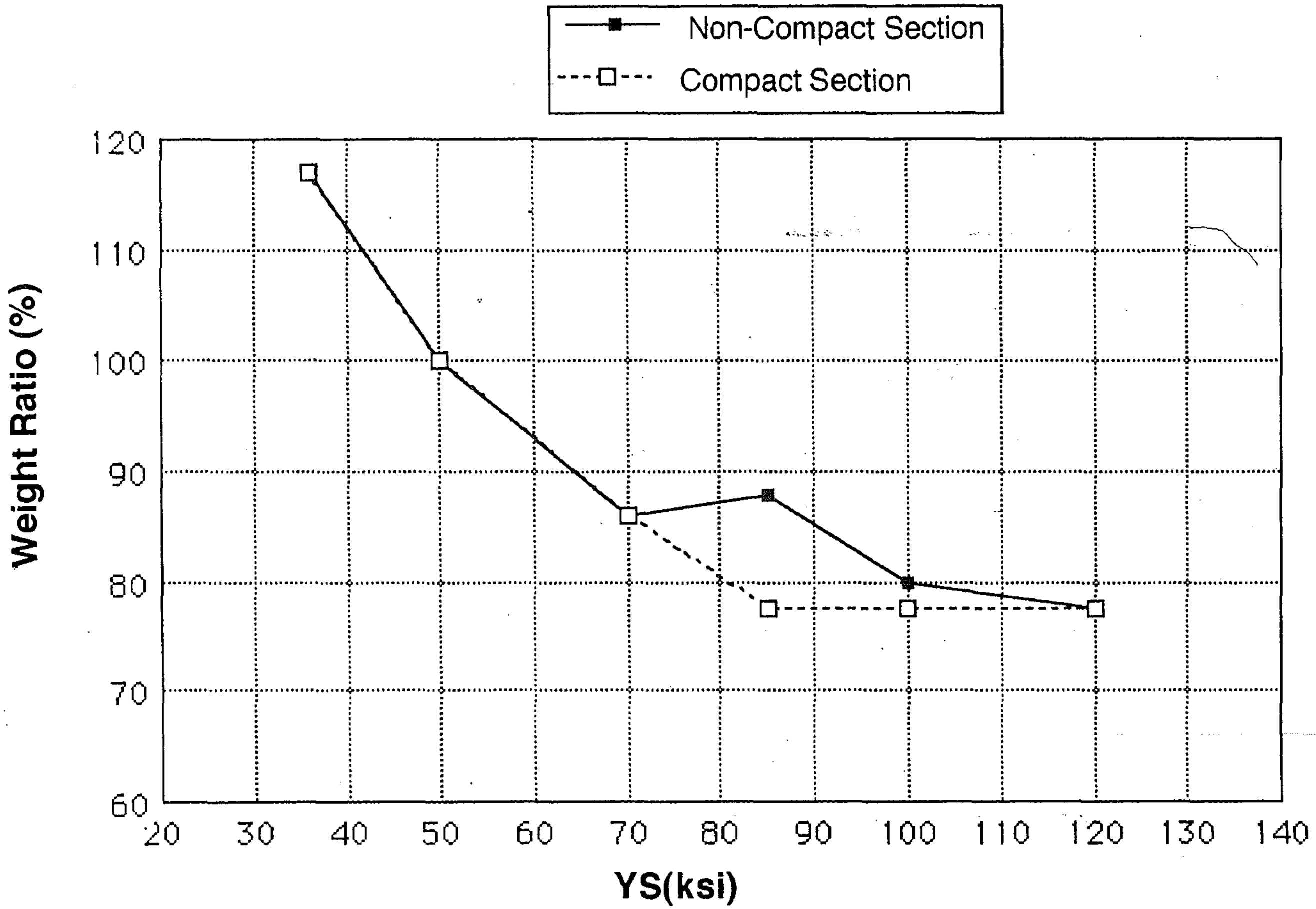


Fig. 5.7 Weight versus Yield Strength (Maximum Positive Bending Section of I-78 over Delaware River)

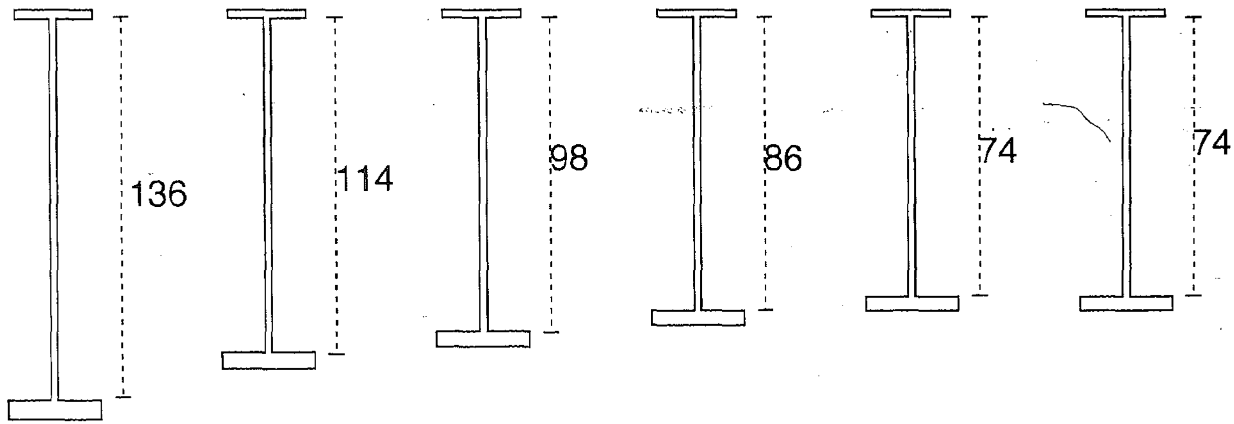
Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
Section	 136	114	98	86	74	74
Weight	117%	100%	82%	75%	68%	62%
Compact Section?	Non-compact	Non-compact	Non-compact	Non-compact	Non-compact	Non-compact

Fig.5.8 Optimized Sections for I-78 over Delaware River
(Maximum Negative Bending Section)

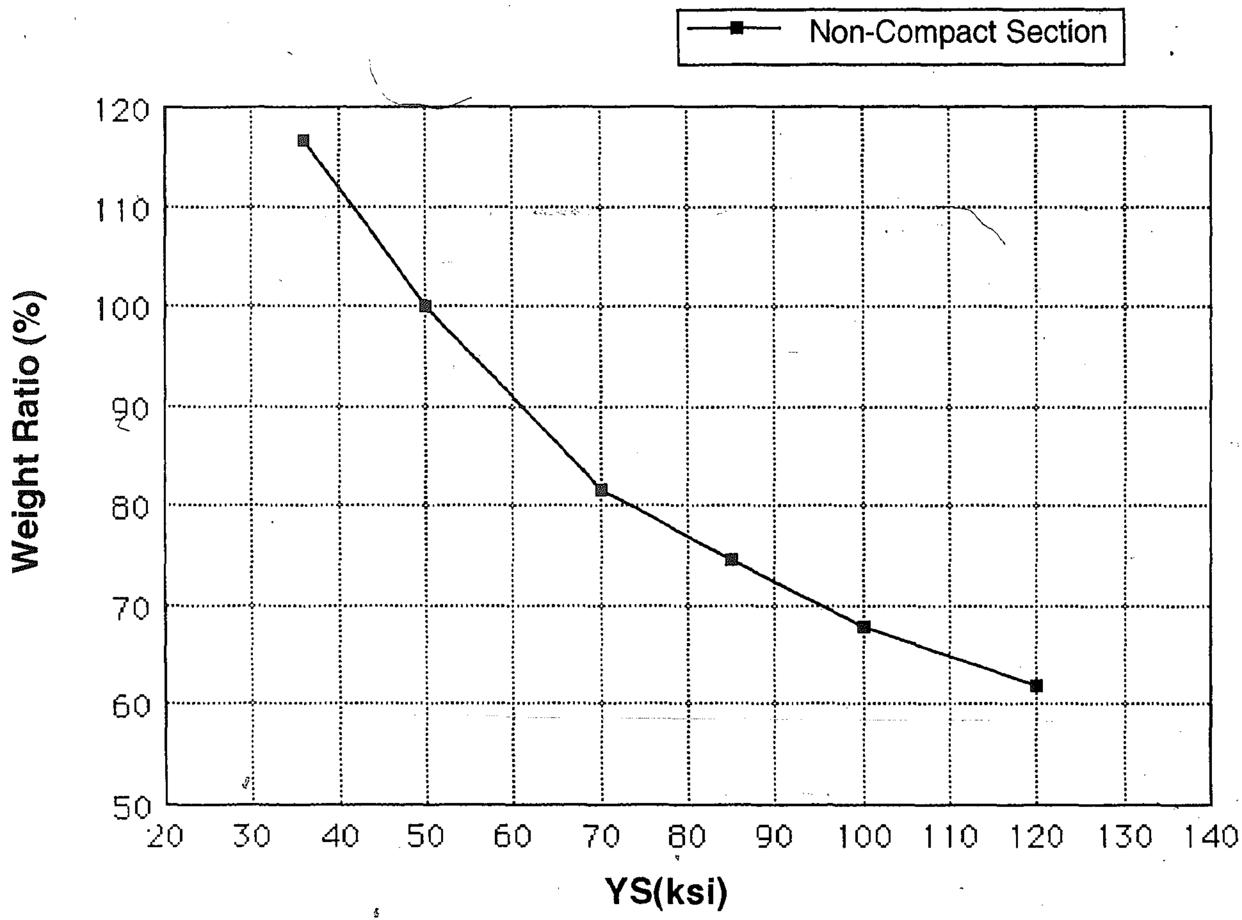


Fig .5.9 Weight versus Yield Strength (Maximum Negative Bending Section of I-78 over Delaware River)

Steel	36ksi	50ksi	70ksi	85ksi	100ksi	120ksi
Section	90	86	74	70	82	82
Weight	119%	100%	84%	86%	80%	80%
Compact Section?	Compact	Compact	Compact	Non-compact	Non-compact	Non-compact

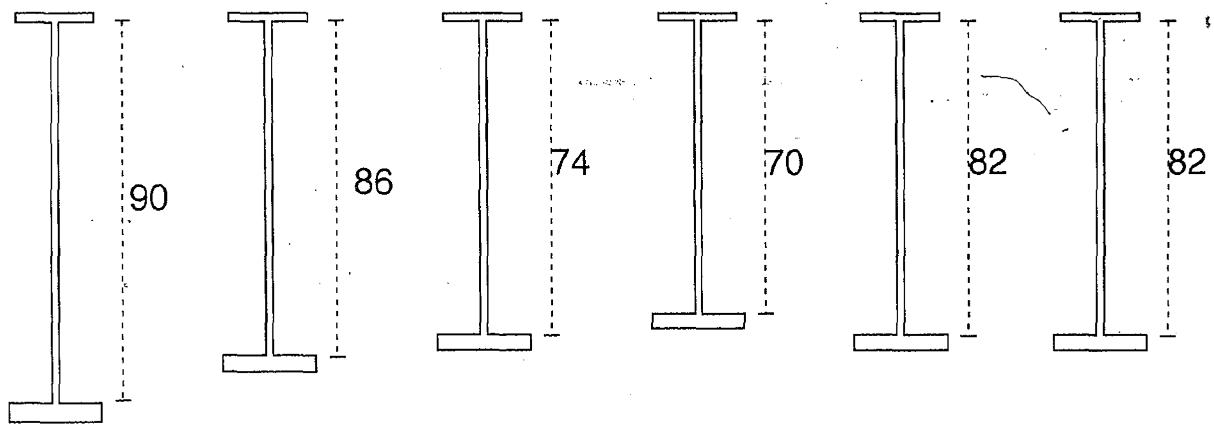


Fig. 5.10 Optimized Section including Fatigue Limit State (Category C) for I-78 over Lehigh Street

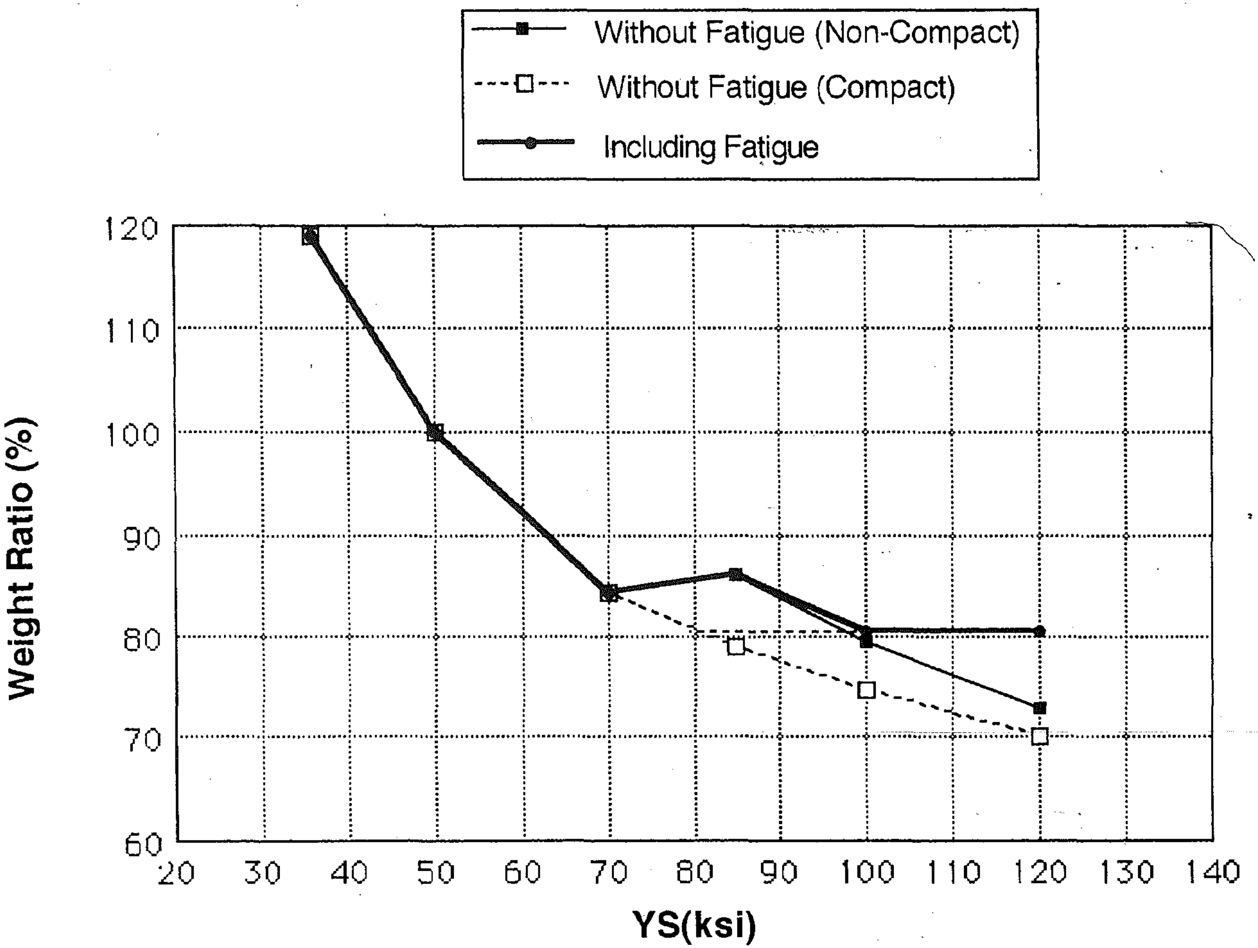


Fig. 5.11 Weight versus Yield Strength including Fatigue Limit State (Maximum Positive Bending Section of I-78 over Lehigh Street)

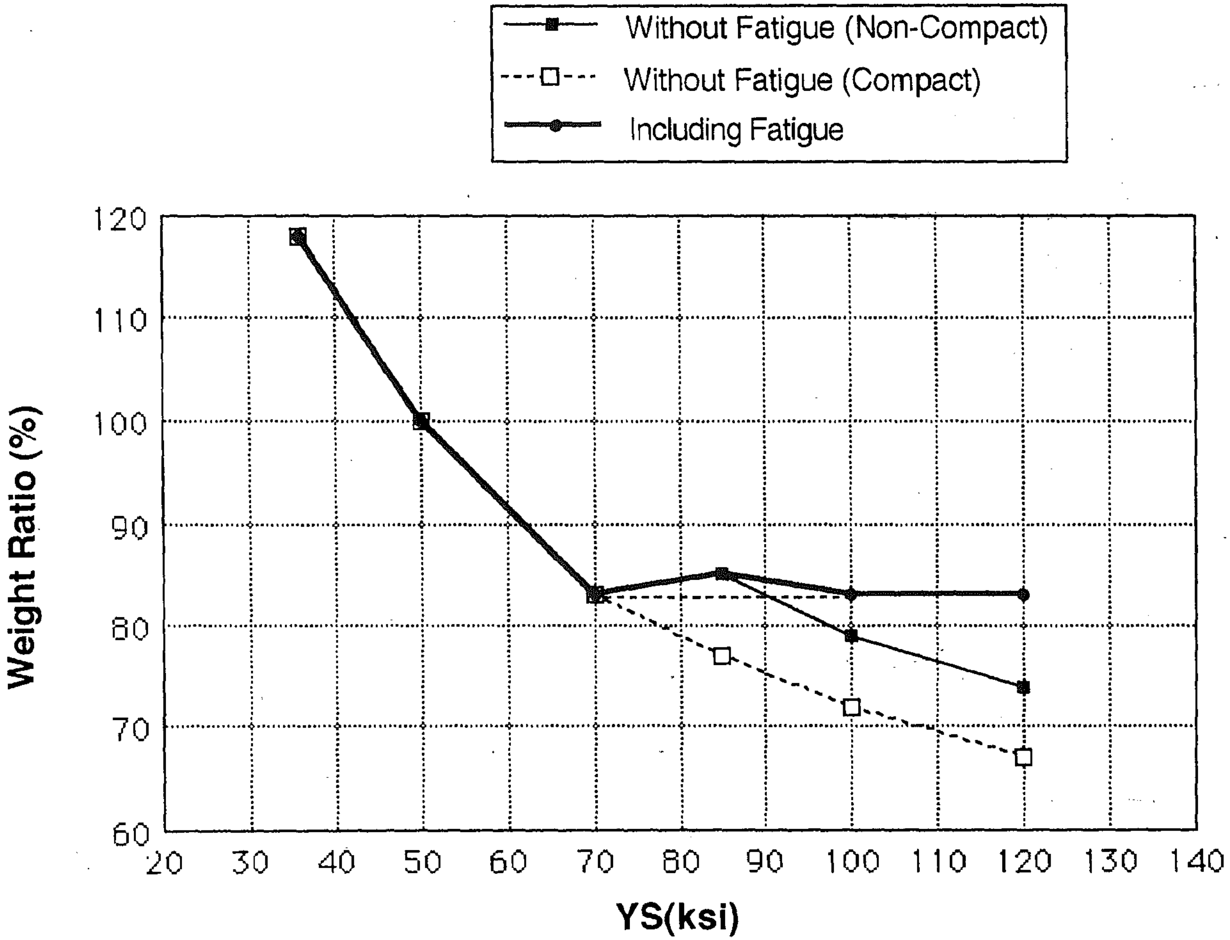


Fig. 5.12 Weight versus Yield Strength including Fatigue Limit State (Plate Transition Section of I-78 over Lehigh Street)

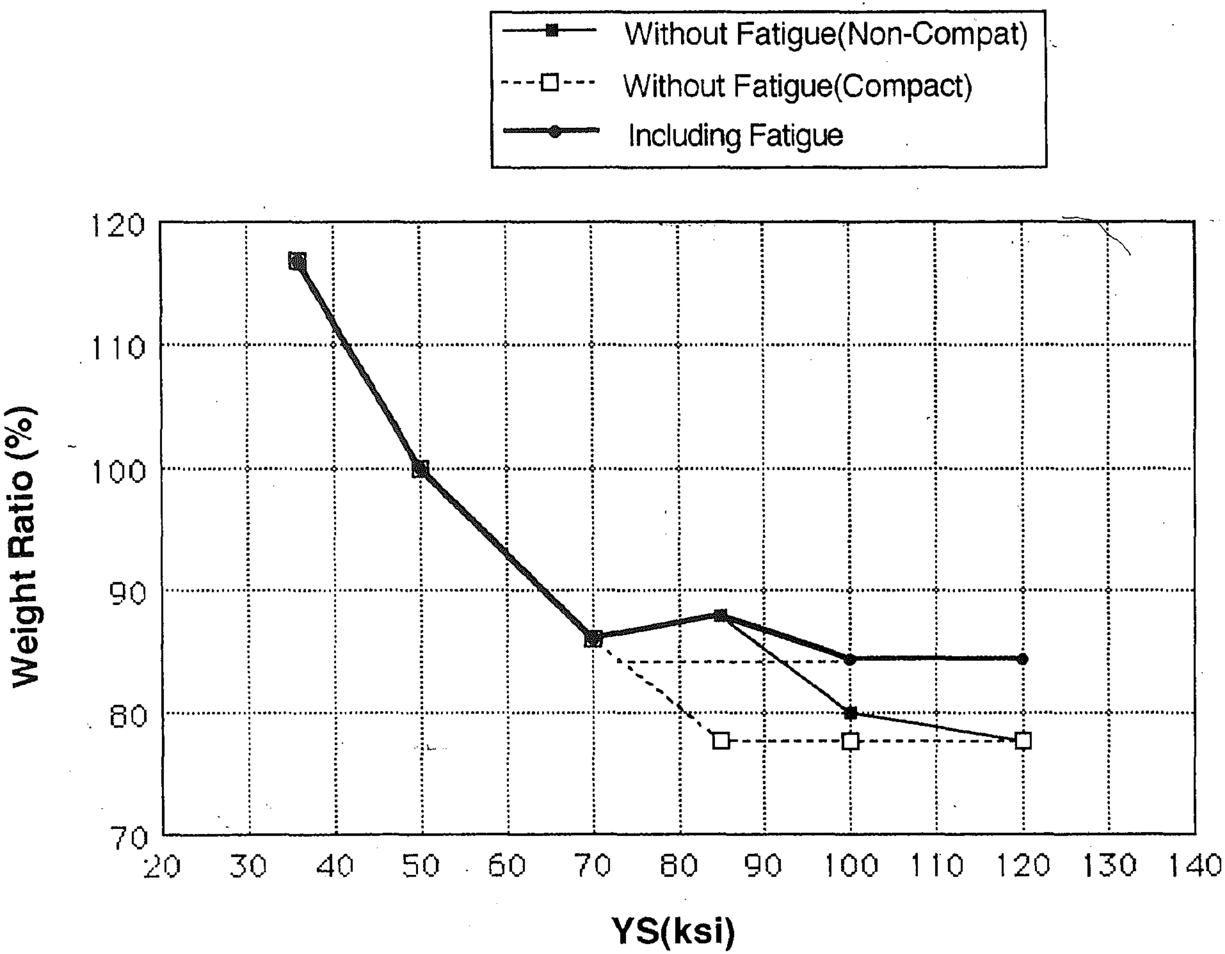


Fig. 5.13 Weight versus Yield Strength including Fatigue Limit State (Maximum Positive Bending Section of I-78 over Delaware River)

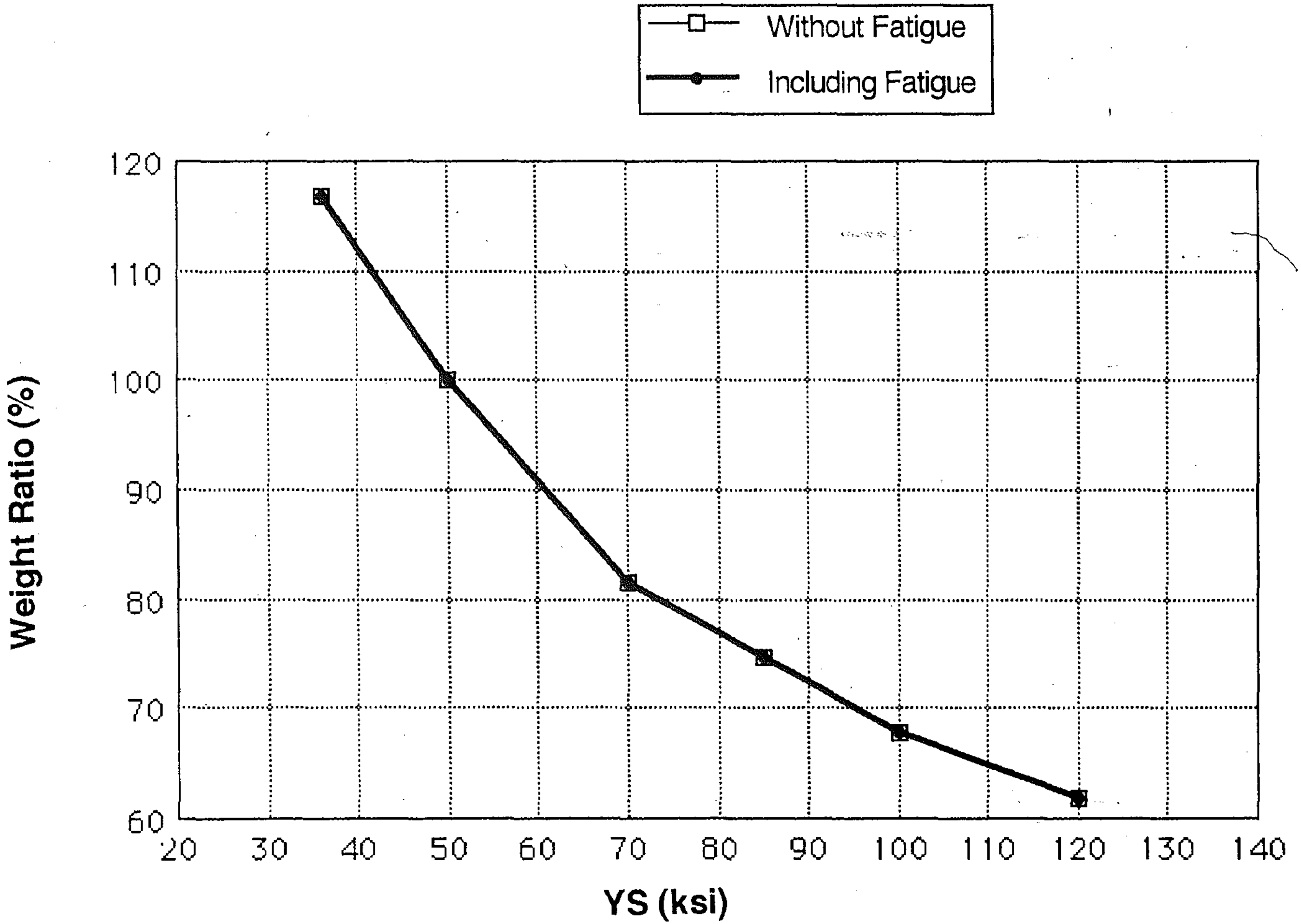


Fig. 5.14 Weight versus Yield Strength including Fatigue Limit State (Maximum Negative Bending Section of I-78 over Delaware River

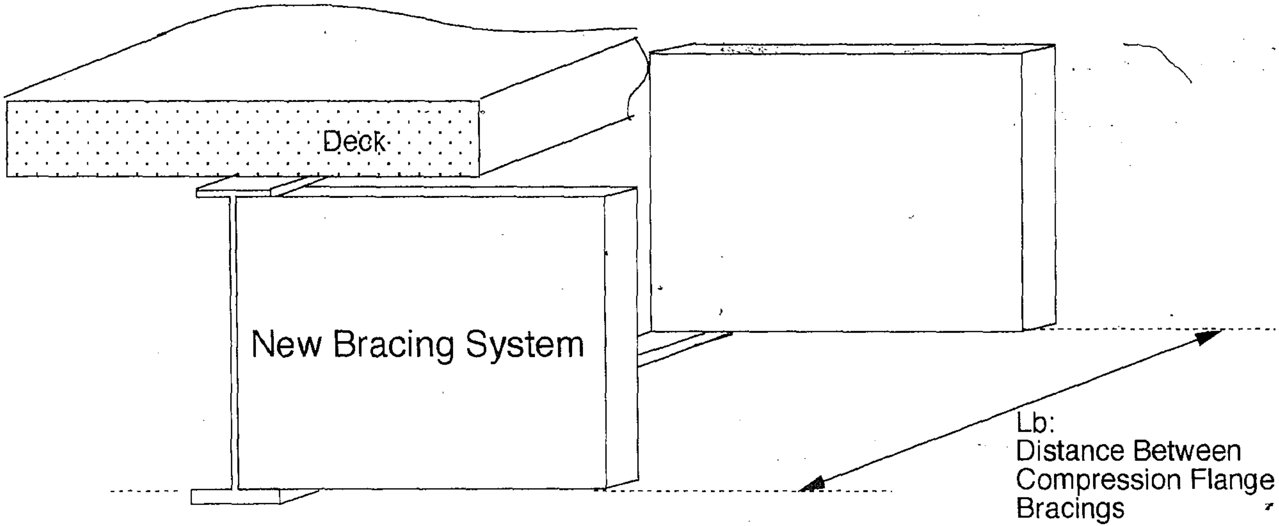


Fig. 6.1 Illustration of New Bracing System

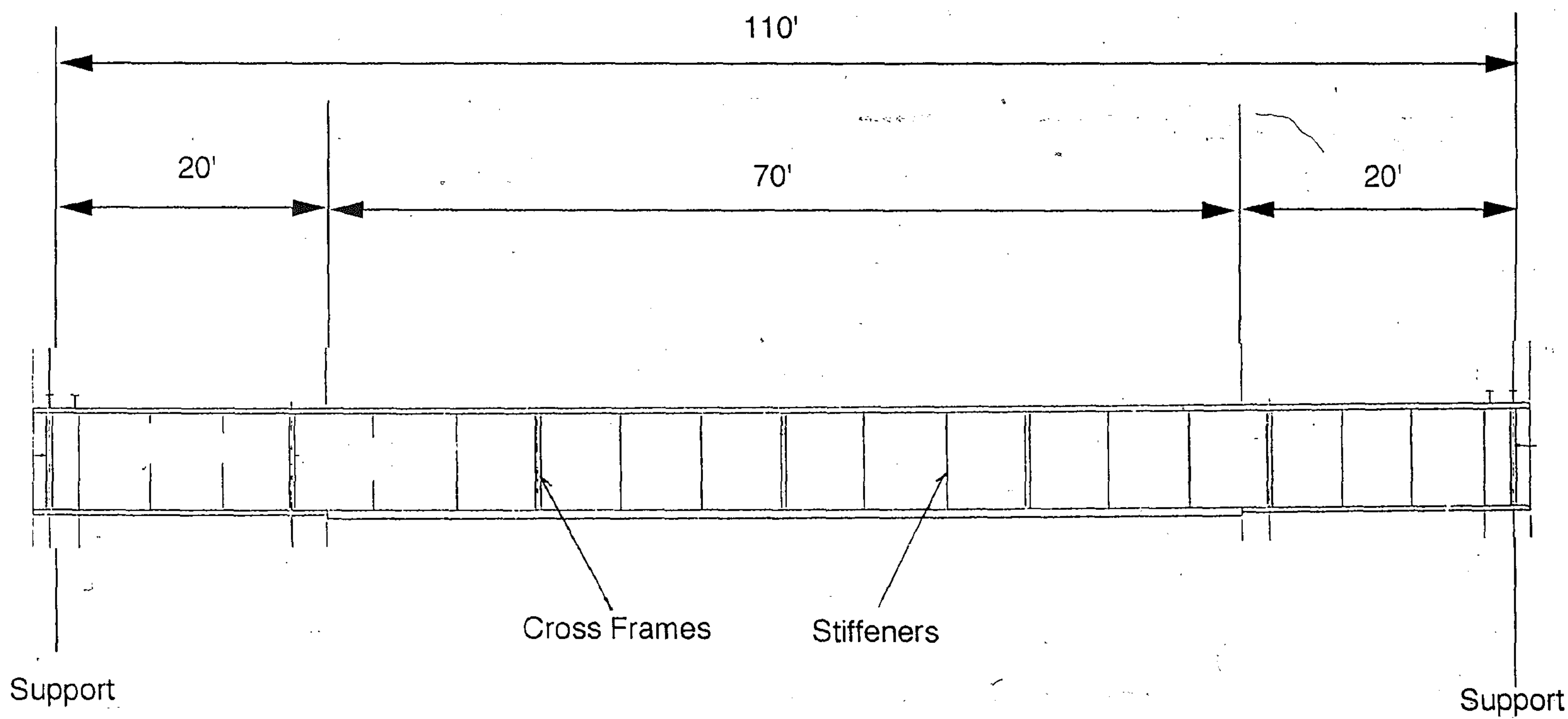
Appendices

Appendix A

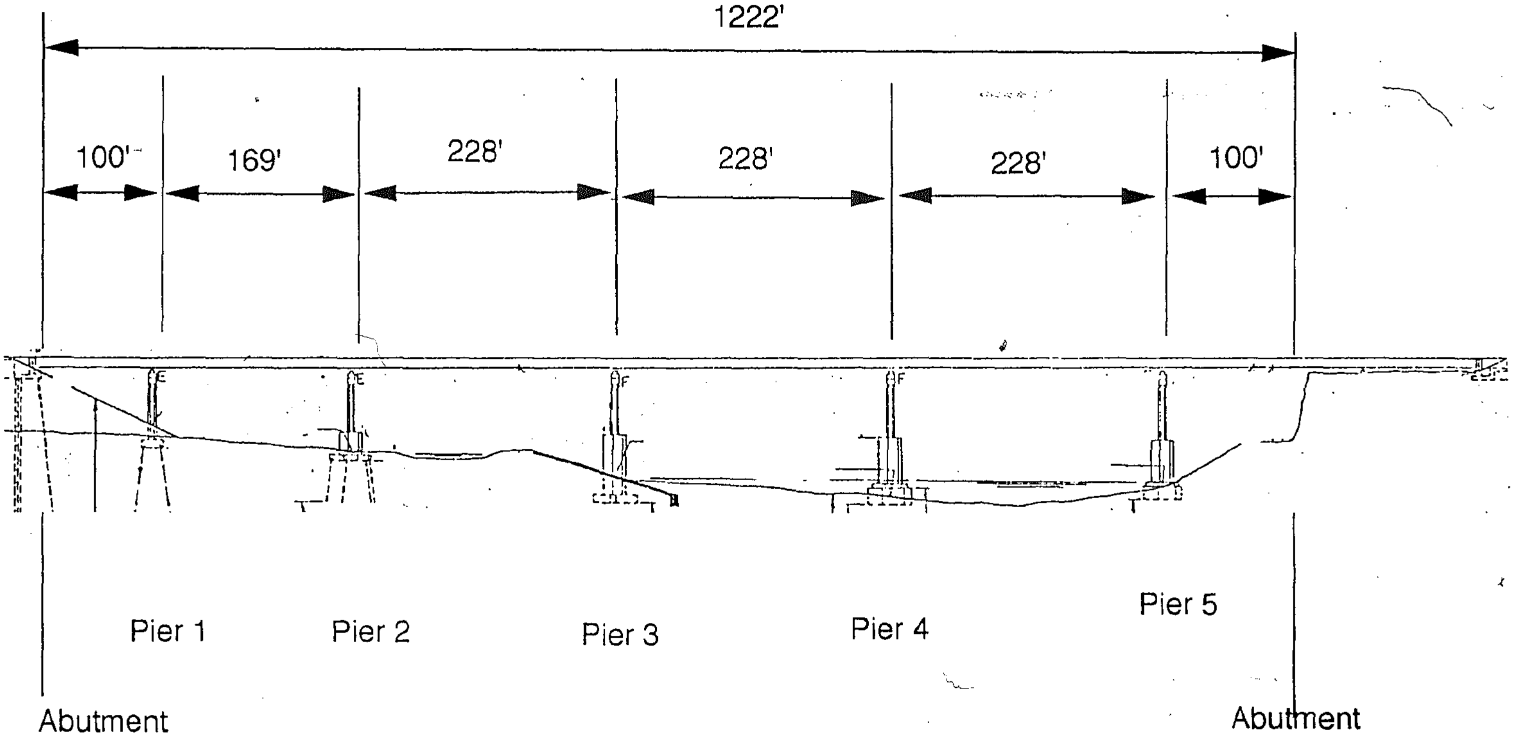
A-1 I-78 over Lehigh Street

A-2 I-78 over Delaware River

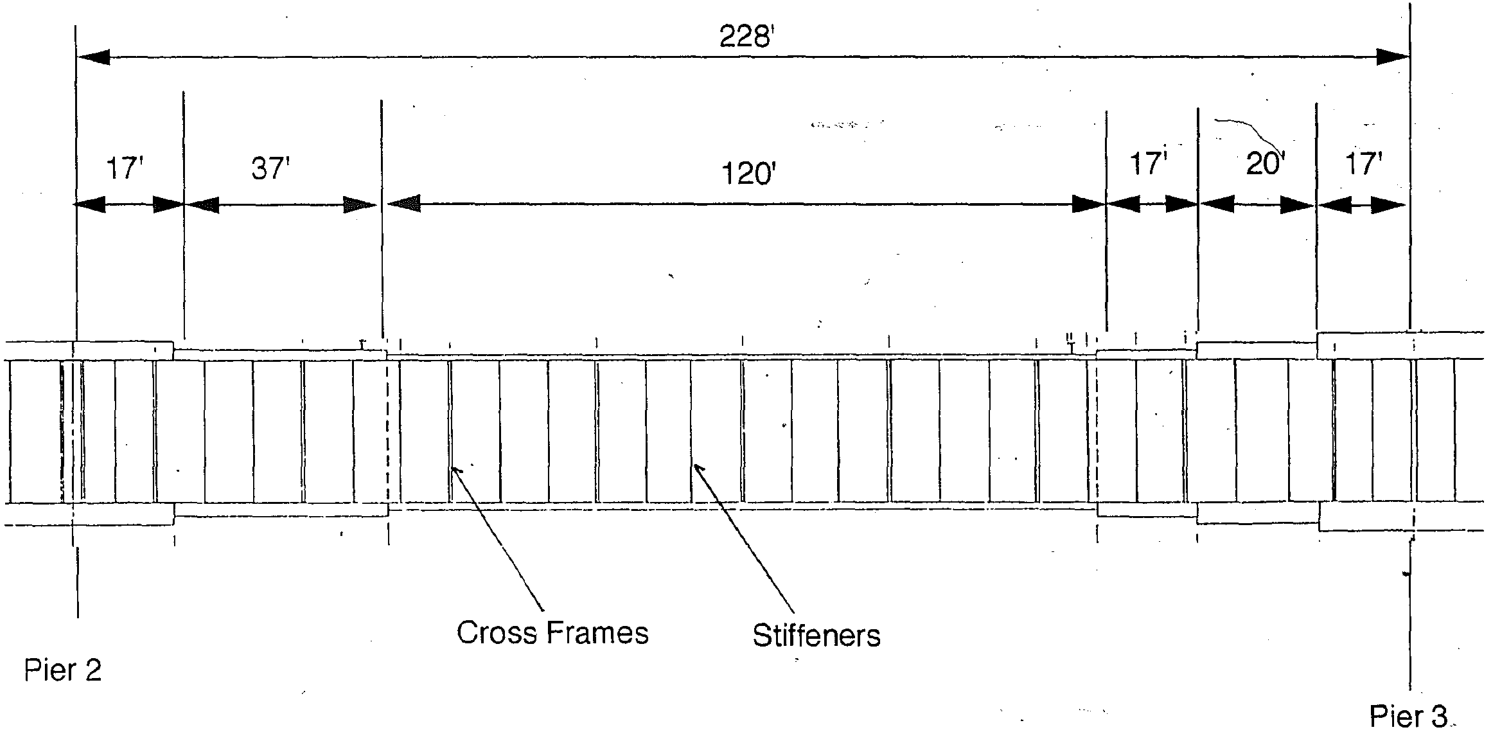
A-3 I-78 over Delaware River (between Pier 2 and Pier 3)



Appendix A-1 I-78 over Lehigh Street



Appendix A-2 I-78 over Delaware River



Appendix A-3 I-78 over Delaware River

(Between Pier 2 and Pier 3)

Appendix B

B-1 Unfactored Moment Envelop
(I-78 over Lehigh Street)

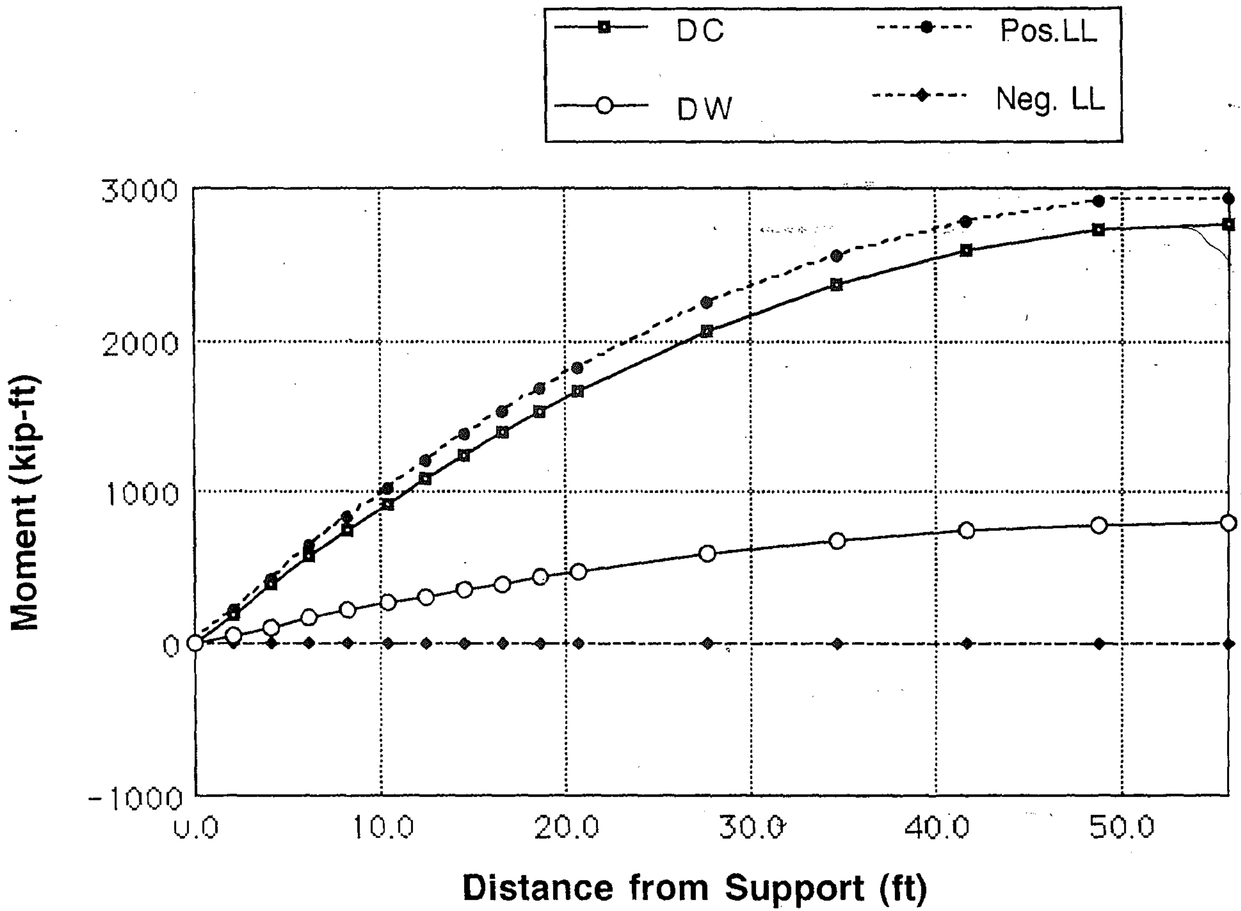
B-2 Live Load Moment Range
(I-78 over Lehigh Street)

B-3 Unfactored Shear Envelop
(I-78 over Lehigh Street)

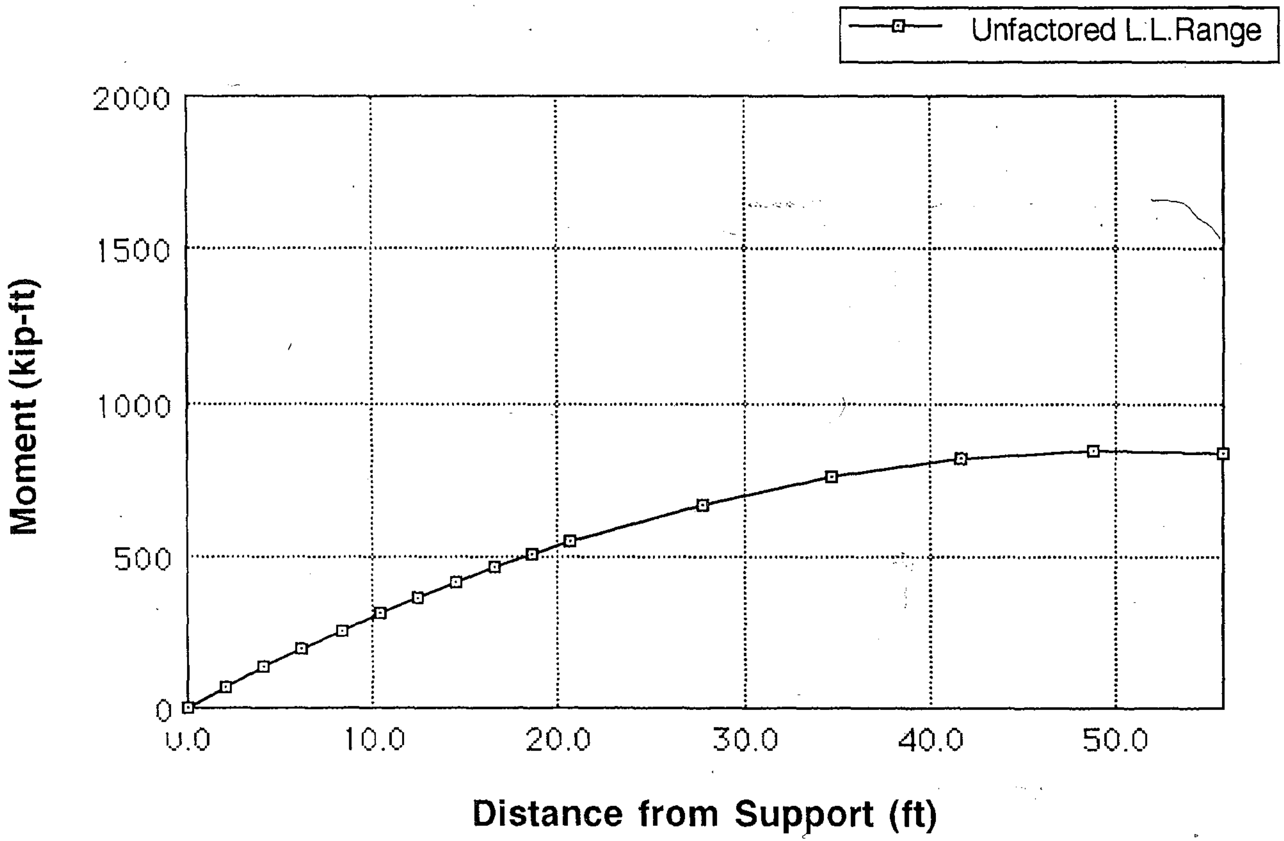
B-4 Unfactored Moment Envelop
(I-78 over Delaware River)

B-5 Live Load Moment Range
(I-78 over Delaware River)

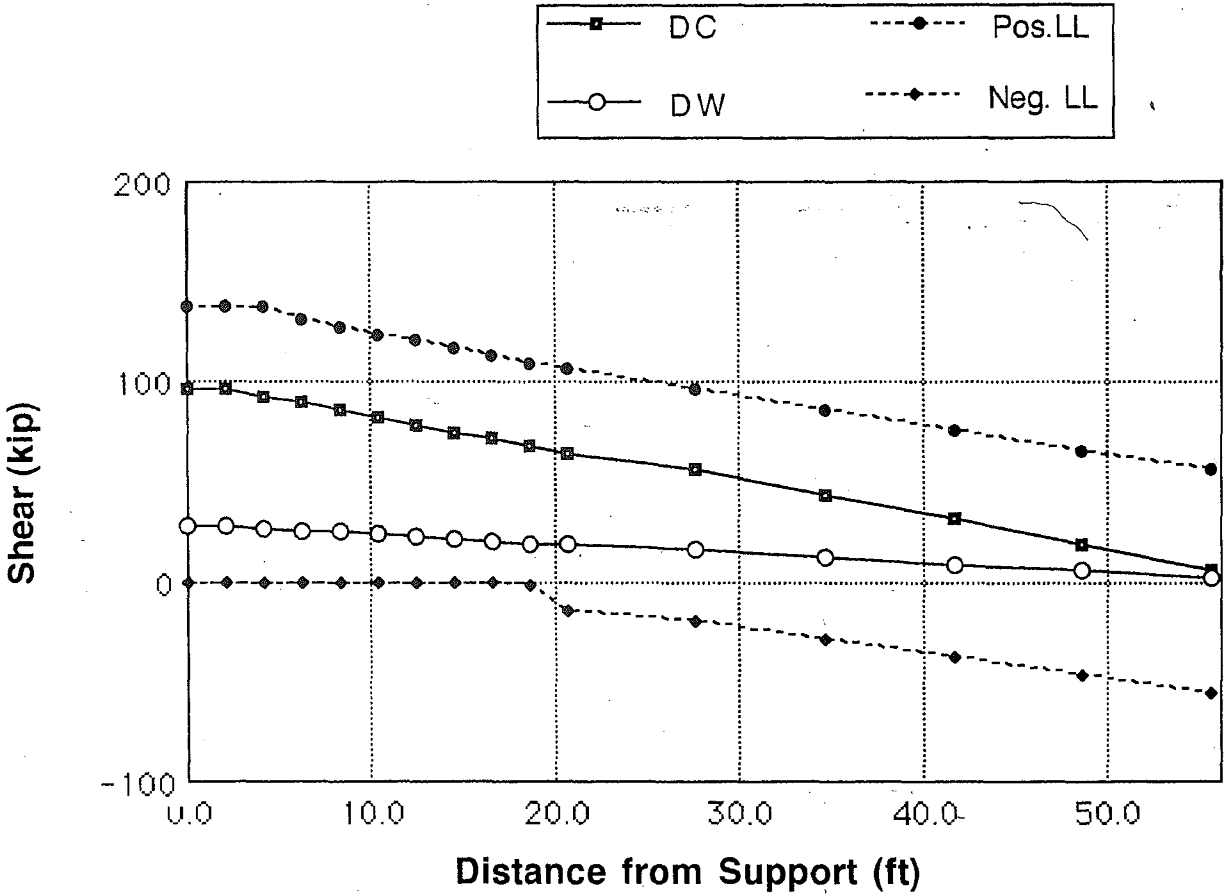
B-6 Unfactored Shear Envelop
(I-78 over Delaware River)



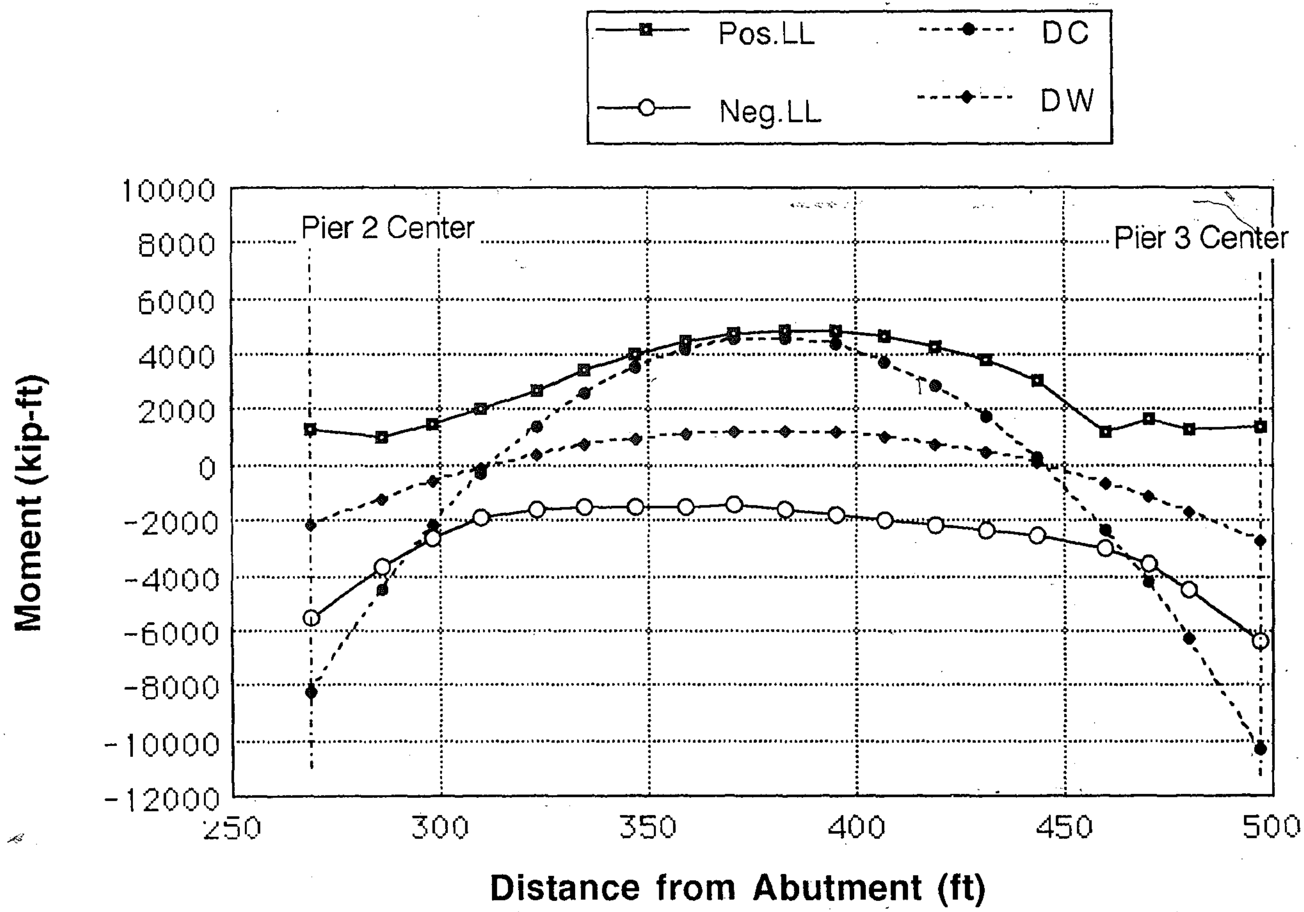
Appendix B-1 Unfactored Moment Envelop
(I-78 over Lehigh Street)



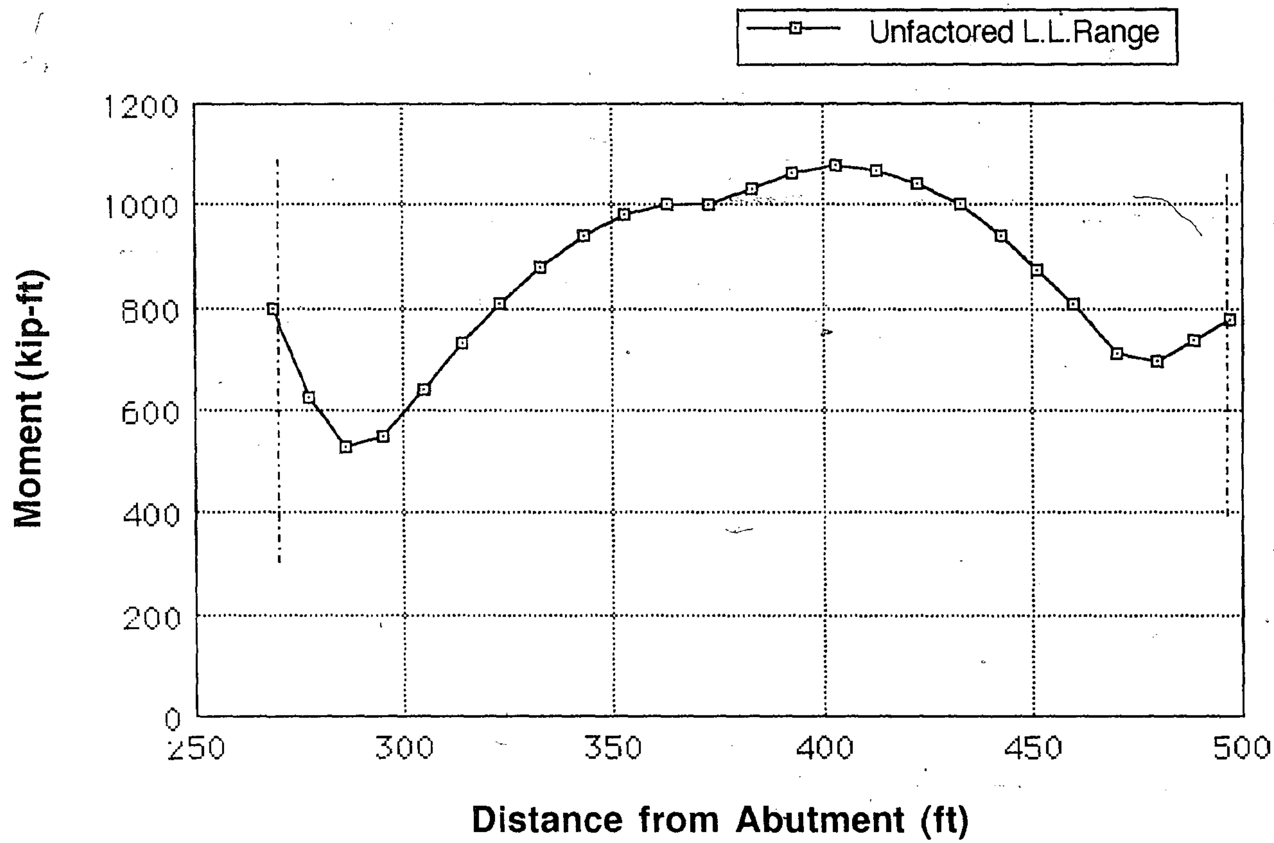
Appendix B-2 Live Load Moment Range
(I-78 over Lehigh Street)



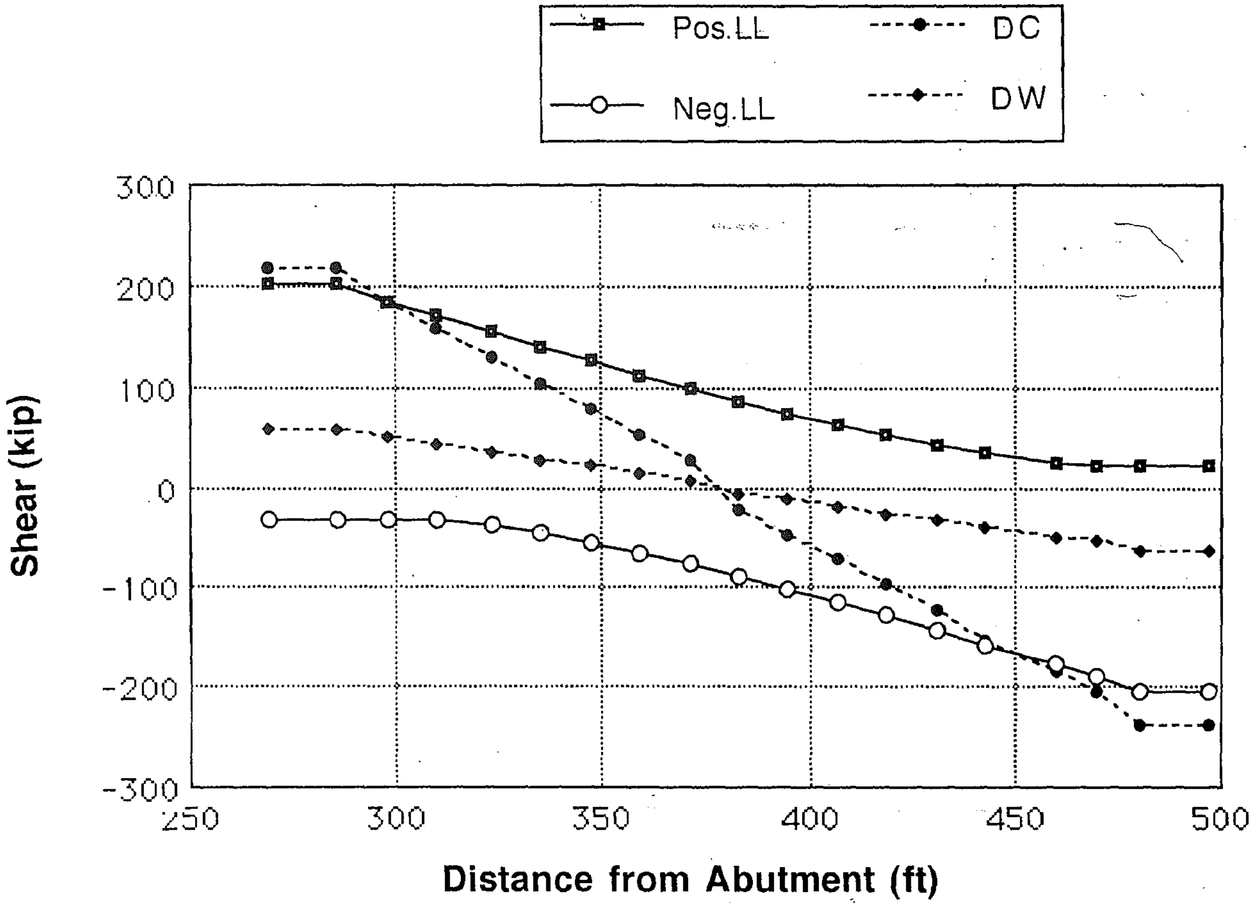
Appendix B-3 Unfactored Shear Envelop
(I-78 over Lehigh Street)



Appendix B-4 Unfactored Moment Envelop
(I-78 over Delaware River)



Appendix B-5 Live Load Moment Range
(I-78 over Delaware Bridge)



Appendix B-6 Unfactored Shear Envelop
(I-78 over Delaware River)

Vita

Koji Homma was born in Tokyo, Japan on February 16, 1960. He is the second son of Mr. Seiichi Homma and Mrs. Sachiko Homma. He has a wife, Yoko Homma and a daughter, Alisa Homma. Koji received his Bachelor of Science degree in Civil Engineering at the University of Tokyo in Japan in March 1984. Immediately after graduating from the university, he joined Nippon Steel Corporation, the world largest steel producer. He received the Nippon Steel President Award for his effort on the development of the corrosion protection method for steel structure using titanium clad steel plate. He has been a Senior Research Engineer in the Steel Structure Development Center in Nippon Steel Corporation since 1992, and he has received a leave of absence from the company for two years. While at Lehigh, Koji has dedicated himself on the research project at the Center for Advanced Technology for Large Structural Systems (ATLSS).

END

OF

TITLE