ANALYSIS AND DESIGN OF A TRIANGULAR CROSS SECTION TRUSS FOR A HIGHWAY BRIDGE

by

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(ABSTRACT)

A review of the available literature on past and present uses and advantages of a triangular cross section truss was made. A span length of 150 feet was selected for study of a truss to carry two lane traffic over a 30 foot wide roadway. The structural analysis program TRUSS1, written by the author, was used for preliminary analysis of various truss configurations. Several primary bracing, sway bracing, and truss depth configurations were analyzed, and final selection for design was based on least weight criteria.

The final design conformed to the AASHTO Bridge Specifications. The design dealt with member selection considering buckling and stress constraints, joint design, deflection criteria, bearings, and secondary stress considerations.

The structural analysis program STRUDL was used to check the final design and verify the accuracy and results of the TRUSS1 program. The paper cited 80 references.
ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>II.</td>
<td>REVIEW OF TRIANGULAR CROSS SECTION STRUCTURAL SYSTEMS</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>A. ANTENNAS AND TOWERS</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>B. ROOF STRUCTURES</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>C. PETROLEUM INDUSTRY STRUCTURES</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>D. MISCELLANEOUS STRUCTURES</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>E. BRIDGES</td>
<td>9</td>
</tr>
<tr>
<td>III.</td>
<td>ANALYSIS OF TRIANGULAR CROSS SECTION BRIDGE TRUSS</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>A. DESCRIPTION AND SCOPE OF STUDY</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>1. SPECIFICATIONS AND DESIGN LOADS</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>2. BRIDGE DECK</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>3. BRIDGE TRUSS</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>4. COMPUTER PROGRAM TRUSS1</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>B. DESIGN LOAD CALCULATIONS</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>1. DEAD LOAD</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>2. LIVE LOAD AND IMPACT</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>3. WIND LOAD</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>4. INITIAL ANALYSIS PARAMETERS</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>C. ANALYSIS OF PRIMARY BRACING</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>D. ANALYSIS OF SWAY BRACING</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>E. ANALYSIS OF TRUSS DEPTH</td>
<td>41</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>IV. DESIGN OF BRIDGE TRUSS</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>A. INITIAL DESIGN ANALYSIS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. DESIGN LOADING CONDITIONS</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>2. DESIGN PARAMETERS</td>
<td>49</td>
<td></td>
</tr>
<tr>
<td>B. FINAL DESIGN ANALYSIS</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>1. TRUSS MEMBERS</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>2. FLOORBEAMS</td>
<td>57</td>
<td></td>
</tr>
<tr>
<td>C. DEFLECTION CRITERIA</td>
<td>58</td>
<td></td>
</tr>
<tr>
<td>D. DESIGN OF CONNECTIONS</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>E. DESIGN OF BEARINGS</td>
<td>63</td>
<td></td>
</tr>
<tr>
<td>F. VERIFICATION OF ANALYSIS</td>
<td>66</td>
<td></td>
</tr>
<tr>
<td>V. SUMMARY</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>VI. CONCLUSIONS</td>
<td>73</td>
<td></td>
</tr>
<tr>
<td>REFERENCES</td>
<td>75</td>
<td></td>
</tr>
<tr>
<td>GENERAL BIBLIOGRAPHY</td>
<td>81</td>
<td></td>
</tr>
<tr>
<td>APPENDIX</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. TRUSS1 PROGRAM LISTING</td>
<td>82</td>
<td></td>
</tr>
<tr>
<td>VITA</td>
<td>101</td>
<td></td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TYPICAL TRIANGULAR CROSS SECTION TRUSS</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>AASHTO HS20 LIVE LOADS</td>
<td>13</td>
</tr>
<tr>
<td>3A</td>
<td>BRIDGE DECK DETAILS - CROSS SECTION</td>
<td>15</td>
</tr>
<tr>
<td>3B</td>
<td>BRIDGE DECK DETAILS - PROFILE</td>
<td>16</td>
</tr>
<tr>
<td>4</td>
<td>TYPICAL TRUSS PANEL POINT</td>
<td>19</td>
</tr>
<tr>
<td>5</td>
<td>GENERAL TRUSS CONFIGURATION</td>
<td>21</td>
</tr>
<tr>
<td>6</td>
<td>WIND LOADS ON BRIDGE</td>
<td>32</td>
</tr>
<tr>
<td>7</td>
<td>PRIMARY BRACING CONFIGURATIONS</td>
<td>37</td>
</tr>
<tr>
<td>8</td>
<td>SWAY BRACING CONFIGURATIONS</td>
<td>42</td>
</tr>
<tr>
<td>9</td>
<td>TRUSS WEIGHT VS. DEPTH</td>
<td>46</td>
</tr>
<tr>
<td>10</td>
<td>FINAL TRUSS CONFIGURATION</td>
<td>47</td>
</tr>
<tr>
<td>11</td>
<td>TYPICAL JOINT DETAILS</td>
<td>61</td>
</tr>
<tr>
<td>12</td>
<td>BEARING DETAILS</td>
<td>65</td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>BRIDGE DECK DEAD LOADS</td>
<td>26</td>
</tr>
<tr>
<td>2A</td>
<td>HS20 TRUCK LIVE LOADS AND IMPACT LOADS</td>
<td>29</td>
</tr>
<tr>
<td>2B</td>
<td>HS20 LANE LIVE LOADS AND IMPACT LOADS</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>INITIAL WIND LOADS</td>
<td>33</td>
</tr>
<tr>
<td>4</td>
<td>TRUSS MEMBER LINKED GROUPINGS</td>
<td>36</td>
</tr>
<tr>
<td>5</td>
<td>COMPUTER ANALYSIS OF TRUSS GEOMETRIES</td>
<td>39</td>
</tr>
<tr>
<td>6</td>
<td>PRIMARY BRACING MEMBER FORCES</td>
<td>40</td>
</tr>
<tr>
<td>7</td>
<td>FINAL WIND LOADS</td>
<td>50</td>
</tr>
<tr>
<td>8</td>
<td>COMPUTER ANALYSIS OF TRUSS DESIGN</td>
<td>53</td>
</tr>
<tr>
<td>9</td>
<td>MAXIMUM DEFLECTIONS AND BEARING REACTIONS</td>
<td>59</td>
</tr>
<tr>
<td>10</td>
<td>RESULTS OF STRUDEL ANALYSIS</td>
<td>68</td>
</tr>
</tbody>
</table>
**LIST OF SYMBOLS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>cross sectional area, in²</td>
</tr>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>A_{min}</td>
<td>minimum cross sectional area, in²</td>
</tr>
<tr>
<td>A_f</td>
<td>cross sectional area from FSD solution, in²</td>
</tr>
<tr>
<td>AWS</td>
<td>American Welding Society</td>
</tr>
<tr>
<td>C</td>
<td>compression</td>
</tr>
<tr>
<td>d</td>
<td>truss depth, ft</td>
</tr>
<tr>
<td>E</td>
<td>modulus of elasticity, ksi</td>
</tr>
<tr>
<td>F_{allow}</td>
<td>maximum allowable stress, ksi</td>
</tr>
<tr>
<td>F_{ax}</td>
<td>axial stress, ksi</td>
</tr>
<tr>
<td>F_b</td>
<td>bearing stress, ksi</td>
</tr>
<tr>
<td>FSD</td>
<td>Fully Stressed Design algorithm</td>
</tr>
<tr>
<td>F_{y}</td>
<td>material yield strength, psi</td>
</tr>
<tr>
<td>f</td>
<td>fiber stress in member, ksi</td>
</tr>
<tr>
<td>f'_{c}</td>
<td>concrete compressive strength, psi</td>
</tr>
<tr>
<td>I</td>
<td>impact load, or moment of inertia, in⁴</td>
</tr>
<tr>
<td>I_{r}</td>
<td>moment of inertia for KL/r requirement, in⁴</td>
</tr>
<tr>
<td>K</td>
<td>effective length factor</td>
</tr>
<tr>
<td>L</td>
<td>live load, or member length, ft</td>
</tr>
<tr>
<td>M</td>
<td>bending moment, kip-ft</td>
</tr>
<tr>
<td>P</td>
<td>load or reaction, kip</td>
</tr>
<tr>
<td>r</td>
<td>radius of gyration, in</td>
</tr>
<tr>
<td>r_{min}</td>
<td>minimum required radius of gyration, in</td>
</tr>
</tbody>
</table>
LIST OF SYMBOLS (cont.)

S - section modulus, in³
T - tension
TS - tubular steel section
t - wall thickness of tubular member, in
\( t_{\text{min}} \) - minimum required wall thickness, in
\( t_w \) - web thickness, in
V - variable axle spacing for HS20 truck loading, ft
W - wind load
WL - wind load on live load
w - distributed load, lb/ft
X,Y,Z - global coordinate axis
\( \phi \) - internal angle, degrees
\( \rho \) - density of material, lb/ft³
\( \Delta \) - deflection, in
1. INTRODUCTION

A triangular cross section truss is simply a truss with three main chord members as shown in Figure 1. These chord members are separated by batten or lattice bracing which varies in configuration and spacing along the truss depending on the type and location of loads the truss must resist. The spacing of the main chord members vary with respect to each other depending on the type of loading, but are usually spaced equidistant to form an equilateral triangle in cross section.

This truss configuration has been used on a limited basis for a number of structures such as crane booms, highway overhead sign structures, portal frames, offshore oil rig platform legs, material transfer and pipeline bridges, roof truss beams, antennas, and pedestrian bridges. There have been few instances in recent history where a bridge truss of this nature has been designed and constructed for heavy loads such as highway and railway traffic.

In structural design, an engineer attempts to minimize cost, weight, construction time, and labor and to maximize efficiency of the structure. Studies\(^{(1,2)}\) of this type of truss under continuously distributed loads and cantilever conditions indicate a weight and material savings of up to
Figure 1. Typical Triangular Cross Section Truss.
13.9% can be gained when using a triangular cross section truss over a conventional rectangular truss of four main chord members. This is due mainly to the elimination of one main chord member and related bracing. The triangular truss required 35.1% fewer joints which would reduce fabrication costs, construction time, and labor costs. Research on the aerodynamics of certain trusses indicates that triangular trusses have an 11.3% smaller drag coefficient than rectangular trusses, due mainly to the reduction in exposed surface area. In design this will be reflected in lighter members required for triangular trusses to resist wind loads, and thus an overall lighter design.

Realizing these advantages, it is the subject of this project report to discuss the past and present uses of this type of truss and to present a design for a highway bridge utilizing a triangular cross section truss. This design will include a computer analysis to determine the most efficient truss configuration, selection of truss member section shapes, and design of truss joints and bearing details. The overall design will conform to the applicable AASHTO specifications.
II. REVIEW OF TRIANGULAR CROSS SECTION STRUCTURAL SYSTEMS

This structural configuration offers many weight saving, aesthetic, and practical advantages over other structural forms and is becoming ever increasingly popular in its application. The following is a discussion of the more prevalent uses of this type of truss.

A. ANTENNAS AND TOWERS

The most familiar application of a three-sided truss is for guyed radio and television antennas or towers for either electrical, telephone, or satellite transmissions.

Because of the extreme height requirements, radio and television antennas are usually guyed towers. The primary loads on this type of structure are wind, ice and member weight. Wind loads are usually the critical loads for design. As stated in the introduction, research has shown that triangular cross section trusses are more aerodynamically efficient than rectangular trusses, and thus are more prevalently used in high guyed tower applications.

There have also been many applications of three-sided trusses for free-standing cantilever towers. This type of tower is usually used to support transmission lines and is more commonly rectangular in cross section. But as noted in the introduction, studies have shown that a
substantial weight savings may be achieved by using a triangular truss. For this reason, and because triangular towers require less right-of-way and are more aesthetically pleasing due to fewer structural members\(^2\), they are becoming increasingly popular in cantilever tower applications.

**B. ROOF STRUCTURES**

The increasing demand for long span roof structures to serve sports arenas, civic centers, and recreation buildings has created a need for trussed roof systems. This demand, coupled with the introduction of computers in the early 1960's which greatly simplified analysis of three-dimensional structures, generated a boon for new forms of roof systems.

Truss girders\(^8,9\) of triangular cross section have become a popular means of fulfilling the needs of new roof systems to have long, unobstructed spans of light weight. In a typical application, a series of simply supported truss girders are set down the length of the structure. Usually the truss is oriented so that the two main compression members are on top. Purlins or joists span the area longitudinally between the truss girders to support the roof. Architects favor this system for its openness and sharp, clean appeal. Engineers favor this system because of its efficiency. A triangular girder requires no additional lateral or wind bracing\(^10\), thus saving material and cost.
Also, the two top compression chords contribute more roof support than a two-dimensional joist, and thus reduce the span length of the purlins or joists between each successive girder.

In some instances truss girders are set close together to eliminate the need of purlins or joists. In other roof systems the truss girders are set side by side with the upper chord members of each girder bolted to the adjoining girders, or the adjoining top chord members from the girders are replaced by one single member. This creates a continuous "W" structure known as a folded plate roof.

The use of triangular truss girders is not limited to straight beams, the configuration having been incorporated into arches. Designs of triangular arches have shown a 10% weight reduction over customary rectangular truss designs. When these arches are spaced close together as to combine top chords, a barrel vault type roof structure is created.

When clearance requirements do not permit the use of an arch, a series of portal frames constructed of triangular truss girders may be erected. Purlins or joints span the area longitudinally between the portal frames. With the high cost of heating and cooling, it is now becoming customary to erect exterior portal frames and
support the roof and walls from the single lower chord. This reduces the volume of the controlled atmosphere in the building.

C. PETROLEUM INDUSTRY STRUCTURES

In the exploration for and transmission of oil and natural gas, the petroleum industry has frequently utilized the triangular truss for many different applications.

In offshore exploration in less than 350 feet of water, the "jack-up" drilling rig is the most commonly used. This is a mobile rig which, when towed to a location, is raised above sea level by jacking down its support legs to the ocean floor. Nearly half of the world's fleet of offshore rigs are of this type. In early rigs, these legs were constructed of cylindrical columns, but as operating depths increased, bending stresses in columns became critical, prompting the use of triangular lattice trusses which have greater strength and reduced wave force impact due to decreased surface area. These truss legs are inclined outward toward the bottom with a single compression chord facing outward and two tension chords facing inward toward the platform. This "canting" minimizes leg bending moments and increases stability.

This type of truss has also been used for tanker mooring masts. In this application, one end of the truss is anchored to the sea floor by a universal joint. The other
end extends above the surface of the ocean, kept vertical by buoyancy tanks attached to the truss just below the surface.\(^{(26)}\) Again, this configuration has been chosen because of reduced wave forces acting on the structure.

In transporting oil and gas to shore, triangular trusses have been used as pipe bridges to support oil, gas and utility lines over large waterborne spans. In this instance, a through-type truss is used with a single top compression chord and two lower tension chords, the piping supported by bracing between the lower chords.\(^{(28)}\) In some instances, tubular steel has been used when constructing the main chords.\(^{(29)}\) These tubular members provide a dual purpose, serving as structural members for the truss and also carrying oil or natural gas. In one case, a triangular truss configuration has been used to build a 358-foot long parabolic through-arch to carry a 16-inch transmission pipe over a gorge. The pipe is supported on bracing between the two lower chords.\(^{(30)}\)

D. MISCELLANEOUS STRUCTURES

Tower cranes\(^{(31)}\) and gantry cranes\(^{(32,33)}\) very often utilize triangular trusses for booms. In almost all cases, a single chord is oriented on top, either in tension or compression, depending on the crane application. The two lower chords are utilized as tracks to carry a traveler which moves the hook along the length of the boom.
In highway structures, triangular trusses have been used extensively for cantilever sign towers and overhead sign bridges which span the entire road width.\textsuperscript{32,34} Generally, two main chord members will be oriented parallel to the plane in which the sign is to be erected. These two chord members also support the sign. The third chord member acts in tension or compression to resist wind loads on the sign.

To construct space stations in the earth's orbit, NASA has conducted studies\textsuperscript{35} for using triangular trusses in space. NASA has already developed a ground model of the "Beam Builder"\textsuperscript{36}, which is a device that can automatically fabricate triangular truss beams of infinite length using lightweight "space age" materials. It requires a minimum of maintenance and can be transported by the space shuttle.

E. BRIDGES

The topic of this project report is to study the utilization of a triangular truss in a bridge design. There have been few instances in modern history where this type of truss has been incorporated in a bridge.

The noted English engineer, I.K. Brunel,\textsuperscript{37,38,39,40} built two unique bridges in England, the Chepstow Bridge in 1852, and the Royal Albert Viaduct in 1859. Both bridges were similarly constructed as through-trusses to support railway loadings. In cross section, they consisted of a
single top compression tube with two lower chords built as plate girders. The railway ran through the truss and was supported on top of these girders. Vertical members were also constructed of built-up plates, while chains were used for diagonal members.

In 1930 the Dueren Bridge in Germany was completed. (41) This was the first truly truss bridge of triangular cross section. It was a through-type truss supporting two railway lines with dimensions of 256 feet long, 44 feet wide between lower chords, and 47-1/2 feet high to the top chord. The bridge was constructed of lattice box girders with diagonal bracing forming a triangular pattern. The bridge was destroyed in 1945. (42)

In 1942 the Army Corps of Engineers developed a "V-type" bridge for use in combat areas to replaced destroyed structures. (43) The bridge could easily be assembled on-site for spans of 30 to 90 feet for Cooper E-40 railway loadings, or up to 180 feet for a 20-ton truck load. The structure consisted of a series of rolled beams connected to gussets to form a triangular pattern between the main chord members on all three sides. The truss was a deck type with a depth of 9 feet 6 inches.

In more recent times, the triangular truss has been used in highway arch bridges. The El Cubo bridge in Spain, which was completed in 1963, consists of a series of
triangular truss arches placed side by side to form a barrel vault arch bridge that spans 213 feet. There have been at least five additional bridges of this type built throughout Europe. (19)

The Cosmos Bridge over the St. Lawrence River in Montreal, Canada, was built to carry monorail traffic for the EXPO pavilions. It has five spans, each of 135 feet, and is made up of two independent truss bridges, each of triangular cross section. The single top compression chord supports the monorail. (44)

In Edmonton, Alberta, a deck truss footbridge has been erected. (45) Two top chords in compression support the floor beams and stringers for the deck. A single lower chord is in tension. This bridge has all welded connections and all structural steel members are rectangular tubing.

Noting that some of these bridges have been built to sustain heavy railway loading, it appears possible that a triangular type deck truss could be economically designed for multi-lane highway traffic.
III. ANALYSIS OF TRIANGULAR CROSS SECTION
BRIDGE TRUSS

A. DESCRIPTION AND SCOPE OF STUDY

1. Specifications and Design Loads

    Unless otherwise noted, all analysis and design methods
conform to the 1977 AASHTO Standard Specifications for
Highway Bridges (46) with the 1981 Interim Bridge
Specifications, (47) henceforth referred to as AASHTO.

    The structure is analyzed and designed for the HS20-44
truck loading or lane loading (AASHTO 1.2.5(C)). These
loads are illustrated in Figure 2. The loading that
produces the maximum stress in the truss member or
structural component under investigation determines whether
truck loading or lane loading is applied to the structure
(AASHTO 1.2.8(B)).

    The loadings that are considered to act on the
structure are dead load, live load, impact, wind load, and
wind load on live load. These loadings are discussed in
detail in other sections of this report.

    The scope of this project report is limited to the
analysis and design of the bridge superstructure (deck slab,
stringers, floorbeams, truss members, and bearings).
Components of the substructure (abutments, wingwalls, and
VARIABLE SPACING = 14 TO 30 FEET TO PRODUCE MAX. STRESS.
W = COMBINED WT. OF FIRST TWO AXLES.

(A) TRUCK LOADING

\[ w = 0.040K / \text{LINEAR FT. OF LOAD LANE} \]

(ROVING CONCENTRATED LOAD)
\[ P = 18K \text{ FOR MOMENT} \]
\[ P = 20K \text{ FOR SHEAR} \]

(B) LANE LOADING

FIGURE 2. AASHTO HS20 LIVE LOADS.
approach slabs) will not be covered.

2. Bridge Deck

In discussing the bridge deck slab, stringers, and floorbeams, design calculations are not presented. There are publications (48, 49, 50) readily available to the engineer which provide details for predesigned bridge decks of fixed dimensions, or give design tables for proportioning any given span length and width of deck. There are also several references (4, 51, 52) that provide step-by-step instructions and examples for designing bridge decks to meet AASHTO specifications. For this reason, this report does not go into detailed discussions of designing the bridge deck. What follows is a description of the resulting design by using these references (4, 51, 52).

Dimensions and details of the bridge deck are shown in Figure 3(A). A roadway width of 30 feet consisting of two 12-foot wide travel lanes and two 3-foot wide shoulders is used. These dimensions will allow the bridge deck to conform to all AASHTO road classifications (53, 54, 55) under minimum conditions. Dimensions and details for curbing, railing, and parapet are taken from AASHTO Figure 1.1.8, giving an overall cross section dimension of 33 feet, 6 inches. It is assumed that this bridge will be erected in rural areas, and thus no sidewalk is required.
The concrete deck slab was proportioned by the Load Factor Design method (AASHTO 1.5.30) using concrete with a compressive strength of $f_c^t = 4,000$ psi, and reinforcing steel with a yield strength of $F_y = 40,000$ psi. The 7-inch thick slab provided is sufficient to satisfy all requirements of this design method. Although normal weight concrete (150 lb/ft$^3$) is suggested in constructing the slab, lightweight concrete (115 lb/ft$^3$) may also be used. This provides an additional 21 lb/ft$^2$ dead load allowance for future bridge deck repaving.

Six stringers, spaced 6 feet 3 inches on center, support the slab. The stringers are designed as two-span, continuous beams, each span of 25 feet supported by floorbeams. Thus, each segment of the concrete deck is 50 feet long, which is a practical length for crack control in reinforced concrete. The stringers are designed as unshored composite beams (AASHTO 1.7.48) using the Allowable Stress Design method (AASHTO 1.7.40). Designing the beams as unshored was selected because labor cost savings far exceed the material savings over the case of the beams being shored during construction. To simplify the design of stringers, the outside stringer is designed as one would design an inside stringer, with dead loads from the slab divided equally among all stringers. This assumption is allowed by AASHTO 1.3.1(B)(2)(a). The composite beams
are fabricated from W16x31 rolled shapes selected from the AISC manual (56). Two 4-3/4-inch wide by 3/8-inch thick by 68 inches long cover plates are provided in the negative moment region of the stringers. A588 steel (56) with a yield stress of \( F_y = 50,000 \) psi is used in fabricating stringers, cover plates, and all welds. Stringer details are shown in Figures 3(A) and 3(B).

3. Bridge Truss

As described in the previous section, the stringers are designed as two-span, continuous beams, 25 feet per span. Thus, the panel points are located throughout the truss at 25-foot intervals. A typical panel point is shown in cross section in Figure 4. The panel points are designed as a deck-type truss consisting of a floorbeam to which the stringers are bolted and transfer loads to the truss. The ends of the floorbeams are assumed pinned. In this manner, no member end moments are developed in the floorbeam to transfer to the truss, and thus the analysis of the truss as consisting of axial forces only is preserved. The length of the floorbeam is 35 feet center-to-center of joints. This provides a clearance of 1 foot 10-1/2 inches from the outermost stringer to the centerline of the joint, and this is an adequate dimension to design and erect the connection. The two truss members which form the remaining sides of the triangular panel point (henceforth referred to
as delta members) are also assumed to be pinned to the floorbeams. Where the delta members meet at the centerline of the cross section, they are connected to a single, lower main truss chord. Two upper main truss chord members connect to the panel point at the floorbeam joints.

It is the intent of this report to investigate a single span length, simply supported, that would be most appropriate for a truss. This span length would be in excess of what could economically be bridged by plate girders, but would not be so long as to be more practical or economical to span with a suspension or cable-stayed bridge. The literature (50,57) deems this range to be of medium span for bridges and suggests a clear span of 150 feet would be appropriate for this study.

The truss configuration to be investigated is shown in Figure 5. Joint numbers (nodes) are represented inside squares, and member numbers (elements) are shown in circles. The truss is oriented such that two upper main truss chord members are in compression and the single lower main truss chord is in tension. The lower main truss chord terminates at the bearings through inclined endposts as suggested by AASHTO 1.7.44. The truss has four bearing points, located at the ends of the two main compression chords. All bearings are simply supported. The bracing member configuration, which is determined later in this report, is
that which produces the lightest structure while conforming to the AASHTO specifications.

The depth of the truss is also determined later in this report, and selection is based on the depth which produces the lightest structure. As an initial dimension for preliminary analysis, a depth (d) of 30 feet is used. This dimension translates into a depth-to-span ratio of $\frac{30}{150} = \frac{1}{5}$, which represents the higher limit of the range of this ratio deemed feasible by the literature.\(^{(57)}\)

As with the stringers and floorbeams, the truss members are proportioned using the Allowable Stress Design method (AASHTO 1.7.40), and fabricated of A588 steel with a yield stress of $F_y = 50,000$ psi. A588 steel is corrosion-resistant, and thus no painting is required.\(^{(56)}\) Welds are also designed using this grade of material.

4. Computer Program TRUSS1

The computer program TRUSS1 is used to analyze the various truss configurations and loading conditions. The program was developed and written by the author for other graduate course work and was modified and improved for use in this report. The program utilizes matrix methods to analyze three-dimensional truss structures for up to 60 members, 30 joints, and 14 loading conditions.

For output, the program computes member forces and stresses, reactions, and joint displacements. When a
multitude of loading conditions are analyzed, the program identifies the loading condition that produces the maximum stress in each member.

For optional output, the program selects member properties to satisfy a given allowable stress ($F_{allow}$) for each member based on the Fully Stressed Design (FSD) algorithm. This algorithm reproportions member properties until the member stress is within set tolerances of the given allowable stress. The user may select any one of three sets of equations that relate cross sectional area ($A$) to moment inertia ($I$) and section modulus ($S$) to solve the algorithm. In this report, equations presented by Arora, Haug, and Rim are used in the FSD solution. These equations are further discussed in Section B-4 of this chapter.

When using the FSD solution, the program can also link truss members into groupings and select member properties for all members of that group based on the maximum stress occurring in any one member. The advantages of this feature are discussed in Section B-2 of this chapter. A minimum cross section area ($A_{min}$) may also be defined for each member to supersede FSD calculations. Finally, after new section properties have been determined, the program computes the total structure weight based on the given
density of the material. A program listing is presented in Appendix 1.

B. DESIGN LOAD CALCULATIONS

1. Dead Load

In this report, dead loads are divided into two categories, dead load from the bridge deck (slab, parapet and railing, stringers, floorbeams) and dead load from the truss (truss members, connections, bearings).

Dead load from the bridge deck is applied to the truss as joint loads at the extreme ends of the floorbeams where they connect to the delta members. The floorbeams, which serve a dual purpose as bridge deck members and truss members, initially are designed for bending moment resulting from deck loads only. Axial loads from truss action are then considered and the sections initially selected are then analyzed for this additional axial load. To provide some flexibility in case the members initially selected are inadequate, an estimated beam weight slightly higher than the actual member weight is used for determining the total bridge deck dead load. The floorbeam members selected, and their estimated weights are:

Floorbeam 1, 17 W33x130 w = 135 lb/ft
Floorbeam 2, 8, 14 W33x201 w = 210 lb/ft
Floorbeam 5, 11 W36x150 w = 160 lb/ft
A summary of bridge deck dead loads then applied to the truss are given in Table 1.

Dead load from the truss consists of the total weight of the truss (minus floorbeam weight) divided equally among, and applied to, the 19 truss joints as joint loads. Since in the sequence of calculations, the truss depth, bracing configuration, and member sizes must be known to make a realistic estimate of the truss weight, to simplify the preliminary analysis dead load from the truss is not considered. It is assumed that dead load from the truss is small in comparison with dead load from the bridge deck and does not significantly affect the results of the preliminary analysis.

2. Live Load and Impact

AASHTO HS20 truck loading (1.2.5(C)) controls by producing the maximum joint live load forces (L) at the ends of the floorbeams, which in turn, are transmitted to the truss. Note that for a two-span, continuous beam of 25 feet per span, the variable axial spacing to produce maximum reactions is \( V = 14 \) feet for all cases (see Figure 2). These beam reactions must be distributed laterally across the width of the bridge deck to produce the maximum reactions at the ends of the floorbeams. In positioning these loads laterally, AASHTO specifies that one truck occupies a width of 10 feet within each design traffic lane.
TABLE 1. Bridge Deck Dead Loads

<table>
<thead>
<tr>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load, Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Z</td>
<td>-22.7</td>
</tr>
<tr>
<td>2</td>
<td>Z</td>
<td>-22.7</td>
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<tr>
<td>3</td>
<td>Z</td>
<td>-71.5</td>
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<tr>
<td>4</td>
<td>Z</td>
<td>-71.5</td>
</tr>
<tr>
<td>6</td>
<td>Z</td>
<td>-43.4</td>
</tr>
<tr>
<td>7</td>
<td>Z</td>
<td>-43.4</td>
</tr>
<tr>
<td>9</td>
<td>Z</td>
<td>-71.5</td>
</tr>
<tr>
<td>10</td>
<td>Z</td>
<td>-71.5</td>
</tr>
<tr>
<td>12</td>
<td>Z</td>
<td>-43.4</td>
</tr>
<tr>
<td>13</td>
<td>Z</td>
<td>-43.4</td>
</tr>
<tr>
<td>15</td>
<td>Z</td>
<td>-71.5</td>
</tr>
<tr>
<td>16</td>
<td>Z</td>
<td>-71.5</td>
</tr>
<tr>
<td>18</td>
<td>Z</td>
<td>-22.7</td>
</tr>
<tr>
<td>19</td>
<td>Z</td>
<td>-22.7</td>
</tr>
</tbody>
</table>

NOTE: Also see Figure 5 when referring to this table.
For a 30-foot wide roadway, two 15-foot wide design traffic lanes are permitted. To produce the maximum reaction at the end of the floorbeam the two trucks are placed as far as possible to the right side of the deck while still remaining in their respective design traffic lane.

Lane load is applied to the structure in a similar manner to that of truck load. To produce the maximum reactions at the end of the floorbeam two lane loads are placed as far as possible to the right side of the bridge deck while still remaining in their respective design traffic lanes. Two roving concentrated loads for shear (26 kips), one per lane load and distributed laterally over a width of 10 feet, are also placed to the extreme edge of the bridge deck within the design traffic lanes. Although lane load may control in producing the maximum stress in some members, truck loading only is used for the preliminary analysis.

Impact load factors (I) are applied to the live load on the structure to account for dynamic, vibratory, and impact effects. AASHTO (1.2.12) expresses impact as an increase in live load applied to the structure. This factor of increase is given by

\[ I = \frac{50}{L + 125} \leq 0.30 \]  

where \( L \) = length (in feet) of the portion of the span which
is loaded to produce the maximum stress in the member. When the deck slab is analyzed, this length corresponds to the stringer spacing \((L = 6.25')\); for the stringers the clear span dimension is used \((L = 25')\); and for the floorbeams the member length \((L = 35')\) is used. For these structural components, Equation 1 gives a value of \(l = 0.30\) (or 30\%). When analyzing the truss members for impact, the total bridge span length \((L = 150')\) is used. Thus, from Equation 1, the impact factor for truss members is

\[
I = \frac{50}{150 + 125} = 0.182 \text{ or } 18.2\%
\]

One should recall from the description of the TRUSS1 program capabilities that truss members can be linked into groups and member properties selected from that group based on the maximum stress occurring in any one member. If the truss members are linked in a symmetrical pattern about the truss centerline (i.e., floorbeams 1 and 17, delta members 3, 4, 15, and 16, etc.) moving live loads need only be applied to half of the truss. The program selects member properties for symmetric members on each side of the truss centerline. Tables 2(A) and 2(B) give the live load plus impact forces applied to one-half of the truss as the truck loading moves across the bridge and for lane load as the roving concentrated load moves across the bridge.
TABLE 2A. HS20 Truck Live Loads and Impact Loads

<table>
<thead>
<tr>
<th>Truck Location</th>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load, Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floorbeam 1</td>
<td>1</td>
<td>Z</td>
<td>-43.6</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Z</td>
<td>-58.1</td>
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<tr>
<td></td>
<td>3</td>
<td>Z</td>
<td>-24.4</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Z</td>
<td>-32.5</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Z</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Z</td>
<td>4.2</td>
</tr>
<tr>
<td>Floorbeam 2</td>
<td>1</td>
<td>Z</td>
<td>-0.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Z</td>
<td>-1.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Z</td>
<td>-57.5</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Z</td>
<td>-76.6</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Z</td>
<td>-14.6</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Z</td>
<td>-19.4</td>
</tr>
<tr>
<td>Floorbeam 5</td>
<td>1</td>
<td>Z</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Z</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Z</td>
<td>-6.1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Z</td>
<td>-8.1</td>
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<td></td>
<td>6</td>
<td>Z</td>
<td>-46.4</td>
</tr>
<tr>
<td></td>
<td>7</td>
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<td>Z</td>
<td>-24.4</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Z</td>
<td>-32.5</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>Z</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>Z</td>
<td>4.2</td>
</tr>
<tr>
<td>Floorbeam 8</td>
<td>6</td>
<td>Z</td>
<td>-0.9</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Z</td>
<td>-1.2</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>Z</td>
<td>-57.5</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Z</td>
<td>-76.6</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>Z</td>
<td>-14.6</td>
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<tr>
<td></td>
<td>13</td>
<td>Z</td>
<td>-19.4</td>
</tr>
</tbody>
</table>

NOTE: Also see Figure 5 when referring to this table.
### TABLE 2B. HS20 Lane Live Loads and Impact Loads

#### Uniform Lane Load

<table>
<thead>
<tr>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load, Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Z</td>
<td>-6.9</td>
</tr>
<tr>
<td>2</td>
<td>Z</td>
<td>-7.3</td>
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<tr>
<td>3</td>
<td>Z</td>
<td>-23.0</td>
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<td>4</td>
<td>Z</td>
<td>-24.3</td>
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<td>6</td>
<td>Z</td>
<td>-13.8</td>
</tr>
<tr>
<td>7</td>
<td>Z</td>
<td>-14.6</td>
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<tr>
<td>9</td>
<td>Z</td>
<td>-23.0</td>
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<td>10</td>
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<td>-24.3</td>
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<tr>
<td>13</td>
<td>Z</td>
<td>-14.6</td>
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<tr>
<td>15</td>
<td>Z</td>
<td>-23.0</td>
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<td>16</td>
<td>Z</td>
<td>-24.3</td>
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<tr>
<td>18</td>
<td>Z</td>
<td>-6.9</td>
</tr>
<tr>
<td>19</td>
<td>Z</td>
<td>-7.3</td>
</tr>
</tbody>
</table>

#### Concentrated Load

<table>
<thead>
<tr>
<th>Load Location</th>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load, Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floorbeam 1</td>
<td>1</td>
<td>Z</td>
<td>-29.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Z</td>
<td>-31.6</td>
</tr>
<tr>
<td>Floorbeam 2</td>
<td>3</td>
<td>Z</td>
<td>-29.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Z</td>
<td>-31.6</td>
</tr>
<tr>
<td>Floorbeam 5</td>
<td>6</td>
<td>Z</td>
<td>-29.8</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Z</td>
<td>-31.6</td>
</tr>
<tr>
<td>Floorbeam 8</td>
<td>9</td>
<td>Z</td>
<td>-29.8</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Z</td>
<td>-31.6</td>
</tr>
</tbody>
</table>

**NOTE:** Also see Figure 5 when referring to this table.
3. Wind Load

AASHTO 1.2.14(A) specifies two types of wind loads on the structure, wind load (W) and wind load on live load (WL).

Wind load (W) is applied to all surfaces in the truss profile. AASHTO denotes for girder members (outside stringer and parapet surface area) a load of 50 lb/ft², but not less than 300 lb/ft of member. For the given deck dimensions, the 300 lb/ft load controls. For truss members the applied load is 75 lb/ft², but not less than 150 lb/ft of member. For all truss members, the 150 lb/ft load controls. Since the exact truss configuration and depth is yet to be determined, wind load on the truss members is not considered at this time. The resulting load on the truss surface area is relatively small, and it is assumed that it does not significantly affect the results of the preliminary analysis. When applying wind loads, AASHTO assumes the bridge deck and stringers offer resistance as a rigid unit, thus wind load reactions are created at both ends of the floorbeams at the joints. This loading is shown in Figure 6. Wind loads applied to the bridge deck (outside stringer and parapet) are used in the preliminary analysis. This load is summarized in Table 3(A).

Wind load on live load (WL) is applied to the structure
WIND LOAD ON LIVE LOAD
100 LB./LINEAR FOOT

WIND LOAD
300 LB./FT. FOR GIRDER & PARAPET
150 LB./FT. FOR TRUSS MEMBERS

FIGURE 6. WIND LOADS ON BRIDGE
TABLE 3. Initial Wind Loads

(A) Wind Load (Bridge Deck Only)

<table>
<thead>
<tr>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Y</td>
<td>-1.9</td>
</tr>
<tr>
<td>2</td>
<td>Y</td>
<td>-1.9</td>
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<tr>
<td>3</td>
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<td>-3.8</td>
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<td>6</td>
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<td>7</td>
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<td>-1.9</td>
</tr>
<tr>
<td>19</td>
<td>Y</td>
<td>-1.9</td>
</tr>
</tbody>
</table>

(B) Wind Load on Live Load

<table>
<thead>
<tr>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Y</td>
<td>-0.6</td>
</tr>
<tr>
<td>2</td>
<td>Y</td>
<td>-0.6</td>
</tr>
<tr>
<td>3</td>
<td>Y</td>
<td>-1.3</td>
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<tr>
<td>4</td>
<td>Y</td>
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<td>16</td>
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<tr>
<td>18</td>
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<td>-0.6</td>
</tr>
<tr>
<td>19</td>
<td>Y</td>
<td>-0.6</td>
</tr>
</tbody>
</table>

NOTE: Also see Figure 5 when referring to this table.
to represent wind forces acting on moving vehicles as they cross the bridge. AASHTO specifies this as a 100 lb/linear foot load acting 6 feet above the bridge deck. This loading is also shown in Figure 6, and Table 3(B) summarizes resulting joint loads from wind load on live load.

4. Initial Analysis Parameters

Along with the structural dimensions and properties already discussed, additional parameters must be determined to analyze the indeterminate structure and to use the FSD solution of the computer program TRUSS1.

Initial cross section area for all truss members is arbitrarily chosen as $A = 10.0$ in.$^2$. A minimum cross section area $A_{\text{min}} = 0.01$ in.$^2$ is also chosen for all truss members. The value of $A_{\text{min}}$ will supercede FSD calculations when area ($A$) is less than 0.01 in.$^2$. Equations presented by Arora, Haug, and Rim$^{(59)}$ are used to solve the FSD algorithm. These equations are

$$A = 0.58 \ (1)^{0.5} \quad (2)$$

$$S = 0.58 \ (1)^{0.75} \quad (3)$$

Solving Equation 2 for $l$ we have

$$l = \left( \frac{A}{0.58} \right)^2 \quad (4)$$
with \( A = 10.0 \text{ in.}^2 \), then

\[
l = \left( \frac{10.0}{0.58} \right)^2 = 297.0 \text{ in.}^4
\]

A Young's Modulus of \( E = 29 \times 10^6 \text{ ksi} \) (AASHTO 1.7.1(A)), and density \( \rho = 490 \text{ lb/ft}^3 \) (AASHTO 1.2.2) are used for the A588 steel truss members with \( F_y = 50,000 \text{ psi} \). To simplify the preliminary analysis, an allowable stress of \( F_{\text{allow}} = 27.0 \text{ ksi} \) (AASHTO Table 1.7.1(A)) will be used for both tension and compression members.

Truss members are linked symmetrically about the center of the structure into groupings. These member groupings are given in Table 4.

C. ANALYSIS OF PRIMARY BRACING

The main function of bracing members is to prevent large deflections and buckling of the structure. For trusses this is accomplished by using diagonal members between joints to give the structure rigidity.

For primary bracing members, which are located on the two diagonal faces of the truss (See Figure 5, X-Z Plane), the main loads which these members must resist are dead load \( (D) \) and truck live load \( (L) \). Figure 7 shows four possible bracing configurations to resist these loads. In proposing various bracing configurations for study, only those orientations that are symmetric about the bridge center line
TABLE 4. Truss Member Linked Groupings

<table>
<thead>
<tr>
<th>Group Number</th>
<th>Members</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Floorbeams 1, 17</td>
</tr>
<tr>
<td>2</td>
<td>Floorbeams 2, 8, 14</td>
</tr>
<tr>
<td>3</td>
<td>Delta members 3, 4, 15, 16</td>
</tr>
<tr>
<td>4</td>
<td>Floorbeams 5, 11</td>
</tr>
<tr>
<td>5</td>
<td>Delta members 6, 7, 12, 13</td>
</tr>
<tr>
<td>6</td>
<td>Delta members 9, 10</td>
</tr>
<tr>
<td>7</td>
<td>Main chord compression members 18, 23, 24, 29</td>
</tr>
<tr>
<td>8</td>
<td>Main chord compression members 19, 22, 25, 28</td>
</tr>
<tr>
<td>9</td>
<td>Main chord compression members 20, 21, 26, 27</td>
</tr>
<tr>
<td>10</td>
<td>Main chord tension members 30, 33</td>
</tr>
<tr>
<td>11</td>
<td>Main chord tension members 31, 32</td>
</tr>
<tr>
<td>12</td>
<td>Endpoint members 34, 35, 36, 37</td>
</tr>
<tr>
<td>13</td>
<td>Primary bracing members 38, 41, 42, 45</td>
</tr>
<tr>
<td>14</td>
<td>Primary bracing members 39, 40, 43, 44</td>
</tr>
<tr>
<td>15</td>
<td>Sway bracing members 46, 51</td>
</tr>
<tr>
<td>16</td>
<td>Sway bracing members 47, 50</td>
</tr>
<tr>
<td>17</td>
<td>Sway bracing members 48, 49</td>
</tr>
</tbody>
</table>
FIGURE 7. PRIMARY BRACING CONFIGURATIONS
in the X-Z plane are considered.

As discussed in Section A-4 of this chapter, the FSD solution of the computer program TRUSS1 is used to determine the member properties necessary to satisfy stress requirements for the given loading conditions, and to determine the total truss weight. Values of D and L are taken from Table 1 and Table 2. As stated in Section A-3 of this chapter, a truss depth of \( d = 30 \) feet is used for analyzing all bracing configurations.

A summary of the computer analysis is given in Table 5(A). It can be seen from the results that bracing configuration (I) produces the lightest weight truss (36.53 Kips), but configuration (I) is only 0.2% lighter than configuration (III) (36.61 Kips). With only a small percentage difference in total weight, no clear choice of configuration (I) over configuration (III) can be made.

Table 6 gives further results of the computer analysis. The total member forces in the main tension and compression members were calculated, and the ratio between the two groups \( \frac{C}{T} \) was computed. It can be seen that bracing configuration (III) is more efficient in distributing loads on the truss equally to main tension and compression members (\( \frac{C}{T} = 1.08 \)). This should result in a lighter overall structure and thus configuration (III) is used in further analysis and design.
TABLE 5. COMPUTER ANALYSIS OF TRUSS GEOMETRIES

(A) PRIMARY BRACING

<table>
<thead>
<tr>
<th>Bracing Configuration</th>
<th>Truss Weight (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>36.53</td>
</tr>
<tr>
<td>II.</td>
<td>38.75</td>
</tr>
<tr>
<td>III.</td>
<td>36.61</td>
</tr>
<tr>
<td>IV.</td>
<td>40.93</td>
</tr>
</tbody>
</table>

(B) SWAY BRACING

<table>
<thead>
<tr>
<th>Bracing Configuration</th>
<th>Truss Weight (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>1.45</td>
</tr>
<tr>
<td>II.</td>
<td>1.70</td>
</tr>
<tr>
<td>III.</td>
<td>1.71</td>
</tr>
</tbody>
</table>

(C) TRUSS DEPTH

<table>
<thead>
<tr>
<th>Truss Depth (in.)</th>
<th>Truss Weight (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>210</td>
<td>53.13</td>
</tr>
<tr>
<td>260</td>
<td>45.38</td>
</tr>
<tr>
<td>310</td>
<td>40.65</td>
</tr>
<tr>
<td>360</td>
<td>37.71</td>
</tr>
<tr>
<td>410</td>
<td>35.85</td>
</tr>
<tr>
<td>460</td>
<td>34.70</td>
</tr>
<tr>
<td>510</td>
<td>34.11</td>
</tr>
<tr>
<td>560</td>
<td>33.89</td>
</tr>
<tr>
<td>610</td>
<td>33.95</td>
</tr>
<tr>
<td>660</td>
<td>34.22</td>
</tr>
<tr>
<td>710</td>
<td>34.67</td>
</tr>
<tr>
<td>760</td>
<td>35.29</td>
</tr>
<tr>
<td>Primary Bracing Configuration</td>
<td>Total Main Compression Forces (C)</td>
</tr>
<tr>
<td>------------------------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>i.</td>
<td>2778.5</td>
</tr>
<tr>
<td>II.</td>
<td>2959.0</td>
</tr>
<tr>
<td>III.</td>
<td>2591.0</td>
</tr>
<tr>
<td>IV.</td>
<td>3144.6</td>
</tr>
</tbody>
</table>
**D. ANALYSIS OF SWAY BRACING**

For sway bracing members, which are located on the horizontal face of the truss (See Figure 5, X-Y Plane), the primary loads which these members must resist are wind load (W) and wind load on live load (WL). Figure 8 shows three possible bracing configurations to resist these loads.

Again, the FSD solution of the computer program TRUSS1 is used to determine required member properties and total truss weight. Two loading conditions, wind load (W), and 30% of (W) plus wind load on live load (0.3W + WL), as discussed in AASHTO 1.2.14 are used to analyze the proposed bracing configurations. Values for W and 0.3W + WL are taken from Table 3(A) and 3(B).

A summary of the computer analysis is given in Table 5(B). It can be seen from the results that bracing configuration (1) produces the lightest weight truss (1.45 Kips), and thus is the configuration used for further analysis and design. It should be noted that configuration (1) was arbitrarily selected for use when analyzing the truss for primary bracing in Section C.

**E. ANALYSIS OF TRUSS DEPTH**

It is obvious that increasing the depth of the truss causes member forces to decrease, and thus a smaller member cross section is required. But as the members become longer
FIGURE 8. SWAY BRACING CONFIGURATIONS
as depth increases, the more dead load weight is contributed to the truss from the members. At some point of increasing depth these two factors of decreased member cross section and increased member length will offset each other.

Dead load (D) and truck live load (L) are used to analyze the various truss depths. Values of D and L are taken from Tables 1 and 2, respectively. An initial truss depth of \( d = 210 \) inches is used. This corresponds to an internal angle at the truss panel points of \( \alpha = 45^\circ \) (See Figure 4). The truss depth is increased by increments of 50 inches up to a final analysis of \( d = 760 \) inches.

In order to introduce the increasing dead load weight of the truss into the analysis without lengthy computations, the weight of the truss computed from the initial analysis of each truss depth is applied as an additional dead load weight distributed equally over the 19 truss joints as joint loads. That particular truss depth is then reanalyzed for the additional dead load. It should be noted that when distributing the dead load weight of the truss to the joints, a load value slightly higher than actually calculated is used (1.1 to 4.5\% increase). This increased dead load weight is to compensate for the increased truss member size required to support the additional dead load.

Table 5(C) gives the result of the analysis. It can be seen that a truss depth of \( d = 560 \) inches produces the
lightest weight truss (33.89 Kips). However, at this point the analysis has not considered slenderness effects which limit the lengths of the members as discussed in AASHTO 1.7.5. Slenderness effects are represented by the slenderness ratio, which is expressed as

$$\frac{KL}{r} \leq \text{AASHTO Max. Value} \quad (5)$$

where $K$ reflects end conditions and is the effective length factor, $L$ is the unbraced length of the member (in inches), and $r$ is the radius of gyration of the member cross section. AASHTO 1.7.5 gives the maximum allowable value for the slenderness ratio as

- 120 for main compression members
- 140 for secondary compression members
- 200 for main tension members
- 140 for main members with stress reversals

Solving Equation 5 for radius of gyration $r$, and assuming pinned end connections for a value of $K = 1.0$ (AASHTO Appendix C, Table C-1), gives

$$r_{\text{min}} \geq \frac{(1.0)L}{\text{AASHTO Max. Value}} \quad (6)$$

Square structural tubing is chosen for the design of truss members. The advantages of structural tubing is discussed later in Chapter 4, Section A-2. Solving Equation 6 for the different members (i.e., main truss chord tension...
and compression members, bracing members, etc.) an $r_{\text{min}}$ value is determined for each type. The lightest tubular cross section to satisfy the $r_{\text{min}}$ value for each member is selected from the AISC manual \cite{56}. This area ($A_r$) necessary to satisfy $KL/r$ requirements is then compared with the cross section area necessary to satisfy stress requirements ($A_f$). A plot of truss weight vs. depth from Table 5(C) is shown in Figure 9. Also shown is the point of increasing truss depth that $KL/r$ requirements control in selecting truss members for various groups. Note that at a depth of 260 inches or less, no tubular sections are available to satisfy stress requirements for groups 10 and 11, and at a depth of 360 inches or greater, $KL/r$ requirements govern for groups 3 and 5.

Based on the results shown in Figure 9, a depth of 360 inches is used for the truss design. This point on the curve represents a location where increasing truss depth begins to have a diminishing effect on producing a lighter truss. Also, at this point the majority of truss members are sized to satisfy stress requirements instead of $KL/r$ requirements, resulting in lighter members, and tubular sections are available to satisfy all member requirements.

The final truss configuration as determined from Sections C, D, and E of this chapter is shown in Figure 10.
Figure 9. Truss Weight vs. Depth
IV. DESIGN OF BRIDGE TRUSS

A. INITIAL DESIGN ANALYSIS

1. Design Loading Conditions

When determining the truss configuration in Chapter 3, the structure was isolated in a particular plane and the principal loads which occurred in that plane were used for the analysis. Now that a final configuration has been determined, all load types can be applied to the truss simultaneously as required by AASHTO.

AASHTO 1.2.22 states the loading combinations for designing the truss. For Allowable Stress Design, ten group loading combinations are given. Group loading I, II, and III control for the types of loads that are being considered in this report (D, L, I, W, WL).

These group loadings are:

1. \( D + (L + 1) \)
2. \( D + W \)
3. \( D + (L + 1) + 0.3W + WL \)

With the truss depth defined, the wind loading on the truss members can be determined. As noted in Chapter 3, Section B-3, for preliminary analysis wind load (W) was applied only to the bridge deck profile, the truss profile being unknown at the time. The controlling wind load for truss members (150 lb/ft) is now applied to the truss.
profile as joint loads. The total wind loading on the truss is the sum of wind load on the bridge deck (Table 3(A)) and wind load on the truss members. This total wind load is summarized in Table 7. Note that wind load on live load (WL) is unaffected by truss depth and these load values are unchanged from Table 3(B).

Although it has been determined that HS20 truck loading governs for the design of the bridge deck, stringers, and floorbeams, it has thus far also been assumed that truck loading would govern the design of truss members. But tables of loading results given in Appendix A of AASHTO indicate that for a simply supported span of 150 feet, HS20 lane load controls over truck loading. To determine the correct loading, effects of both truck load and lane load are used for the remaining computer analysis. HS20 truck load and lane load values are taken from Table 2(A) and 2(B) respectively.

2. Design Parameters

Computer analysis for the truss design uses the same material properties and analysis parameters as discussed in Chapter 3 with the exception of member cross section properties. With the truss configuration determined, truss member shapes can now be selected from the AISC manual.
Table 7. Final Wind Loads

(A) Wind Load (Bridge Deck and Truss Members)

<table>
<thead>
<tr>
<th>Joint Number</th>
<th>Direction</th>
<th>Joint Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Y</td>
<td>-1.9</td>
</tr>
<tr>
<td>2</td>
<td>Y</td>
<td>-6.7</td>
</tr>
<tr>
<td>3</td>
<td>Y</td>
<td>-3.8</td>
</tr>
<tr>
<td>4</td>
<td>Y</td>
<td>-9.8</td>
</tr>
<tr>
<td>5</td>
<td>Y</td>
<td>-10.0</td>
</tr>
<tr>
<td>6</td>
<td>Y</td>
<td>-3.8</td>
</tr>
<tr>
<td>7</td>
<td>Y</td>
<td>-12.7</td>
</tr>
<tr>
<td>8</td>
<td>Y</td>
<td>-8.9</td>
</tr>
<tr>
<td>9</td>
<td>Y</td>
<td>-3.8</td>
</tr>
<tr>
<td>10</td>
<td>Y</td>
<td>-15.6</td>
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<tr>
<td>11</td>
<td>Y</td>
<td>-6.0</td>
</tr>
<tr>
<td>12</td>
<td>Y</td>
<td>-3.8</td>
</tr>
<tr>
<td>13</td>
<td>Y</td>
<td>-12.7</td>
</tr>
<tr>
<td>14</td>
<td>Y</td>
<td>-8.9</td>
</tr>
<tr>
<td>15</td>
<td>Y</td>
<td>-3.8</td>
</tr>
<tr>
<td>16</td>
<td>Y</td>
<td>-9.8</td>
</tr>
<tr>
<td>17</td>
<td>Y</td>
<td>-10.0</td>
</tr>
<tr>
<td>18</td>
<td>Y</td>
<td>-1.9</td>
</tr>
<tr>
<td>19</td>
<td>Y</td>
<td>-6.7</td>
</tr>
</tbody>
</table>

(B) Wind Load on Live Load

SEE TABLE 3(B)

NOTE: Also see Figure 5 when referring to this table.
Square structural tubing is used for the design of truss members. Structural tubing is an efficient section for carrying compression and torsional forces, having equal properties in both major directions. Weight savings of up to 30% can be achieved for compression members using square tubing rather than standard rolled shapes. For equal weight members, structural tubing has the largest radius of gyration value among structural shapes. Rectangular tubing offers a 30-40% decrease in exposed surface area over other shapes, resulting in reduced wind loads. In general, structural tubing offers modern, clean, aesthetically pleasing lines, is easy to detail and fabricate, and is versatile in its application.

Structural tubing is also very efficient in meeting KL/r requirements (AASHTO 1.7.5) of truss members. To satisfy this requirement, the same procedure is followed as in Chapter 3. Equation 6 is solved for each truss member, and the resulting value ($r_{min}$) is used to select the lightest square tube member. The corresponding cross sectional area $A_r$ and moment of inertia ($I_r$) of the tube selected is then used as the new member parameters for solving the FSD algorithm. Floorbeam member sections have previously been determined, and thus member section properties have been used directly (member groups 1, 2 and 4).
Table 8(A) shows the values of \( r_{\text{min}} \) determined from Equation 6, and the cross sectional area \( A_r \) of the tube sections for each truss member grouping. The required cross sectional area determined by the FSD solution \( A_f \) of the initial design analysis, is also shown in Table 8(A).

Comparing the value given by the FSD solution \( (A_f) \) with that given by KL/r requirements \( (A_r) \), it can be seen that main chord tension, compression, and the endpost members (member groups 7 to 12) are governed by stress requirements. The remaining truss members are governed by slenderness requirements with the exception of the floorbeams (member groups 1, 2 and 4), which are governed by bending stresses.

B. FINAL DESIGN ANALYSIS

1. Truss Members

Recall from Chapter 3, Section B-4, that to simplify calculations during the preliminary analysis, an allowable stress of \( F_{\text{allow}} = 27.0 \) ksi was used for both tension and compression members. But AASHTO sets a lower limit on the allowable stress for compression members (columns). This reduced allowable stress, given by AASHTO Table 1.7.1(A) is when \( KL/r \leq C_c \):

\[
F_{\text{allow}} = \frac{F_y}{2.12} \left[ 1.0 - \frac{(KL/r)^2 F_y}{4 \pi^2 E} \right]
\]  
(7)
### TABLE 8. Computer Analysis of Truss Design

#### (A) Slenderness Ratio Analysis

<table>
<thead>
<tr>
<th>Group Number</th>
<th>Min. Radius of Gyration $r_{\text{min}}$ (in.)</th>
<th>Slenderness Ratio $\frac{r_{\text{min}}}{A}$</th>
<th>FSD Req'd. Area $A_f$ (in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.10</td>
<td>38.30</td>
<td>38.30</td>
</tr>
<tr>
<td>2</td>
<td>2.10</td>
<td>59.10</td>
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<td>3</td>
<td>3.47</td>
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<td>9.59</td>
</tr>
<tr>
<td>4</td>
<td>2.10</td>
<td>44.20</td>
<td>44.20</td>
</tr>
<tr>
<td>5</td>
<td>2.08</td>
<td>4.27</td>
<td>4.27</td>
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<td>7</td>
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<td>5.02</td>
<td>7.23</td>
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<td>8</td>
<td>2.50</td>
<td>5.02</td>
<td>7.43</td>
</tr>
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<td>9</td>
<td>2.50</td>
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<td>1.50</td>
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<td>2.77</td>
<td>24.80</td>
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<td>12</td>
<td>2.57</td>
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<tr>
<td>17</td>
<td>3.69</td>
<td>9.59</td>
<td>9.59</td>
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</tbody>
</table>

#### (B) Truss Member Selection

<table>
<thead>
<tr>
<th>Group Number</th>
<th>Member Section</th>
<th>Allow. Stress $F_{\text{allow}}$ (ksi)</th>
<th>Final Member Stress (ksi)</th>
<th>Controlling Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W33X130</td>
<td>±27.00</td>
<td>+0.00</td>
<td>Truck</td>
</tr>
<tr>
<td>2</td>
<td>W33X201</td>
<td>±27.00</td>
<td>+1.36</td>
<td>Truck</td>
</tr>
<tr>
<td>3</td>
<td>TS10X10X1/2</td>
<td>-11.46</td>
<td>-9.60</td>
<td>Truck</td>
</tr>
<tr>
<td>4</td>
<td>W36X150</td>
<td>±27.00</td>
<td>+1.45</td>
<td>Truck</td>
</tr>
<tr>
<td>5</td>
<td>TS12X12X1/4</td>
<td>+27.00</td>
<td>+9.06</td>
<td>Truck</td>
</tr>
<tr>
<td>6</td>
<td>TS10X10X1/4</td>
<td>+27.00</td>
<td>+0.89</td>
<td>Truck</td>
</tr>
<tr>
<td>7</td>
<td>TS10X10X3/8</td>
<td>-17.49</td>
<td>-14.52</td>
<td>Lane</td>
</tr>
<tr>
<td>8</td>
<td>TS10X10X3/8</td>
<td>-17.49</td>
<td>-15.07</td>
<td>Lane</td>
</tr>
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<td>9</td>
<td>TS10X10X5/8</td>
<td>-17.09</td>
<td>-14.42</td>
<td>Lane</td>
</tr>
<tr>
<td>10</td>
<td>TS16X16X3/8</td>
<td>+27.00</td>
<td>+26.90</td>
<td>Lane</td>
</tr>
<tr>
<td>11</td>
<td>TS16X16X1/2</td>
<td>+27.00</td>
<td>+23.51</td>
<td>Lane</td>
</tr>
<tr>
<td>12</td>
<td>TS8X8X1/2</td>
<td>+27.00</td>
<td>+24.99</td>
<td>Lane</td>
</tr>
<tr>
<td>13</td>
<td>TS12X12X1/2</td>
<td>-11.12</td>
<td>-9.30</td>
<td>Truck</td>
</tr>
<tr>
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<td>-7.55</td>
<td>-6.69</td>
<td>Truck</td>
</tr>
<tr>
<td>15</td>
<td>TS10X10X1/4</td>
<td>±7.95</td>
<td>-6.11</td>
<td>Truck</td>
</tr>
<tr>
<td>16</td>
<td>TS10X10X1/4</td>
<td>±7.95</td>
<td>+4.37</td>
<td>Truck</td>
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<td>TS10X10X1/4</td>
<td>±7.95</td>
<td>-2.25</td>
<td>Truck</td>
</tr>
</tbody>
</table>

* See Table 4.  

NOTE: "+" denotes tension.
when $KL/r \geq C_c$:

$$F_{\text{allow}} = \frac{\pi^2 E}{2.12 (KL/r)^2}$$  \hspace{1cm} (8)

where  \hspace{1cm} $C_c = \left(2 \pi^2 E/F_y\right)^{0.5}$  \hspace{1cm} (9)

The truss member shapes are chosen from the AISC manual\(^{(56)}\) with a cross sectional area ($A$) greater than the required area given by the FSD algorithm ($A_f$) for each member group. When selecting member shapes, the lightest square tube section possible to satisfy the required area ($A_f$) is chosen. Consideration is also given to keeping member outside dimensions uniform within a member type (i.e., main tension members, etc.). If the member is in compression, a reduced allowable stress ($F_{\text{allow}}$) is calculated for the shape selected using the governing AASHTO equation (equation 7 or 8). The resulting maximum allowable stress for the compression member is later checked against the actual stress determined from the final TRUSS1 analysis. The member shapes selected to satisfy the requirements are shown in Table 8(B). Also shown is the maximum allowable stress for each member group.

For corrosion protection, AASHTO 1.7.7 sets minimum thickness requirements for structural shapes. This
thickness is \( t = \frac{5}{16} \) inches for rolled shapes, and \( t_w = 0.23 \) inches for webs of rolled beams. Since the entire perimeter of the tube ends is welded in constructing the truss joints, the interior of the tube members is sealed and no corrosion will occur on the inside surface. With this condition, it is customary to use a value of one-half the required minimum thickness. All truss member shapes selected meet this requirement of \( t_{\text{min}} = \frac{1}{2} (\frac{5}{16}) = \frac{5}{32} \) inches, as well as the minimum web thickness.

The final design analysis is now performed for the member shapes selected using the program TRUSS1. An additional load of -4.4 kips is applied to each joint in the Z-axis direction. This additional load represents the total dead load weight of the member shapes selected distributed equally over all truss joints. Table 8(B) gives the final maximum member stress in each group for the structural shapes selected and also shows the controlling loads which produced the maximum stresses. It can be seen that the maximum stress in each member group is less than the allowable stress.

Note from Table 8(B) that in member groups 7 through 12, HS20 lane load controls in producing the maximum stress in these members. The increased stress produced by HS20 lane load over HS20 truck load varies from 0.2 to 5.2%. With such a small increase, it is not believed that the
outcome of previous truss configuration results in Chapter 3, based on HS20 truck loading only, is significantly affected.

Note also from Table 8(B) that all sway bracing members (member groups 15 to 17) are grossly understressed. This is because sway bracing resists wind loads which are small, and member selection is controlled by slenderness requirements which produced a large cross section. Since the bridge deck stringers are composite in construction with the bridge deck, stringers and deck act as a rigid unit and will be able to accommodate these small stresses produced by wind loads. Thus, the sway bracing members may be eliminated from the design as major structural members. Lighter members such as angles, as suggested by AASHTO 1.7.17, may be used instead. It should be noted that some form of temporary sway bracing must be provided to resist wind loads until the stringers can be erected and the bridge deck slab constructed with the concrete cured sufficiently to have obtained the 28-day compressive strength.

Delta members 6, 7, 12, 13 (member group 5) and delta members 9 and 10 (member group 6), are also grossly understressed (see Table 8(B)). The large sections provided are to meet AASHTO slenderness requirements, and to provide sufficient area to accommodate the large primary bracing members and main compression members meeting at the delta
2. **Floorbeams**

In Chapter 3, Section A-2, it was noted that calculations for designing the floorbeams would not be presented for there are several references (4, 51, 52) available for designing these members. As noted in Chapter 3, Section B-1, structural shapes were selected for the floorbeams based on bending stresses from member loads transmitted through the stringers, the axial stress from truss action had not been considered.

For combined axial and bending stress, the maximum fiber stress occurs at the outer flange surface and is given by

\[ f = F_{ax} \pm \frac{M}{S} \quad (10) \]

where \( F_{ax} \) = axial stress. Solving for the axial stress, and noting that for this type of truss geometry it is always in tension, we have

\[ F_{ax} = F_{allow} - \frac{M}{S} \quad (11) \]

Solving equation 11 for the three floorbeam member groups (1, 2, and 4), the following allowable axial stresses are calculated so as to not exceed a total stress of \( F_{allow} = 27.0 \text{ ksi} \).
Comparing these values of maximum allowable axial stress with the axial stress produced by truss action and given in Table 8(B), it can be seen that the total stress in the floorbeams does not exceed the allowable axial stress ($F_{ax}$).

C. DEFLECTION CRITERIA

AASHTO 1.7.6 gives the maximum allowable deflections due to live loads plus impact loads. Since it is assumed that this bridge will not be erected in urban areas, and no sidewalk has been provided (thus no pedestrian traffic), the maximum allowable deflection is given by

$$\Delta_{\text{max}} = \frac{\text{Span Length}}{800}$$

which, for the span under investigation, is

$$\Delta_{\text{max}} = \frac{(150)(12)}{800} = 2.25 \text{ inches}$$

Table 9(A) shows the maximum deflection occurring in the three principal directions due to live loads plus impact loads as calculated by the program TRUSS1. It can be seen from Table 9(A) that all deflections that occur are less than the AASHTO maximum allowable of 2.25 inches.
TABLE 9. Maximum Deflections and Bearing Reactions

(A) Maximum Live Load Deflections

<table>
<thead>
<tr>
<th>Direction</th>
<th>Joint Number</th>
<th>Deflection (in.)</th>
<th>Controlling Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>18, 19</td>
<td>-0.27</td>
<td>Lane</td>
</tr>
<tr>
<td>Y</td>
<td>9, 10</td>
<td>0.20</td>
<td>Truck</td>
</tr>
<tr>
<td>Z</td>
<td>9, 10</td>
<td>-0.82</td>
<td>Lane</td>
</tr>
</tbody>
</table>

(B) Maximum Bearing Reactions

<table>
<thead>
<tr>
<th>Direction</th>
<th>Joint Number</th>
<th>Reaction (kips)</th>
<th>Controlling Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>1, 2</td>
<td>60.45</td>
<td>Truck</td>
</tr>
<tr>
<td>Y</td>
<td>1, 2, 18, 19</td>
<td>149.78</td>
<td>Lane</td>
</tr>
<tr>
<td>Z</td>
<td>1, 2, 18, 19</td>
<td>293.23</td>
<td>Truck</td>
</tr>
</tbody>
</table>

Note: Also see Figure 5 when referring to this table.
D. ***DESIGN OF CONNECTIONS***

Up to this point in the report, the design of the truss has adhered to the AASHTO Bridge Specifications. For the design of tubular connections, AASHTO offers little guidance.

A review of the literature indicates that sufficient material has been published abroad on this subject, but the use of tubes as structural members is relatively limited in the United States; thus, subject matter is limited.

The American Welding Society's Structural Welding Code for Steel (henceforth referred to as AWS) is the only domestic source which provides an in-depth procedure and the specifications needed to design connections for rectangular tubing. The AWS code, as modified and approved by the AASHTO publication "Standard Specifications for Welding of Structural Steel Highway Bridges", serves as a guide for the welding of bridges. The AASHTO Welding Specification has yet to review and approve the section of the AWS code which deals with the design of tubular joints. For lack of any AASHTO or other domestic specification on the subject, the AWS code is used to design the tubular truss joints along with design aides based on this code.

Figure 11 gives details of some of the truss connections. Those joints not shown are similar in configuration and weld requirements. The main concern in
FIGURE II. TYPICAL JOINT DETAILS
designing the joints is to prevent local failure due to punching shear stress. If the member wall thickness is not sufficient to provide the necessary allowable shear stress, a cover plate is provided to increase capacity or internal stiffeners are used to provide transfer of shear stresses. Where access can be made to the interior of the joint during fabrication, stiffeners are inserted and held in place by one-sided fillet welds, otherwise cover plates are used. An additional section, a TS14X14X5/16 bearing member, has been added at each of the four support joints to provide a vertical reaction to the bearings (see Figure 11, joint no. 2). These bearing members have a resulting maximum stress (P/A) of -17.35 ksi.

Simple plate theory\(^{(73)}\) is used to design end plates and floorbeam bearing plates. Note from Figure 11 that the floorbeam joints are designed as a single lower flange connection and not a pin as discussed earlier. It is assumed that this connection acts as a pin (i.e., no moment develops). This assumption is discussed further in Section F. Also, note that this type of joint creates a small eccentricity between the floorbeam joint and truss tube joint, creating a torsional stress in the main compression members. Although every attempt is made to avoid eccentric joints when designing the connections, where this is not
possible the calculated torsional shear stress is within AASHTO allowables (AASHTO Table 1.7.1A).

Since some field welding is required, the joint welds are designed as a Shielded Metal Arc Weld process (AWS 4.5). The AASHTO welding specifications require the use of E8XX electrodes for A588 weathering steel in order to provide the same corrosion resistant properties. In general, the joint welds are designed in accordance with the AWS code with the exception of complying with AASHTO sections 1.7.16, 1.7.21(B), and 1.7.41(B), which specify the minimum required weld strength, the minimum required fillet weld size, and the allowable weld stress, respectively.

E. DESIGN OF BEARINGS

Throughout the analysis, the truss has been considered simply supported, with one end of the structure restrained against movement except rotation (joints 1, 2), and the other end free to move longitudinally (but not laterally) and to rotate (joints 18, 19). There are presently three distinct types of bearings available which are typically used to achieve this desired type of support condition. They are hinge/roller, elastomeric pad, and expansion plate bearings.

While investigating each bearing type, it was found that the hinge/roller type would be relatively large. A design of this type conforming to the specifications (AASHTO
1.7.41(D)) would require a roller pin diameter of 19 inches if a single roller was used, a 9.5-inch pin if two rollers were used, and so forth. An elastomeric pad type bearing proved inadequate, for it could not accommodate the large longitudinal movements of the structure. Thus, an expansion plate type bearing is chosen. This type of bearing is modest in size, requires no forging, and is easy to design, fabricate, and install.

There are several references (4, 48, 49, 51, 52) that provide examples for designing bridge bearings to meet the AASHTO specifications, thus, detailed calculations are not presented. What follows is a description of the resulting design. Tables 9(A) and 9(B) give the maximum deflections and reactions the bearing must accommodate as determined by the program TRUSS1. In addition, movement from thermal expansion and contraction of the structure (AASHTO 1.7.1(A)) is considered. This gives a total of 1.67 inches longitudinal movement the bearings must allow for.

Figure 12 shows the details for the expansion bearing. The fixed bearing (not shown) is similar in detail, except that the bronze expansion plate is eliminated, a thicker (2 7/8") back-up plate being welded directly to the bearing plate. Design of various plates that make up the bearing assembly are based on plate theory (73) subject to allowable
FIGURE 12. BEARING DETAILS
stresses and loadings given by AASHTO 1.7.1(F), 1.7.41(D), and Table 1.7.1(A). The expansion plate contact surface is sufficient in length to provide for the expected range of longitudinal movement, and is lubricated to provide for a nearly friction-free surface. The bearing plate, which sits directly on the abutment, has sufficient area to limit the bearing reaction stress \(F_b\) on the concrete to within the allowable stress of \(F_b \leq 0.30 f'c\) (AASHTO 1.5.26(A)(3)). Finally, the anchor bolts are designed to resist a combination of lateral loads and to meet minimum dimensions set forth by AASHTO 1.7.37.

F. VERIFICATION OF ANALYSIS

Throughout this report, the program TRUSS1 has been used to analyze the structure. Although the author has thoroughly debugged and tested the program for several truss configurations and loading applications, it still remains to be proven to the reader that no errors have been committed, either by the author in inputing data, or by the program in computing the results. The commercially available structural analysis program STRUDL \(^{(74)}\) was used to verify the results of the TRUSS1 program, as well as to prove certain design assumptions. Three individual computer analyses were performed for the structure loaded to produce maximum stress in each member.
The first STRUDL analysis models the structure as a three-dimensional truss, the same model as analyzed by the TRUSS1 program. A partial output of the results, giving maximum stress occurring in each member group, is shown in Table 10. The maximum stress values given are the same results as determined by the TRUSS1 program and shown in Table 8(B). Thus, the TRUSS1 program has been accurate and correct in analyzing the truss configurations.

The second STRUDL analysis models the structure as a frame with the floorbeams being hinged at their joints. It was assumed in Section D that the floorbeam joints would behave in this manner. Analyzing the structure as a frame gives stresses due to axial load (truss action), as well as secondary stresses from deflection. AASHTO allows for secondary stresses in trusses to be neglected when the total member stress does not exceed an additional 3 ksi for compression members, and 4 ksi for tension members over and above the allowable stress (AASHTO 1.7.44(C)). It can be seen from the results shown in Table 10 that total stresses in the members are less than the maximum additional allowable stress.

The third and final STRUDL analysis models the structure as a frame with rigid connections at all joints. From Table 10, it can be seen that the majority of these
TABLE 10. Results of STRUDL Analysis

Maximum Stress (ksi)

<table>
<thead>
<tr>
<th>Group* Number</th>
<th>Truss Analysis</th>
<th>Frame Analysis with Hinged Floorbeams</th>
<th>Frame Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+0.00</td>
<td>+23.44</td>
<td>+15.13</td>
</tr>
<tr>
<td>2</td>
<td>+1.36</td>
<td>+25.84</td>
<td>+24.65</td>
</tr>
<tr>
<td>3</td>
<td>-9.60</td>
<td>-10.79</td>
<td>-18.66</td>
</tr>
<tr>
<td>4</td>
<td>+1.45</td>
<td>+25.62</td>
<td>+23.89</td>
</tr>
<tr>
<td>5</td>
<td>+9.05</td>
<td>+11.32</td>
<td>+13.56</td>
</tr>
<tr>
<td>6</td>
<td>+0.89</td>
<td>+0.98</td>
<td>+8.55</td>
</tr>
<tr>
<td>7</td>
<td>-14.52</td>
<td>-19.15</td>
<td>-19.09</td>
</tr>
<tr>
<td>8</td>
<td>-15.07</td>
<td>-18.42</td>
<td>-19.03</td>
</tr>
<tr>
<td>9</td>
<td>-14.42</td>
<td>-17.76</td>
<td>-17.29</td>
</tr>
<tr>
<td>10</td>
<td>+26.90</td>
<td>+30.04</td>
<td>+29.61</td>
</tr>
<tr>
<td>11</td>
<td>+23.51</td>
<td>+25.60</td>
<td>+25.47</td>
</tr>
<tr>
<td>12</td>
<td>+24.99</td>
<td>+26.78</td>
<td>+26.78</td>
</tr>
<tr>
<td>13</td>
<td>-9.50</td>
<td>-10.08</td>
<td>-16.05</td>
</tr>
<tr>
<td>14</td>
<td>-6.69</td>
<td>-7.58</td>
<td>-13.58</td>
</tr>
<tr>
<td>15</td>
<td>-6.11</td>
<td>-6.96</td>
<td>-10.06</td>
</tr>
<tr>
<td>16</td>
<td>+4.37</td>
<td>+4.60</td>
<td>+6.98</td>
</tr>
<tr>
<td>17</td>
<td>-2.25</td>
<td>-3.06</td>
<td>-7.08</td>
</tr>
</tbody>
</table>

* See Table 4.

NOTE: "+" denotes tension.
member stresses are also within the maximum additional allowable stress, the exception being member groups 3, 13 and 14. Thus, if the floorbeam joints are not considered to act as pinned connections, and develop and transmit a fully-fixed joint moment, sections with a larger moment of inertia (I) must be selected for these member groups in order for stresses in the truss to be within the specified allowable.
V. SUMMARY

A review of the available literature on the subject of triangular cross section trusses was made. On a limited basis, this truss configuration has been used for a number of structures such as crane booms, offshore oil rig platform legs, material transfer and pipeline bridges, roof truss beams, antennas, and pedestrian bridges. Its selection for use over other structural forms is usually because of its aerodynamic efficiency, aesthetics, and weight-saving characteristics. Few instances were found where a bridge truss of this configuration has been designed and constructed for heavy loads such as highway and railway traffic in recent history.

A simply supported deck type truss bridge with a clear span of 150 feet was selected for investigation. The bridge deck consisted of a 7-inch thick concrete slab with composite construction over six stringers spaced 6 feet 3 inches on center. The stringers were two-span continuous beams connected to truss floorbeams at 25-foot intervals. The bridge deck width was 33 feet 6 inches overall, with a roadway width of 30 feet curb-to-curb.

Preliminary analyses were made to determine the most efficient truss configuration based on least weight criteria. The structural analysis program TRUSS1 was used to analyze and compare the various proposed configurations.
The primary bracing configuration, sway bracing configuration, and truss depth were selected in this manner. The structure was analyzed for HS20 truck loading only in the preliminary analysis.

Based on the configuration determined in the preliminary analysis, a truss was designed to conform to the requirements of the AASHTO bridge specifications. The FSD solution of the computer program TRUSS1 was used to determine the member section properties to satisfy the given KL/r and allowable stress requirements. Square structural tubing was used for the truss members because of its uniform section properties about both axes and because of its light weight in comparison with other structural shapes. Deflections based on the truss members selected were within the specified allowable. The truss was designed for the greater of HS20 truck loading and lane loading.

The American Welding Society code was used for the design of the tubular truss joints. It was found that punching shear stress controlled the design of the joints. Either cover plates were used to increase joint capacity, or internal stiffeners were provided to transfer shear loads.

Expansion plate type bearings were designed for supporting the truss at the four reaction points. Compared with other types of bearings, the expansion plate was
considered the most feasible in achieving the desired support conditions.

The structural analysis program STRUDL was used to verify the accuracy of the TRUSS1 program. STRUDL was also used to determine the secondary stresses in the truss members, which were found to be less than the maximum allowable stress.
VI. CONCLUSIONS

From the review of the literature, it is evident that a triangular cross section truss is an efficient structural form. Triangular sections have a smaller drag coefficient (up to 11.3% less) than rectangular truss shapes. Generally speaking, a triangular truss provides for a lighter structure (as much as 13.9%) when used in place of other structural configurations.

A triangular truss configuration proved efficient in resisting lateral loads. This point was made in the review of the literature, but also became apparent during the truss design. In the scope of the design, lateral bracing (sway bracing) members were grossly understressed.

For the span length under investigation, HS20 lane load controlled in producing maximum stress in the main truss chord tension and compression members and in the endpost members (member groups 7 through 12). HS20 truck load controlled in producing maximum stress in the other truss members, as well as the bridge deck slab, stringers, floorbeams, and bearings.

Satisfying slenderness requirements (KL/r) controlled in selecting structural shapes for the delta members. Stress requirements controlled in selecting the remaining truss members. Square structural tubing proved to be the most efficient section shape in satisfying buckling...
requirements. Having equal properties in both major directions, it provided the highest radius of gyration value with the lowest weight per foot than any other type of section.

Secondary stresses in the truss members, either from floorbeam member loads or truss distortion, are small and can be neglected. Torsional moment in the main compression members caused by floorbeam end connection eccentricity is also small and can be neglected.

The structural analysis program TRUSS1 is an accurate, useful, and inexpensive means of analyzing truss configurations. Although not as versatile as the computer program STRUDL, it does have several analysis options not available to STRUDL.

Finally, a triangular cross section truss of modest span can be designed to carry HS20 vehicle traffic loading and conform to the AASHTO Bridge Specifications.
REFERENCES


30. "Pipe Bridge," Tubular Structures, No. 5, April, 1966, p. 27.


APPENDIX 1

TRUSS1 PROGRAM LISTING
C THIS PROGRAM USES MATRIX METHODS TO ANALYZE THREE DIMENSIONAL TRUSSES
C FOR A MAXIMUM OF SIXTY MEMBERS AND THIRTY JOINTS WITH A MAXIMUM OF 14
C LOADING CONDITIONS
C COMMON/MEMBER/A(6),X(6),E(6),S(6),GAMMA(6),C1(6),C2(6),C3(6),MC
C ORDER(6),NE,MINE(6),2*,X(6),D(6),FALLLW(6),FL(2,50,14),STRESS
C 2(60,14),SH(6),DENSITY(6),AM(N(6),LCOE(6),1)
C COMMON/JOINT/X(3),X(3),X(3),JCODE(3),NJ
C COMMON/SYSTEM/NODF,IM(J,3),NODF,ASTRES(6),SD(6)
C ILINK(30),STRESSL(30),LCO(60)
C DIMENSION SS(30,30),P(90)
C DATA 0.1260,4,1260,4,1260,4,1260,4,1260,4,1260,4
C KCOUNT=PARAMETER TO SUPPRESS OUTPUT DURING KSO ITERATIONS
C KCOUNT=5
C CALL DATA (MFSO,LMAX)
C CALL PROCES (CTP)
C 0) 5 =NACT
C CALL ACTION (I)
C CONTINUE
C 8 CALL STIFF (55,KCOUNT)
C DO 5 =1,NACT
C WRITE(6,5) I
C 12 KCOUNT=1
C IF(KCOUNT=NE,5) 3 TO 12
C WRITE(6,5) I
C 14 KCOUNT=1
C IF(KCOUNT=NE,5) 14 TO 19
C WRITE(6,55)
C DO 15 KZ=1,NODF
C DO 14 II=1,NJ
C DO 14 JJ=1,NJ
C 15 WRITE (6,56) II,JJ,Z(KZ)
C C EQUATE JOINT FORCE VECTOR TO GENERALIZED DISPLACEMENT VECTOR FOR
C COMPUTATIONS IN SUBROUTINE SOLVE
C 18 DO 19 K=1,NODF
19 P(K)=W(K,I)
   CALL SOLVE (SS,P,MCOUNT,KCOUNT)
   DO 20 J=1,NJ
   DO 20 K=1,3
  20 PJF(J,K)=0.
   IF(KCOUNT.EQ.1) WRITE (6,7)
   DO 22 M=1,NE
      CALL FORCE (M,I,P)
      CALL SCHEME (M,I)
      IF(KCOUNT.EQ.1) WRITE (6,5,H,M,FL(K,M,I),K=1,2)
   22 CONTINUE
      IF(KCOUNT.EQ.1) GO TO 57
      WRITE(6,7)
   DO 25 J=1,NJ
      DO 25 K=1,3
      IF(JCODE(J,K).EQ.1) WRITE (6,4,J,K,PJF(J,K))
      IF(JCODE(J,K).EQ.2) WRITE (6,75,J,K,PJF(J,K))
   25 CONTINUE
      WRITE(6,77)
   DO 27 M=1,NE
  27 WRITE(6,79) M,STRESS(M,I)
   50 CONTINUE
      IF(KCOUNT.EQ.1) GO TO 55
      WRITE (6,81)
   DO 52 K=1,NE
  52 WRITE(6,82) K,ASTRES(K),LC(K)
   55 CONTINUE
      IF(MFSQ.EQ.1) GO TO 110
      IEND=1
      CALL FSD (MFSQ,KCOUNT,GTP,IEND,LMAX)
   C IEND=PARAMETER TO EXIT FSD SUBROUTINE FROM ITERATION LOOP AFTER
   C CONVERGENCE
      IF (IEND.EQ.1) GO TO 8
      FORMAT(*1*,TS*RESULTS FROM LOADING CONDITION NO.*,I2,2x,********)
1****850 FORMAT(T5, 'ACTUAL JOINT FORCE VECTOR (KIP)'/1T1, 'JOINT NO.'/ T2, 'VALUE'/
   1, 'DIRECTION'/ T4, 'VALUE'/
   65 FORMAT(T13, I2, T24, I2, I3, F12.3)
   66 FORMAT(T5, 'LOCAL FORCES'/ T75, 'END'/ T35, 'END'/)
   67 FORMAT(T10, 'LOCAL ELEMENT END FORCES FOR ELEMENT NO.'/ T13, 'KIPS'/
   1, '26X/F14.3')
   70 FORMAT(T5, 'JOINT FORCES (KIPS)'/ T15, 'JOINT NO.'/ T27, 'DIRECTION'/
   1T42, 'VALUE'/ T6, 'TYPE'/)
   74 FORMAT(T19, I2, T31, I2, I3, F10.3, T55, 'REACTION'/)
   75 FORMAT(T19, I2, T31, I2, F39, F10.3, T55, 'ACTION'/)
   77 FORMAT(T5, 'MAXIMUM STRESS'/ T12, 'MEMBER NO.'/ T25, 'VALUE (KIP/SQ. IN.)'/
   1T620)
   79 FORMAT(T14, I2, T31, F10.3)
   80 FORMAT(T1, T5, 'MAXIMUM MEMBER STRESS--ALL LOADING CONDITIONS'/ T11, 'MEMBER NO.'/ T25, 'VALUE (KIP/SQ. IN.)'/ T620, 'LOADING CONDITION NO. 2'/)
   82 FORMAT(T14, I2, T31, F10.3, T53, I2)
   83 WRITE(6, 105)
   105 FORMAT('14')
   STOP
END
!
SUBROUTINE DATA (MAX, LMAX)
C THIS SUBROUTINE READS AND WRITES DATA TO DEFINE THE STRUCTURE
C COMMON/MEMBER/A61, X(26), E(65), CGMA(65), C1(65), C2(65), C3(65), M
C DENSITY, ZIN, ZL, M/CAL(65), ZL(65), ACAL(65), Al(65), FALLOW(65),
C 2(65), 3(65), DENSITY, ZIN, M/CAL(65), ZL(65), ACAL(65), Al(65), FALLOW(65),
C COMMON/JOINT/X(13), Y(33), Z(33), JCODE (33), NJ
C COMMON/SYSTEM/NACT, NO. OF EMB, 2(30, 14), P(30, 3), ASTRES(65), S(65),
C 1LINK(30), STRES(30), L(65)
CHARACTER PRIND(3)
LMAX=30
C READ AND WRITE CONTROL PARAMETERS AND CONTROL FLAGS
C NE = NUMBER OF ELEMENTS, NJ = NUMBER OF JOINTS
C NACT = NUMBER OF ACTIONS (LOADING CONDITIONS)
C MFSO = PARAMETER TO DETERMINE IF THE FSO ALGORITHM IS TO BE USED
C PROBID = STRING OF 30 CHARACTERS TO IDENTIFY PROBLEM OUTPUT
READ(6, 31) NE, NJ, NACT, MFSO, PROBID
WRITE(6, 33) PROBID
WRITE(6, 35) NE, NJ, NACT
C READ AND WRITE JOINT COORDINATES
C X1 = GLOBAL-1 COORDINATE, X2 = GLOBAL-2 COORDINATE, X3 = GLOBAL-3 COORDINATE
WRITE(6, 37)
DO 10 I = 1, NJ
READ(5, 42) X1(I), X2(I), X3(I)
WRITE(6, 45) X1(I), X2(I), X3(I)
10 CONTINUE
C READ AND WRITE MEMBER DATA
C AI(J) = X-SECTION AREA VECTOR, V1(J) = MOMENT OF INERTIA VECTOR
C C(J) = YOUNG'S MODULUS VECTOR, JA(J) = JOINT NO. AT A AND B END OF ELEMENT
C MINC = MEMBER INCIDENCE MATRICES, DIC = MEMBER DEPTH
C FALLOW(J) = ALLOWABLE STRESS, DENSTY(J) = MATERIAL DENSITY
C AMINC = MINIMUM REQUIRED CROSS-SECTION AREA, LCODE = MEMBER LINKING CODE
WRITE(6, 51)
DO 15 I = 1, NE
READ(5, 52) AI(I), V1(I), C(J), FALLOW(I), DENSTY(I), AMINC(I), JA(J), LCODE(I)
15 CONTINUE
C SET MINIMUM VALUE OF AMINC(I)
IF (AMINC(I) .LT. 0.1) AMINC(I) = 1
C DETERMINE LMAX = NUMBER OF MEMBER LINKED GROUPS
IF (LCODE(I) .EQ. LMAX) LMAX = LCODE(I)
WRITE(6, 52) IA(I), X1(I), X2(I), X3(I), D(J), FALLOW(I), DENSTY(I), AMINC(I), JA(J) LCODE(I)
INCI(J) = JA(J) INCI(J+1) = JB
15 CONTINUE
C INITIALIZE JOINT CODE MATRIX (LCODE) TO UNITY

DO 25 I=1,NJ
  DO 2 J=1,3
  JCODE(I,J)=1
  CONTINUE
C READ JOINT CONSTRAINTS
C NUMBER-JOINT NUMBER, J01, J02, J03=JOINT CONSTRAINT IN GLOBAL-1,2,3
C DIRECTION (I.E., X, Y, Z DIRECTION)
DO 25 I=1,NJ
  READ(5,6) JNUM,JOI1,JOI2,JOI3
  IF (JOI1.NE.0) JCODE(I,1)=1
  IF (JOI2.NE.0) JCODE(I,2)=1
  IF (JOI3.NE.0) JCODE(I,3)=1
  CONTINUE

30 FORMAT(13,2X,A3)
31 FORMAT(*13,2X,A3)
32 FORMAT(*13,2X,A3)
33 FORMAT(*13,2X,A3)
34 FORMAT(*13,2X,A3)
35 FORMAT(*13,2X,A3)
36 FORMAT(*13,2X,A3)
37 FORMAT(*13,2X,A3)
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163 FORMAT(*13,2X,A3)
164 FORMAT(*13,2X,A3)
165 FORMAT(*13,2X,A3)
166 FORMAT(*13,2X,A3)
167 FORMAT(*13,2X,A3)
168 FORMAT(*13,2X,A3)
169 FORMAT(*13,2X,A3)
170 FORMAT(*13,2X,A3)
SUBROUTINE PROCES (CTP)
C THIS SUBROUTINE GENERATES JOINT CODE MATRIX, MEMBER CODE MATRIX,
C AND COMPUTES MEMBER PROPERTIES AND THE HALF-BAND WIDTH
C
COMMON/MEMBER/A(6),K(60),E(60),SIGMA(5),C1(6),C2(6),C3(6),MC
JODEC(6),NE,MINC(60,2),XL(6),D(6),F(6),ALLW(6),FL(2,60,14),STRESS
2(-1,1),SM(6),DENSTY(60),AMIN(60),LACODE(60)
COMMON/JOINT/X1(30),X2(30),X3(30),JCODE(3,3),NJ
COMMON/SYSTEM/NACT,NOOF,IHBW,4(3,14),PJE(3,3),ASTRES(6),SD(6),
1,LI(N6),STRESL(3),LC(6)
C GENERATE JOCODE
C NOOF=NUMBER DEGREES OF FREEDOM
NCOUNT=0
DO 10 I=1,NOOF
  DO 10 J=1,3
    IF(JCODE(I,J),EQ.0) GO TO 10
  NCOUNT=NCOUNT+1
  JCODE(I,J)=NCOUNT
10 CONTINUE
NOOF=NCOUNT
C GENERATE MEMBER CODE (MCODE)
DO 15 I=1,NE
  JA=MINC(I,1)
  JH=MINC(I,2)
  DO 15 J=1,3
    MCODE(I,J)=JCODE(JA,J)
    MCODE(I,J+3)=JCODE(JH,J)
15 CONTINUE
C COMPUTE MEMBER PROPERTIES, SIGMA(I)=A*XL, C1*C2*C3=DIRECTIONAL COSINES
WRITE(6,35)
35 FORMAT(1X,10I3)
DO 20 I=1,NE
  XL1=MINC(I,1)-X1(MINC(I,1))
  XL2=MINC(I,2)-X2(MINC(I,1))
  XL3=MINC(I,3)-X3(MINC(I,1))
  XL(I)=SQRT(XL1**2+XL2**2+XL3**2)
20 CONTINUE
C1(I)=XLI/XL(I)
C2(I)=XLI/XL(I)
C3(I)=XLI/XL(I)
GAMMA(I)=PA(I)*E(I)/XL(I)
WRITE(6,41) I+C1(I)+C2(I)+C3(I)+XL(I)+GAMMA(I)
20 CONTINUE
C CTX=CONVERGENCE TOLERANCE PARAMETER
CTX=.98
C COMPUTE HALF-BAND WIDTH (IHW)
IHW=:
00 22 I=1+NE
00 22 J=1+6
ILOW=MCGDE(I,J)
IF(IF(IFLOW.EQ.1))GO TO 22
J1=1
21 J1=J1+1
IHIGH=MCGDE(I,J1)
IF(IF(IHIGH.EQ.1))GO TO 21
IHW1=IHIGH-ILOW+1
IF(IF(IHW1.EQ.0))IHW1=IHW1
22 CONTINUE
WRITE(6,45) IHW1,NOUJ
350 FORMAT(/I5,'MEMBER PROPERTIES',/I5,'MEMBER NO.',/5I4,'DIRECTINAL CO',/5I4,'LENGTH',/5I4,'ANGLE',/5I4,'T27',/5I4,'T44',/5I4,'C2',/5I4,'T61',/5I4,'C3',/5I4,'T75'/)
2 FORMAT(INCHES)'/
400 FORMAT(I14,I2,T21,1PE14.7,T33,1PE14.7,T55,1PE14.7,T63,1PE14.2,T89,1PE14.7)
450 FORMAT(/I5,'HALF-BAND WIDTH=',I3/I5,'NUMBER OF DEGREES OF FREEDOM=1',/I5'/)
RETURN
END
SUBROUTINE ACTION (I)
C THIS SUBROUTINE READS APPLIED JOINT FORCES AND DEPENDS THE JOINT
C FORCE VECTOR
C COMMON/ MEMBER/A(1:6),I(6),E(6),GAMMA(60),C1(60),C2(60),C3(60),MC
10,0,6,1,N,0,0,0,2,N,E(6),K(6),X(6),FL(2,60,14),STRESS
2(60,14),S(6),DENSITY(6),AMIN(60),LCODE(60)
C COMMON/JOINT/A(3),X(3),Z(3),JCODE(3,3),NJ
OC COMMON/SYSTEM/ACT,NOF,THB,THB(3,3),PJP(30,3),ASTRES(6),SD(60),
l1NK(30),STRESL(1),LC(60)
C GENERATE JOINT FORCE VECTOR
C Q(J,J)=JOINT FORCE VECTOR (INITIALIZED TO ZERO IN MAIN PROGRAM)
C JNUM=JOINT NUMBER
C JDIR=FORCE VECTOR DIRECTION, VALUE=FORCE VALUE
10 READ(5,25) JNUM,JDIR,VALUE
C TEST FOR BLANK CARD IN DATA TO EXIT FROM LOOP
IF(JNUM.EQ.0) GO TO 12
K=JCODE(JNUM,JDIR)
Q(J,K)=Q(J,K)+VALUE
GO TO 11
12 CONTINUE
25 FORMAT(213,213,3)
RETURN
END
SUBROUTINE STIFF (SS,KCOUNT)
C THIS SUBROUTINE COMPUTES THE BANDED STIFFNESS MATRIX
C COMMON/ MEMBER/A(1:6),I(6),E(6),GAMMA(60),C1(60),C2(60),C3(60),MC
10,0,6,1,N,0,0,0,2,N,E(6),K(6),X(6),FL(2,60,14),STRESS
2(60,14),S(6),DENSITY(6),AMIN(60),LCODE(60)
C COMMON/JOINT/A(3),X(3),Z(3),JCODE(3,3),NJ
OC COMMON/SYSTEM/ACT,NOF,THB,THB(3,3),PJP(30,3),ASTRES(6),SD(60),
l1NK(30),STRESL(1),LC(60)
C DIMENSION SS (NOF,THB,THB(3,3),PJP(30,3),ASTRES(6),SD(60)
C ZERO OUT STIFFNESS MATRIX
DO 2 J=1,THB
DO 1 I=1,NOF
2 SS(I,J)=0.
C INITIALIZE INDEX MATRIX
C SS=SYSTEM STIFFNESS MATRIX, INDEX=COEFFICIENT MATRIX OF GLOBAL ELEMENT
C STIFFNESS MATRIX
C DO DATA INDEX/1,2,3,-1,-2,3,2,4,5,-2,4,-3,5,6,-3,5,-6,-1,2,-3,1
     1,2,3,-2,-4,5,2,4,3,4,5,6,7,5,6/;
C COMPUTE G(I).....G(6)
    DO 1 I=1,NE
      G(I)=GAMMA(I)*C1(I)**2
      G(2)=GAMMA(I)*C1(I)*C2(I)
      G(3)=GAMMA(I)*C1(I)*C3(I)
      G(4)=GAMMA(I)*C2(I)**2
      G(5)=GAMMA(I)*C2(I)*C3(I)
      G(6)=GAMMA(I)*C3(I)**2
C GENERATE GLOBAL MEMBER STIFFNESS MATRIX AND TRANSFER TO SYSTEM
C STIFFNESS MATRIX
C SS=SYSTEM STIFFNESS MATRIX
    DO 10 JM=1,6
      J=CODE(I,JM)
      IF(J.EQ.5) GO TO 10
      DO 7 K=J+1,6
        K=CODE(I,K)
        IF(K.EQ.4) GO TO 7
        KB=K-J+1
        L=INDEX(JM,K)
        IF(L.GT.0) GO TO 5
      7      L=L
        SS(J,KB)=SS(J,KB)-G(L)
        GO TO 7
      5      SS(J,KB)=SS(J,KB)+G(L)
    10 CONTINUE
    IF(COUNT.NE.0) GO TO 27
    IF(IHW.GT.9) WRITE(6,26)
    IF(IHW.GT.9) GO TO 27
    WRITE*('6,23')
DO 15 I=1,NDOF
15 WRITE(6,25) (SS(I,J),J=1,IMB)
20 FORMAT(T5, SYSTEM STIFFNESS MATRIX (BANDED))
C NOTE: PRESENT OUTPUT FORMAT FOR SS LIMITED TO IMB=9
25 FORMAT(5x,9(2x,IP=12.5))
26 FORMAT(T5, SYSTEM STIFFNESS MATRIX NOT PRINTED -- OUTPUT EXCEEDS AVAILABLE PRINTED SPACE *****)
27 RETURN
END
SUBROUTINE SOLVE (SS, NDOF, COUNT, KCOUNT)
C THIS SUBROUTINE COMPUTES THE GENERALIZED DISPLACEMENTS
C COMMON/MEMBER/A(40) X(60) E(40) GAMMA(60) C1(60) C2(50) C3(60) NC
100/E(60) NE(WING (6) 0) KXL(60) ANG(60) FALL(60) FL(2,50,14) STRESS
2(60,14), SM(60), DENSITY(60) AMIX(60) LCODE(60)
C COMMON/Joint/x(30) X(30) X(30) JCODE(30) RES
C COMMON/SYSTEM/NACT,NDOR,IH2D,3X(40),14,JPF(35,3),ASTR(60),SD(60)
C COMMON/VECTOR/L(50), STRESL(30), LC(60)
C DIMENSION SS, NDOF, IH2D, JPF, RES
C PND=GENERALIZED DISPLACEMENTS. NOTE: THIS PARAMETER ENTERS THIS
C SUBROUTINE AS JOINT FORCE VECTOR AND EXITS AS GENERALIZED DISPLACEMENT
C VECTOR
C IF (NDOF.GT.1) 50 TO 40
C REDUCE STIFFNESS MATRIX
700 DO 79: N=1, NDOF
710 DO 78: L=1, IMB
720 IF (SS(N,L).EQ.0.) 50, 740
730 I=N+L-1
740 C=SS(N,L)/SS(N,1)
750 J=0
760 DO 75: K=L+1, IMB
770 J=J+1
780 SS(I,J)=SS(I,J)-C*SS(N,K)
790 SS(N,L)=C
740 CONTINUE
790 CONTINUE
C REDUCE FORCE VECTOR
810 DO 93 N=1,NNDF
820 DO 92 J=1,IMBW
     IF(SS(N,J)*EQ.0) GO TO 920
     I=N+J-1
     P(I)=P(I)-SS(N,J)*P(N)
920 CONTINUE
930 P(N)=P(N)/SS(N,N)
C BACK-SOLVE FOR DISPLACEMENTS
DO 850 M=2,NNDF
   N=NNDF+1-M
   DO 840 J=1,IMBW
      IF(SS(N,J)*EQ.0) GO TO 855
      K=N+J-1
      P(N)=P(N)-SS(N,J)*P(K)
850 CONTINUE
855 CONTINUE
IF (KCOUNT*NE.*7) GO TO 975
WRITE(*,970)
DO 890 N=1,NNDF
   DO 880 J=1,3
      DO 880 JJ=1,3
880 WRITE(*,890) N,J, JJ
900 IF(JCODE(JJ,JJ)*EQ.4) GO TO 920
910 WRITE(*,910) N,J, JJ
920 FORMAT(T5,'GENERALIZED DISPLACEMENTS (INCHES)'/T10,'JOINT NO.'/ T12,'DIRECTION'/)
930 FORMAT(T13,I2,T23,I2,T23,I2,T23,I2) RETURN
975 CONTINUE
END
C THIS SUBROUTINE COMPUTES THE ELEMENT ENF FORCES AND THE STRUCTURE
C JOINT FORCES
C COMMON/MEMBER/A(60),XI(60),E(60),GAMMA(60),C1(60),C2(60),C3(60),AC
C COMPUTE COMPONENTS TO GLOBAL DISPLACEMENT VECTOR (DG1...DG6) BY MCODE
C P(NDOF)=GENERALIZED DISPLACEMENT VECTOR
DO 10 J=1,6
DG(J)=0.
K=MCODE(J,J)
IF(K.EQ.1) DG(J)=P(K)
10 CONTINUE
C COMPUTE COMPONENTS TO LOCAL DISPLACEMENT VECTOR (DL1,DL2) BY
C TRANSFORMATION MATRIX
DL1=C1(M)*DG(1)+C2(M)*DG(2)+C3(M)*DG(3)
DL2=C1(M)*DG(4)+C2(M)*DG(5)+C3(M)*DG(6)
C COMPUTE THE LOCAL ELEMENT END FORCES (FL(1,4,1),FL(2,4,1)) BY LOCAL
C STIFFNESS MATRIX
FL(1,4,1)=GAMMA(4)*DL1+GAMMA(5)*DL2
FL(2,4,1)=GAMMA(4)*DL1+GAMMA(5)*DL2
C COMPUTE THE GLOBAL ELEMENT END FORCES BY TRANSPOSED TRANSFORMATION
C MATRIX AND COMPUTE THE CONTRIBUTIONS TO THE STRUCTURE JOINT FORCES
J=MINC(M+1)
K=MINC(M+2)
PJF(J,1)=PJF(J,1)+C1(M)*FL(1,4,1)
PJF(J,2)=PJF(J,2)+C2(M)*FL(1,4,1)
PJF(J,3)=PJF(J,3)+C3(M)*FL(1,4,1)
PJF(K,1)=PJF(K,1)+C1(M)*FL(2,4,1)
PJF(K,2)=PJF(K,2)+C2(M)*FL(2,4,1)
PJF(K,3)=PJF(K,3)+C3(M)*FL(2,4,1)
RETURN
END
SUBROUTINE SCHEME (M,1)
C THE PURPOSE OF THIS SUBROUTINE IS TO COMPUTE THE MAXIMUM NORMAL STRESS
C FOR EACH ELEMENT AND LOADING CONDITION AND LOCATE THE CRITICAL MEMBER
C FOR EACH LINKED GROUP (NOTE: NUMBER OF LINKED GROUPS LIMITED TO 3)
C COMMON/MEMBER/A(30),X(30),E(30),GAMMA(30),C1(30),C2(30),C3(30),MC
C 10DE(50),WEIGHT(60),21*XL(30),D(30),FALLW(60),FL(2,50,14),STRESS
C COMMON/JOINT/X(30),X3(30),JOCODE(30),NU
C COMMON/SYSTEM/NACT,Y0DF,INH(30,14),PJF(30,3),ASTRES(6),SD(60),
C 1LINK(30),STRESL(30),LC(60)
C COMPUTE STRESS FOR EACH ELEMENT M (STRESS(M,I))
STRESS(M,I)=ABS(FL(1,4,I)/A(M))
IF(STRESS(M,I) .LT. ASTRES(M)) GO TO 10
C COMPUTE MAXIMUM STRESS FOR EACH MEMBER UNDER ALL ACTIONS I (ASTRES(M))
ASTRES(M)=STRESS(M,I)
C STORE LOADING CONDITION NUMBER THAT PRODUCES MAXIMUM STRESS (LC(M))
LC(M)=I
C IDENTIFY CRITICAL MEMBER IN EACH LINKED GROUP AND STORE ID IN LINK(J)
C STRESL=STRESS IN CRITICAL MEMBER OF EACH LINKED GROUP
IF(LCODE(1).EQ.0) GO TO 10
LC=LCODE(4)
IF(ASTRES(M),LT,STRESL(J)) 6 TO 10
STRESL(J)=ASTRES(M)
LINK(J)=M
10 RETURN
END
SUBROUTINE SFSD (HFSD,KCOUNT,CTP,END,LMAX)
C THIS SUBROUTINE SELECTS MEMBER PROPERTIES BY THE USE OF THE FULLY
C STRESSED DESIGN ALGORITHM (FSD)
C COMMON/MEMBER/A(30),X(30),E(30),GAMMA(30),C1(30),C2(30),C3(30),MC
C 10DE(50),WEIGHT(60),21*XL(30),D(30),FALLW(60),FL(2,50,14),STRESS
C COMMON/JOINT/X(30),X3(30),JOCODE(30),NU
C COMMON/SYSTEM/NACT,Y0DF,INH(30,14),PJF(30,3),ASTRES(6),SD(60),
C 1LINK(30),STRESL(30),LC(60)
T 40300
T 41100
T 41100
T 41200
T 41300
T 41300
T 41400
T 41400
T 41500
T 41500
T 41600
T 41700
T 41800
T 41900
T 42000
T 42100
T 42200
T 42300
T 42400
T 42500
T 42600
T 42700
T 42800
T 42900
T 43000
T 43100
T 43200
T 43300
T 43400
T 43500
T 43600
T 43700
T 43800
T 43900
T 44000
T 44100
T 44200
C WFSO=PARAMETER TO DETERMINE IF THE FSO ALGORITHM IS TO BE USED AND
C WHICH EQUATIONS TO COMPUTE THE DEPENDENT DESIGN VARIABLES WILL BE
C USED
C LIMIT THE NUMBER OF ITERATIONS TO THE NUMBER DEGREES OF FREEDOM
IF(KCOUNT.EQ.000) GO TO 49
C INITIALIZE WARNING MESSAGE FLAG
IF(KCOUNT.EQ.1) MESSE1=MESSE2=MESSE3=1
C SUPPRESS FSO ITERATION OUTPUT IF FSO IS LESS THAN 10
IF(WFSD-LT.10) GO TO 17
IF(KCOUNT.EQ.3) WRITE(6,1)
IF(KCOUNT.NE.1) WRITE(6,11) COUNT
10 FORMAT(T5,'INITIAL CONDITIONS'/T5,'MEMBER NO.'/T5,'MAXIMUM STRESS'
11 FORMAT(T5,'ITERATION NO.'/T5,'MEMBER NO.'/T5,'MAXIMUM STRESS'
D) 15 M=1*NE
15 WRITE(6,15) M,ASTRES(M),SM(M),XI(M),A(M)
17 CONTINUE
DO 20 M=1*NE
20 SD(M)=C*
SOMAX=D*
DO 25 M=1*NE
25 C BYPASS MEMBERS WITH X-SECTION=M[MINIMUM X-SECTION
IF(A(M).LE.0.5) AND ASTRES(M).LE.FALLOw(M)) GO TO 25
IF(LCODE(1).EQ.0) 5 TO 24
C BYPASS MEMBERS WITHIN A LINKED GROUP THAT ARE NOT THE CRITICAL MEMBER
J=LCODE(M)
K=LINK(J)
IF(J.NE.K) GO TO 25
C COMPUTE THE MAXIMUM STRESS DIFFERENCE (SOMAX)
24 SD(M)=ABS(1.0-FALLOw(M))
IF(SD(M).GE.SOMAX) SOMAX=SD(M)
25 CONTINUE
  IF(SDMAX*LE.*1.-CTP) GOTO 95
  KCOUNT=KCOUNT+1
  GO TO 39
C BYPASS MEMBERS WITHIN A LINKED GROUP THAT ARE NOT THE CRITICAL MEMBER
  IF(LCDE(1)*EQ.0) GOTO 26
  J=CODE
  K=LINK(J)
  IF(M*NE.K) GO TO 39
  26 IF(MFSD*LT.10) GO TO 28
       MFSD=MFSD-10
       GO TO (30,33,35)*MFSD
  28 GO TO (33,33,35,)*MFSD
C INDEPENDENT DESIGN VARIABLE-SECTION MODULUS
C DEPENDENT DESIGN VARIABLES-CROSS SECTIOANAL AREA AND MOMENT OF INERTIA
C COMPUTE *8 & *4 BY EQUATIONS DERIVED BY KEVIN TRAYNOR. VPS85B 82-2
  30 IF(KCOUNT*EQ.1) SM(4)=.2*X(4)/X(3)
      SM(4)=1./CTP*SM(4)/ASTRESS(4)/ALLOW(4)
      SM(4)=3.993864+0.1453455*SUMW(-1.35561E-4*SM(4)**2-8.95937E-4*SM(4)**4)
  10 IF(SM(4)*LT.500*X(4)*SM(4)*LT.5) SUMW(-1.58737+4.499167*SM(4)+4.945434*2*SM(4)**2)
      SUMW(-1.22451E-5+SM(4)**4)
  12 IF(SM(4)*LT.1200.X) MSECGE=1
      GO TO 38
C INDEPENDENT DESIGN VARIABLE-MOMENT OF INERTIA
C DEPENDENT DESIGN VARIABLES-SECTION MODULUS AND X-SECTIONAL AREA
C COMPUTE *5 & *2 BY EQUATIONS
  33 X(4)=1./CTP*X(4)/ASTRESS(4)/ALLOW(4)
  36 IF(X(4)*LT.1000.X) AND X(4)*LE.1000.X) SUMW(-1.4+1.58/490.*SRT(X(4)))
      SUMW(-1.44/490.*((2303+X(4))/75.3)
  39 IF(X(4)*LT.1000.X) AND X(4)*LE.1000.X) SUMW(-SRT(62.4*X(4))+3410-2)
      SUMW(-SRT(62.4*X(4))+410-2)
1. IF(XI(M) .GE. 92000. AND X(M) .LE. 20300.) SM(M) = (XI(M) - 9360.3) / 1.876
   IF(XI(M) .LT. 20300.) MSEGE = 1
   GO TO 33
C INDEPENDENT DESIGN VARIABLE-MOMENT OF INERTIA
C DEPENDENT DESIGN VARIABLES-SECTION MODULUS AND CROSS SECTIONAL AREA
35 XI(M) = 1./CTP*X(M)*ASTRES(M)/FALLOW(M)
   A(M) = .53*SQRT(XI(M))
   SM(M) = 0.33*XI(M)**0.75
C SET MINIMUM VALUE OF I AND S
38 IF(XI(M) .LT. .01) XI(M) = .01
   IF(SM(M) .LT. .01) SM(M) = .01
   CONTINUE
C EQUATE MEMBER PROPERTIES IN LINKED GROUP TO MAXIMUM STRESS MEMBER
   DO 41 M = 1, NE
   IF(LCODE(1) .EQ. 0) GO TO 41
   J = LCODE(M)
   K = LINK(J)
   A(M) = A(K)
   XI(M) = XI(K)
   SM(M) = SM(K)
C SET LOWER LIMIT ON X-SECTION AREA
40 IF(A(M) .LT. A(MIN(M))) A(M) = A(MIN(M))
41 GAMA(M) = A(M) * E(M) * X(M) / XL(M)
   IF(LCODE(1) .EQ. 0) GO TO 33
C ZERO OUT STRESL(J), STRESS(M,I) AND ASTRES(M) ARRAYS
   DO 42 J = 1, LMAX
   STRESL(J) = 0.
   DO 44 I = 1, NACT
   DO 44 M = 1, NE
   STRES(M+1) = 0.
   ASTRES(M) = 0.
   DO TO 95
45 IF(MFSD.GT.10) MFSD=MFSD-10
  IF(MFSD.EQ.1) WRITE(6,50)
  IF(MFSD.EQ.2) WRITE(6,55)
  IF(MFSD.EQ.3) WRITE(6,50)

*00FORMAT//T5,*STRUCTURAL OPTIMIZATION BY FSD//T10,*INDEPENDENT DES
1IGN VARIABLE-SECTION MODULUS//T10,*CROSS SECTIONAL AREA AND MOMEN
2T OF INERTIA COMPUTED FROM TRAYOR EQUATIONS//T10,*MEMBER NO.**
329,*MAXIMUM STRESS*T52,*SECTION MODULUS*T76,*MOMENT OF INERTIA*
4T105,*AREA/T29,*(KIPS/IN.)*T54,*CURIC IN.)*T73,*QUARTIC I
5HIC IN.)*T113,*(SQ. IN.)*//

*50FORMAT//T5,*STRUCTURAL OPTIMIZATION BY FSD//T10,*INDEPENDENT DES
1IGN VARIABLE-MOMENT OF INERTIA//T10,*CROSS SECTIONAL AREA AND SEC
2TION MODULUS COMPUTED FROM BROWN AND ANG EQUATIONS//T10,*MEMBER
3NO.*T29,*MAXIMUM STRESS*T52,*MOMENT OF INERTIA*T78,*SECTION MOD
4ULUS*T105,*AREA/T29,*(KIPS/IN.)*T54,*CURIC IN.*T73,*QUARTIC I
5HIC IN.)*T113,*(SQ. IN.)*//

60FORMAT//T5,*STRUCTURAL OPTIMIZATION BY FSD//T10,*INDEPENDENT DES
1IGN VARIABLE-MOMENT OF INERTIA//T10,*CROSS SECTIONAL AREA AND SEC
2TION MODULUS COMPUTED FROM APRA, HAUS, AND RIM EQUATIONS//T10,*
3MEMBER NO.*T29,*MAXIMUM STRESS*T52,*MOMENT OF INERTIA*T78,*SEC
4TION MODULUS*T105,*AREA/T29,*(KIPS/IN.)*T54,*CURIC IN.*T73,*QUARTIC I
5HIC IN.)*T113,*(SQ. IN.)*//

DO 65 M=1,NE
  IF(MFSD.EQ.1) WRITE(6,70),ASTRES(M),SMC(M),XI(M),A(M)
  65 IF(MFSD.EQ.2,3,MFSD.EQ.3) WRITE(6,75),ASTRES(M),XI(M),SMC(M),A(M)

70 FORMAT(T13,I2,T23,F10.3,T53,F10.2,T79,F10.2,T100,F10.2)
75 FORMAT(T13,I2,T23,F10.3,T53,F10.2,T79,F10.2,T100,F10.2)
WRITE(6,80) CTP,KCOUNT
H0FORMAT//T5,*CONVERGENCE TOLERANCE PARAMETER=*,F5.5//T5,*NUMBER OF
1 DESIGN ITERATIONS=*,I3)
C COMPUTE THE TOTAL STRUCTURE WEIGHT
  WEIGHT=0.
DO 82 K=1,NE
42  WEIGHT = (A(K) + KL(K) + DENSITY(K)/1729) + WEIGHT
        WRITE(6,93) WEIGHT
43  FORMAT (//T5,'TOTAL STRUCTURE WEIGHT (Lbs.) =',F10.2)
C STATE WARNING MESSAGES
   IF (KCOUNT.EQ.NDOF) WRITE(6,95)
   IF (MESGE1.EQ.1) WRITE(6,97)
   IF (MESGE2.EQ.1) WRITE(6,91)
   IF (MESGE3.EQ.1) WRITE(6,92)
500 FORMAT (//T5,'NOTE: NUMBER OF ITERATIONS HAS BEEN LIMITED TO THE NUMB
1ER OF DEGREES OF FREEDOM (KCOUNT=NOOF)*')
500 FORMAT (//T5,'WARNING: VALUE OF S IS LESS THAN 5 DURING ITERATIVE P
1ROCCESS; EQUATION FOR I NOT VALID*/)
510 FORMAT (//T5,'WARNING: VALUE OF S HAS EXCEEDED 1200 DURING ITERATI
VE PROCESS; EQUATION FOR I NOT VALID*/)
520 FORMAT (//T5,'WARNING: VALUE OF I HAS EXCEEDED 2000 DURING ITERAT
IVE PROCESS; EQUATIONS FOR A AND S NOT VALID*/)
    IEND = 1
35  RETURN
END
//DATA
VITA

The author was born on December 7, 1953 in Newport, Rhode Island. He received his high school diploma in June, 1972 from Middletown High School in Middletown, Rhode Island. Having attended Boston University in Boston, Massachusetts, he received his Bachelor of Science degree in Civil Engineering from Clarkson College of Technology in Potsdam, New York in December, 1976.

After working for a period at the New Hampshire Department of Public Works and Highways in Concord, New Hampshire and then with S.E.A. Consultants, Inc., of Boston, Massachusetts, he began graduate studies at Virginia Polytechnic Institute and State University in September, 1980. He pursued a Master of Engineering degree in Civil Engineering where he concentrated in the field of Structural Engineering.

Robert Huntington Durfee
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