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Strengthening of existing timber stringer bridges by the addition of steel stringers

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by

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CHAPTER 1. INTRODUCTION

Timber, a abundant, versatile and easily available material has been used to construct buildings, bridges etc. since the evolution of mankind. From prehistoric times through the Middle Ages, our ancestors adapted available materials, such as logs and vines, to span streams. From the end of the Middle Ages through the 18th century, scientific knowledge developed and influenced the design and construction of timber bridges. In the 19th century, the sophistication and use of timber bridges increased in response to the growing need for public works and transportation systems associated with the industrial revolution. With the 20th century, came major technological advances in wood design such as laminating and preservative treatments, which made it easy to build timber bridges.

Timber's strength, light weight and energy absorbing properties are features which make timber desirable for bridge construction. Wood is abundantly available at a very competitive price when compared to other materials and is capable of supporting short term over loads without adverse effects. Timber bridges can be constructed in virtually any weather conditions, without detriment to the material. Deterioration of timber bridges due to de-icing agents is of very little significance when compared to the deterioration of concrete and steel bridges resulting from de-icing agents. Also these bridges do not require special equipment for installation and can normally be constructed without highly skilled labor. Using modern chemical preservatives and application techniques, timber can now be effectively protected from deterioration; timber treated with preservatives requires minimal maintenance and painting. The advent of glued-laminated timber, (glulam) 40 years ago provided designers with several competitive alternatives. Glulam, the most widely used modern timber bridge material, is manufactured by bonding sawn lumber laminations together with waterproof structural adhesives. Thus, glulam members are virtually unlimited in depth, width, and length and can be manufactured in a wide range of shapes. Glulam provides higher design strength than sawn lumber and provides better utilization of available timber by permitting the manufacture of large wood structural elements from smaller lumber sizes. Technological advances in laminating over the past four decades have further increased the suitability and performance of timber in modern highway bridges. Finally, timber bridges present a natural and aesthetically pleasing appearance, particularly in natural surroundings.

From the early stages of road construction until the beginning of this century or even the late fifties, timber bridges have been used extensively for short and medium span bridges. Because of increased frequency or/and heavier truck loads it became very difficult for timber bridges to accommodate present day traffic, as timber bridges have been designed for live loads that are only a fraction of present commercial vehicle weights. Construction of timber bridges for spans greater than 36 ft. [2] and heavy loads is not advantageous and very expensive when compared to structural steel, prestressed concrete, and reinforced concrete bridges. Hence, construction of timber bridges has been discontinued except in rural and remote areas.

Many of the timber bridges built a few decades ago are still serving as important traffic carriers. According to the 1991 Federal Highway Administration's national bridge inventory [1], 23 percent (of all timber, concrete, steel and other bridges) are classified as structurally deficient, and 16 percent are classified as functionally obsolete. Because of limited resource and the high costs involved with their replacement, maintenance and upgrading these bridges to present day traffic standards is of prime importance. The present economic situation requires these bridges to be utilized as long as is safely possible. Hence, a procedure for strengthening such structurally deficient bridges has been developed.

Objectives

The purpose of this investigation is to investigate the behavior of the timber bridge structural components when subjected to present day AASHTO standard loads and to develop procedures for strengthening them. The objectives of this work are:

- To study the behavior of timber bridge components.
- To determine the overstresses in the components of the bridge when subjected to present day AASHTO standard loads.
- To develop techniques to strengthen the overstressed structural components of the timber bridge to meet the present day AASHTO standards which thus reduce the overstresses in bridge components.

Literature survey

Bridges are perhaps the largest category of structures which are in need of rehabilitation in this country. According to the Federal Highway Administration's national bridge inventory about 40 percent of the bridges in America are structurally deficient and/or functionally obsolete [1, 6]. The structural deficiency can result from deterioration, damage or increased load requirements in excess of the original design capacity. Also bridges may become functionally deficient when the roadway width, vertical clearance, or geometry are inadequate for current traffic requirements. Hence a large number of structurally deficient bridges are posted i.e. restricted to lesser loads or particular type of vehicle or the number of lanes are reduced. The question is whether to rehabilitate or replace a bridge is complex. In some instances, the total cost of rehabilitation is far more than what it takes to erect a new bridge [7]. The costs associated with demolishing the old bridge, erecting a temporary bridge for emergency vehicles if required, traffic detours, new waterway studies, permits, and new substructures also have to be considered. New property may also have to be purchased for widening or modifying the existing approaches. The costs of disrupting commercial activities (i.e., loss of business) also cannot be overlooked.

Faced with statistics that indicates that about 40 percent of our bridges have fallen into a state of disrepair, some bridge engineers have coined a phrase to describe a new concept in bridge rehabilitation: "recycling" [6]. Rehabilitation is done by reinforcing or replacing a few structural components and thereby extending the life and rejuvenating old structures. Hence, rehabilitation is most commonly performed on such older bridges that are built to lesser geometric or loading standards than those required for today's modern traffic.

Background information on timber, its properties, the design and construction, preservation and protection of timber bridges and some information and case studies about inspection, maintenance, rehabilitation and, replacement of timber bridges is available in ref. [2].

Figure 1.1 shows typical timber deck, timber stringer bridges.

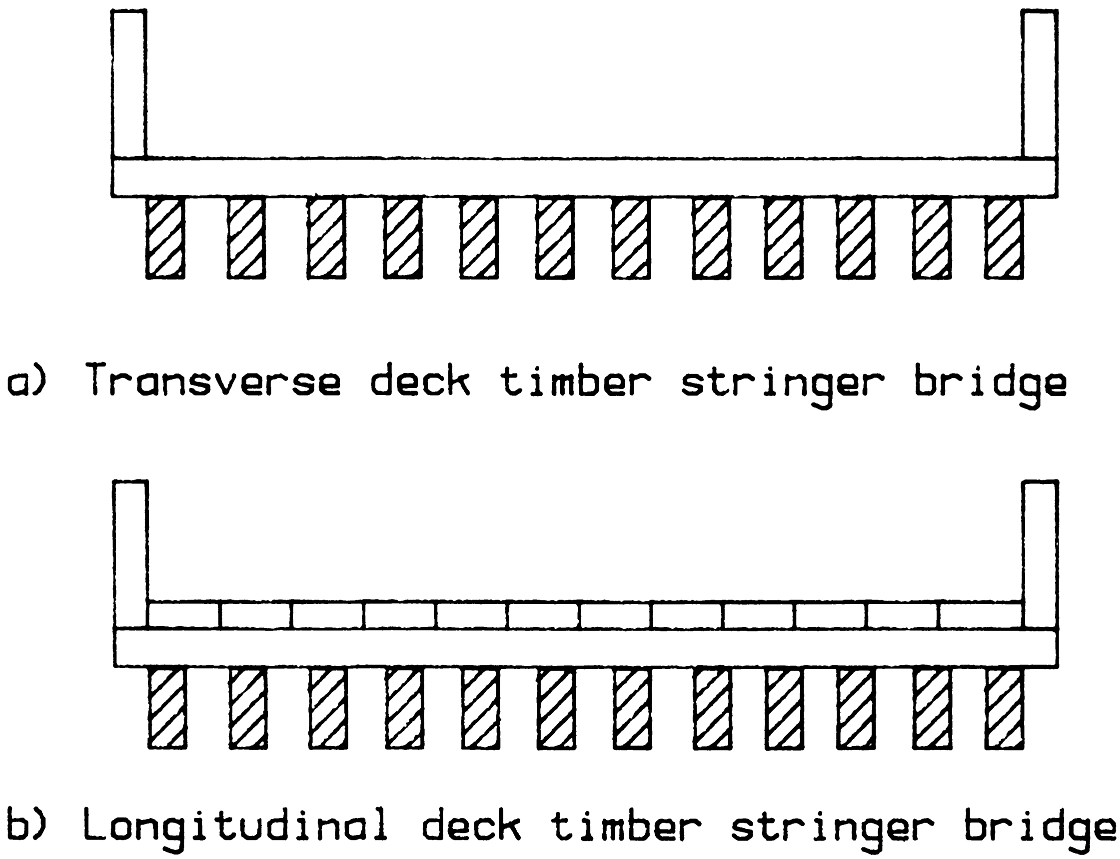

Figure 1.1: Timber deck, timber stringer bridges

Several working drawings of timber bridges built in America have been illustrated in ref. [3]. It provides information about span, width, stringer size and spacing, thickness of deck, connection details, etc.

In the past several techniques have been used for increasing the load carrying capacity of existing timber, concrete and steel bridges. These include:

- Strengthening weak members by replacing or by adding additional material
- Modifying the structural system
- Reducing the dead load through the installation of light weight deck system
- External prestressing
- Adding exterior reinforcing plates to concrete beams by adhesives or bolts
- Making a series of simple spans continuous
- Erecting additional supports to reduce the span length, etc.

Strengthening by replacing or adding critical members

Strengthening deficient or critical members requires adding new material to the existing member, or replacing the entire member, or replacing a portion with new material. In steel bridges, cover plates are added to steel beams, or girders or, structural shapes are added to steel truss members to increase the available section. In many instances, connections are the critical part of the structure. This deficiency can be corrected by adding additional connectors or by replacing the entire connection.

Strengthening by modifying the structural system

Strengthening by modifying the structural system includes in adding few structural members to the existing system. In situations where underclearance permits, the strengthening of either a stringer or a floor beam by adding a kingpost truss system provides an excellent means for increasing live load capacity. In certain other cases structural members are added to maintain continuity and thereby increase the live load capacity. Another method of strengthening by modifying the structural system is by developing a composite action between the deck slab and girder system, wherein the deck slab and beam act together in resisting live loads.

Strengthening by reducing dead load

Dead load reduction can most easily be accomplished by removing the existing deck and providing a lightweight substitute. A number of deck systems have been developed to provide a lightweight yet structurally adequate system. The most common of these are:

- Open steel grid
- Concrete filled steel grid
- Corrugated metal
- Laminated timber
- Metal plate

Also dead load can be reduced by eliminating certain features of the roadway cross section. Concrete parapets can be replaced by lightweight railing [8]. Curbs and median barriers can be replaced with lightweight sections.

Strengthening by external prestressing

Post tensioning or external prestressing truss tension members can be done effectively for steel truss bridges. Cables are strung along the truss member and attached to the end of the member or to the connecting pin. Turnbuckels are introduced in the cables to provide tensioning. The compression stresses thus induced or the resulting reduction in tensile stress in the member permit the member to carry additional live load.

Strengthening by adding exterior reinforcing plates to concrete beams

In United Kingdom and South Africa [8], few concrete bridges are strengthened by external steel reinforcing plates either to the beam flange for added flexural capacity or/and to the web for added shear capacity. Connection of the steel plates to the concrete can be made by bolting with expansion type anchors or by epoxy adhesives.

Strengthening by making a series of simple spans continuous or reducing the span length

One other strengthening method is by making a series of simple spans continuous and thereby reducing the positive moment in the middle of the span and a corresponding increase in the live load capacity. This can be accomplished by establishing continuity of the beam or girder at the support. Also, bridge spans can be reduced by erecting additional supports and thereby increasing live load capacity.

Strengthening timber stringer bridges

Of all these strengthening techniques, strengthening timber bridges is most easily done by adding additional members. Structurally inadequate floor systems on truss and girder bridges can be rehabilitated by positioning additional members between the existing stringers to provide a increase in capacity. Also external reinforcing is another technique used routinely in strengthening timber bridges. This normally consists of steel plates or shapes attached to the substandard members with bolts or lag screws, thus forming composite steel-timber members.

As a result of field inspection and through information and discussions with bridge maintenance personnel in several county's of Iowa, it was determined that several county engineers strengthen timber stringer bridges by adding additional timber stringers between the existing timber stringers.

The addition of the new stringers and respacing of existing stringers reduces the magnitude of the load carried by the individual stringers. In this investigation, a procedure for increasing the load carrying capacity of a given existing timber bridge, by replacing a few of the existing timber stringers with steel stringers has been developed. The added steel stringers reduces the magnitude of the load carried by timber stringers, because of the higher stiffness of the steel stringers. The amount of load carried by steel and timber stringers depends on the relative stiffness of the various stringers. The stiffer the steel stringers, the less load carried by timber stringers.

CHAPTER 2. TIMBER BRIDGE STRENGTHENING

Numerous types and configurations of timber bridges built in the past decades exist today. All timber bridges consist of two basic components, the superstructure and the substructure. The superstructure is the framework of the bridge span and includes the deck, floor system, main supporting members, railings, and other incidental components. The substructure is the portion of the bridge that transmits loads from the superstructure to the supporting rock or soil.

Longitudinal beam superstructures are the simplest and most common types of timber bridges. Longitudinal beam superstructures consist of a deck system supported by a series of timber beams between two or more supports. Bridge beams are constructed from logs, sawn lumber, glued-laminated timber, or laminated veneer lumber. Transverse deck or slab superstructures are constructed of sawn lumber planks, glulam or nail-laminated lumber placed transversely between supports, with the wide dimension of the laminations vertical. From the literature survey it was found that longitudinal stringer bridges are economical and practical for maximum clear spans up to approximately 36 feet.

Timber stringer, timber deck bridge

The timber bridge considered in this investigation was a single span longitudinal beam superstructure. The longitudinal or transverse deck is supported on timber stringers, which in turn were supported at the ends by an abutment. An example of this type of timber bridge with transverse timber deck and longitudinal timber stringers is shown in Figure 2.1.

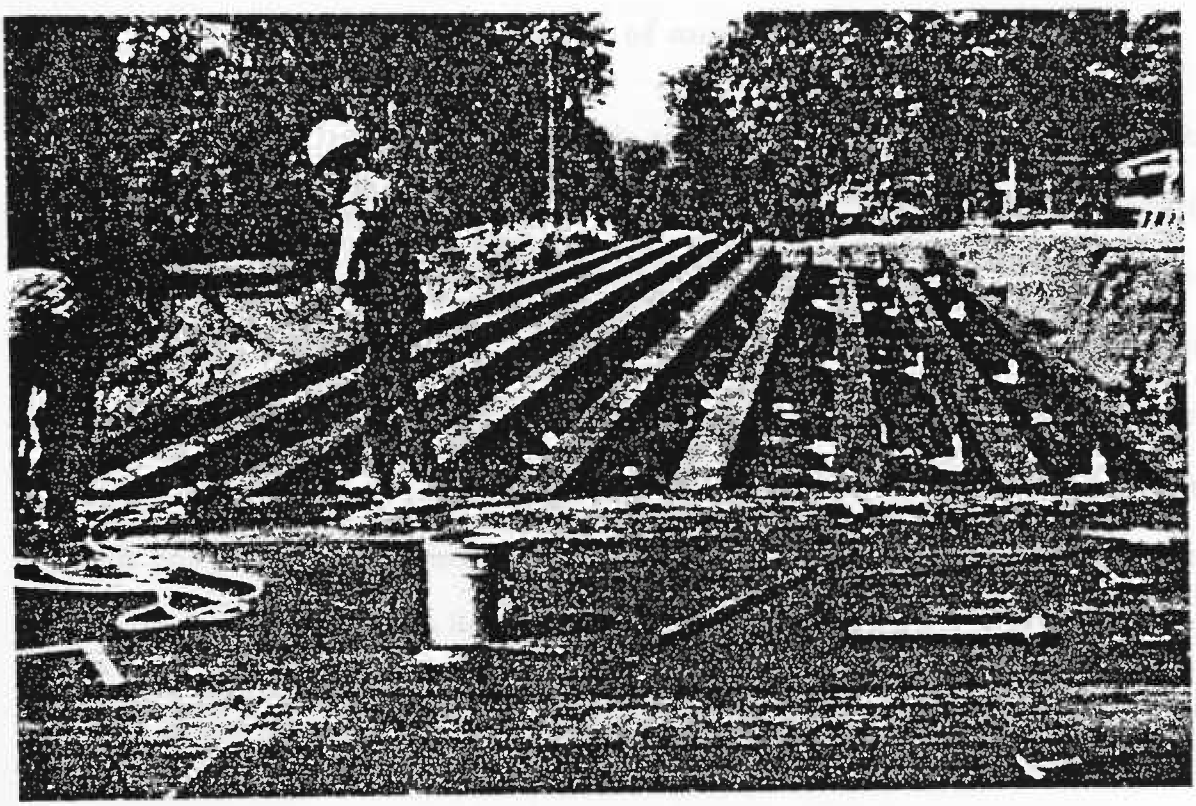

Figure 2.1: Timber bridge with longitudinal timber stringers

From the data obtained from several Iowa countys, it was found that the timber deck was usually made of sawn lumber plank 3 to 4 in. in thickness and 10 to 12 in. wide. The size of the timber stringers vary from 3 to 6 in. in width and 12 to 16 in. in depth, depending on the span, width of the bridge, timber stringer spacing and loading conditions. The bridge span varies from 12 to 25 ft. and in few cases it may reach 30 ft. The bridge width was either approximately 16 ft. (single/one lane bridge) or 24 ft. (double/two lane bridge). The timber stringer spacing varies from 8 to 16 in., again depending on the span, width, stringer size and loading conditions.

Method of analysis

Whether or not an existing bridge is capable of supporting the modern traffic can be ascertained only after a rigorous analysis of the structure has been completed.

Several simple procedures have typically been used in the past for the analysis of timber bridges. The strength of a timber bridge can be easily determined when the stringer spacing is uniform. But there are no accurate methods available to determine the strength when the stringer spacing varies. Also the above theories or analysis methods fail to take into consideration the interaction between the deck, stringers, and the loads. Until now, the bridges were analyzed or designed with constant stringer spacing. In the strengthening technique presented in this thesis, steel stringers were positioned between the existing timber stringers. Available analysis methods are inadequate to calculate the strength of a bridge with timber and steel stringers together.

Numerical solution techniques, like finite strip method, finite element method, etc. are needed when either the geometry or material property vary. In this investigation, the finite element method of analysis which is suitable for the analysis of a bridge with timber and steel stringers, has been used. A general purpose finite element analysis package called ANSYS (ANalysis SYStems) [9] was used. The package accommodates a wide selection of elements, material properties, real constants, loading conditions and has good graphics output capabilities.

Finite element model

Due to symmetry in geometric and loading conditions, it was only necessary to model one half of the bridge in the analysis. The deck has been modeled using a three dimensional plate element with six degrees of freedom at each node, i.e., three translations in x, y, and z directions and three rotations about x, y and z axes. This element has both bending and membrane capabilities. Figure 2.2 shows the plate bending element with co-ordinate directions, displacements and rotations. The timber and steel stringers were modeled as three dimensional elastic beams. The three dimensional beam element has six degrees of freedom at each node, i.e., translations in the nodal x, y and z directions and rotations about the nodal x, y and z axes. Figure 2.3 shows the beam element used to model steel and timber stringers in the analysis.

The deck is supported over the stringers which in turn are supported at the abutments. The deck is made up of timber boards and is usually nailed, spiked or glued to the stringers. There will be joints in the deck, either in transverse or longitudinal direction depending on the type of deck. In the analysis the joints were neglected and the deck was considered as a continuous deck in both ways for simplicity of analysis and the validity is illustrated in Appendix I. Analysis was

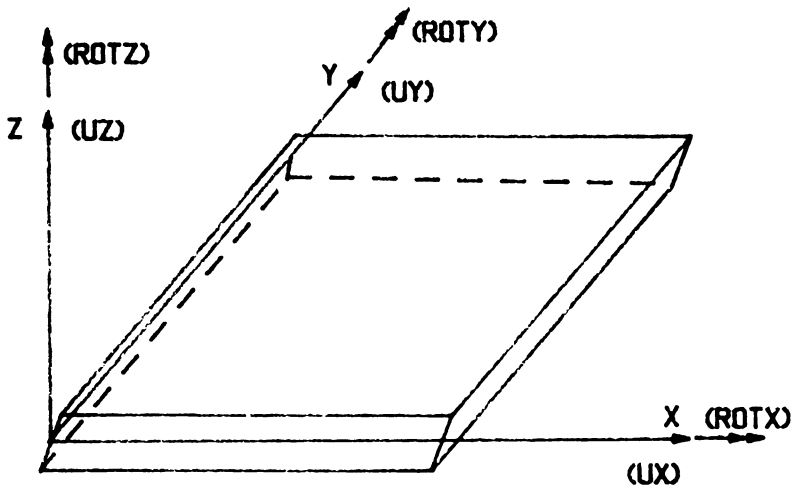

Figure 2.2: Finite element model of a plate bending element

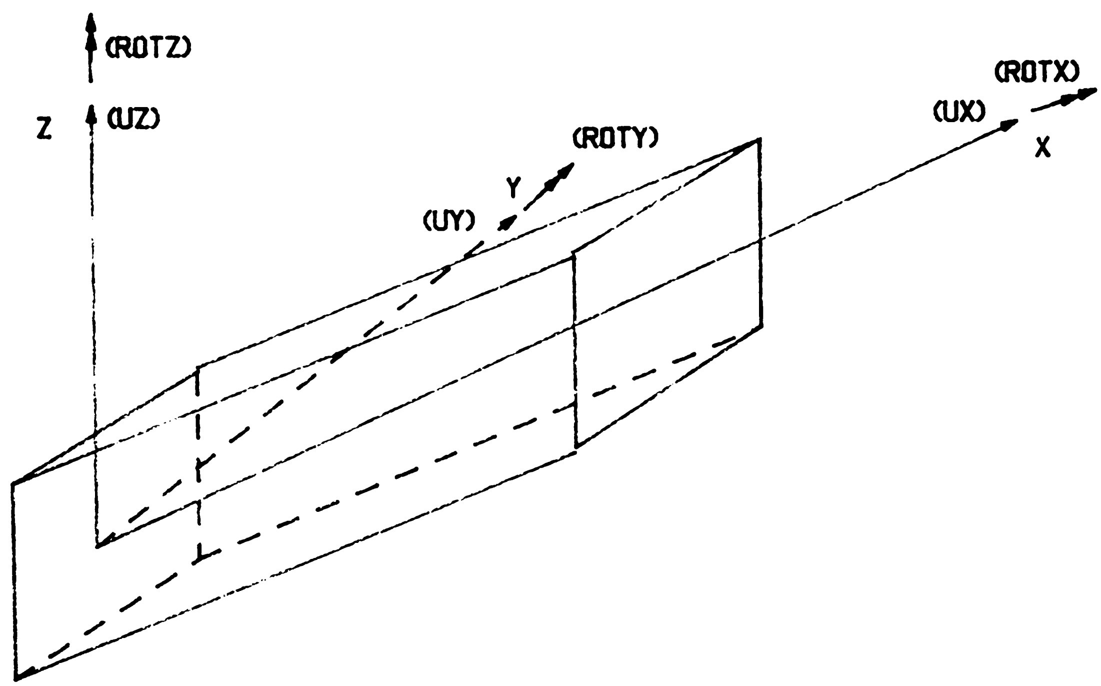

Figure 2.3: Finite element model of a beam element

performed on a deck with and without joints. It was found that the longitudinal stresses were about 5 percent higher for a continuous deck than for a discontinuous deck with joints. The compatibility between the deck and stringer was established by allowing the same amount of vertical displacement in both the deck and stringer. The horizontal movement of the deck was not restrained in either direction. Figure 2.4 shows the finite element model of the longitudinal timber stringer bridge considered for investigation. The number of elements considered for analysis varied in each case and depended on the length of the bridge, width of the bridge and stringer spacing. Depending on the above factors, the size of the plate element varied from 8 in. x 12 in. to 12 in. x 16 in. and the size of the beam element varied from 12 in. to 16 in.

The validity of the finite element method of analysis and the finite element model is illustrated in Appendix II. For comparision the longitudinal stringer stresses were determined by different simplified analysis. It was found that there was a difference in stresses between the different methods when compared with finite element method. This is because several different rigidities are considered in one or two parameters in the simplified method of analysis and hence the difference in results.

Geometric and material properties

Detailed information about the geometric dimensions of timber deck, timber stringer bridges constructed in the country is given in [2]. It provides information about the bridge span, bridge width, stringer sizes, stringer spacing and other details about the bridge geometrics. More data was obtained through discussion with bridge maintenance personnel from several Iowa countys. Several actual existing timber bridge details were obtained from the Iowa countys and field inspection. Ma-

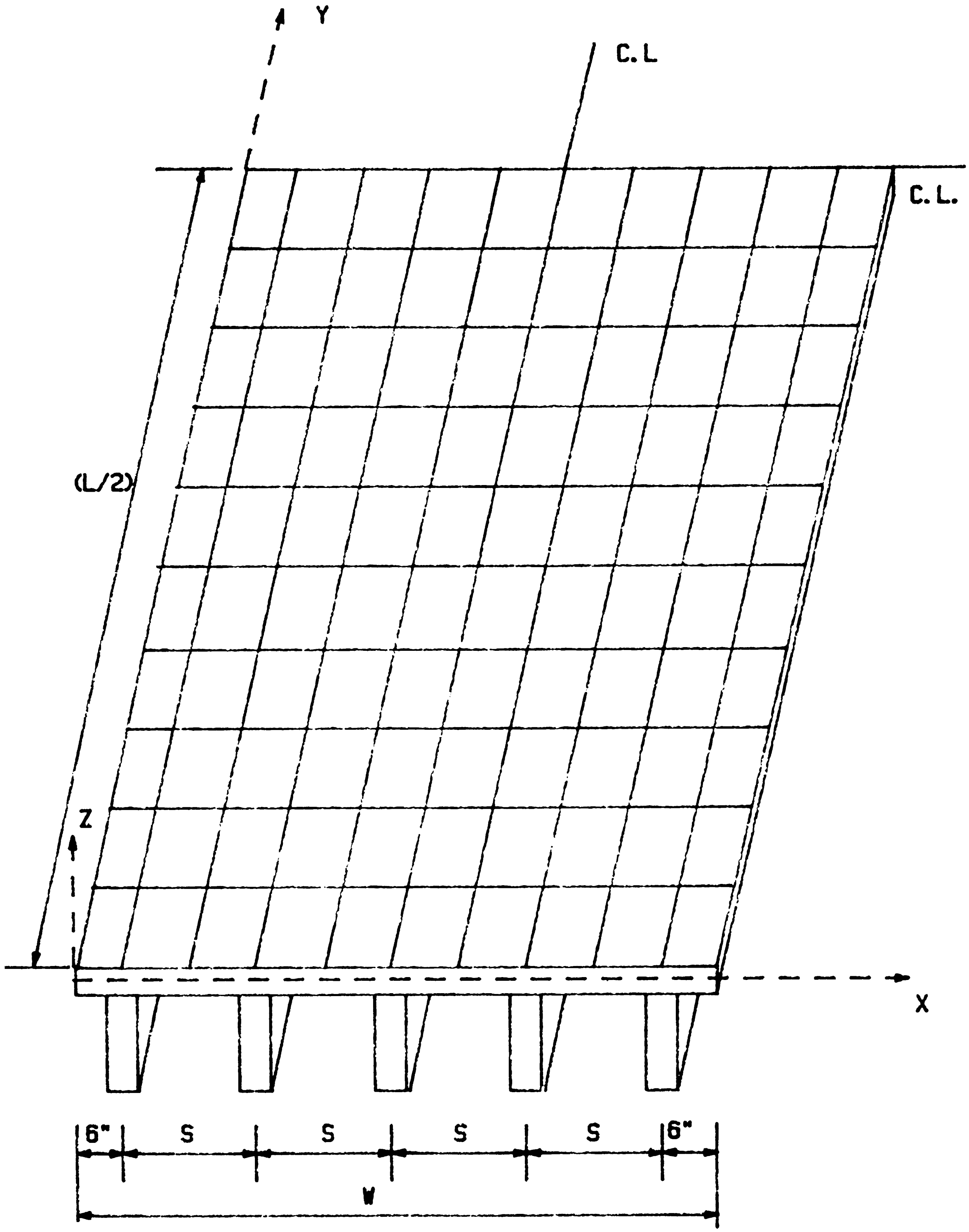

Figure 2.4: Finite element model of a longitudinal stringer bridge

terial properties of the bridge components like deck and stringers were obtained from reference [2].

Due to the vast differences in bridge geometrics, it was decided to analyze for different conditions of loading with different geometric combinations.

Figure 2.5 shows the various parameters considered in the anlaysis. Different combinations were used in the analysis to obtain the realistic behavior of numerous existing bridges.

Load configuration

A bridge must be designed to safely resist all loads and forces that may reasonably occur during its life i.e., dead loads, live loads, impact and so on.

The dead loads are the permanent weight of all structural and non-structural components of a bridge, and were assumed to be uniformly distributed along the length of a structural element (stringer, deck). The dead load of timber (treated or untreated) was assumed as 50 lb/ft^3 and that of steel as 490 lb/ft^3 (AASHTO 3.3.6) [4].

The live loads are the weights of the vehicles that cross the bridge. Each vehicle consists of a series of moving concentrated loads that vary in magnitude and spacing. The live loads which were used previously in the design of the bridges in question were significantly lighter than present day standard loads. AASHTO provides two system of standard vehicle loads - H loads and HS loads (AASHTO 3.7.5 and 3.7.6). Each system consists of individual truck loads and lane loads. The load which produces highest stress should be used for design. AASHTO standard load (HS 20-44) considered in this investigation is given in Figure 2.6.

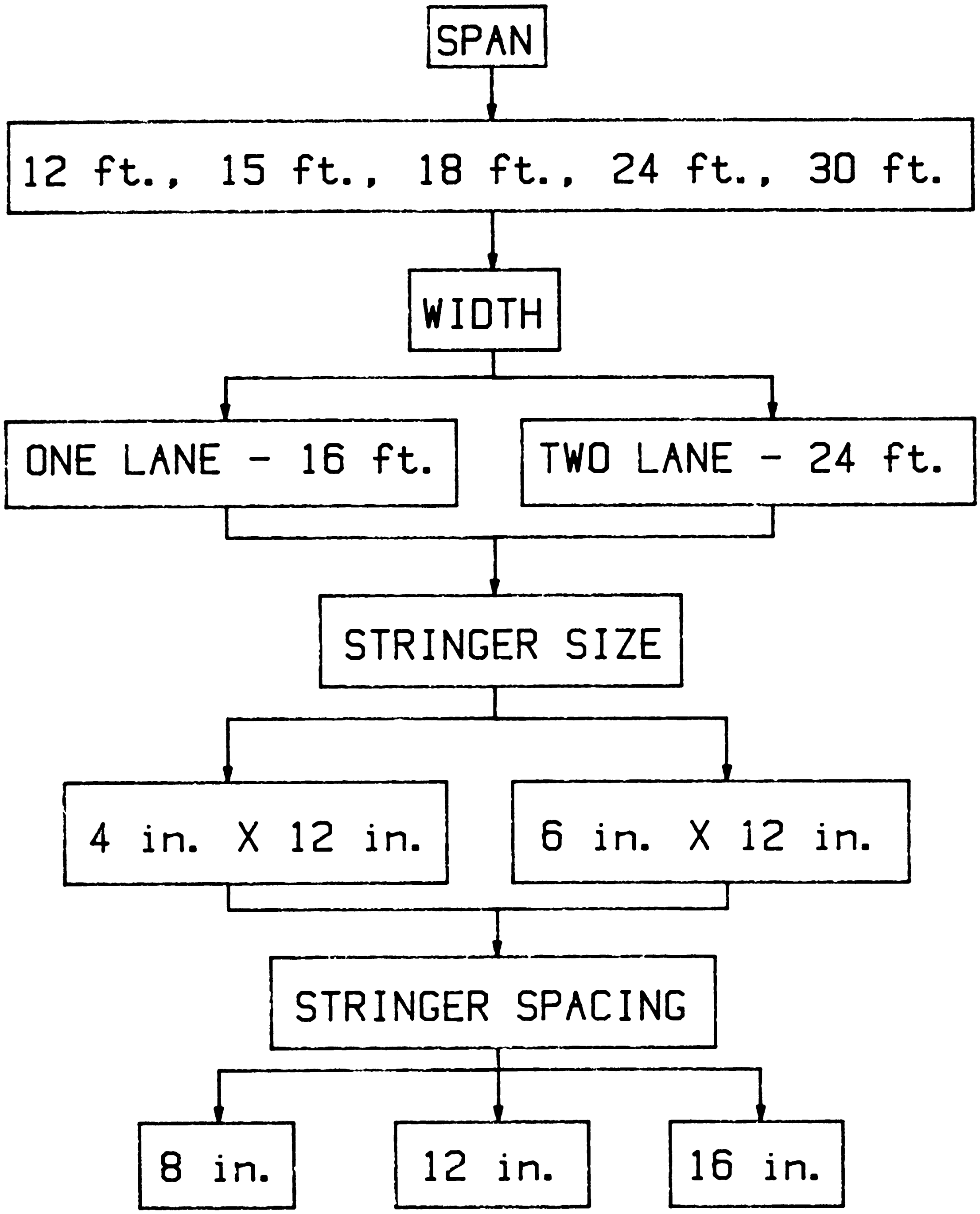

Figure 2.5: Various geometric parameters considered in the bridge analysis

There are six different legal dual axle truck load configurations in Iowa. Figure 2.7 and Figure 2.8 illustrates these loads.

It was required to determine which loads are likely to occur and the magnitude and combination of loads that produce maximum stress. For a set of rolling loads to have a maximum moment, the center of gravity of all the loads and the load nearest to the center of gravity should be positioned equidistant from the center of the span. Then the maximum moment occurs under the load nearest to the center.

This principle is illustrated in Figure 2.9 for a HS 20-44 truck on a 24 ft. span.

The procedure is repeated for all the load cases and the maximum load case was considered for analysis.

AASHTO specifications require that impact be included in the design of bridges. The impact is included as a fraction of vehicle live load. Because of timbers ability to absorb shocks and loads of short duration, AASHTO does not require an impact factor for timber bridges (AASHTO 3.8.1).

Live load position

One of the critical steps in determining the live load effect on a given bridge was to determine the load position. Three possible truck load positions shown in Figure 2.10 have been used in the preliminary analysis.

They are as follows:

- Load case 1: One wheel load of a truck at a distance of 2 ft. from the curb.
- Load case 2: Wheel loads of a truck placed symmetrically about the centerline.
- Load case 3: Two truck loads positioned as per AASHTO on a two lane bridge.

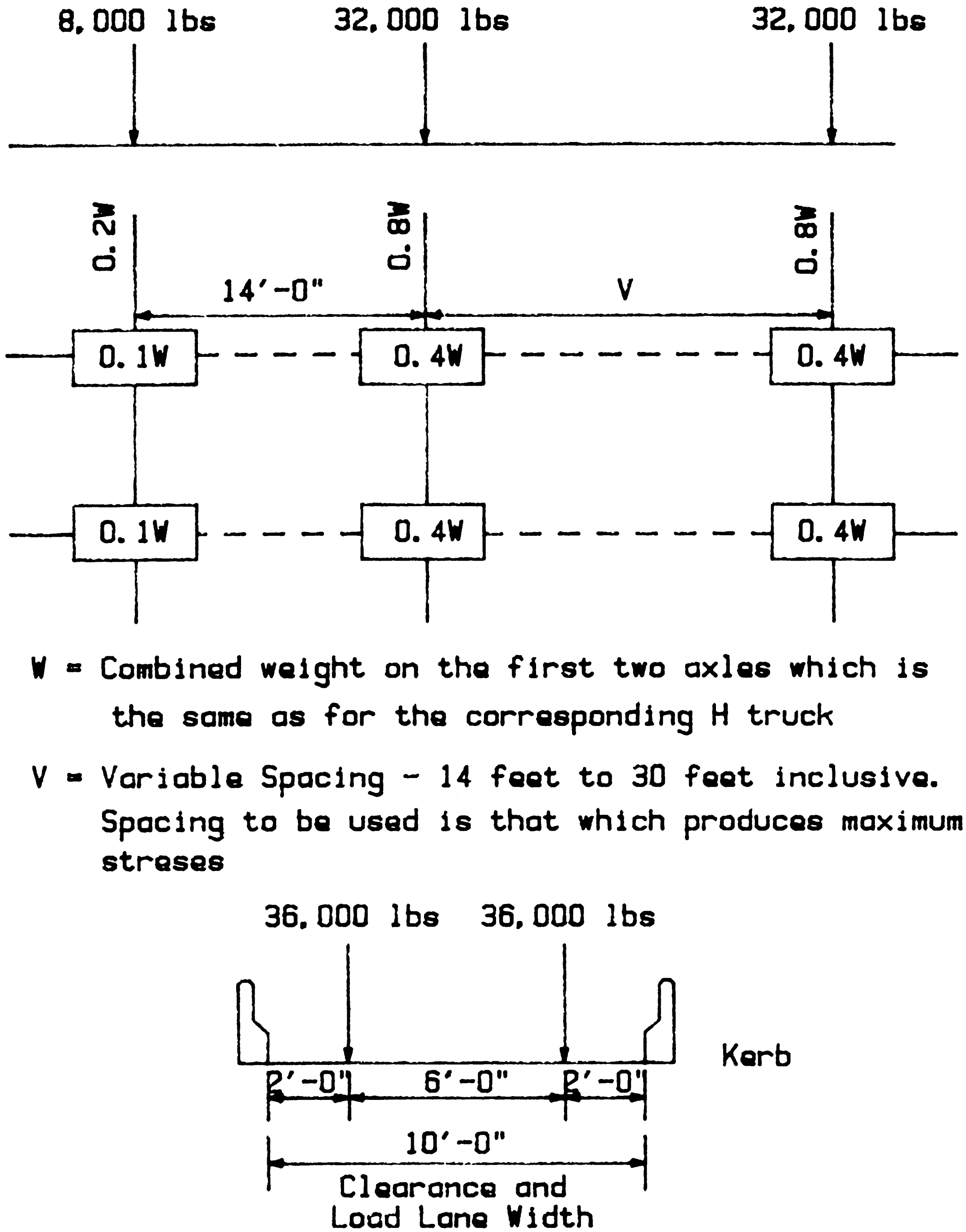

Figure 2.6: AASHTO standard loads - HS 20-44

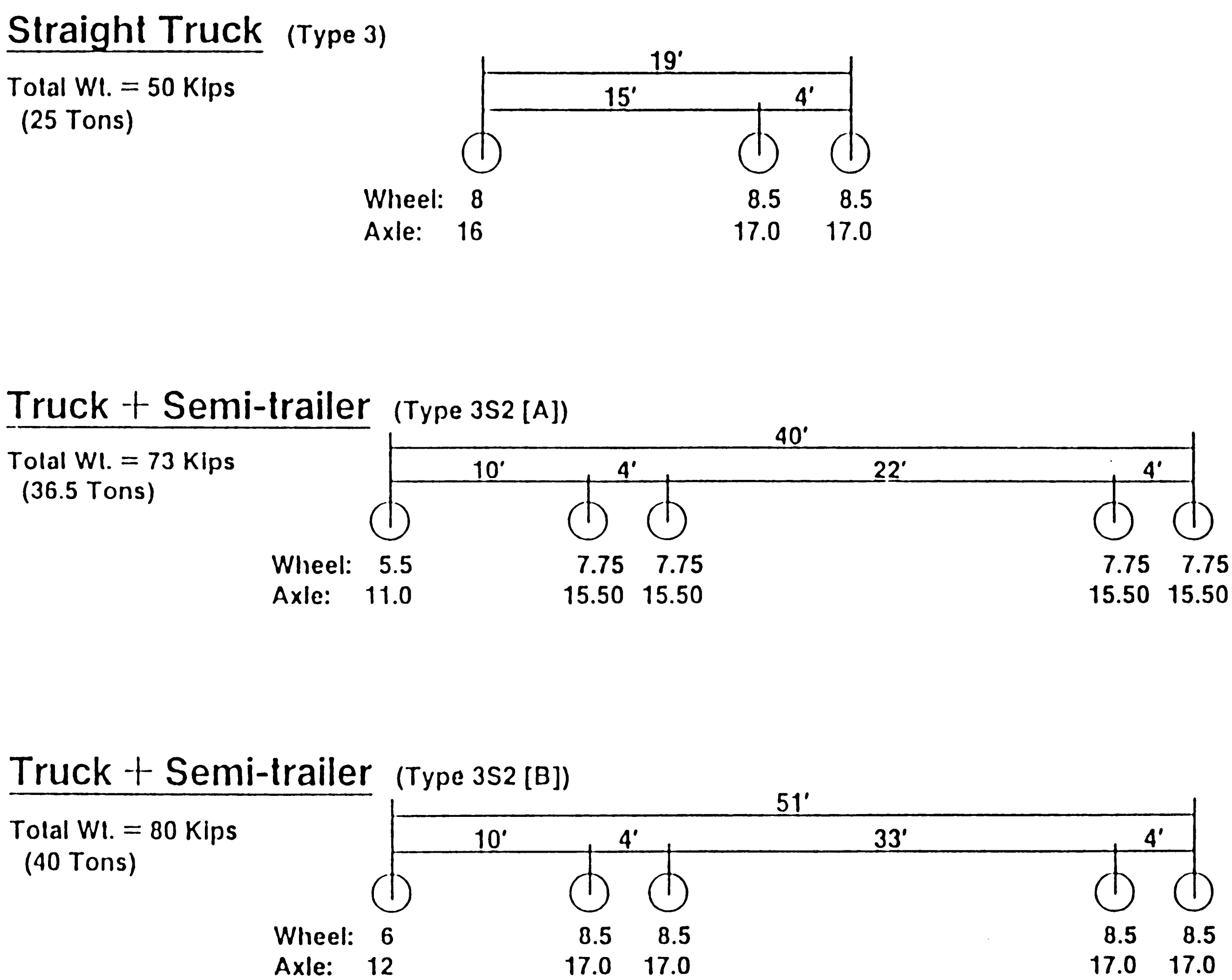

Figure 2.7: Iowa Department of Transportation legal dual axle truck loads (Wheel and axle loads are shown in Kips)

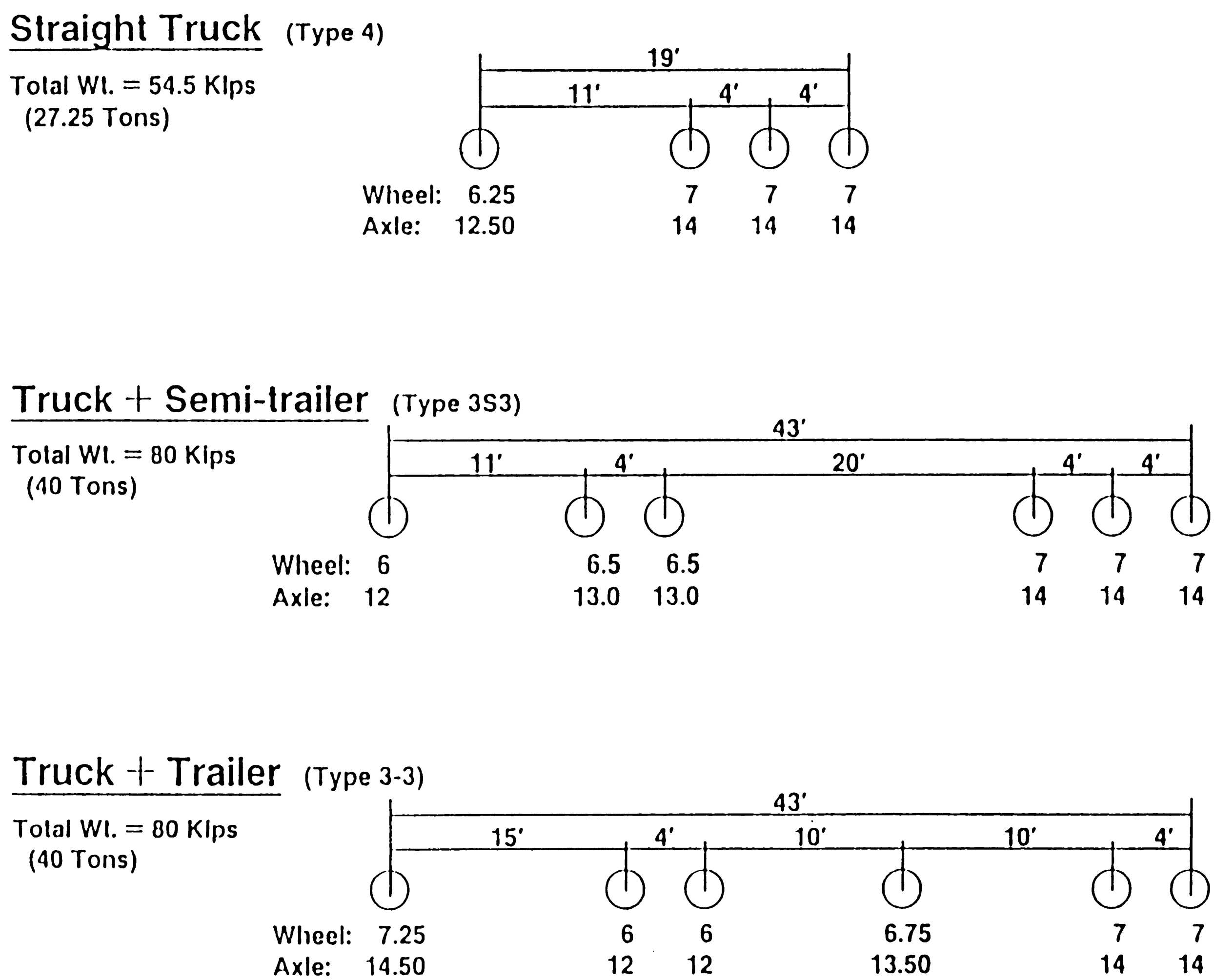

Figure 2.8: Iowa Department of Transportation legal dual axle truck loads (Wheel and axle loads are Shown in kips)

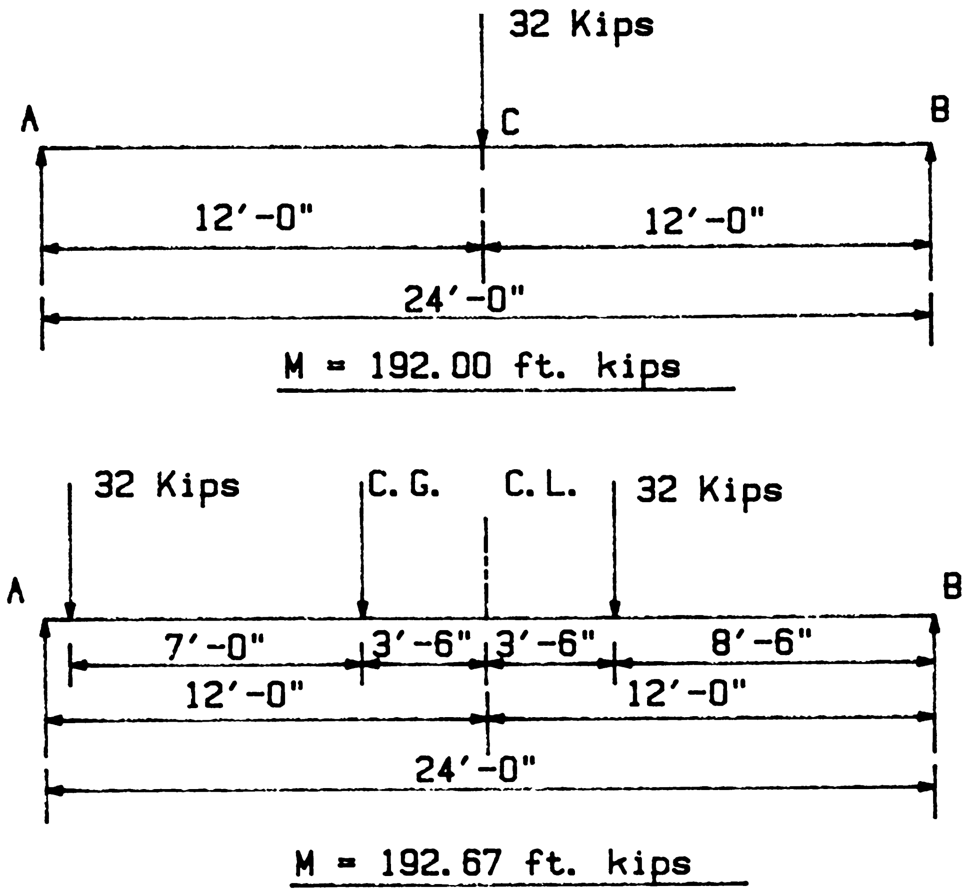

Figure 2.9: Determination of maximum moment in a span due to moving loads

Load case 1 refers to a wheel line of a truck positioned at a distance of 2 ft. from the curb or rail as per AASHTO (Figure 3.7.6A). In load case 2, the wheel lines of a truck were positioned symmetrically about the center line in longitudinal direction to obtain a maximum effect. Load case 3 is for a two way or double lane bridge in which two trucks were positioned as per AASHTO (3.6.2). A preliminary study was performed (on a 18 ft. span and 24 ft. width bridge with stringer size 4 in. x 12 in. and stringer spacing 12 in. center to center) to determine the most severe loading

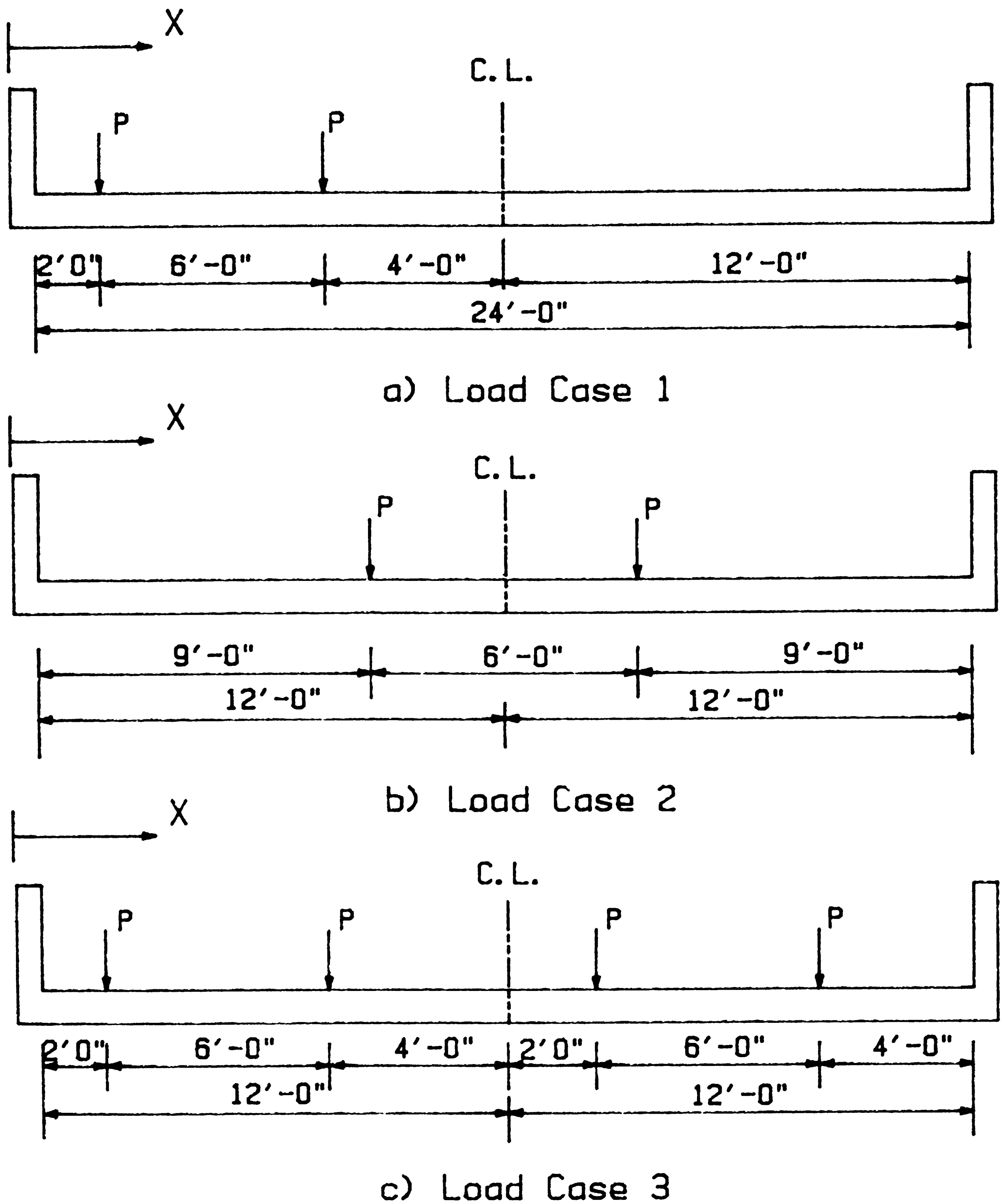

Figure 2.10: Different truck load positions

condition. Figure 2.11 shows the plot of the longitudinal bending stresses in different timber stringers (without any steel stringers) across the width of the bridge due to the three different load conditions.

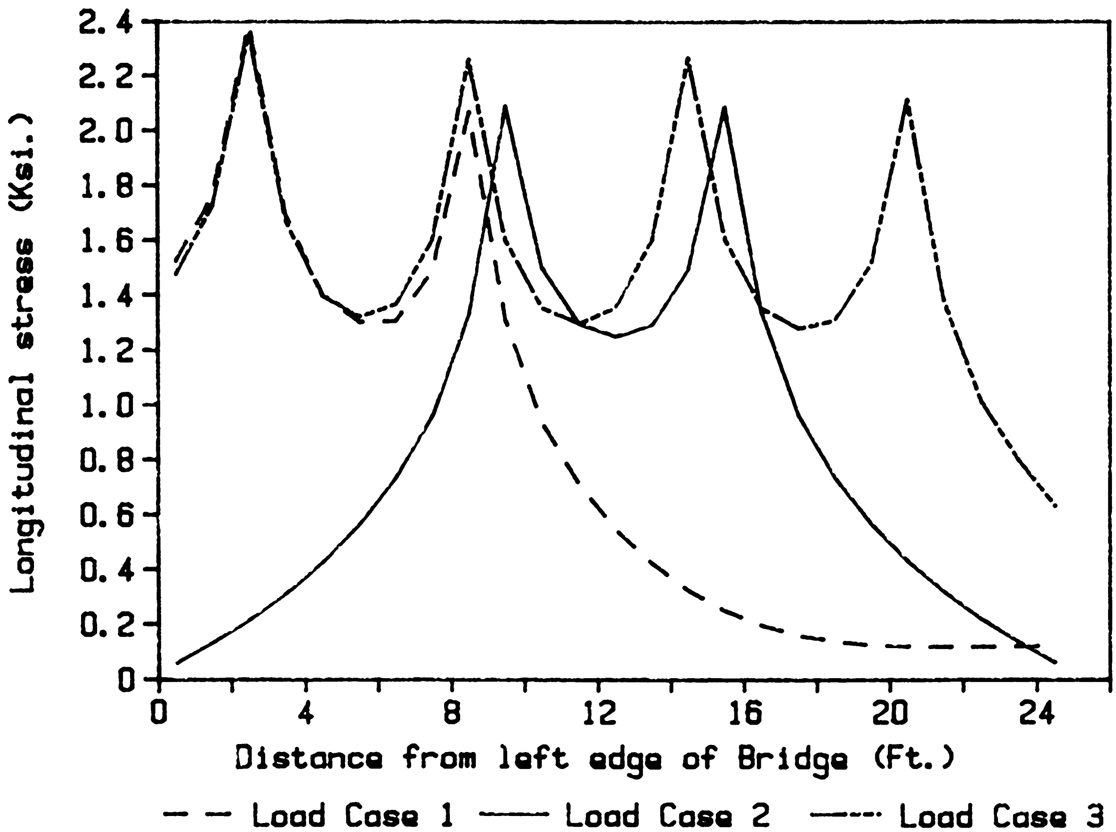

Figure 2.11: Maximum longitudinal stresses in timber stringers for different live load positions

It can be seen that the timber stress pattern is different for each pattern of loading, but the maximum timber stresses were almost the same. Hence in the remaining analysis, only one load case (load case 1), i.e., a wheel load of a truck placed 2 ft. from the curb has been used to simplify the analysis.

Position of steel stringers

The primary objective of the work was to determine strengthening techniques for timber stringer bridges. This was accomplished by replacing several existing timber stringers as shown in Figure 2.12. A thorough preliminary analysis was carried out to determine the required number, position and size of steel stringers.

Data obtained from Iowa Department of Transportation (Iowa DOT), several references [1, 3] and field inspection revealed that the timber stringers used in the existing timber bridges in the field were usually 12 in. or 16 in. in depth. Hence, to simplify construction of adding or replacing stringers, the depth of steel stringers was limited to either 12 in. or 16 in.

Figure 2.12 shows the position of steel stringers. Steel stringers of different sizes were used in this investigation to determine the variations in the timber and steel stringer stresses. An analysis of the timber bridge with above load and geometric configuration was carried out; results are presented and discussed in the following sections.

Strengthening techniques

Timber bridges can be strengthened either by adding steel stringers between existing timber stringers or by replacing few existing timber stringers with steel stringers.

The choice of the above two methods depends on the existing timber stringer spacing. If the stringers are spaced very closely, say between 8 in. and 16 in. it is

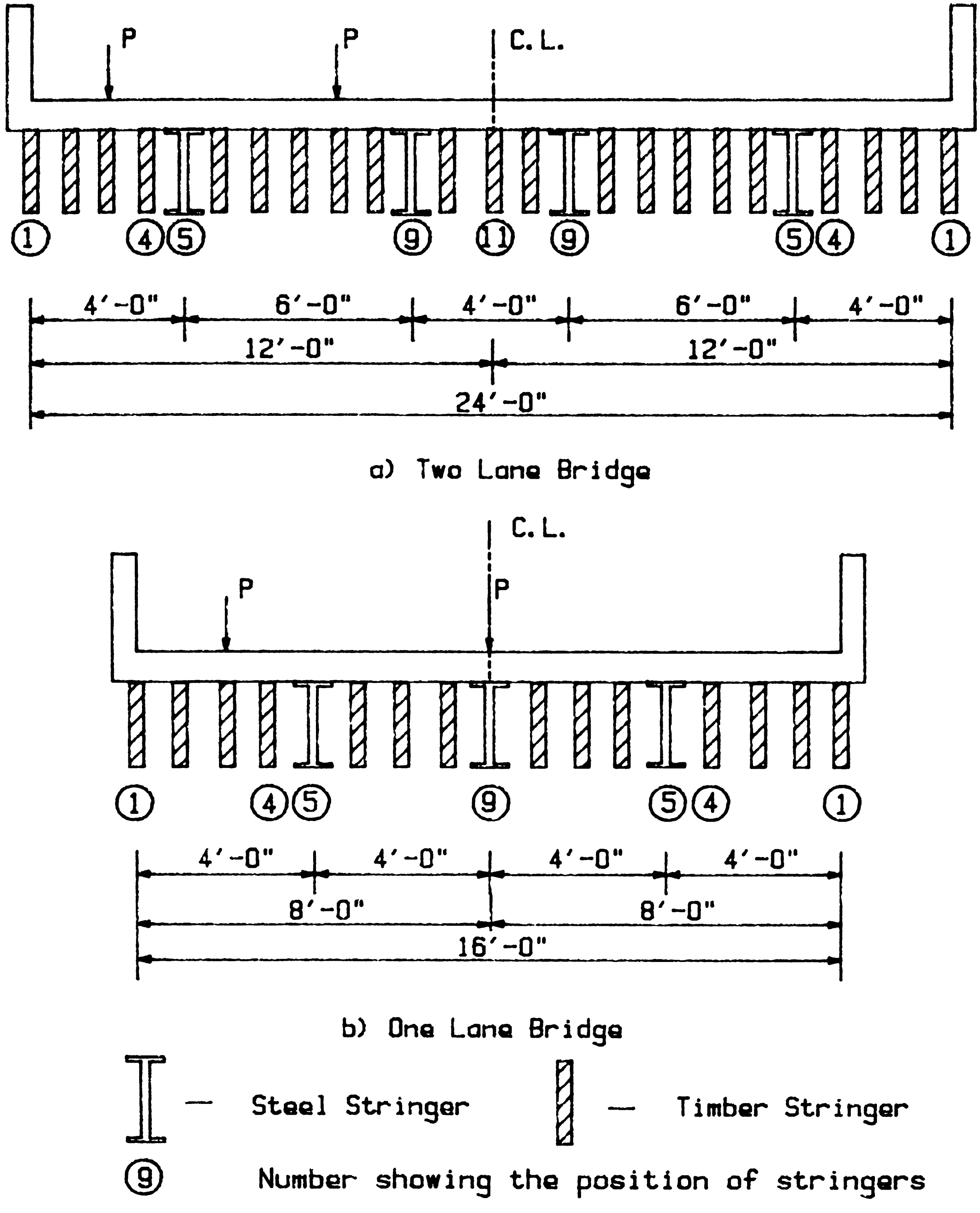

Figure 2.12: Position of steel stringers in double lane and single lane bridge

advisable to replace few timber stringers with steel stringers. If the timber stringers spacing is greater then 16 in., it is better to add few steel stringers.

The position and number of steel stringers were determined from several analysis, which are discussed in the following sections. In this investigation only the second case i.e., replacing few timber stringers with steel stringer was investigated.

Strengthening by replacing timber stringers with steel stringers

In this technique certain timber stringers were replaced with steel stringers. The method is advisable when timber stringer spacing is less i.e., between 8 in. to 16 in. Also adding steel stringers between timber stringers is not convenient when spacing is less and hence the replacing technique is more suitable.

Several trial runs were performed to determine the position, size and number of steel stringers. The depth of the steel stringer was assumed to be a constant, equal to the depth of timber stringers usually 12 in. or 16 in. A 18 ft. span bridge with 24 ft. (two lane) width was considered in the preliminary analysis. The stringer size used was 4 in. x 12 in. and spacing was assumed as 12 in. center to center. The bridge with above geometry was subjected to three different loading conditions (as in Figure 2.10). The loads were positioned as per AASHTO, i.e., one truck load at a distance of 2 ft. from the curb (load case 1). Also other load combinations like one truck load at the center (load case 2) and two truck loads in either lane (load case 3) were considered for analysis.

The bridge with timber stringers alone in the first case and then with timber and steel stringers at different locations and different sizes was subjected to three different loading conditions. Symmetry about the centerline in transverse direction is assumed to simplify the analysis. Several different load and steel stringer combinations were used to determine the position, size and number of steel stringers and are described in the following sections.

Steel stringers at 3 ft. from the curb and subjected to load case 1

In the first case, the bridge with timber stringers only was subjected to load case 1. Later two timber stringers were replaced with steel stringers at position 4 (see Figure 2.12) on either side of the centerline i.e., 3 ft. from the curb and again analyzed for the same load case.

Longitudinal bending stresses in timber and steel stringers along the transverse direction of the bridge before and after replacing two timber stringers with steel stringers at position 4 are plotted in Figure 2.13. The curve 'a' shows the longitudinal bending stresses in timber stringers without the steel stringers, where as the curves 'b' and 'c' are the plot of stresses in timber and steel stringers after replacing timber with steel stringers at position 4.

From Figure 2.13 it can be concluded that, the longitudinal stresses in timber stringers were reduced to a certain extent by the addition of steel stringers. At the critical section, the longitudinal timber stringer stresses were reduced by about 20 percent. Also it can be seen that there was not much reduction in timber stringer stresses with an increase in the size of steel stringer, except that there was a reduction of steel stringer stress itself. The longitudinal bending stress in timber stringers depends on the position of steel stringers.

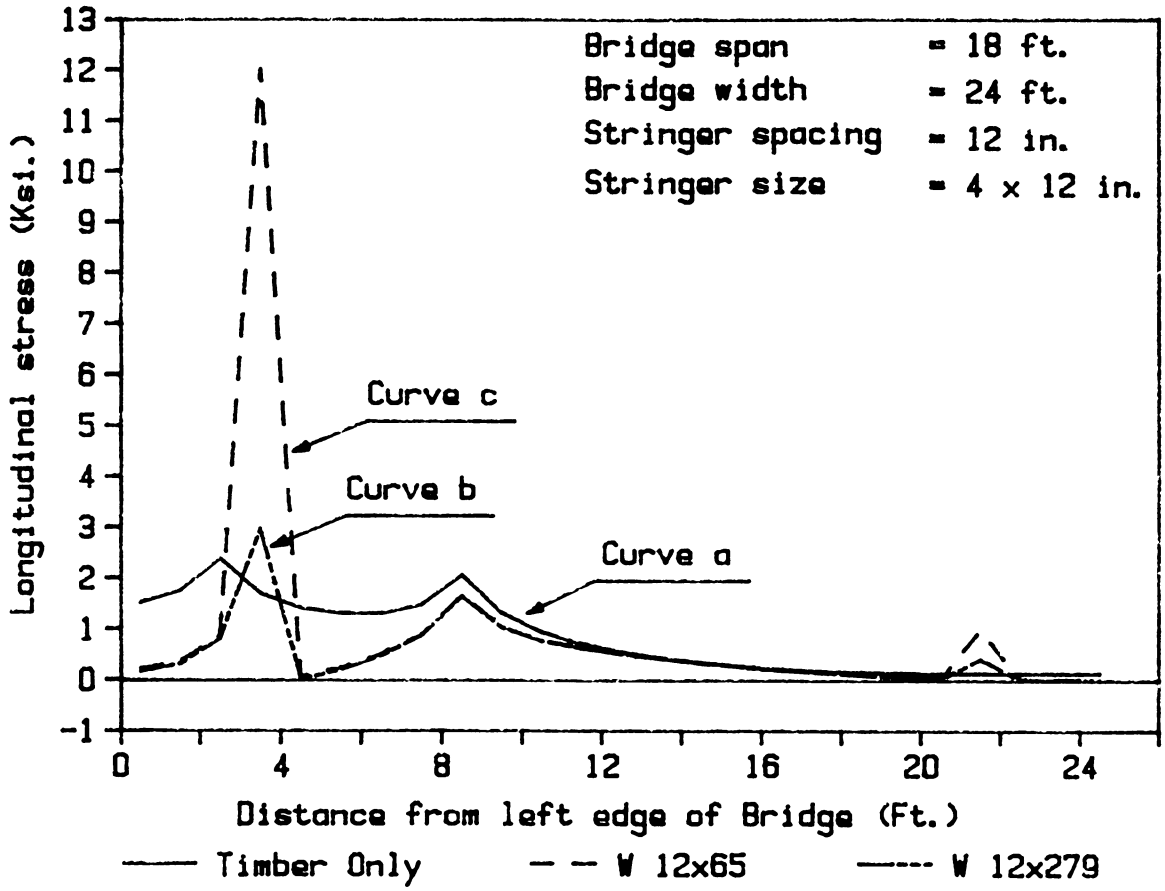

Figure 2.13: Maximum longitudinal stresses in timber and steel stringers - steel at position 4 and load case 1

Steel stringers at 4 ft. from the curb and subjected to load case 1

In this case, investigation is extended to determine further stress reduction in timber stringers. The steel stringers were changed from position 4 to position 5, (see Figure 2.12) one each on either side of the centerline, i.e., 4 ft. from the curb. This configuration was again subjected to load case 1.

Longitudinal bending stresses in timber and steel stringers along the transverse direction of the bridge before and after replacing two timber stringers with steel

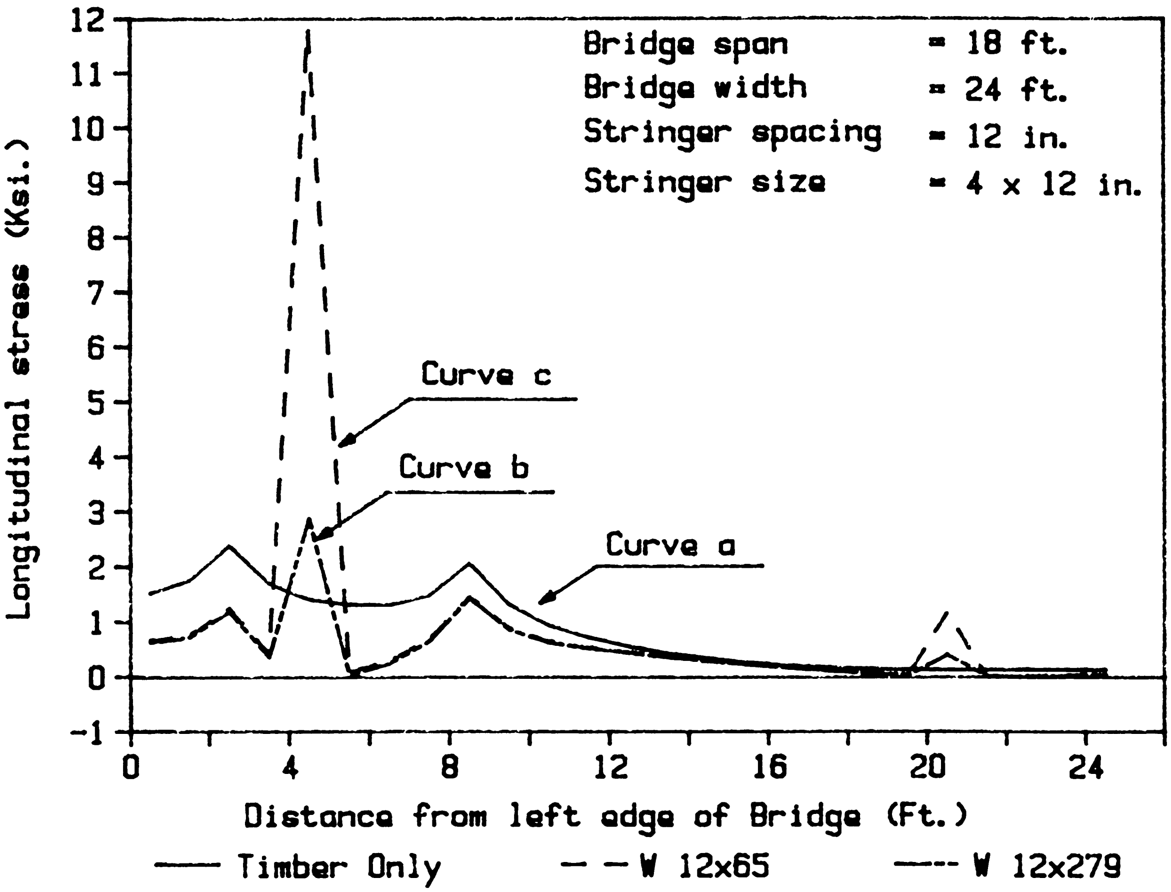

Figure 2.14: Maximum longitudinal stresses in timber and steel stringers - steel at position 5 and load case 1

stringers at position 5 are plotted in Figure 2.14. The curve 'a' is with timber stringers only and the curves 'b' and 'c' are for timber and steel stringers with steel stringers at position 5. Again the same type of behavior was observed i.e., similar to the previous case.

A plot of longitudinal bending stresses against the transverse width as in Figure 2.15 and Figure 2.16 with and without steel stringers, reveals that there was more stress reduction in timber stringers when the steel stringers were positioned at 4 ft.

from the curb rather than at 3 ft. from the curb. At the critical point the timber stress reduction for a W 12 x 65 was 20 percent when the stringers were at position 4 and the timber stress reduction when the stringers were at position 5 was 30 percent. The same figures for a W 12 x 279 were 21 percent and 31 percent. Hence it can be concluded that there is more stress reduction in timber when the steel stringers were at position 5 than at position 4. Also there was not much stress reduction in timber stringers when the size of steel stringer was increased from W 12 x 65 to W 12 x 279.

Also from Figure 2.15 and Figure 2.16, it can be found that the steel stringer stresses were reduced to a considerable extent where as there was not much change in timber stringer stresses, with the increase in size of steel stringers. The steel stringer stresses were reduced by 75 percent with an increase in steel stringer size from W 12 x 65 to W 12 x 279. But the timber stringer stresses were reduced from 20 percent to 21 percent only. Hence an increase in steel stringer size has more effect on steel stringer stresses than on timber stringer stresses.

In a similar way several trials were carried out by varying steel stringer size and position. Finally it was determined that two steel stringers at a distance 4 ft. from the curb gives the desired timber stress reduction for this load case. With this steel stringer position the timber bridge was analyzed for load case 2 and load case 3.

Steel stringers at 4 ft. from the curb and subjected to load case 2

In the previous two configurations, (steel stringer at 3 ft. from the curb on either side and steel stringer at 4 ft. from the curb on either side) a wheel load of a truck was placed at a distance of two feet from the curb (load case 1). But in reality, trucks can take different paths across the bridge. The loads may be offset from the above

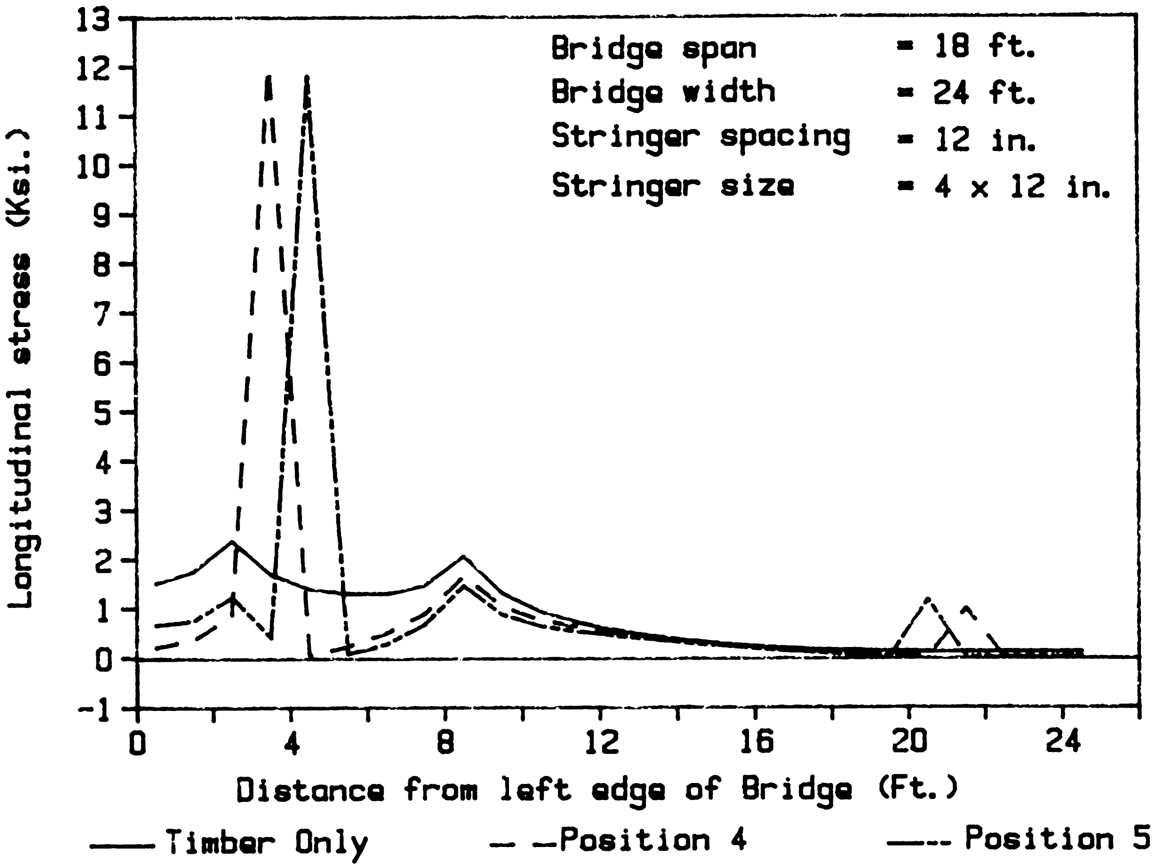

Figure 2.15: Maximum longitudinal stresses in timber and steel stringers - steel W 12x65 and load case 1

position either towards the curb or center line. Usually trucks tend to be close to the centerline and further away from the curb. The load condition was severe when the truck was exactly positioned in the center as in Figure 2.10 i.e., load case 2. This load configuration was applied to the bridge with timber stringers alone and then with timber and steel stringers at position 5.

Figure 2.17 shows the variation of longitudinal bending stress in the timber and steel stringers along the transverse direction at the centerline of the bridge in both

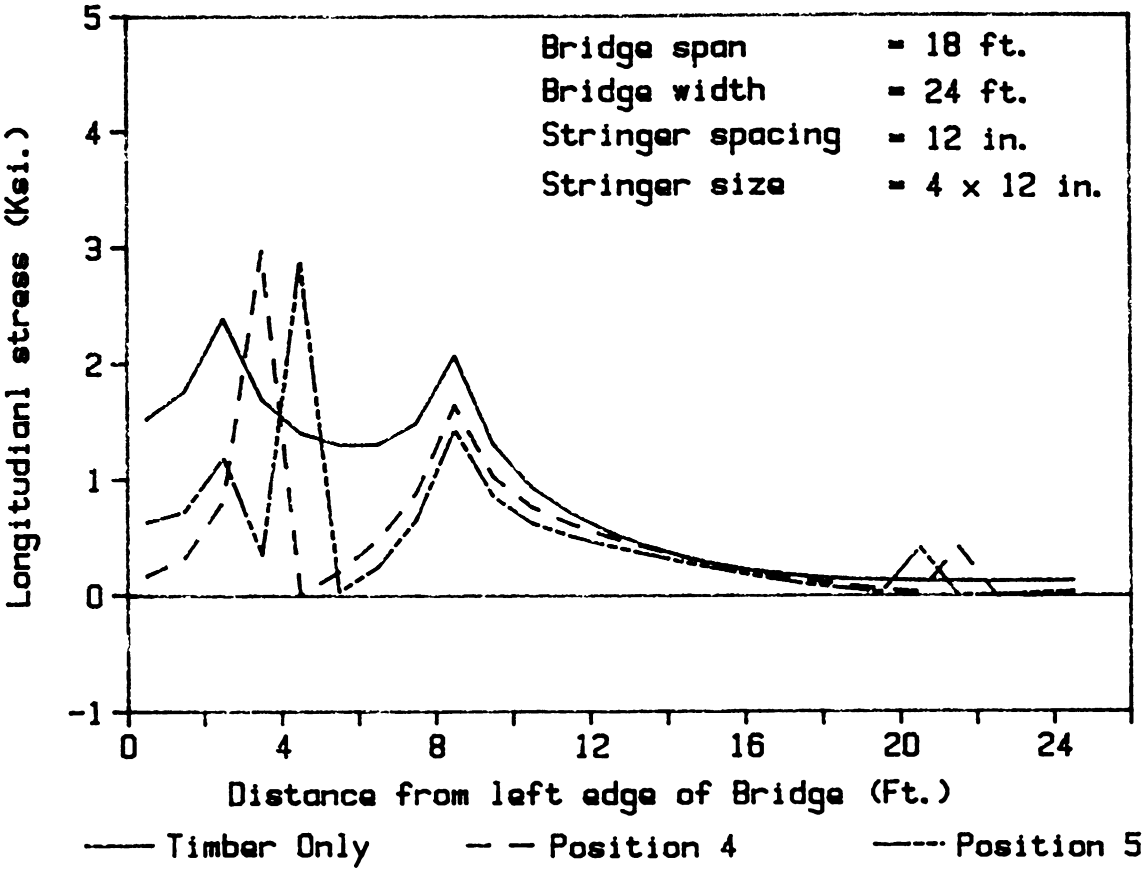

Figure 2.16: Maximum longitudinal stresses in timber and steel stringers - steel W 12x279 and load case 1

the cases. It was determined that in this case steel stringer position does not have much effect in reducing the timber stringer stresses.

By comparing Figure 2.16 and Figure 2.17, it can be found that the timber stringer stresses were reduced by almost 20 percent and the steel stringer stresses were reduced by 67 percent with a change in loading pattern from load case 1 to case 2.

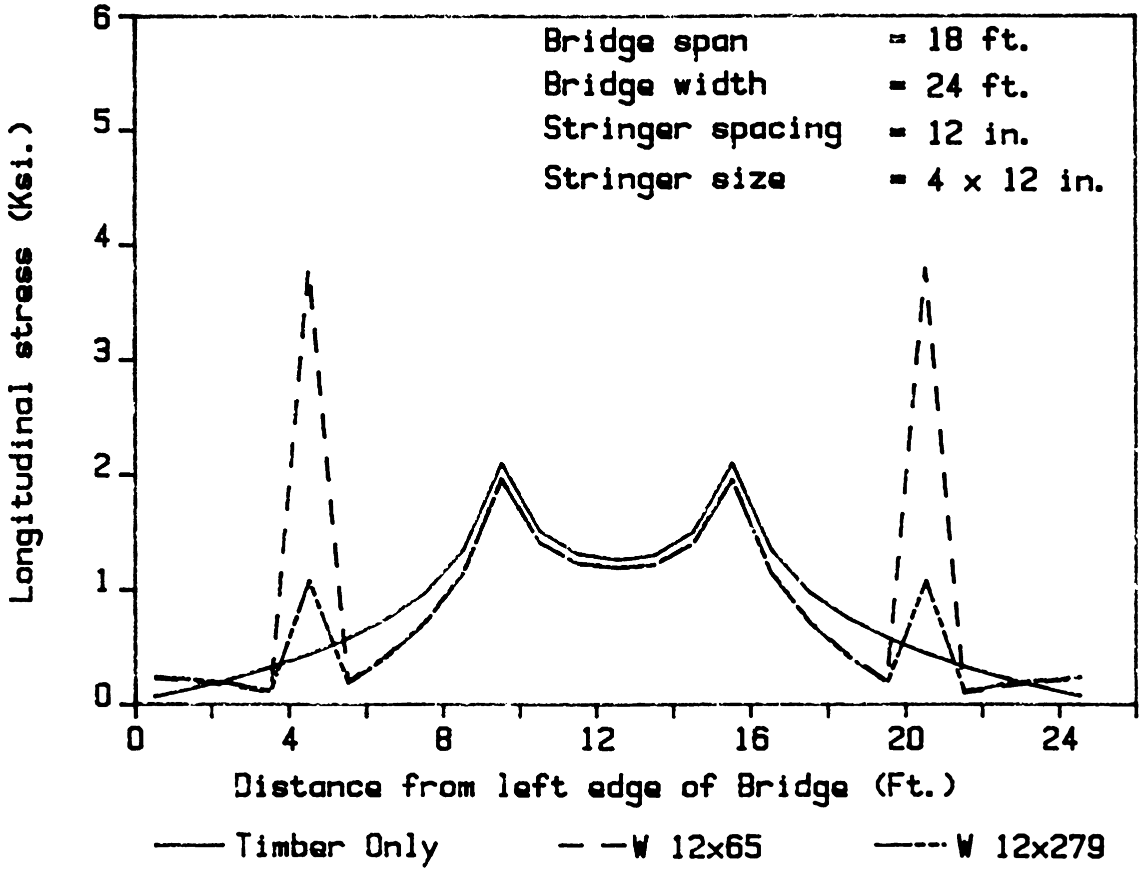

Figure 2.17: Maximum longitudinal stresses in timber and steel stringers - steel at position 5 and load case 2

Steel stringers at 4 ft. from the curb and subjected to load case 3

In this case, load case 3 was applied on the bridge with timber stringers alone and then replacing two timber stringers with steel stringers at position 5. This load case does not occur very frequently.

From Figure 2.18, it can be seen that the timber stringer stresses were reduced to a considerable extent (60 percent) at the edges, but to a very small extent (27 percent) at the critical point due to the addition of steel stringers. Also it can be

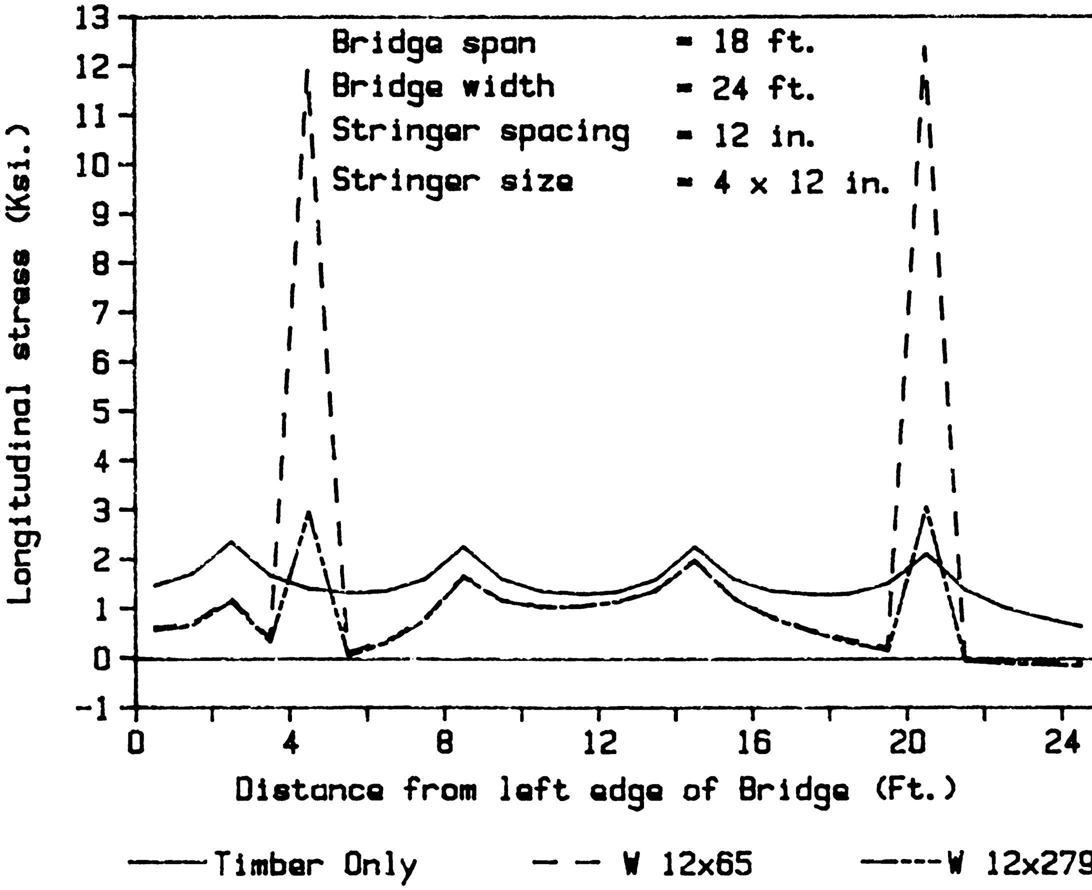

Figure 2.18: Maximum longitudinal stresses in timber and steel stringers - steel at position 5 and load case 3

found that the increase in steel stringer size has not much effect on the timber stringer stresses, but there was a considerable decrease in steel stringer stresses (75 percent). This was because there were no steel stringers under or near the wheel loads.

Steel stringers at 4 ft. from the curb and one at the center line and subjected to load case 3

The position 4 or position 5 of steel stringers were not very effective in reducing the timber stringer stresses due to load case 2 or load case 3. Hence investigation was extended by adding one more steel stringer at position 11 to reduce the timber stringer stresses when the timber bridge (the geometric parameters considered were the same as considered in the previous cases, i.e., 12 ft. span, 24 ft. width bridge with 4 in. x 12 in. stringers spaced at 12 in. center to center) was subjected to load case 2 or load case 3.

The stresses in timber stringers were not reduced at the center as there were no steel stringers near or under the wheel load. Hence in this case an additional timber stringer was replaced with a steel stringer exactly at the center i.e., position 11 (see Figure 2.12). The bridge was subjected to a very rare but heavy load configuration, i.e., load case 3, first with timber stringers only and then three timber stringers replaced with steel stringers at position 5 and position 11.

As earlier, stresses in timber and steel stringers along the transverse directions at the center line are plotted in Figure 2.19, Figure 2.20 and Figure 2.21. Figure 2.19 is with W 12 x 65 steel stringers at position 5 and position 11 and Figure 2.20 is with W 12 x 279 steel stringers again the steel stringers are at the same position, where as Figure 2.21 is a combination of the above two figures. From comparing Figure 2.18 and Figure 2.21 it was found that there was a better stress reduction in timber stringers due to the addition of steel stringer at position 11. At the critical point (about 8 ft. and 13 ft. from the edge) the timber stringer stresses were reduced by 45 percent in this case when compared to 27 percent in the previous case.

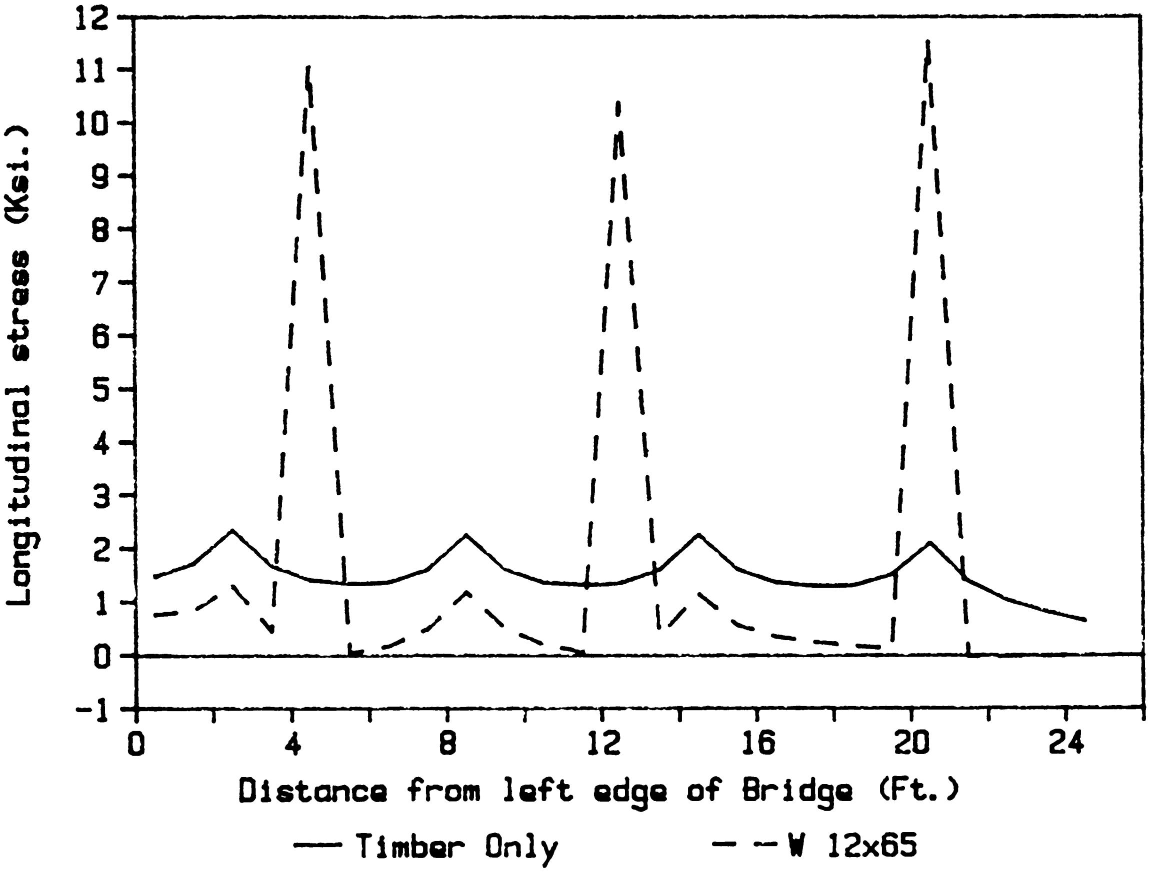

Figure 2.19: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 11 and load case 3

Steel stringers at 4 ft. and 10 ft. from the curb and subjected to load case 1

In an attempt to further reduce timber stringer stresses, four timber stringers were replaced (in the same bridge model) with steel stringers as shown in Figure 2.10 i.e., at 4 ft. and 10 ft. from the curb on either side of the centerline. In this case there will be at least one steel stringer at a distance less than 2 ft. from one of the wheel loads in any of the load configurations considered. Again this bridge model

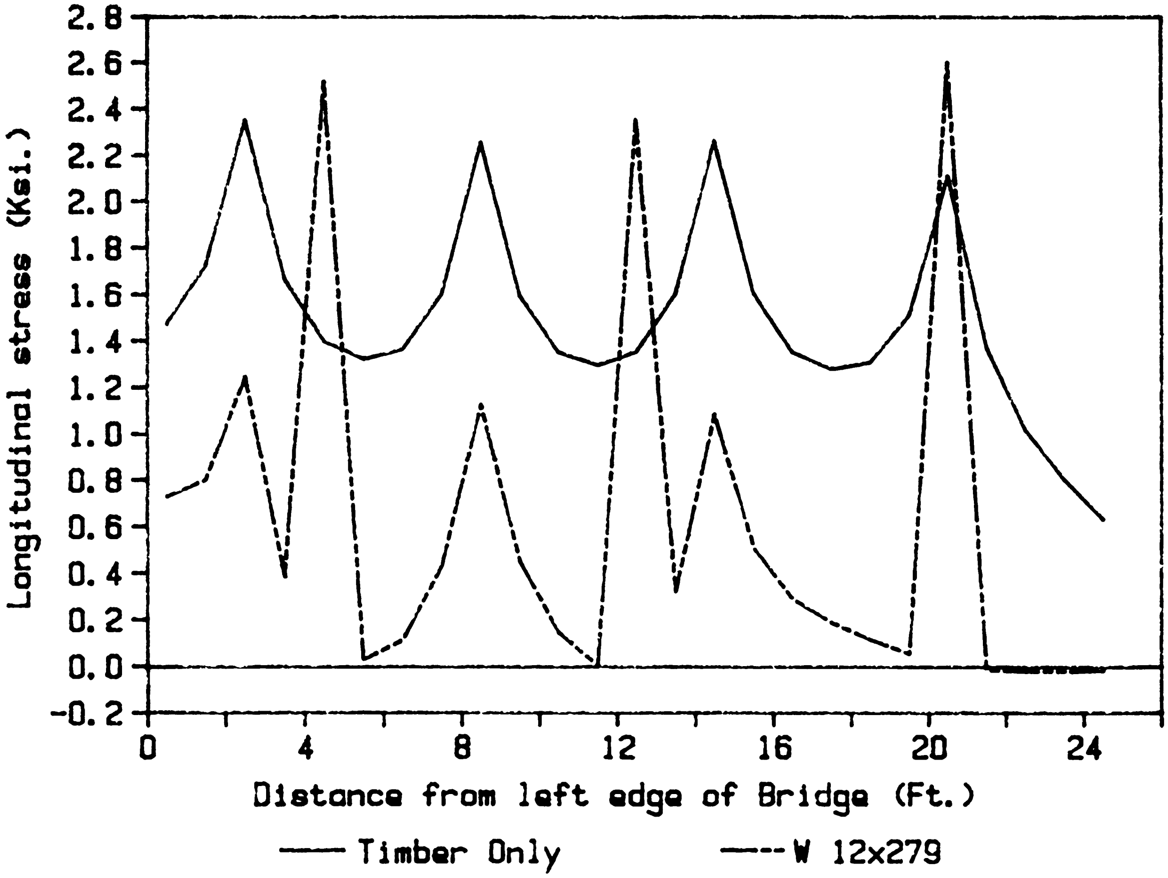

Figure 2.20: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 11 and load case 3

was subjected to all the three load cases.

Bending stresses in the timber and steel stringers are plotted in Figure 2.22, Figure 2.23 and Figure 2.24, along the transverse direction of the bridge. Figure 2.22 is with a steel stringer of size W 12 x 65, Figure 2.23 is with a steel stringer of size W 12 x 279 and Figure 2.24 is a combination of above two figures. The stress reduction was found to be very effective with this combination of steel stringers. By comparing Figure 2.21 and Figure 2.24, it was found that the timber stringer stresses

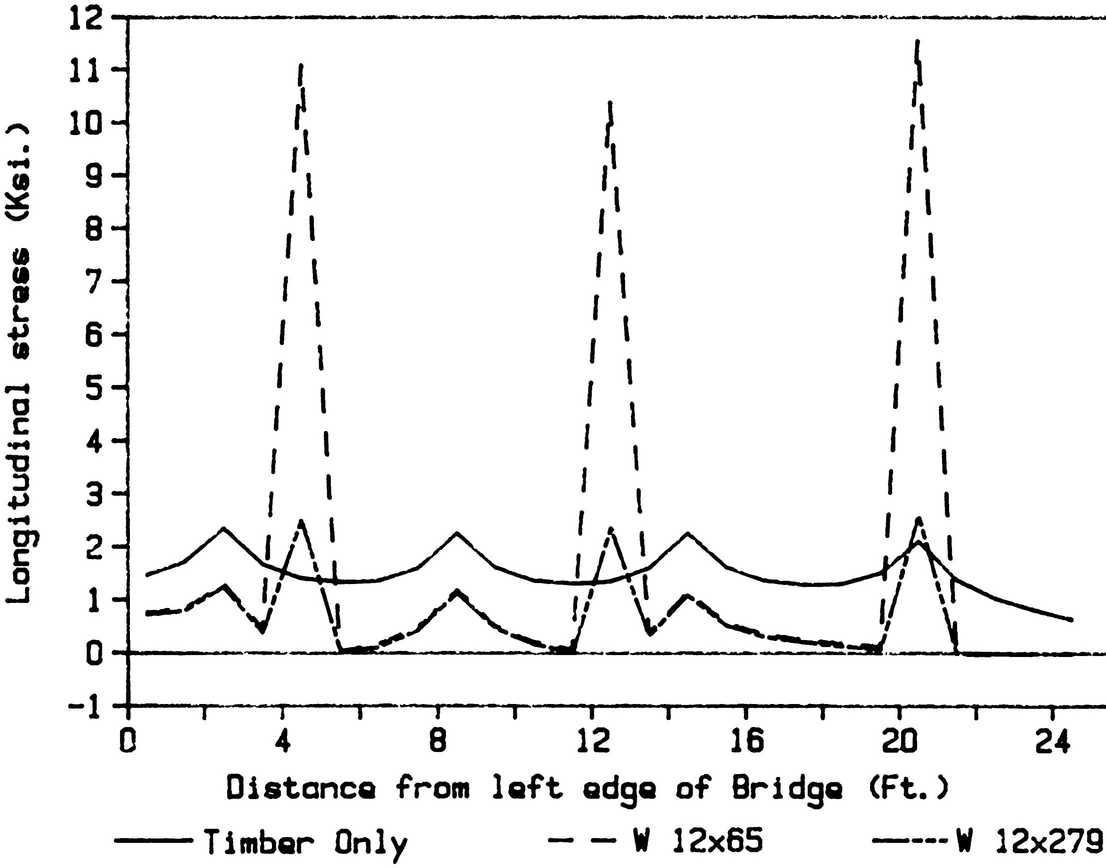

Figure 2.21: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 11 and load case 3

were reduced from 40 percent to 47 percent with the addition of a steel stringer at position 9 instead at position 11. But there was not much reduction in steel stringer stresses.

After several preliminary analysis (discussed in the above sections) with different combinations of steel stringer and load positions, it was determined that four steel stringers for a double/two lane bridge and three steel stringers for a single/one lane bridge increases the load carrying capacity of the bridge.

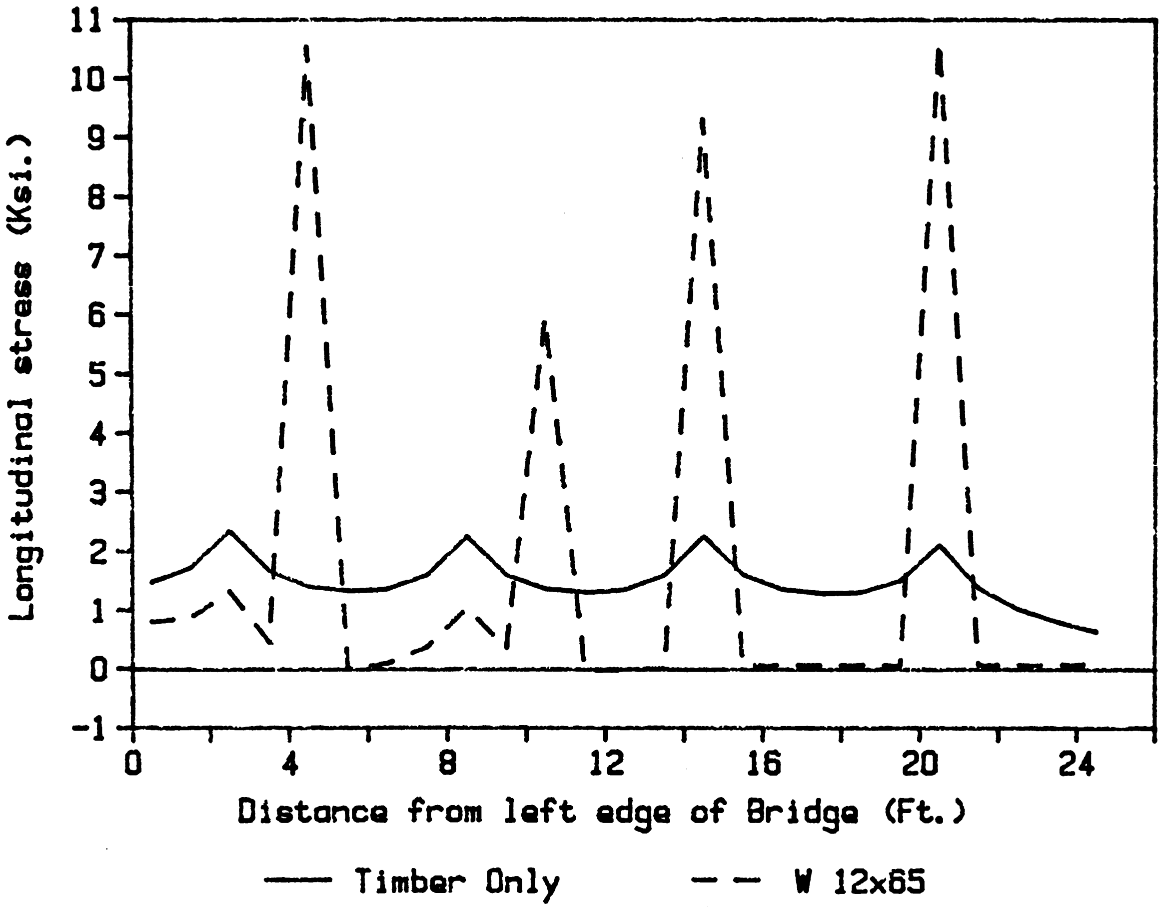

Figure 2.22: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 9 and load case 3

Figure 2.12 shows the position of steel stringers. Steel stringers were placed four ft. from the edge in either type of the bridges. Two steel stringers were placed two ft. away from the center in a two lane bridge and one steel stringer was placed at the center in a single lane bridge.

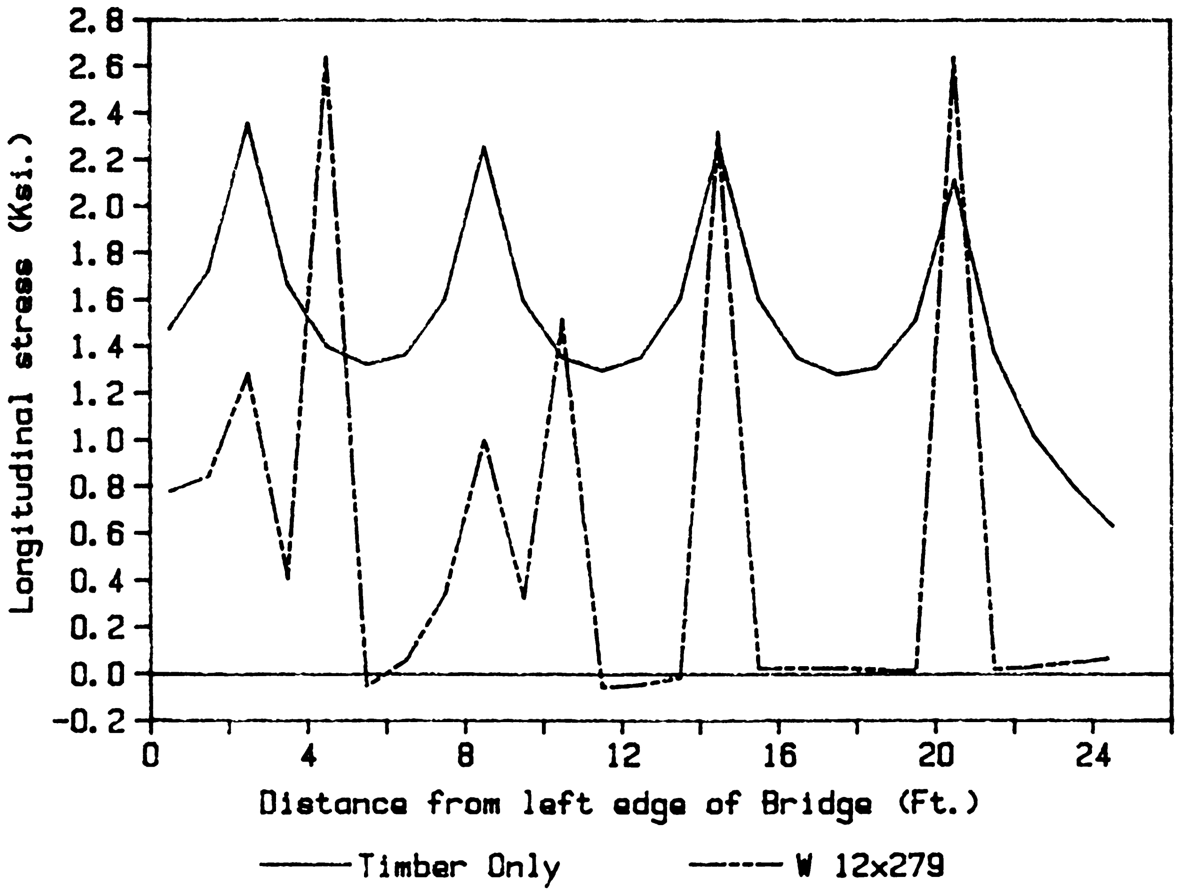

Figure 2.23: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 9 and load case 3

Determination of timber and steel stringer stresses for different span lengths, width, stringer spacings and stringer sizes

In this case the timber stringer size (4 in. x 12 in.), stringer spacing (12 in.) and width (24 ft.) were fixed. The span was varied from 12 ft. to 30 ft. (12 ft., 15 ft., 18 ft., 24 ft., and 30 ft.). At first the bridge with timber stringers only was subjected to load case 1 and then four timber stringers (position 5 and 11) were replaced with steel stringers of different size (depth same as that of timber stringers).

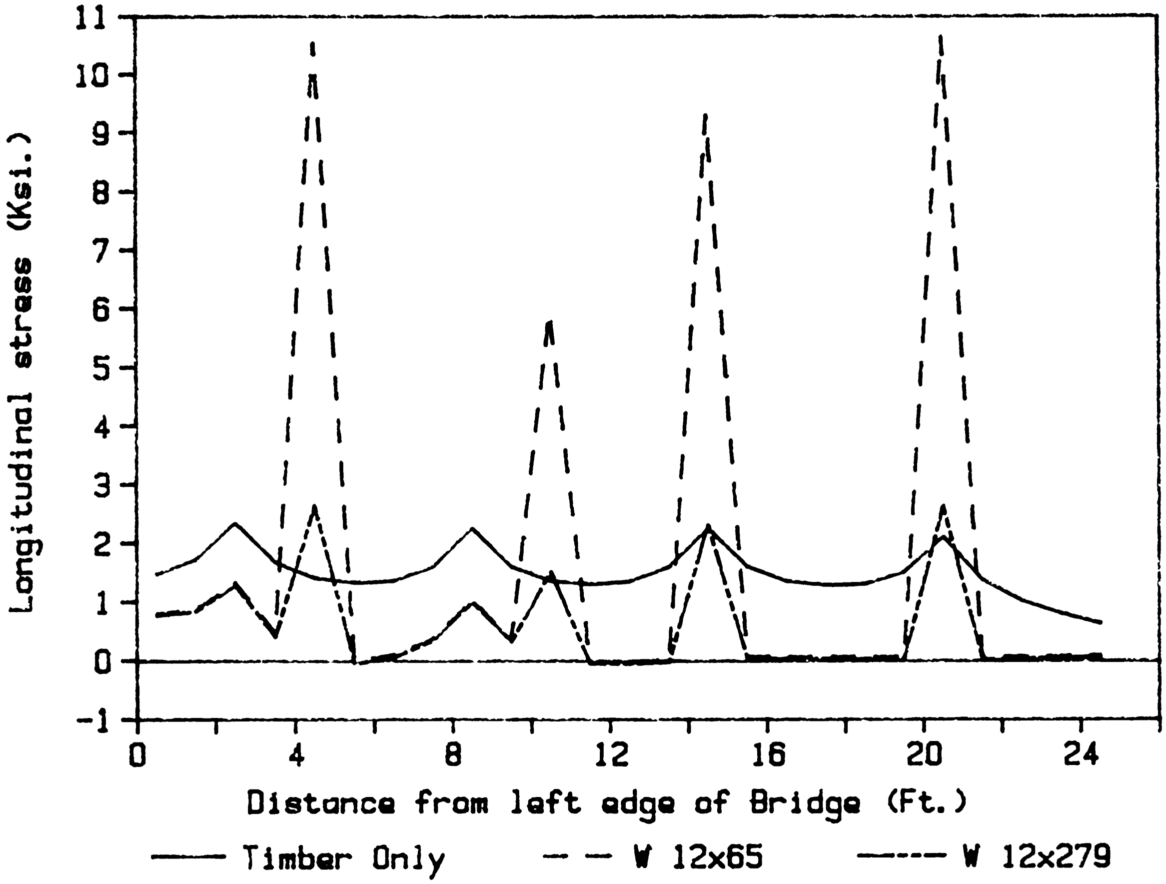

Figure 2.24: Maximum longitudinal stresses in timber and steel stringers - steel position 5 and position 9 and load case 3

Figure 2.25 shows the longitudinal critical stress in timber stringers. It can be seen that the stresses in timber stringers varies almost linearly with the span. The timber stringer stresses were reduced to a considerable extent by the addition of few steel stringers. Additional curves for other stringer sizes, i.e., W 12 x 120, W 12 x 210 and W 18 x 211 lie between the two bottom curves and are not shown here for clarity. The HS 20-44 load condition gives the maximum stresses for spans less than 24 ft., whereas Iowa legal truck loads governs the load condition for spans greater

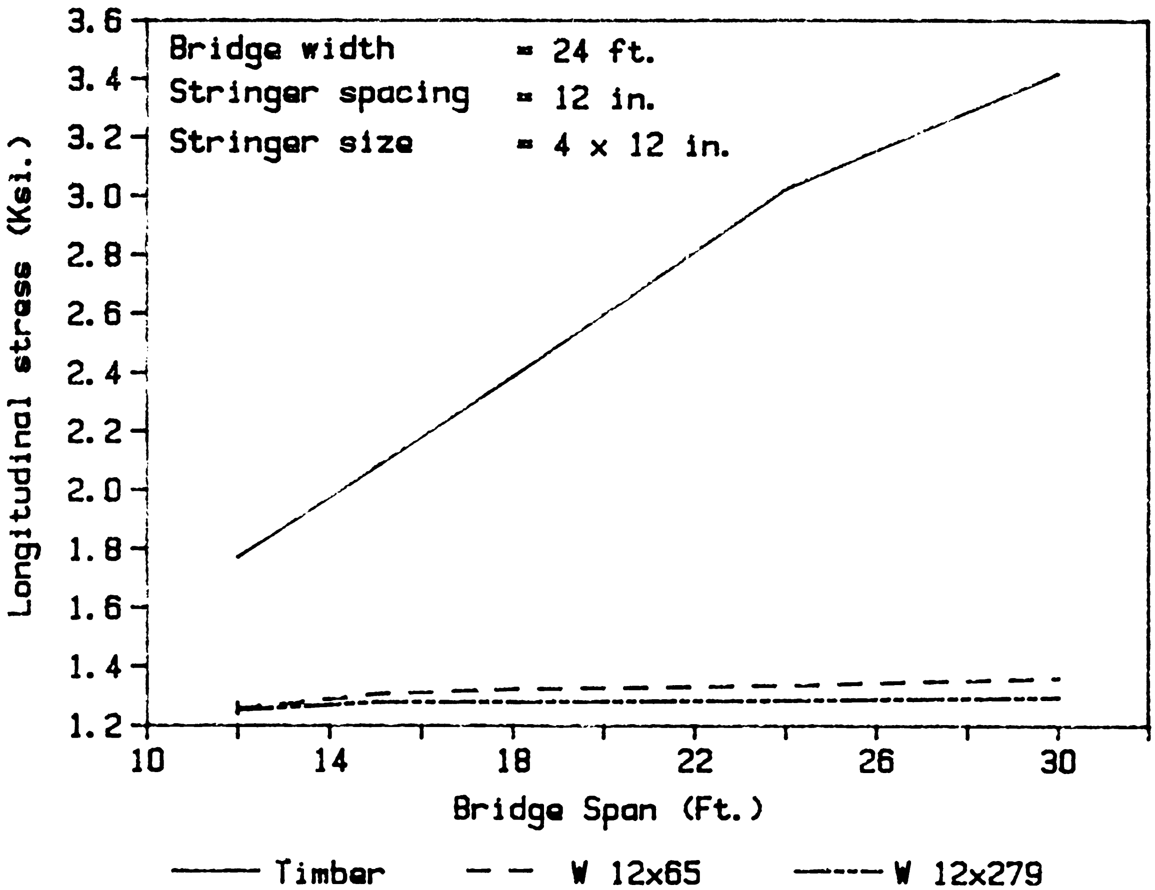

Figure 2.25: Maximum longitudinal stresses in timber stringers

than 24 ft. Because of this change in load condition there is a non-linear behavior in the curve.

It was found that there was more stress reduction in timber bridges of longer spans than that due to shorter spans.

Steel stringers of different moment of inertia were used to replace the timber stringers. The reduction of timber stresses were found to be very less with the increase in size (stiffness) of steel stringer. The stress reduction for a 18 ft. span bridge due to W 12 x 65 was 45 percent and that due to W 12 x 279 was 47 percent. Hence it was found that size of steel stringer does not have much effect on timber stringer stresses. But stress in timber stringers depends on the position of steel stringers.

Stresses in steel stringers

There was not much variation in the timber stringer stresses due to the increase in moment of inertia of steel stringers. But there was a decrease in steel stringer stresses as the moment of inertia was increased.

Figure 2.26 is the plot of stresses in steel stringers for bridges with various spans and different size steel stringers. From the Figure 2.26 it can be seen that the steel stringer stresses decreases with the increase in moment of inertia or size of steel stringer and increases with the increase in span length. For example for a bridge with 18 ft. span, the stress in a W 12 x 65 (I = 533 in^4) was about 10.5 Ksi. and the stress in a W 12 x 279 (I = 3110 in^4) was about 2.5 Ksi. a reduction of 8.0 Ksi.

Figure 2.26 can be used (illustrated in Appendix III) to determine the required moment of inertia or size of steel member so that the stresses in steel stringers were reduced to permissible values. For example W 12 x 65 cannot be used if the span is greater than approximately 22 ft. as the steel stress exceeds 18 ksi.

Figure 2.27 shows the decrease in steel stringer stresses with the increase in moment of inertia of steel stringer for a particular span length of a bridge.

Figure 2.28 shows the variation of steel stringer stresses with the increase in moment of inertia of steel stringer for a particular stringer spacing. Three different commonly used stringer spacings were used in the investigation, i.e.. 8 in., 12 in. and 16 in. center to center. For clarity only curves for 8 in. and 16 in. are shown and

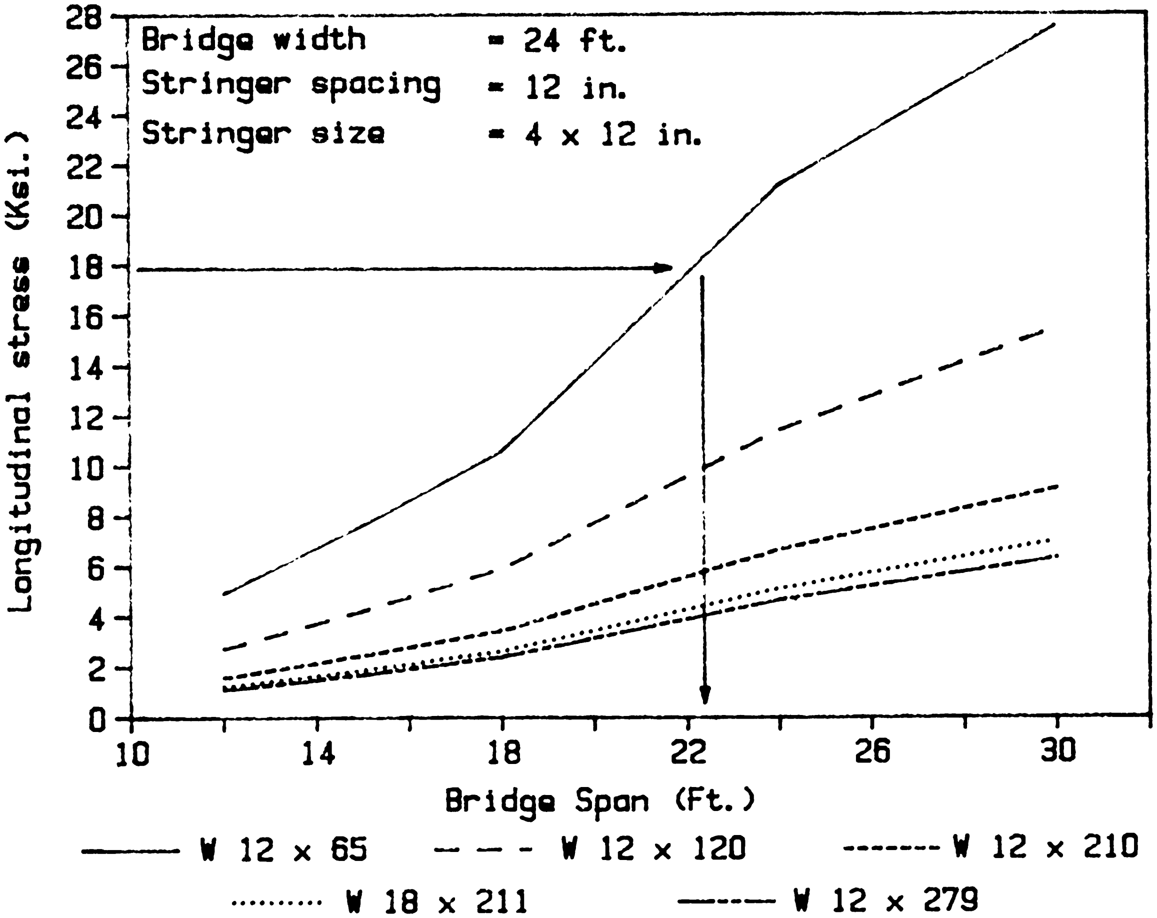

Figure 2.26: Maximum longitudinal stresses in steel stringers

the curve for 12 in. spacing lies between the two curves.

It was found from the investigation that the stresses in steel stringer decreases with the increase in moment of inertia of steel stringer as in the earlier case. But the stringer spacing had a very little effect on the steel stringer stresses.

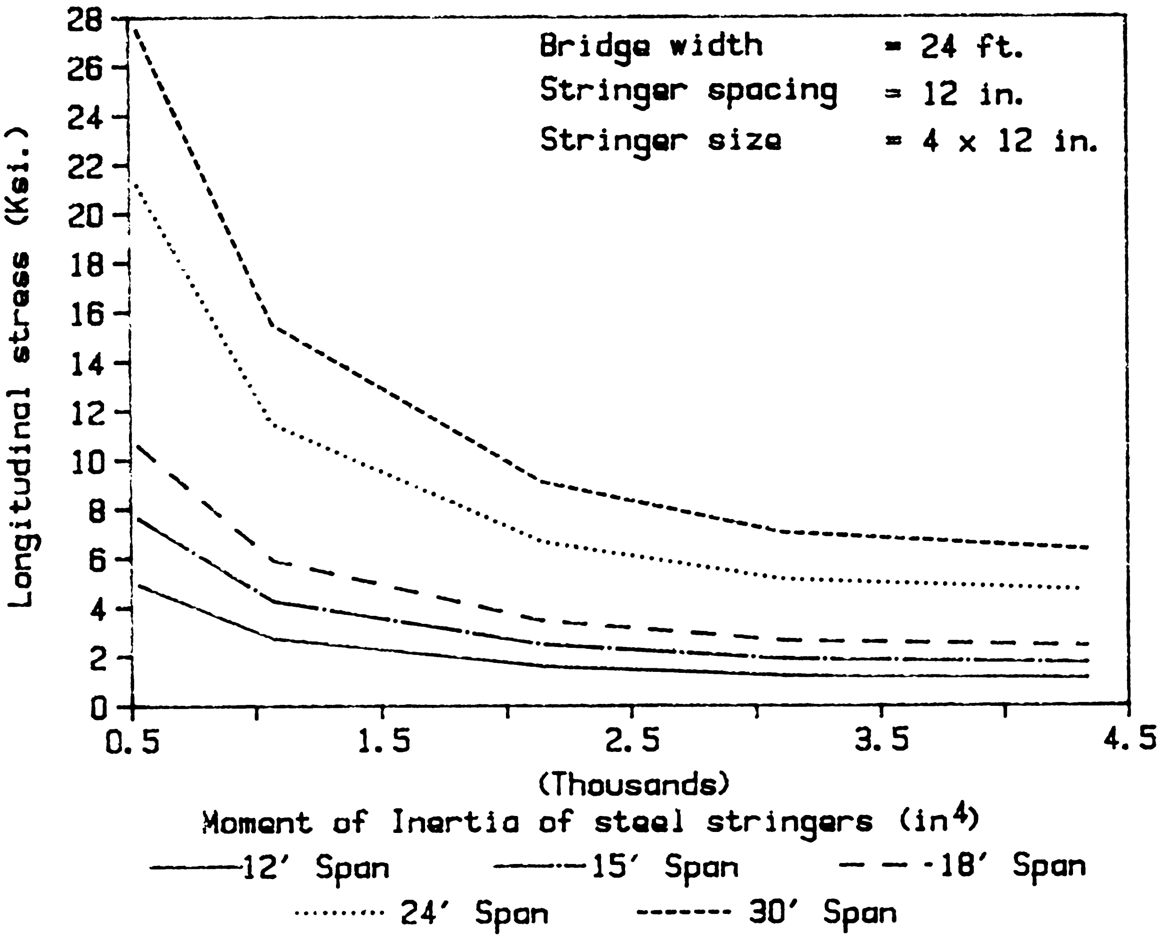

Figure 2.27: Maximum longitudinal stresses in steel stringers

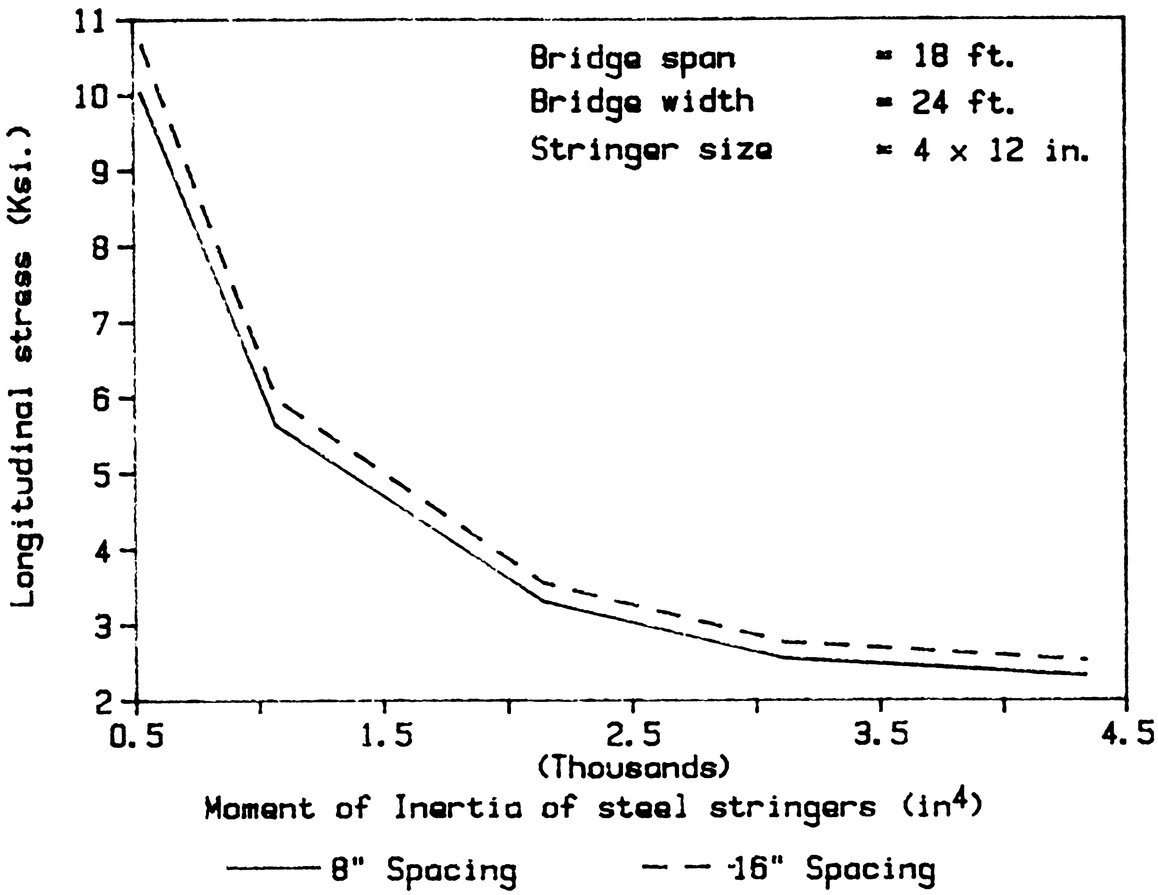

Figure 2.28: Maximum longitudinal stresses in steel stringers

Figure 2.29 shows the variation of steel stringer stresses with the increase in stringer spacing for a particular moment of inertia. Steel sections of different moment of inertia were used to replace the timber stringers.

As in the earlier cases it was found that there was not much increase in steel stringer stresses with the increase in stringer spacing, but there was found to be a decrease in stress in steel stringer with the increase in moment of inertia. There was a little decrease in steel stringer stress with the increase in stringer spacing from 12 in. to 16 in., because of the loading condition. In bridge model with 8 in. and 12 in. stringer spacing the load was placed exactly over a timber stringer, but in case of a bridge with 16 in. stringer spacing the load was positioned exactly midway between two timber stringers. Figure 2.30 shows the position of loads and stringers with different stringer spacing.

Stresses in timber stringers

The primary aim of the work was to achieve a reduction of stress in timber stringers. This was achieved by replacing few steel stringers in place of timber stringers.

Figure 2.31 shows the variation of stress in timber stringers with an addition of steel stringer for a particular spacing. The stress in timber stringer drops all of a sudden due to the addition of steel stringers. But there was not much reduction with the increase in moment of inertia of steel stringer. It can be seen that the timber stringer stresses were more for a bridge with larger stringer spacing.

Figure 2.32 is a plot of timber stringer stresses against stringer spacing for different steel sections. It can be seen that timber stringer stresses were almost constant

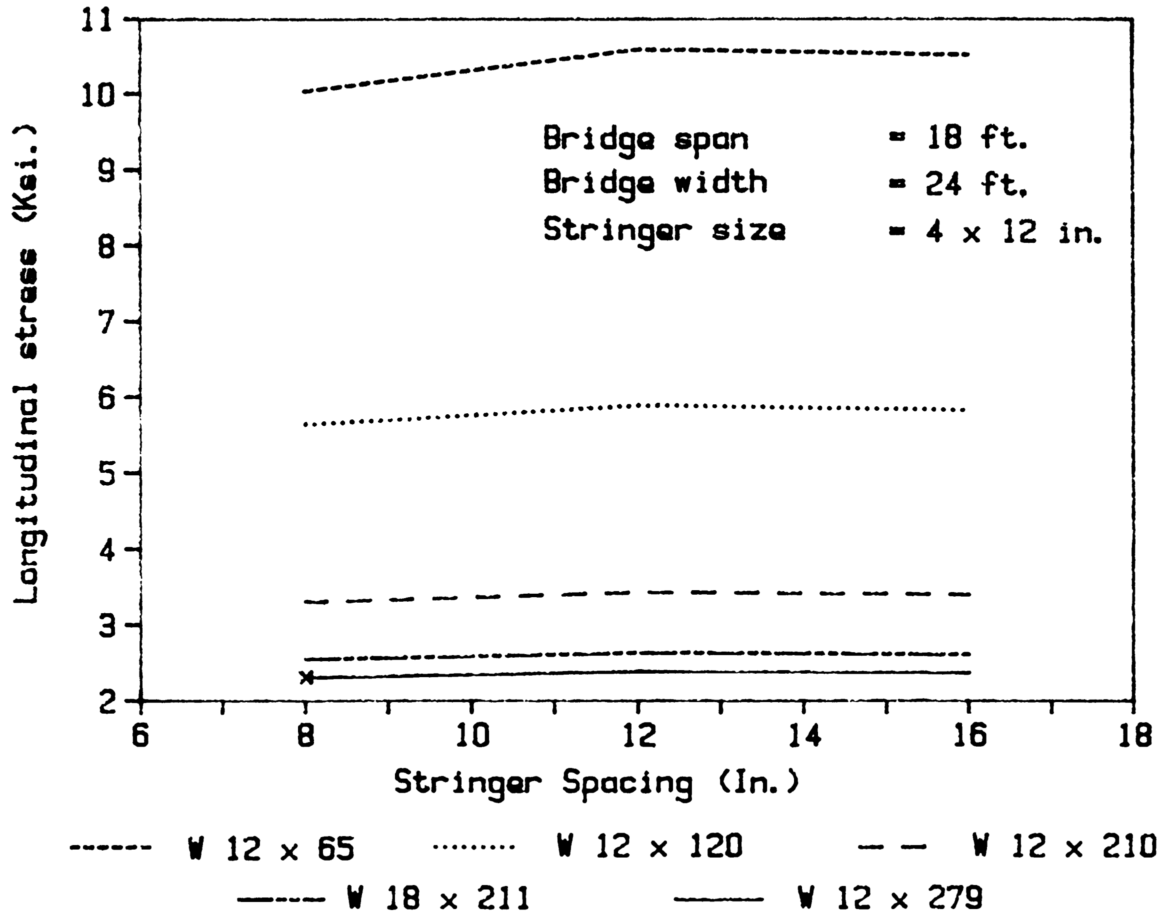

Figure 2.29: Maximum longitudinal stresses in steel stringers

with the increase in moment of inertia of steel stringer, but the increase in timber stringer stress was found to be very little with the increase in timber stringer spacing. Again there was a drop in stress in timber stringers with 16 in. stringer spacing, because of the position of load and stringer condition (as discussed in previous article).

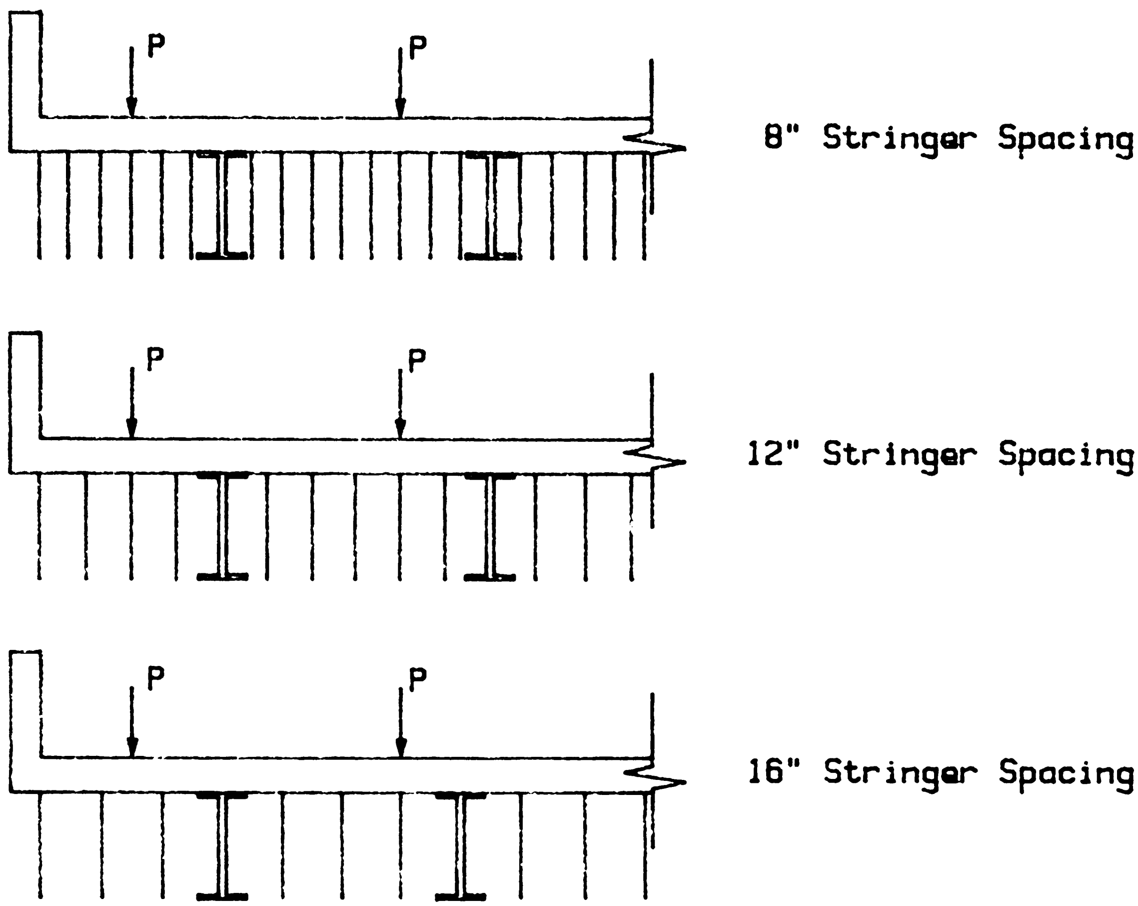

Figure 2.30: Position of loads and stringers with different stringer spacing

Timber and steel stringer displacements

Figure 2.33 is a plot of timber and steel stringer displacements. The span of the bridge was 18 ft. and the width of the bridge was 24 ft. The stringers 4 in. x 12 in. were spaced at 12 in. center to center. Four timber stringers were replaced with steel stringers of different sizes at position 5 and position 9. The bridge was subjected to HS20-44 loads.

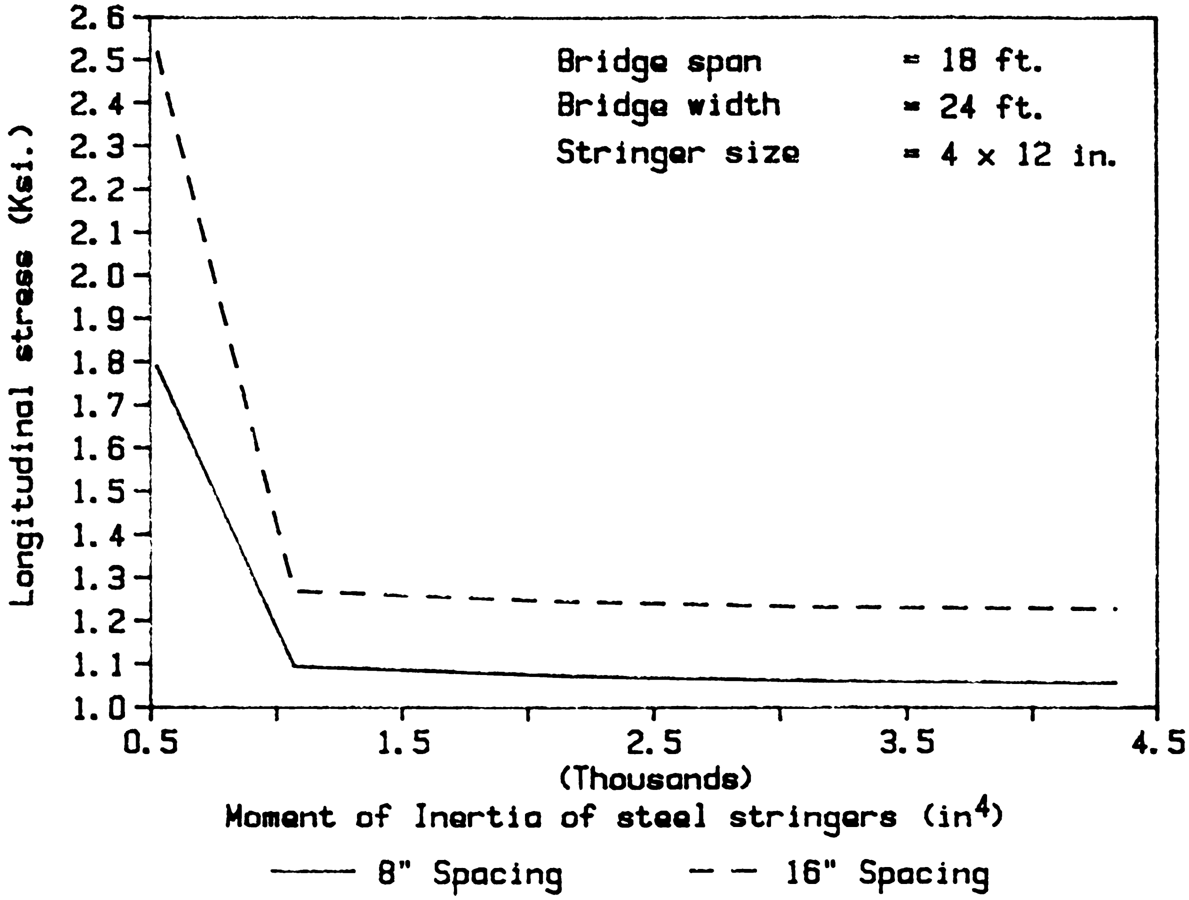

Figure 2.31: Maximum longitudinal stresses in timber stringers

The bottom curve is a plot of timber stringer displacements. The two upper curves are the plots of timber and steel stringer displacements. The displacements were drastically reduced in timber stringers by the addition of four steel stringers. At the left edge where the displacement was maximum, the displacement was reduced by about 50 percent. At intermediate locations there was a 70-80 percent reduction. As a whole the timber stringer displacements were reduced by the addition of steel stringers. Also it can be seen that there was not much reduction in timber stringer

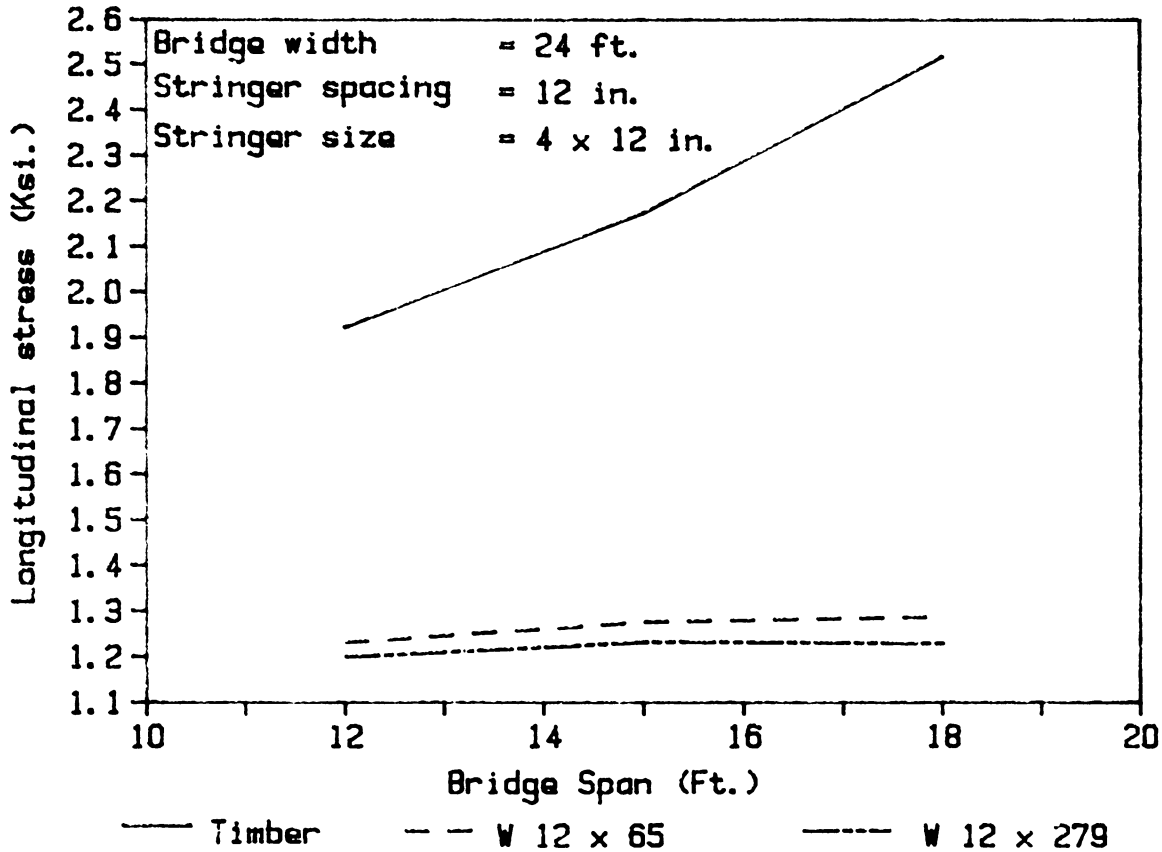

Figure 2.32: Maximum longitudinal stresses in timber stringers

displacements with an increase in steel stringer size. By increasing the steel stringer size from W 12 x 210 to W 12 x 279, the timber stringer displacement reduction was only 5 percent.

Abutment reactions

Figure 2.34 is a plot of the reactions at the abutment. The bridge with standard dimensions as in the previous section was considered for analysis.

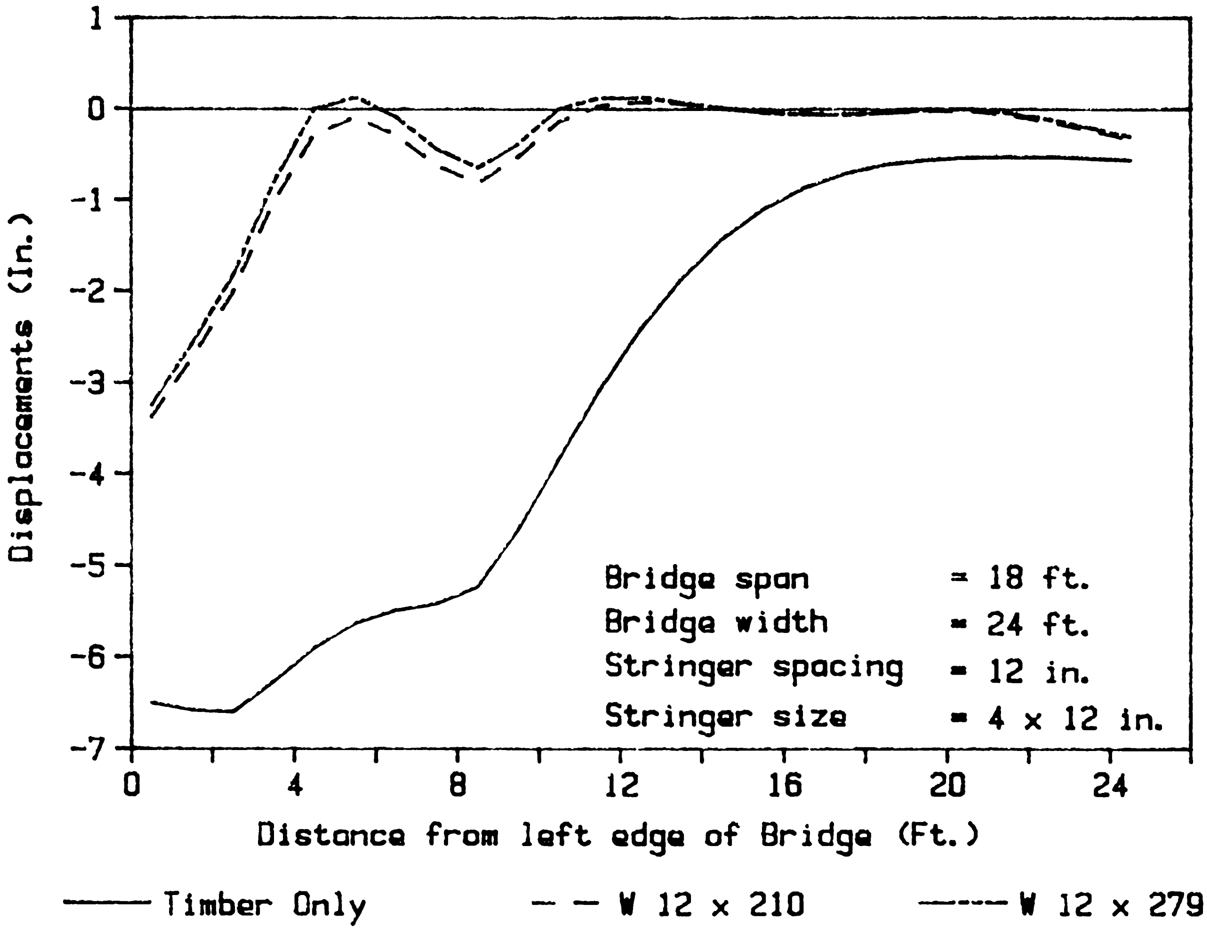

Figure 2.33: Displacements in timber and steel stringers

The Figure shows plot of the reactions without and with timber stringers. With the addition of steel stringers the loads carried were reduced in the timber stringers and at the same time the loads carried were very high in the steel stringers. The load carried by the steel stringer was directly proportional to the size (stiffness) of the stringer. The heavier (stiffer) the stringer, more was the load carried by the stringer and hence the higher reaction at the abutment. By the addition of four W 12 x 65 steel stringer at position 5 and position 9, the reaction was reduced by about 50

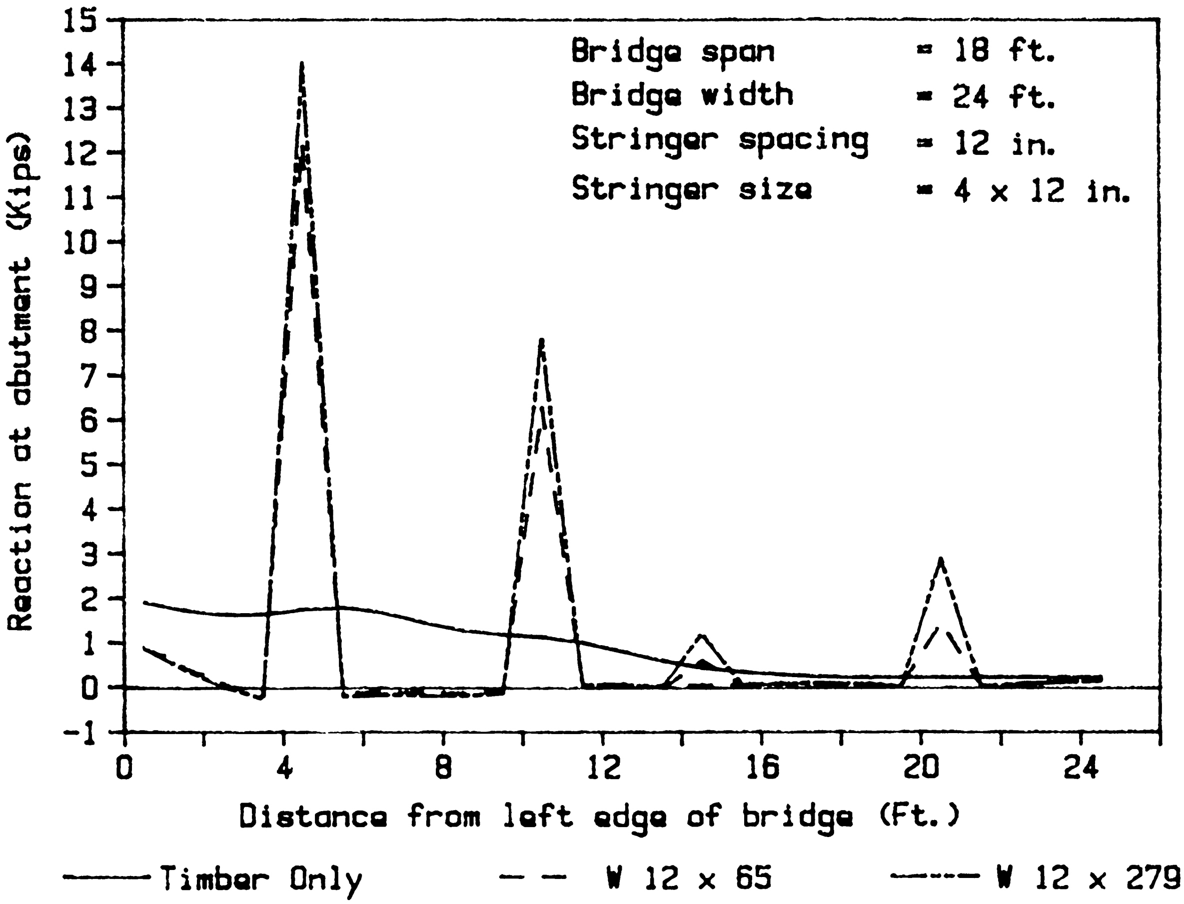

Figure 2.34: Maximum reactions at the abutment

percent. But there was not much reduction by increasing the steel stringer size from W 12 x 65 to W 12 x 279.

As the abutments are designed for small uniform continuous loading, the abutments should be redesigned or reinforced to take care of the heavy loads due to the addition of steel stringers. The Figure shows the abutment reactions for load case 1. The load carried by steel stringers varies with the position of load. Hence to accomodate all possible load combinations, the abutment should be designed for the maximum value of reaction (i.e., about 14 kips point load in this case).

Stringer stresses for different stringer sizes and different stringer spacings

The following figures are the plot of stresses in timber and steel stringers for different stringer size and stringer spacings. These curves can be used to determine the size of steel stringers so that the timber stringer stresses are reduced to permissible values.

Figure 2.35 is for bridges with span varying from 12 ft. to 30. depending on the stringer spacing. For spans varying from 12 ft. to 18 ft. the stringer spacing varies from 12 in. to 16 in. and for spans varying from 18 ft. to 30 ft. the stringer spacing varies from 8 in. to 12 in. The upper three curves are the plot of stresses in timber stringers without steel stringers and the bottom three curves are the plot of stresses in corresponding timber and steel stringers due to the addition of four steel stringers of size W 12 x 65 at position 5 and position 9.

Spans 18 ft. to 30 ft. were not considered for analysis with 16 in. stringer spacing as the timber stringer stresses were very high and spans 12 ft. to 18 ft. were not considered for analysis with 8 in. stringer spacing as the timber stringer stresses were very low even without the steel stringers.

Figure 2.36 is the plot of steel stringer stresses for spans between 12 ft. and 18 ft. The stringer spacing was 16 in. and the size was 4 in. x 12 in. The figure shows the variation of steel stringer stresses for different bridge spans.

Figure 2.37 is a repeat of the above plot, except that the bridge span varies from 18 ft. to 30 ft. The stringer spacing was 8 in. and the stringer size was 4 in. x 12 in.

Figure 2.38 is a plot of timber stringer stresses for different stringer sizes. The bridge span varies from 12 ft. to 30 ft. and the stringer spacing was a constant 12 in. The two upper curves are stresses in timber stringers without steel stringers for 4 in. x 12 in. and 6 in. x 12 in. stringers and the two bottom curves are due to the addition of steel stringers (W 12 x 65). It can be found that the timber stringer stresses reduces with an increase in timber stringer size.

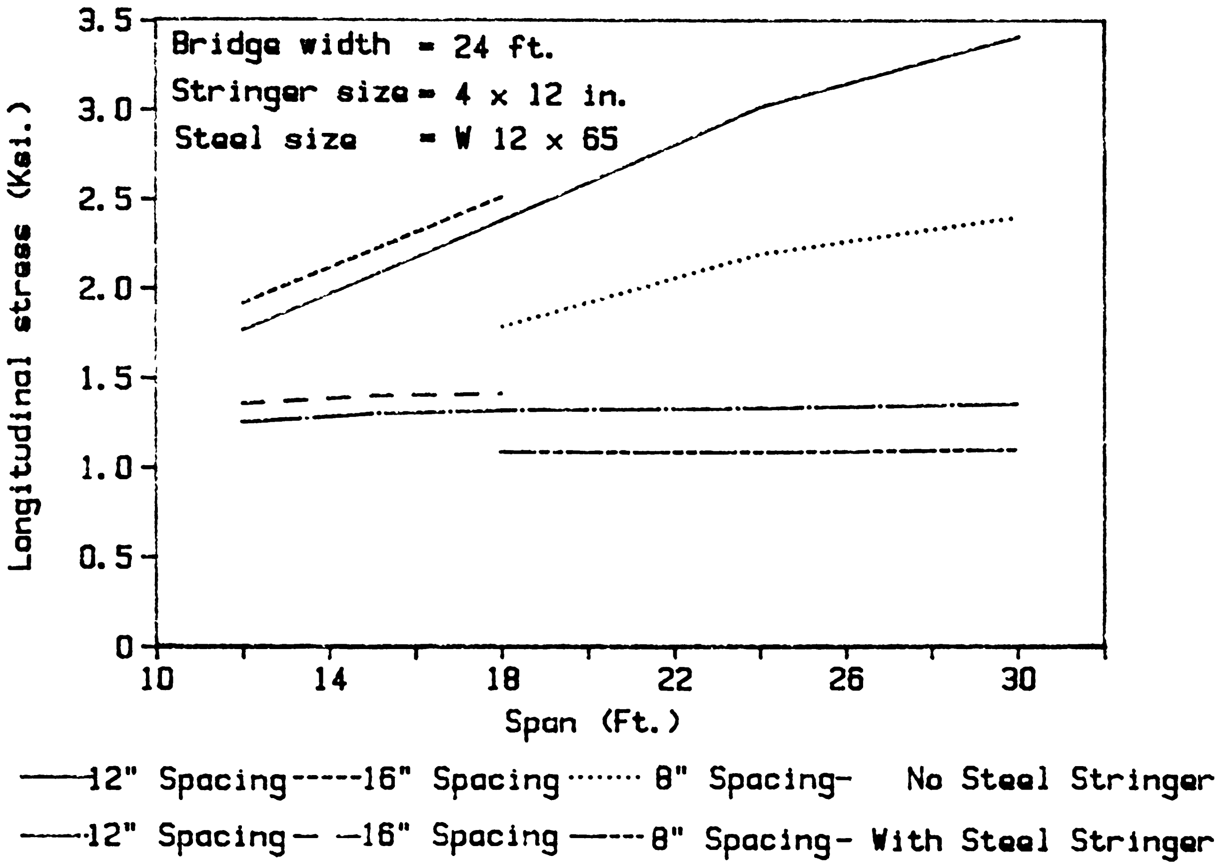

Figure 2.35: Maximum longitudinal stresses in timber stringers for different stringer spacing

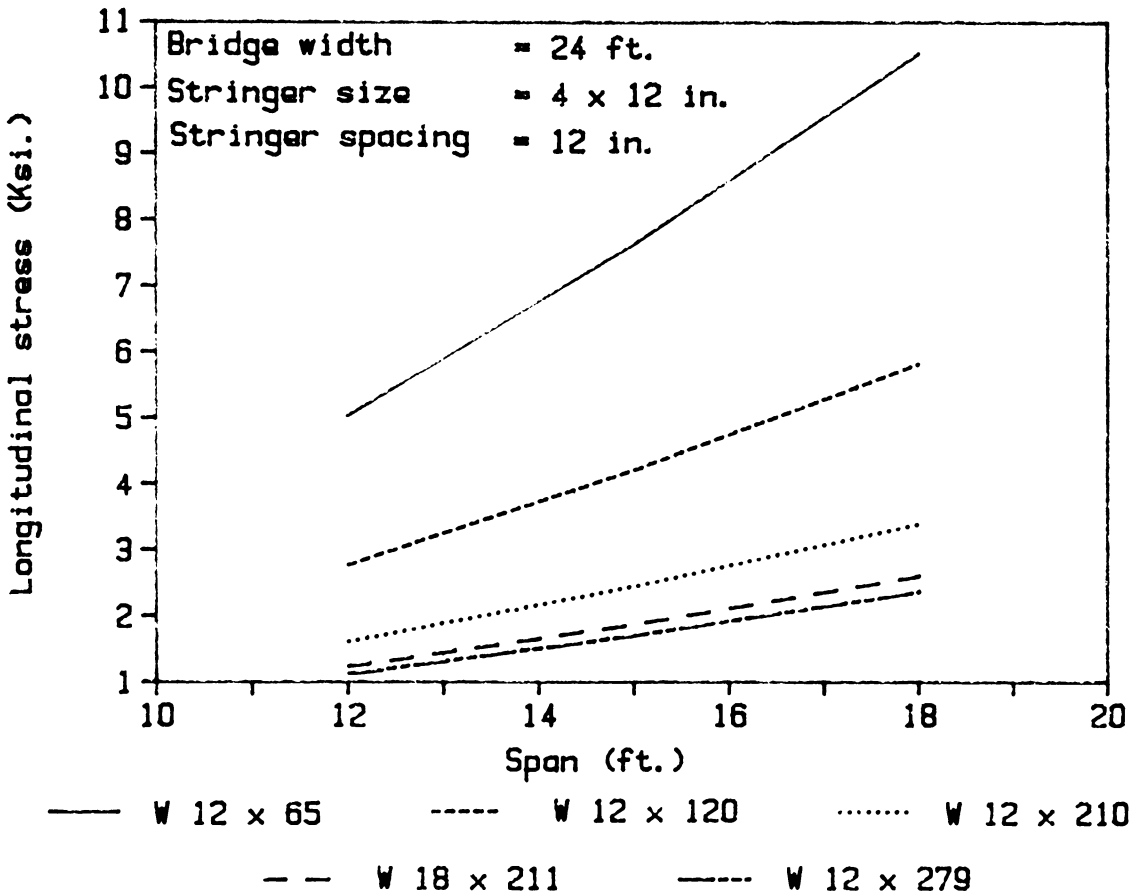

Figure 2.36: Maximum longitudinal stresses in steel stringers for 16 in. stringer spacing

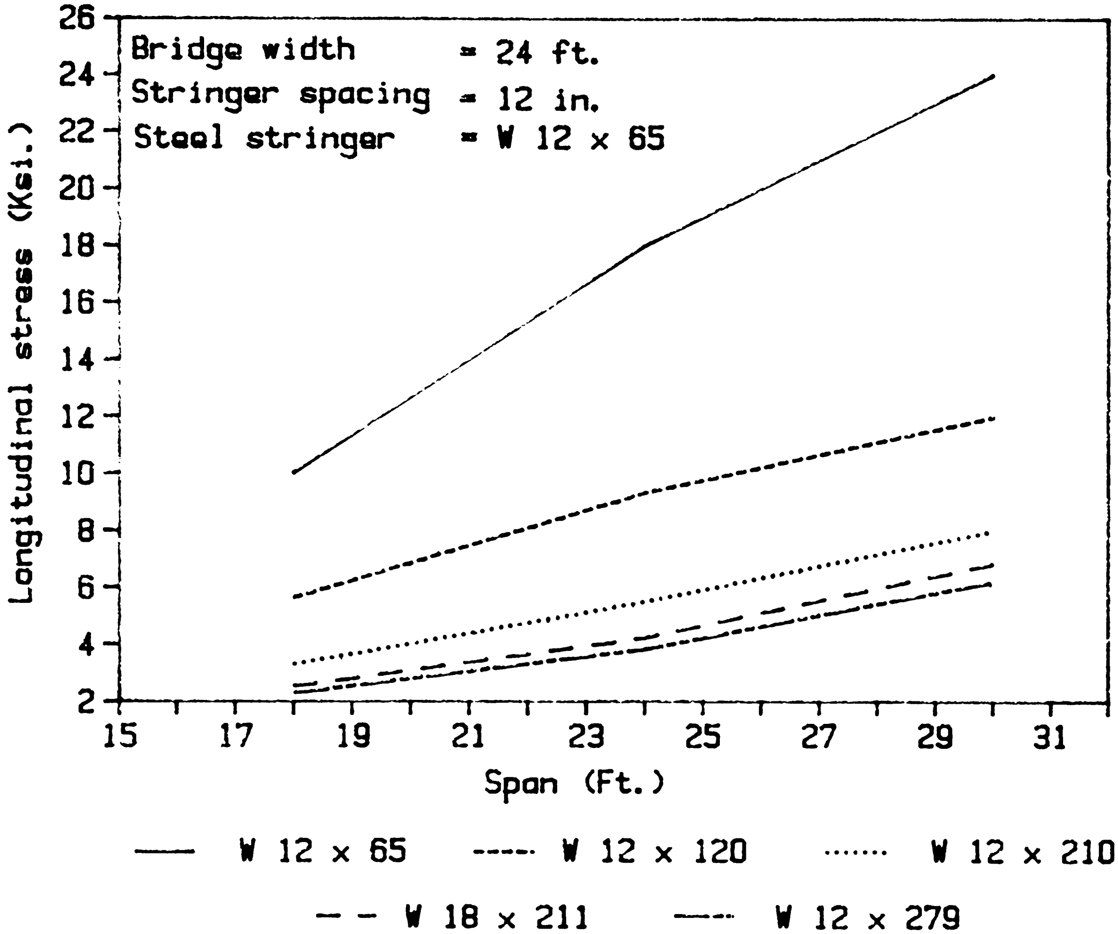

Figure 2.37: Maximum longitudinal stresses in steel stringers for 8 in. stringer spacing

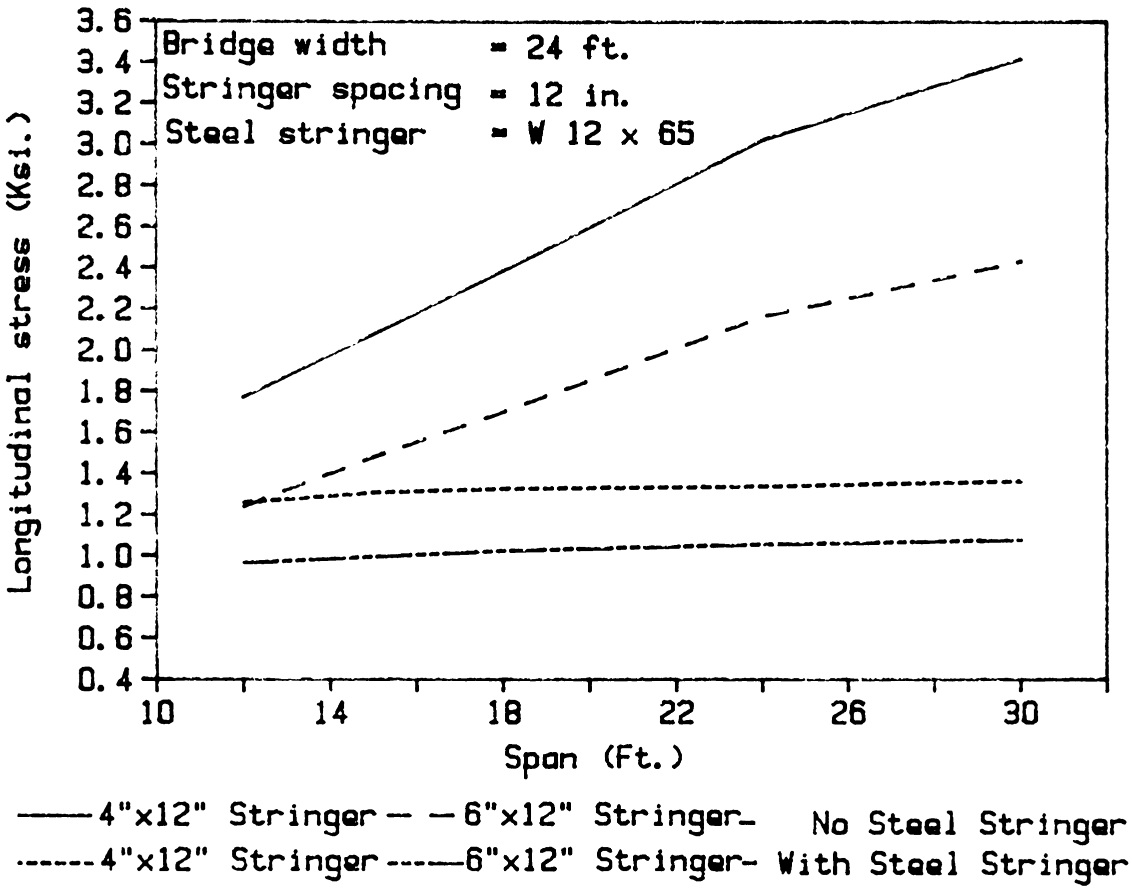

Figure 2.38: Maximum longitudinal stresses in timber stringers for different timber stringer sizes

CHAPTER 3. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

Timber stringer, timber deck bridges have been used widely in the past and the existing bridges are used extensively. As a result of increased frequency or/and higher truck loads the bridges have been posted i.e. restricted to lesser loads or particular type of vehicle or the number of lanes have been reduced.

An attempt has been to rehabilitate timber bridges to meet the present day AASHTO standards. The investigation aimed at determining the overstresses in the timber stringers and to determine the techniques to reduce overstresses to a permissible value or allowable value.

The technique of addition or replacement of timber stringers with steel stringers can be used to reduce the overstresses in timber stringers. The addition or replacement technique depended on the timber stringer spacing, size, span, etc. The present investigation was carried out using the replacement technique, i.e. removing certain timber stringers and replacing with steel stringers.

The available methods cannot predict the true behavior or exact stresses in timber bridge components when several timber stringers are replaced with steel stringers. Hence in order to determine a more realistic stress distribution, finite element method of analysis was used in the study. ANSYS, a general purpose finite element program was used in this work.

A three dimensional finite element model was used to predict the behavior and to determine the stresses in timber and steel stringers. The model predicts the actual behavior of the complex bridge model and gives the stresses in timber or/and steel stringers due to present day loading standards.

Conclusions

Based on the results, the following conclusions can be drawn:

- Some of the existing timber bridges cannot carry present day truck loads or increased frequency of traffic. Hence the bridges should be posted.

- To increase the load carrying capacity, the timber bridges should be strengthened. The strengthening can be done by adding steel stringers or replacing some of the timber stringers with steel stringers. The technique of adding or replacing depends on timber stringer spacing, timber stringer size, span, load, etc.

- The best position for steel stringer was at a distance 4 ft. and 10 ft. from the curb on either side of center line in a two lane bridge and at a distance of 4 ft. from the curb on either side of center line with one at the centerline in case of a one lane bridge.

- By the addition of steel stringers, the stresses in timber stringers were reduced by about 47 percent (for the standard bridge considered). There was not much reduction in timber stringer stress with the increase in steel stringer size, but the steel stringer stresses were reduced by about 75 percent.

- The timber and steel stringer stresses depend on the position, depth, size and type of steel stringers.

- The abutment seating and the abutment must be designed for the extra load it is subjected to due to the addition of steel stringers.

Recommendations

Several areas need further investigations related to this study. They are:

- In the investigation, it was assumed that the deck is nailed to stringers and the displacement of deck and stringer is assumed to be same. But in actual condition this is not true and there is a slip between the deck and stringer, which should be considered in the study.

- Experimental study should be conducted on a small-scale model bridge in a laboratory or on a full-size bridge in the field to validate the theoretical behavior.

- Other loading cases like temperature loading, ice loading, wind loading, centrifugal loading depending on the location should be considered in the analysis.

- The deck is made up of timber boards and has joints in both directions (depending on the type of deck). The deck should be modeled to incorporate these joints.

BIBLIOGRAPHY

[1] U. S. Department of Transportation. *Highway bridge replacement and rehabilitation program.* Eighth annual report of the secretary. Document 100-22, Washington, D.C.: U. S. Department of Transportation.

[2] Ritter, Michael A. *Timber bridges: Design, construction, inspection and maintenance.* 1st ed. Washington, D.C.: U. S. Department of Agriculture, 1990.

[3] Federal Highway Administration, Bureau of Public Roads. *Standard Plans for Highway Bridges.* Washington, D.C.: U. S. Department of Transportation, 1979.

[4] AASHTO. American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges.* 14th ed. Washington, D.C.: AASHTO, 1989.

[5] Bakht, Baider, and Jaeger, Leslie. G. *Bridge Analysis Simplified.* 1st ed. New York: McGraw-Hill, 1985.

[6] Kim, Jai. B., Brungraber, Robert J., and Kim, Robert. H., "Recycling bridges" Civil Engineering Journal, (November 1988). 58-59.

[7] Fargo, B. "Rehabilitation of steel truss bridges in Ontario." Downsview, Ontario: Ministry of Transportation of Ontario.

[8] Ronald, Berger H. *Extending the service life of existing bridges by increasing their load carrying capacity.* Report No. FHWA-RD-78-133. Washington, D.C.: U. S. Department of Transportation, 1978.

[9] DeSalvo G.J., and Swanson J.A. *ANSYS Engineering Analysis System - User Manuals I and II, Theoretical Manual and Examples Manual.* Houston, Pa.: Swanson Analysis Systems Inc., 1985.

[10] Cook, R.D. *Concepts and application of finite element analysis.* 2nd ed. New York: John Wiley, 1981.

[11] Grandin, Hartley. *Fundamentals of the finite element method.* 4th ed. New York: Macmillan, 1986.

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APPENDIX I: VALIDATION OF FINITE ELEMENT MODEL

The timber stringer timber deck bridge was modeled using finite element method in this investigation.

In reality the deck is made up of large number of small timber boards, nailed to the stringers and hence the deck is discontinuous in one direction. In transverse timber deck the deck is discontinuous in longitudinal direction and in longitudinal timber deck the deck is discontinuous in transverse direction. For simplicity in analysis, the deck was assumed to be continuous in both directions without any joints. The longitudinal stringers are modeled using beam elements and the transverse deck is modeled using plate bending elements.

An analysis was performed to validate the modeling of continuous timber deck instead of the discontinuous timber deck with joints. In the first case, the deck was modeled as discontinuous deck with transverse joints. This was achieved by removing the continuity in one direction, either transverse or longitudinal, depending on the type of timber deck. In the second case, the deck was modeled as continuous in both the directions irrespective of the type of deck. This is illustrated in Figure I.A.

The maximum longitudinal timber stringer stresses were plotted against the transverse direction of the bridge in Figure I.B.

From Figure I.B it can be found that the stresses are reduced by about 5 percent throughout, due to the continuous deck model instead of the discontinuous deck model with joints.

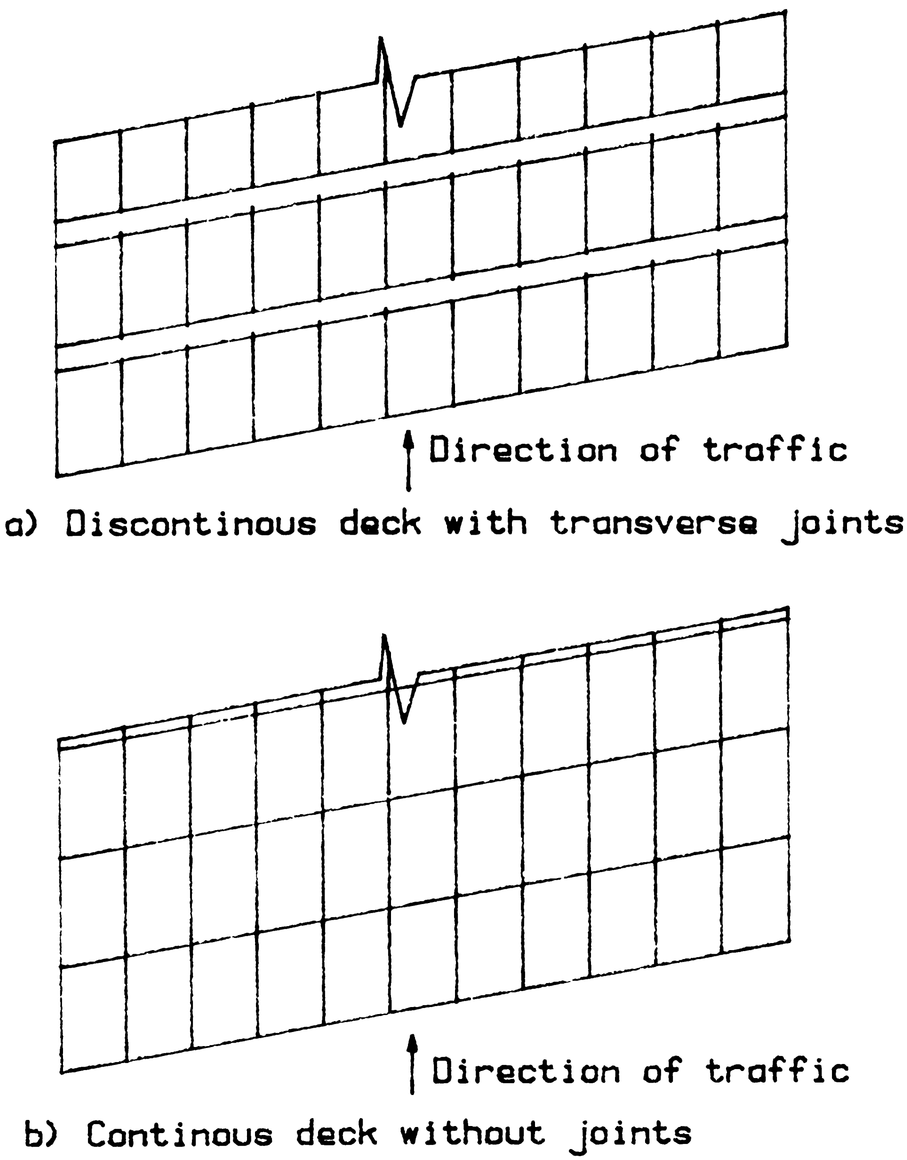

Figure I.A: Finite element model of timber deck with and without joints

Hence to simplify the analysis, the deck was modeled as continuous in both the directions without any joints.

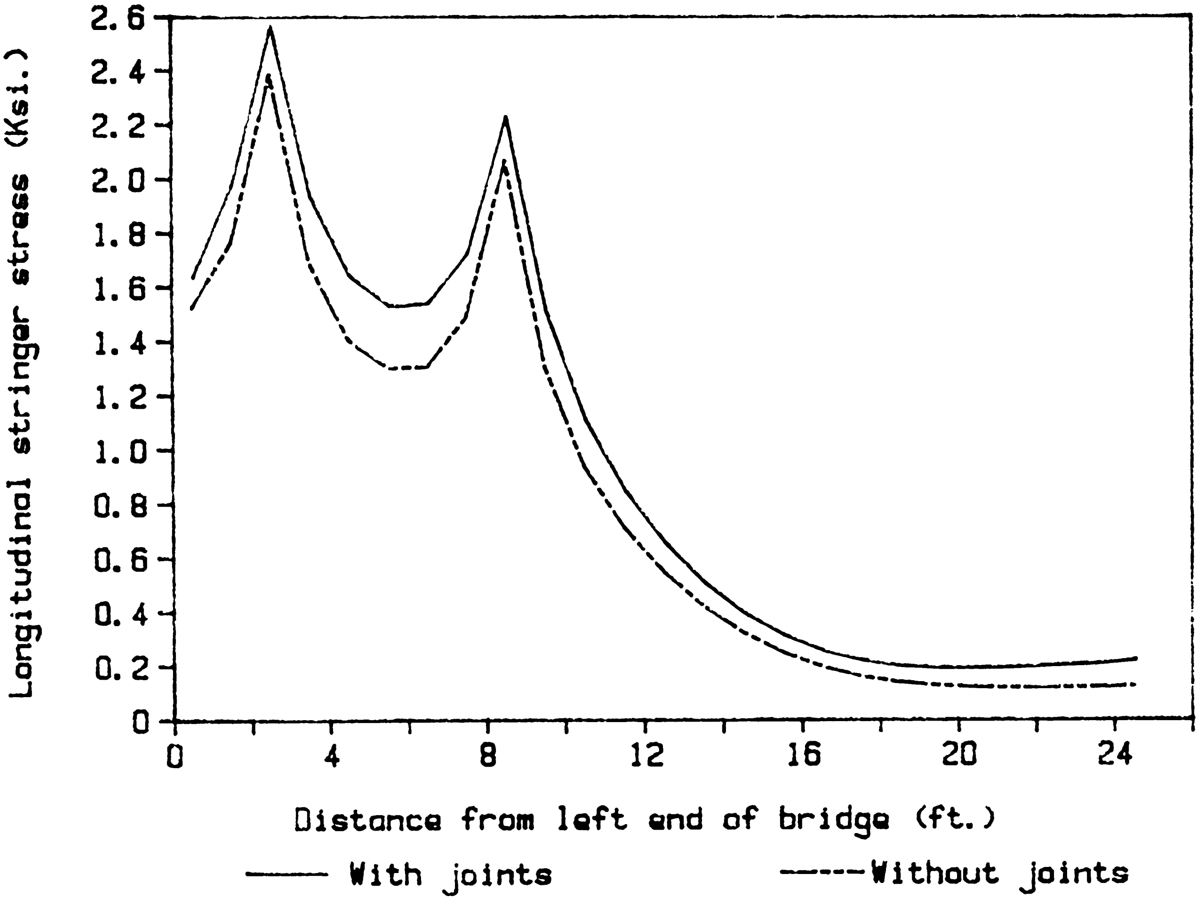

Figure I.B: Longitudinal timber stringer stresses with and without joints

APPENDIX II: VALIDATION OF THE FINITE ELEMENT RESULTS

To validate the finite element results, the longitudinal timber stringer stresses obtained from finite element analysis were compared with the longitudinal timber stringer stresses obtained from simple analytical methods.

In the simplified method the longitudinal timber stringer stresses were obtained by assuming that a stringer plus its associated portion of the slab is subjected to a load comprising one line of wheels of the design vehicle, with the wheel loads multiplied by a fraction (S/D).

In this simplified method the bridge is idealized as an orthotropic plate and the distribution pattern of intensity of longitudinal moments across a transverse cross section is independent of the longitudinal position of the load and the transverse section considered.

A timber bridge with 18 ft. span and 24 ft. width was considered for the analysis. The depth of the deck was 3 in. The stringer size was 4 in. x 12 in. and the stringer spacing was 12 in. center to center. The analysis was performed for a HS 20-44 load.

The longitudinal stresses obtained from different simplified analysis were compared with finite element analysis.

- Baider Bakht's analysis - 1.83 ksi.
- Iowa DOT method using AASHTO - 2.55 ksi.

- Finite element analysis - 2.37 ksi.

The longitudinal timber stringer stress was found to be less in case of Baider Bakht's simplified analysis and more in case of Iowa DOT's simplified analysis when compared to finite element anlaysis.

The lesser value in Baider Bakht's simplified analysis may be due to the factor "D", called as a measure of load distribution. Load distribution is a function of longitudinal and transverse flexural and torsional rigidities of the bridge, ratio of span to width of the bridge, type of load, position of load, etc. All these factors are accounted in a single factor "D" and is a constant for a given type of bridge. This oversimplification might be the cause for the lesser value of longitudinal timber stringer stress when compared to a more realistic value obtained from finite element method of analysis.

The longitudinal timber stringer stress obtained by Iowa DOT procedure based on AASHTO is in close proximity with the finite element results. This is because of the modified "D" factor. The "D" factor obtained from Bakht's analysis was 4.75 ft. whereas the "D" factor used in Iowa DOT analysis procedure in 3.75 ft. and is based on AASHTO (AASHTO 3.23.2).

Hence a more realistic finite element method of analysis is used in this investigation.

APPENDIX III: ILLUSTRATION OF THE USE OF THE GRAPHS AND CURVES

The use of the several curves and graphs is illustrated for a specific longitudinal timber stringer bridge. From the investigation it was found that the overstress in timber stringers can be reduced to permissible limits by replacing several timber stringers with steel stringers.

If the overstress in timber stringers is known, the size of steel stringers to be added to reduce the timber stringer stress to permissible limits can be determined from the curves and graphs.

The procedure is illustrated for a longitudinal timber stringer bridge with following dimensions.

- Bridge span = 18 ft.
- Bridge width = 24 ft.
- Stringer size = 4 in. x 12 in.
- Stringer spacing = 12 in.
- Curb to curb width = 24 ft.
- Number of lanes = 2

- Design lane width = 12 ft.
- Slab thickness = 3 in.
- Youngs modulus of stringers = 1800 ksi.
- Poisson's ratio = 0.33
- Truck loading = HS20-44
- Bending stress = 1.5 ksi (Douglas Fir-larch)

Analysis was done for the maximum longitudinal timber stringer stresses by three different methods and the stresses are tabulated below.

- Baider Bakht's analysis - 1.83 ksi.
- Iowa DOT method using AASHTO - 2.55 ksi.
- Finite element analysis - 2.37 ksi.

From all the three analysis methods it was found that the longitudinal stresses were more than the permissible bending stress. The longitudinal bending stresses can be reduced by the addition of steel stringers. The size of the steel stringer and the stresses in timber and steel stringers after replacement of timber stringers with steel stringers can be determined using figure III.A and III.B.

From figure III.A it can be found that by the addition of four W 12 x 65, the timber stringer stresses can be reduced to about 1.3 ksi. and to about 1.28 ksi. by the addition of four W 12 x 279, thereby reducing the stresses to less than there permissible value of 1.5 ksi. For stringer sizes in between these sizes, the timber stringer stresses lies between 1.28 ksi. and 1.5 ksi. The size of steel stringer does not have much effect on timber stringer stresses.

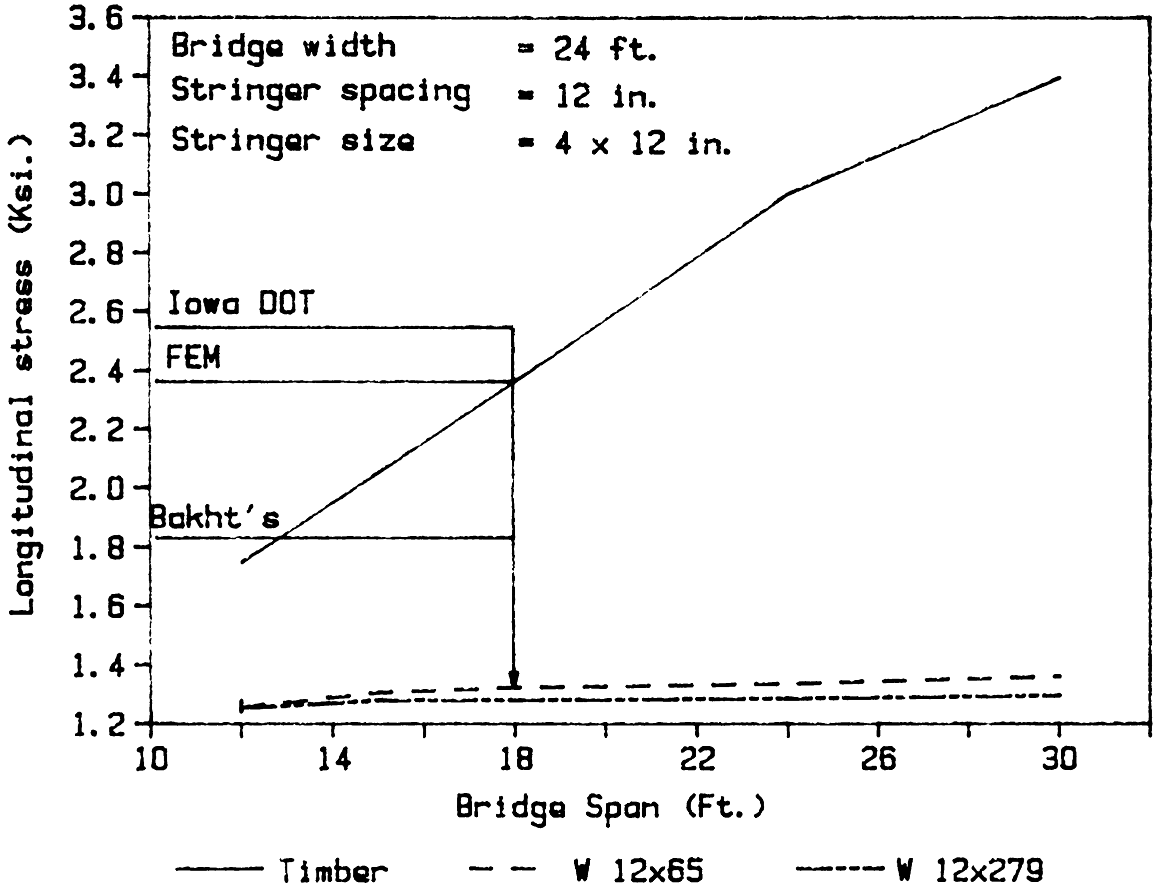

Figure III.A: Maximum longitudinal timber stringer stresses with and without steel stringers

Figure III.B can be used to determine the stresses in steel stringers. It can be found that for a W 12 x 65 the steel stringer stress is about 11 ksi. and for a W 12 x 279 the steel stringer stress is reduced to 2 ksi. The stress in steel stringer has not much effect on timber stringer stress but the steel stringer stress is reduced drastically with the increase in size of the steel stringer.

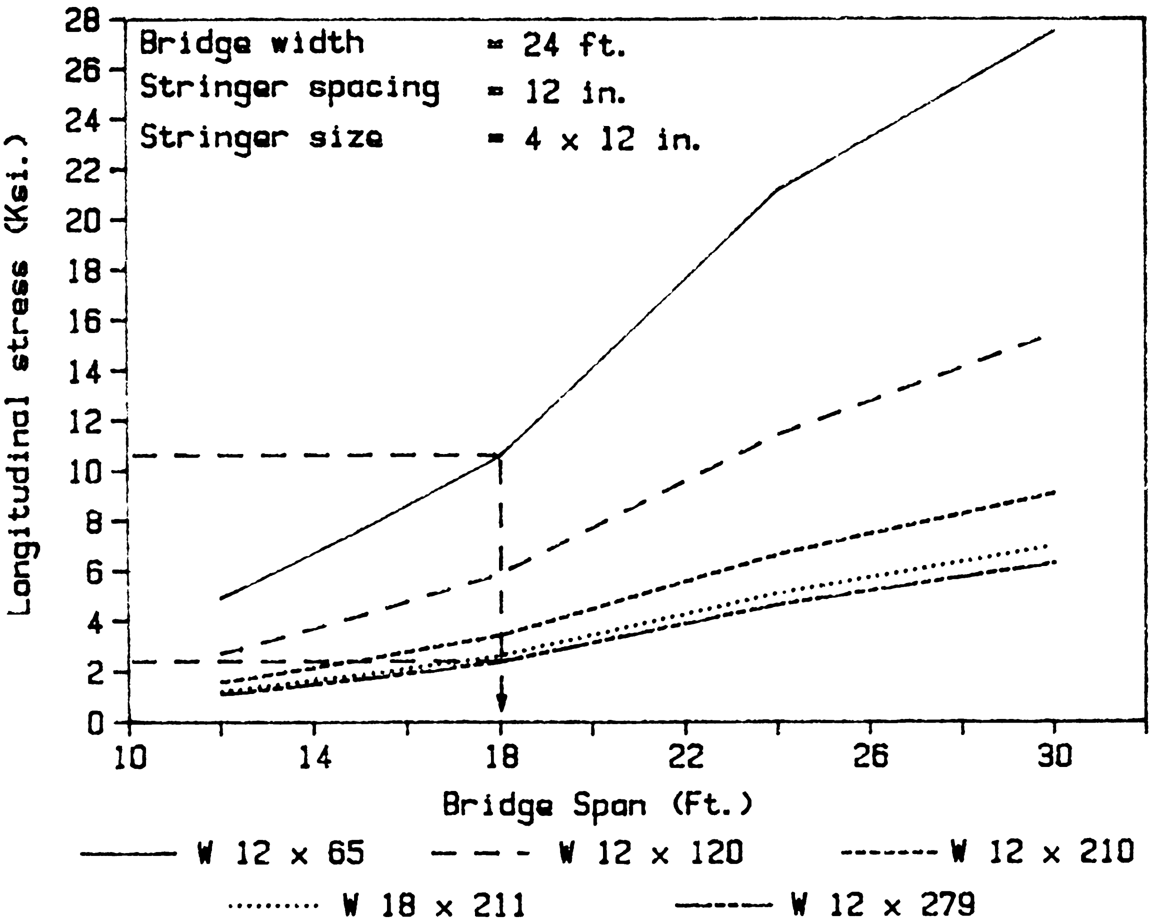

Figure III.B: Maximum longitudinal steel stringer stresses

Hence depending on the timber and steel stringer stresses the size of steel stringers can be determined.