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STRESS ANALYSIS OF THE HAUNCH REGION IN A RIGID FRAME BRIDGE

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

Virginia Highway & Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia Department of Highways & Transportation and the University of Virginia)

Charlottesville, Virginia

August 1977
VHTRC 78-R8

ABSTRACT

The purpose of this study was to obtain an understanding of the behavior and stress distribution in the haunch region of a rigid frame highway bridge. A finite element model of the haunch of the bridge was developed to permit the prediction of stress levels within the haunch region. Loadings on this region were determined using an analytical model of the total frame.

Results from an analytical model of the haunch region were compared with the results obtained from previously made field tests of the studied bridge. Stress contours of minimum principal stress in the haunch were plotted for live loading by a test vehicle placed at three locations. Areas of probable high stress concentrations were identified, and the effect of varying web and flange thicknesses on live load stress levels in these regions was determined.

Current analysis procedures of the region are generally based on a method proposed by Olander. A comparison between stresses calculated in this study and those determined by Olander's method indicated that Olander's procedure may be on the nonconservative side and designers should be extremely cautious in its use.

Though only one bridge configuration and haunch geometry was considered, it is believed that the results from this study can be readily extrapolated to haunch configurations in other rigid frame bridges.

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INTRODUCTION

The use of rigid frame structures for highway bridges is becoming increasingly popular in many states, particularly in Virginia. Bridges of this type generally consist of multi-span, welded rigid frames, so-called because the supporting legs are framed integrally with the welded haunched girders which support the deck. The particular three-span rigid frame highway bridge considered in this study carries Interstate Route 64 over U. S. Route 250 about three miles (4.8 km) east of Charlottesville. Another recently constructed rigid frame bridge is a seven-span delta-leg bridge which carries Interstate Route 64 over the Maury River approximately five miles (8.0 km) west of Lexington. Both structures, shown in Figures 1 and 2, have received national attention through advertisements and brochures.

This type of structure possesses a number of significant advantages over other structural configurations when used as a highway bridge. Three of the primary attributes of this design are (1) the structural efficiency and material economy resulting from the continuity between all elements of an individual frame; (2) safety benefits to traffic by virtue of the interior supports being inclined away from the lower roadway pavement; and (3) the considerable aesthetic value resulting from its slender lines, arch-like appearance and wide, clear span. For these and other reasons, it is likely that this type of bridge structure will be utilized more and more in the future.

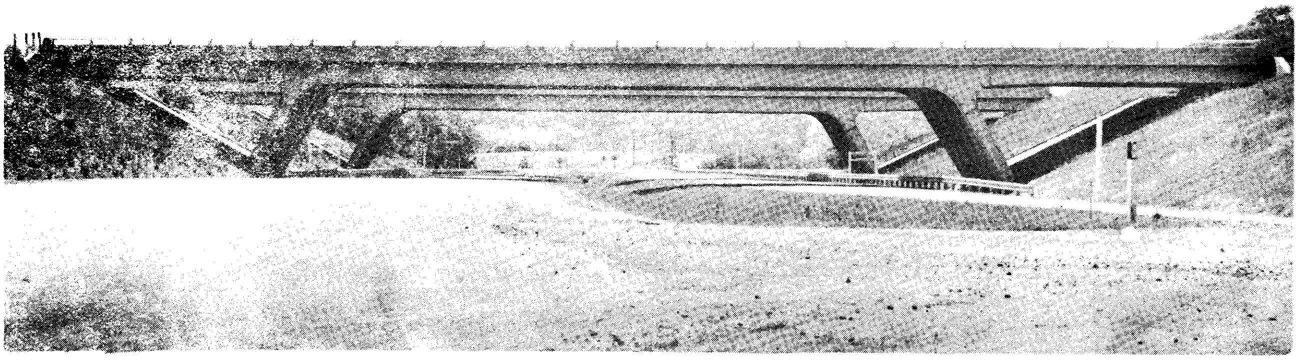


Figure 1. Rigid Frame Bridge on Which Study Was Based.

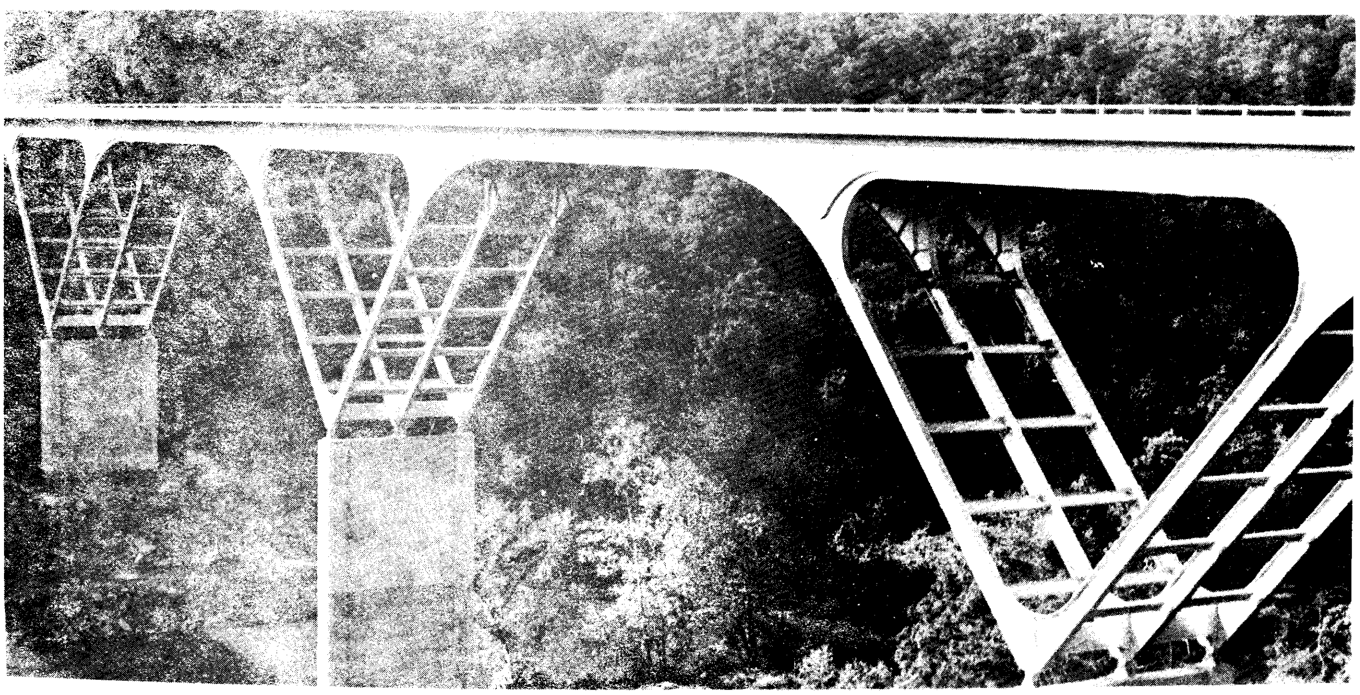


Figure 2. Typical Delta-Leg Rigid Frame Bridge.

PROBLEM

In the analysis and design of a rigid frame highway bridge, it is necessary to make certain idealizing assumptions so that a reasonably uncomplicated solution may be achieved. Such assumptions include selecting effective lengths and effective stiffnesses for nonprismatic members; idealizing support conditions as either pinned, fixed, or roller supported; and making rather substantial simplifications to permit the analysis of stresses in the haunch regions of the frame. While certain of these assumptions can be made on a rational basis and, based on experience, yield reliable results, others are less reliable and the resulting solution is subject to question.

Uncertainty in the results definitely is encountered when a stress analysis of the haunch region is attempted. In the design of the haunch, accurate predictions of stresses in the web under design load are essential to ensure that AASHTO specifications against buckling are satisfied and to determine the need and location for stiffeners. To determine stress levels in the haunch, many current design procedures employ a stress analysis based on a procedure proposed by Olander over 20 years ago.(1) Since this procedure is approximate, even for beam geometries much simpler than those found in rigid frame haunches, web stresses determined by this approach are questionable.

With current analysis capabilities, such as the finite element method, it was deemed desirable to investigate the feasibility of developing a technique more accurate than those currently used for predicting haunch stresses and, if possible, to evaluate the reliability of the more approximate, but simpler, existing procedures. Such an investigation appears especially timely since an experimental study of a rigid frame bridge, which included measurements of stresses in the haunch region, has recently been completed.(2)

OBJECTIVE

The broad objective of the study was to develop a realistic model of the haunch region in a rigid frame bridge to permit an accurate and reliable determination of stresses within the haunch. Such a model could be used to make parametric stress analysis studies to evaluate the effects of geometry, stiffener location, and flange and web thicknesses on stress levels and locations of peak stresses within the haunch. The results of experimental stress measurements in the haunch region of a rigid frame bridge, determined from field tests, were available as a basis of comparison for verifying the model.

Within the broad objective of the investigation, the following specific tasks were established:

1. Development of a realistic analytical model of the haunch region of a rigid frame highway bridge to permit accurate determination of stresses.

2. Determination of loadings on the haunch, corresponding to actual vehicle loads, using standard modeling techniques for the entire bridge.

3. Calculation of the stress levels throughout the haunch for various vehicle locations.

4. Comparison of these analytical values of stress with those determined experimentally, to provide some measure of verification of the haunch model.

5. Evaluation of the effects of haunch parameters such as stiffeners, web and flange thicknesses on stress levels and peak stress locations.

6. Comparison, on a limited basis of stresses obtained by the haunch model with those obtained by existing design procedures.

The investigation was limited to the study of one bridge configuration and haunch geometry, namely those of the three-span rigid frame bridge on Interstate 64 near Charlottesville. This particular choice was made because an extensive field study had recently been conducted on this bridge and the results of experimental stress and deflection measurements made on it were available. The haunch configuration in the studied bridge is similar to that in other rigid frame structures, and it is believed the results from the study can be readily extrapolated to other haunch configurations.

Because all of the experimental stress and deflection measurements correspond to a single vehicle load, the same live load was used throughout the study. The haunch parameters included in the investigation were the thicknesses of the web and flange and the haunch with and without stiffeners.

DESCRIPTION OF BRIDGE

The bridge, shown in Figures 1 and 3, is 216 ft. (65.83 m) long and consists of five 3-span welded rigid frames. The two interior supports of each frame are inclined I-shaped columns framed integrally with the welded haunched girders and supported on concrete footings with anchor bolts attached to the web in such a manner as to allow free rotation. The ends of the frames are simply supported on shelf abutments with allowance for longitudinal movements. The structure was designed for an HS 20-44 live load using A-36 structural steel in accordance with AASHTO Specifications, 1965. The structure was completed in the latter part of 1969.

An extensive experimental load testing study of the bridge used as the model for the present study was conducted in September 1972. Measured strains and deflections were recorded for a variety of test vehicle locations. Experimental measurements of haunch stresses and subsequent analytical modeling of the bridge and haunch were made with respect to the first interior frame from the outside, which was taken to be representative of all frames. In particular, 8 rosette gages were placed on the web of the haunch, and 19 SR-4 strain gages were located on the flanges and stiffeners of the same haunch. A sketch of the haunch instrumentation is presented in Figure 4 and further details of the instrumentation may be found in a report describing the study.(2)

The test vehicle used for the experimental study was a 3-axle diesel semitrailer loaded to simulate an HS 20-44 loading. A sketch showing the wheel loadings and dimensions between wheels and axles is shown in Figure 5. This same vehicle loading was utilized in the analytical investigation reported here. Although the field study utilized test runs at various speeds and various lane locations, only data from crawl (static) runs in which the vehicle was centered over the test frame were used in the analytical study.

When comparisons between experimental and analytical stresses are given in subsequent sections of this report, the location of the test vehicle is given as a percentage of the distance between pneumatic hoses positioned 50 ft. (15.2 m) ahead of the east abutment and 75 ft. (22.9 m) beyond the west abutment. All locations are referenced from the front axle of the vehicle and are measured from east to west.

In the design of a bridge girder, total vehicle loads are distributed to each girder in accordance with AASHTO specifications. In this analytical study, the distribution factors used were based on experimental measurements. One set of distribution factors was based on midspan moments and the other on midspan deflections. The distribution factor for the vehicle centered over the test frame was 0.295 based on measured deflections and 0.3887 based on moments determined from measured strains. Both distribution factors were evaluated in the analytical studies of the bridge and haunch described in the next section of this report. Results using the factor of 0.3887 agreed more closely with the experimental findings, and only this factor was used in the remainder of the study.

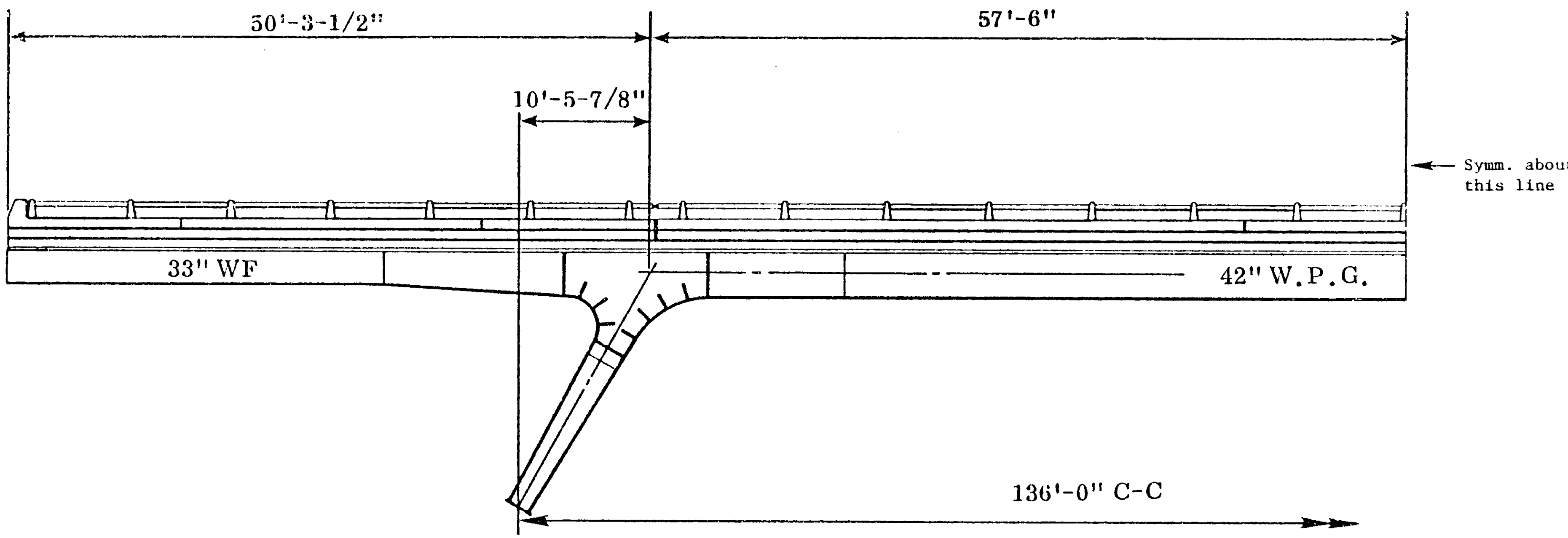


Figure 3. Partial Elevation of Test Structure. (From Ref. 2)

(1 in. = 0.0254 m.; 1 ft. = 0.3048 m)

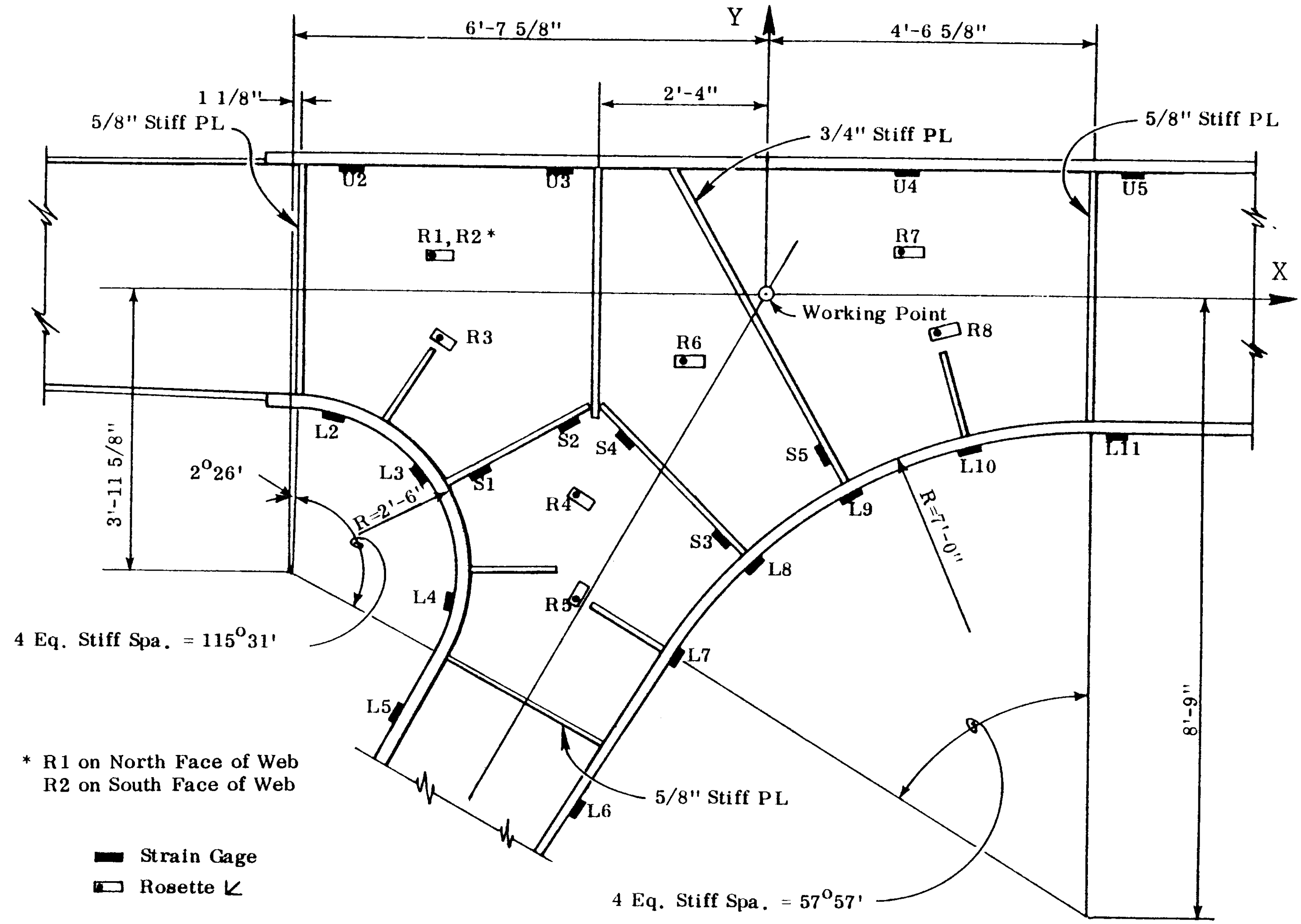


Figure 4. Sketch of Haunch Showing Gage Locations. (From Ref. 2)
(1 in. = 0.0254 m.; 1 ft. = 0.3048 m)

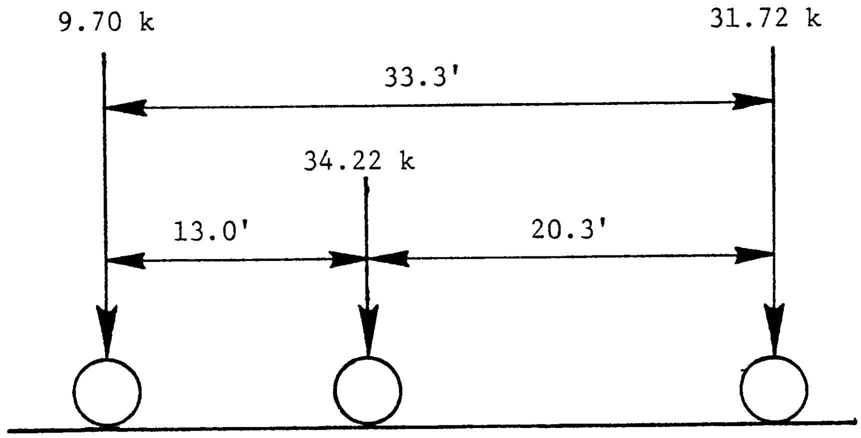


Figure 5. Axle Weights and Spacing, (From Ref. 2)

(1 ft. = 0.3048 m ; 1 kip = 4448.222 N.)

DEVELOPMENT OF HAUNCH MODEL

Description of Haunch

The geometry and dimensions of the haunch region of the rigid frame were given in Figure 4. For convenience in subsequent discussion, the three extremities of the haunch will be designated as follows: (1) the portion of the haunch toward the centerline of the bridge structure will be denoted as haunch interior, (2) the portion of the haunch toward the shelf abutments will be denoted as haunch exterior, and (3) the portion of the haunch connected to the inclined legs will be denoted as haunch leg (see Figure 6). Reference to internal forces acting on the haunch extremities will also be denoted in a like manner.

The haunch consists of a straight upper flange and curved lower flanges, all 2 in. (0.0508 m) thick and 14 in. (0.3556 m) wide. The radius of the interior lower flange is 7 ft. (2.1336 m) and that of the exterior lower flange is 2 ft. 6 in. (0.7620 m). The thickness of the web region is 1/2 in. (0.0127 m) while the stiffeners vary in thickness from 5/8 to 3/4 in. (0.159 to 0.0191 m). An 8-in. (0.2032 m) concrete deck overlays the top flange and is connected by studs to ensure composite action.

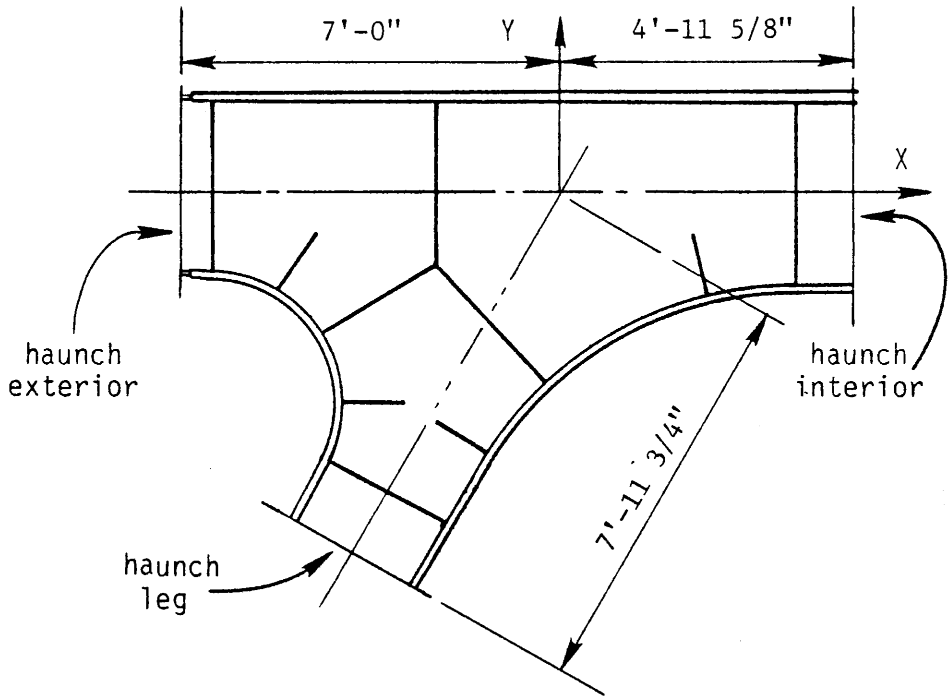


Figure 6. Haunch Extremity Designation.

(1 in. = 0.0254 m ; 1 ft. = 0.3048 m)

The stiffeners in the haunch region consist of pairs of bearing and radial stiffeners while others pairs of stiffeners give stability to the web region to resist buckling. The vertical stiffener that meets the two radial stiffeners in the central portion of the haunch is also connected to a diaphragm that joins the interior frames to provide lateral stability. The outer faces of the exterior frames have pairs of radial stiffeners and bearing stiffeners only at the outer portions of the haunch.

The bearing and interior stiffeners are fillet welded on both sides to the web and are milled to bear against the flanges. The radial stiffeners are fillet welded on both sides to the web but also are groove welded to the lower flanges.

Analytical Model of Haunch Region

To provide for modeling flexibility and detailed stress and displacement computations, a finite element model was adopted as the analytical representation of the haunch. In the initial phase of the study, a number of mesh sizes and configurations were examined. The finite element model finally adopted is depicted in Figure 7. The small crosses at the centroids of certain web elements indicate strain gage locations on the actual bridge. Only four basic elements were used in the finite element model of the haunch. Beam elements were used to represent the top steel flange and concrete upper deck. The concrete was transformed into an equivalent steel section using a value of n (ratio of E_S/E_C) = 6. In this model, 22 beam elements were used. Bar elements were used to represent the stiffeners. The minimum number of bars used to model a stiffener was 2 elements and the maximum number was 8. The total number of bar elements used was 40.

The web region of the haunch was modeled using membrane elements which can take in-plane deformation but cannot resist any out-of-plane twisting. Since the study considered only vehicle loads centered over the instrumented frame, the assumption of no torsion, or out-of-plane bending of the web seems reasonable. The web region is made up almost entirely of triangular elements formulated on a constant stress formulation. One quadrilateral membrane element was used in the web to model the region where the two lower radial stiffeners almost joined. In the haunch model, there were 490 triangular membrane elements and 1 quadrilateral membrane element. The lower flanges of the haunch were represented by plate membrane elements. This element is a combination of a membrane element and a plate element, and thus is able to resist both in-plane deformation and bending. A total of 48 plate membrane elements were used for the lower flanges. Details of these elements may be found elsewhere.(3)

Since the haunch is always in equilibrium under the external and internal applied loads, the finite element model of the haunch should, theoretically, require no support conditions for equilibrium. But, as in any structure in equilibrium, a means of supporting the structure must be provided to prevent rigid body motion. Therefore, two support conditions of the haunch region were chosen: (1) a pinned support at the middle node at the leg extremity (point A in Figure 7), and (2) a roller support at the lowest node of the haunch interior (point B in Figure 7). This method of supporting the haunch model seems rational since the legs are assumed to be pinned and the bearings at the abutments are taken as rollers in the bridge finite element program.

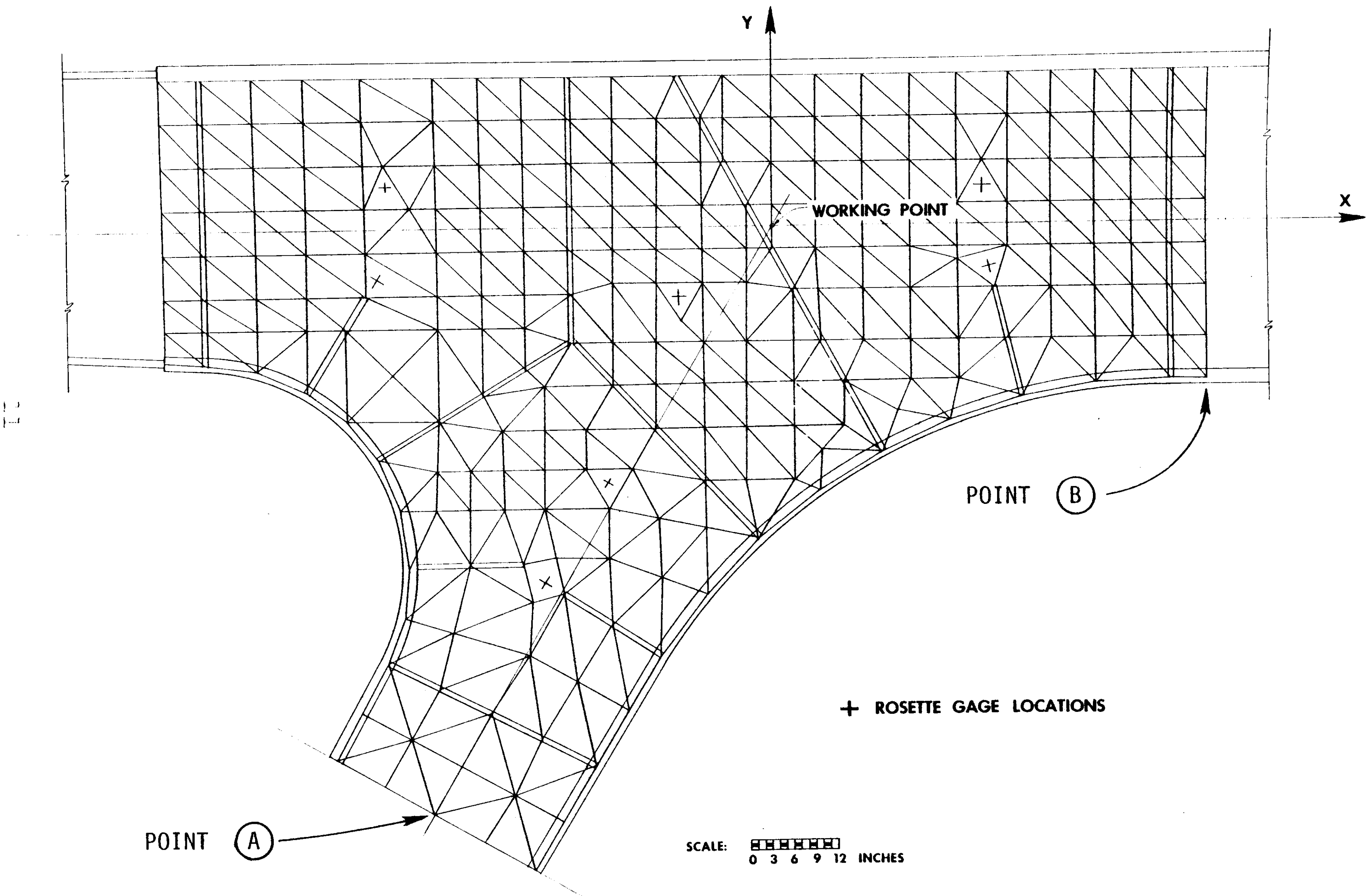


Figure 7. Sketch of Finite Element Model of Haunch Region.

(1 in. = 0.0254 m)

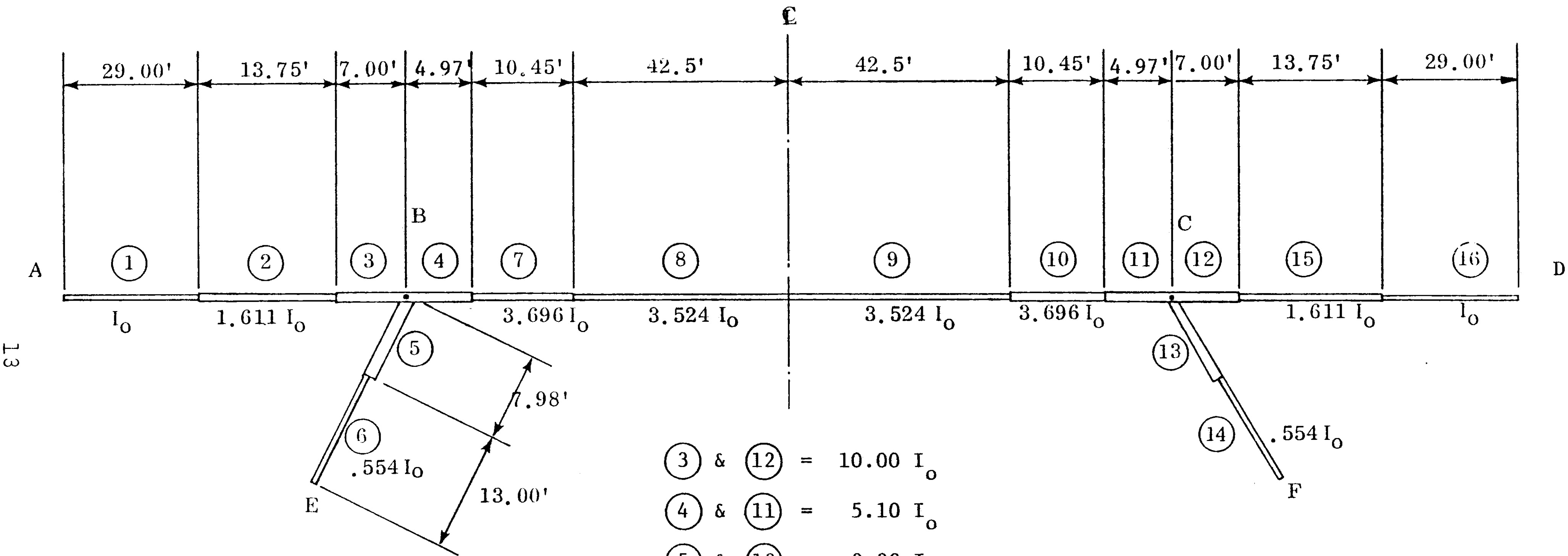
Determination of Loads on Haunch Model

The external loads to be applied to the haunch extremities correspond to internal forces at the haunch locations in the frame. Accordingly, for a prescribed magnitude and location of live load, the corresponding loadings on the haunch required an analysis of the entire frame and a determination of the resulting forces in the haunch region.

An analysis of a typical frame of the bridge structure was performed using standard finite element modeling in which the frame was represented as a series of prismatic beam elements. The total frame structure was subdivided into 16 separate beam elements, with 2 elements representing each end span, 4 elements modeling the center span, 1 element for each inclined leg, and 3 elements representing each haunch. A sketch of the idealized structure used in this analysis is shown in Figure 8.

This particular bridge model was developed as part of an earlier study on this same rigid frame bridge.[2] In this model, the flexural characteristics of the actual structure were modeled as closely as possible. The concrete deck was taken into account by transforming it to an equivalent area of steel using a value of n equal to 6, and an effective slab width prescribed by AASHTO specifications. With regard to support conditions, each of the inclined legs was assumed to be pinned at the base, and the bearings at the abutments were treated as roller supports. Reported results from the analytical study compared favorably with the experimental results, thus indicating a reliable model.[2] For both midspan moments and deflections due to live load, the analytical results compared within 4% of the experimental results, with both showing good agreement. The analytical results used in this comparison were based on the vehicle centered over the test frame with a transverse distribution factor of 0.3887. This factor was mentioned earlier as being the experimentally measured distribution factor based on the midspan moments.

Using a live load corresponding to an AASHTO HS 20-44 vehicle loading, internal forces were calculated at frame locations corresponding to the haunch extremities for a variety of load locations, and these forces were used as external loading on the haunch model. This same bridge model was also used to define the loading at the haunch extremities due to the dead load of the bridge. For dead load, a weight/unit length for each element was found by multiplying the transformed area of that element by the specific weight of steel and adding a correction factor due to the additional weight of concrete. This weight/unit length defined a loading on each element and the internal forces at the haunch extremities were then determined.



(3) & (12) = 10.00 I_o

(4) & (11) = 5.10 I_o

(5) & (13) = 3.30 I_o

I_o = 18,352 in.^{4}

1 inch^4 = 416.3 x 10^{-9} m^4

1 ft. = 0.3048 m

Figure 8. Idealized Model of Rigid Frame Structure. (From Ref. 2)

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Once the external forces on the haunch were obtained, they were transformed to nodal loads for use with the finite element model. The shear and thrust forces were distributed evenly among the nodes at the haunch extremities. For the moments, an equivalent force at each node was obtained by calculating the tributary area about the node in question multiplied by the average of the stresses at adjacent node points.

As part of this phase of the study, the effect of assumed support conditions on the internal reactions at the haunch extremities was studied. The internal reactions affected the most were the moments at the interior and leg and the thrusts at all three locations. In the finite element study of the haunch region described in a later section, the effect is considered when comparisons with the experimental data are used.

Additional information obtained from an analysis of the entire frame was concerned with regions of tension and compression in the welded frame, where fatigue considerations were of interest. The routine inspection of bridges requires a detailed inspection of all welds in all regions where tensile stresses exist. At the request of the Department of Highways and Transportation, the analytical model for the bridge on I-64, and a similar model developed for a 5-span delta-leg bridge on I-77, were used to predict regions of tension and compression in the girder and legs due to live load and dead load combination. In almost all cases, the dead load was the controlling factor. The results of this supplemental study are given in Figures 9 and 10. Although these results are for two specific bridges, the information should be indicative of regions of tension in other rigid frame configurations and thus offer guidance to inspectors.

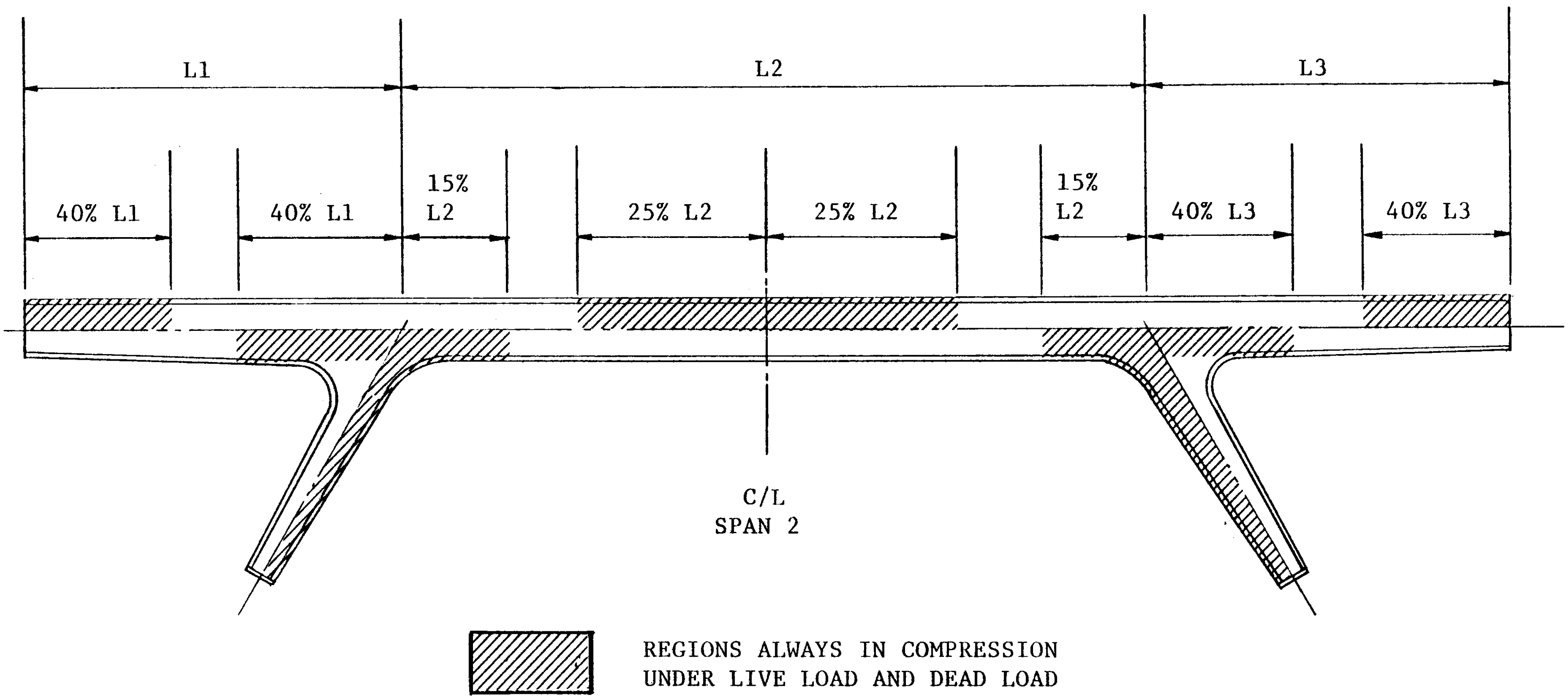

Figure 9. Tension and Compression Regions in a Three-Span Rigid Frame Bridge on I-64.

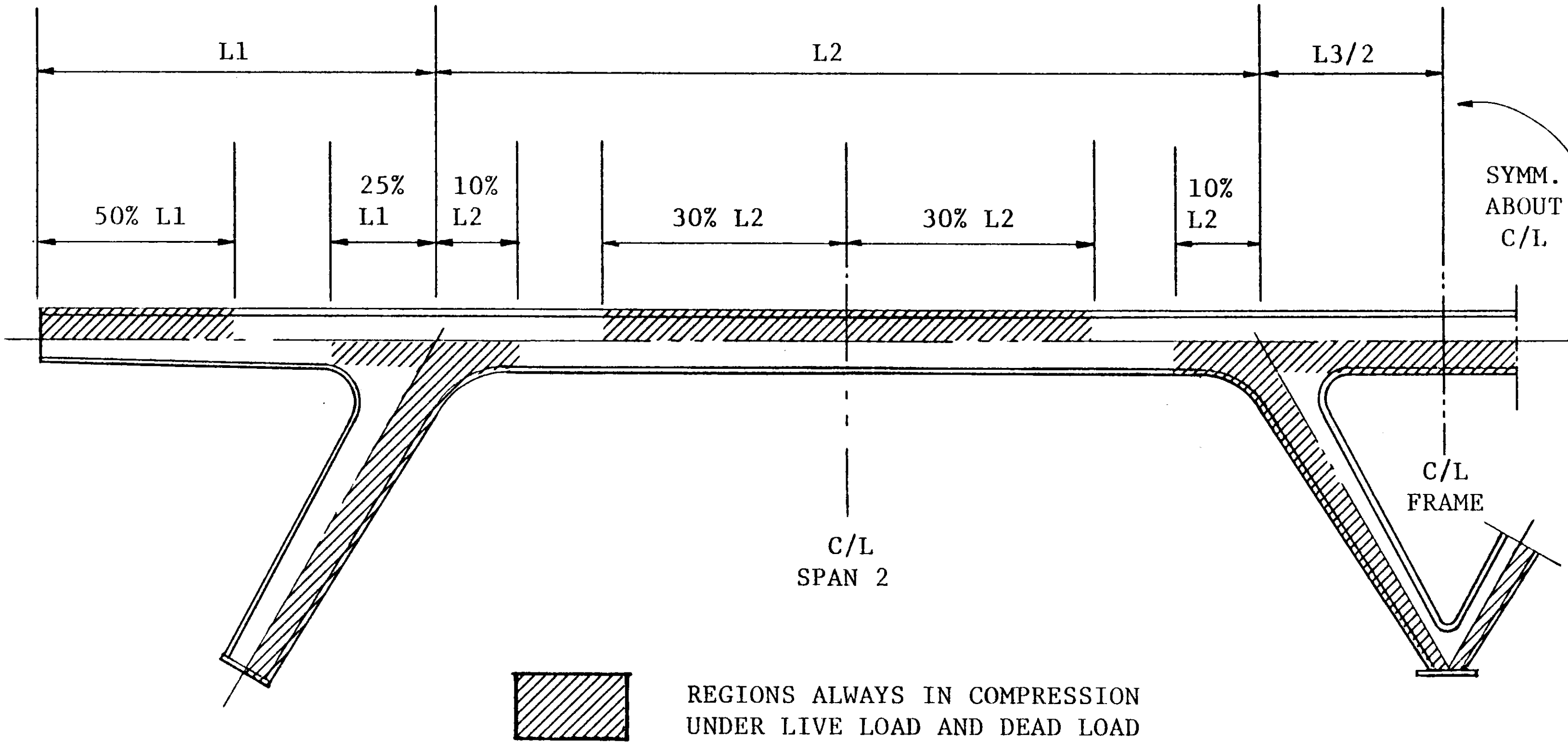

Figure 10. Tension and Compression Regions in a Five-Span Rigid Frame Bridge on I-77.

METHOD OF ANALYSIS

The finite element program used in the analysis of the haunch model was a large general purpose finite element code call SPAR.[3] SPAR is a finite element program made up of a system of related programs which may be operated either in batch or demand (teletype) mode.

SPAR uses the displacement method of finite element analysis in which a high-order system of linear equations is obtained. The system stiffness matrix is regarded as an array of submatrices, with each submatrix corresponding to the connection of one joint to another. Each submatrix is n by n, where n is the number of degrees of freedom at each joint. For general shells and frames n = 6 (three displacements, three rotations); for a plane frame n = 3 (two displacements, one rotation); etc. The only nonzero submatrices are those corresponding to pairs of joints connected by elements. Accordingly, in all but the smallest finite element models, only a tiny fraction of the submatrices are nonzero — usually less than 1%.

SPAR takes account of this small fraction of submatrices being nonzero by solving a large system of linear equations by a so-called sparse matrix solution method. The characterizing feature of this method of solution is that it operates exclusively with data contained in the nonzero submatrices, virtually eliminating the unessential arithmetic (multiplying, adding zeros) and wasted data storage (storing zeros) associated with conventional band matrix techniques. Consequently, by using sparse matrix techniques, low computer execution costs and large size capacity are achieved.

RESULTS FROM STRESS ANALYSIS OF HAUNCH

Stresses Due to Live Load

The analysis of the haunch model by the SPAR computer program provided nodal displacements and element stresses (σ_x, σ_y, and τ_{xy}) for all elements in the web region. Since the stress parameter of primary interest in the web of the haunch was the absolute maximum stress, the output from the SPAR program was subsequently transformed into principal stresses. This value not only provided the best indication of critically stressed regions in the haunch, but also permitted direct comparison with previously determined experimental stresses, which were available only in the form of principal stresses.

For the live load study, three vehicle locations were chosen as shown in Figure 11. Based on influence lines and experimental data, it was believed that these three locations at 28%, 35%, and 42% would produce maximum stresses in the haunch region. As indicated in the figure, the location of the front axle of the vehicle is given as a percentage of the distance between air hoses that were placed at the ends of the bridge during the experimental testing. Thus, 28% of the distance between air hoses is equivalent to the front axle of the vehicle being a distance of 40 ft. and 3 in. (12.17 m) from the left abutment. As observed from Figure 11, at location 28% the vehicle is in the first span and has not entered the haunch portion of the bridge. At 35%, the vehicle has one wheel load in the haunch region, and at 42% the vehicle has just left the haunch region and entered the second span.

Shown in Table 1 are the internal forces about the haunch region for the respective live load locations at 28%, 35%, and 42%. Shown in Figure 12 is the sign convention used in the table for the moment, shear, and thrust at the haunch exterior, interior, and leg designated by subscripts 1, 2, and 3.

Stress Contours of Minimum Principal Stress

The representation of the stress levels within the haunch region is perhaps best given in the form of stress contours. Thus, stress contours of the minimum principal stress were plotted for the three vehicle locations at 28%, 35%, and 42%. The plot of stress contours was accomplished by obtaining the minimum principal stress (extracted from the SPAR computer output of stresses within the analytical model of the haunch) at the centroid of each element of the finite element model of the haunch (see Figure 7); and interpolating a smooth curve between values of equal stress to obtain a contour line. All stress contours plotted in the figures are for the principal minimum stress only, and all stresses are in compression; note also that the direction of minimum stress is normal to the contour line. Stress contours were plotted for the haunch region with and without stiffeners to observe if any significant changes in stress levels were induced. In all cases the haunch loadings were calculated assuming the frames were on roller supports at the end abutments.

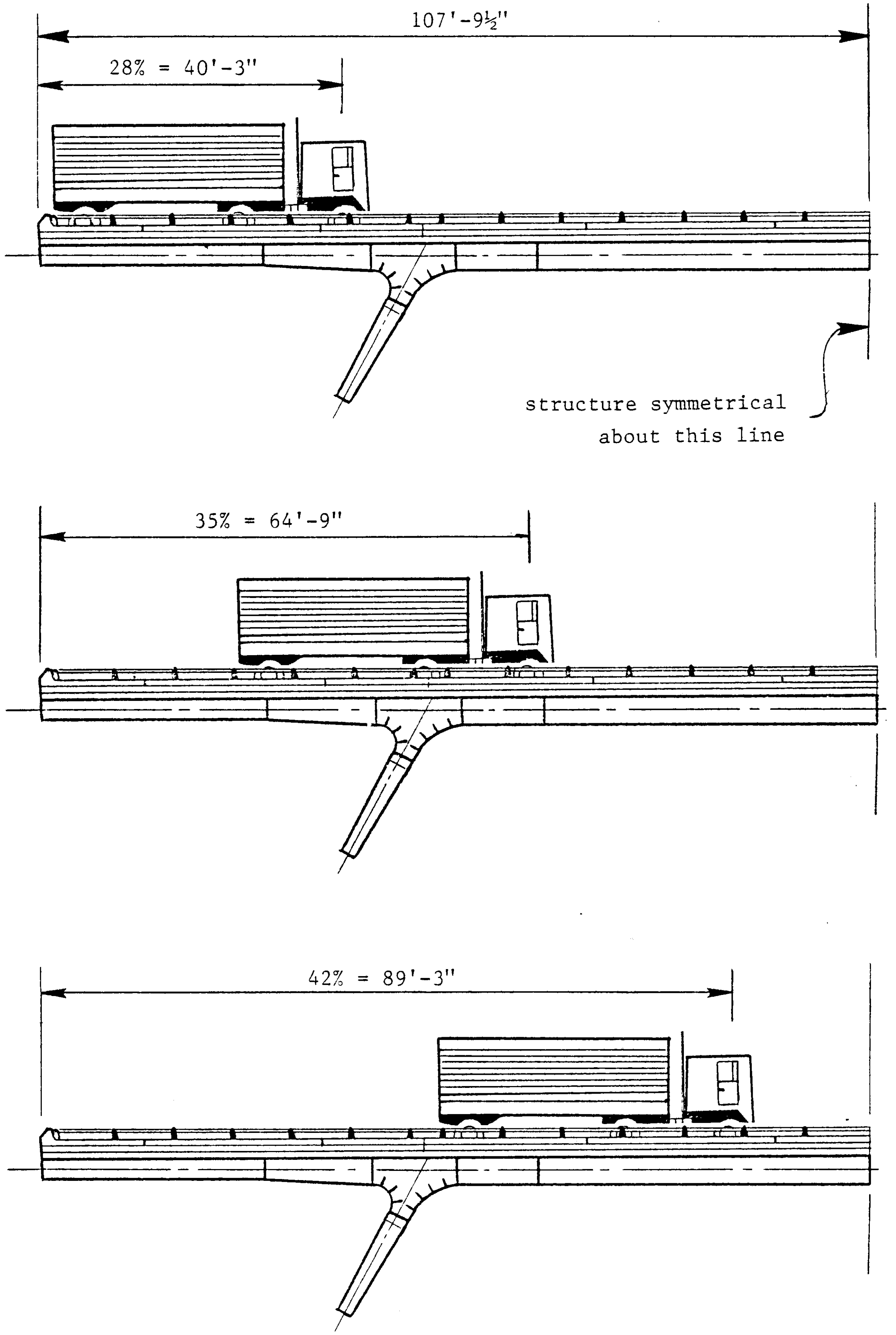

Figure 11. Locations of Vehicle as Percentage of Distance Between Air Hoses.
(1 in. = 0.0254 m ; 1 ft. = 0.3048 m)

TABLE 1

SUMMARY OF APPLIED LOADINGS ON
HAUNCH EXTERIORS (LIVE LOAD)
(1 kip = 4.448 x 10^3 N)
(1 ft-kip = 1.356 x 10^3 N·m)

Force*	Vehicle Location		
	28%	35%	42%
M1 (ft-kips)	-104.62	-81.12	-103.57
V1 (kips)	-16.46	-10.97	-2.42
T1 (kips)	0.00	0.00	0.00
M2 (ft-kips)	45.73	-38.99	19.65
V2 (kips)	-0.44	-2.26	-23.48
T2 (kips)	-0.29	-6.56	-15.82
M3 (ft-kips)	106.54	98.58	-9.82
V3 (kips)	8.20	7.59	-0.75
T3 (kips)	14.78	26.25	30.34

* See Figure 12 for force notation.

Note: Bridge on roller supports at end abutments.

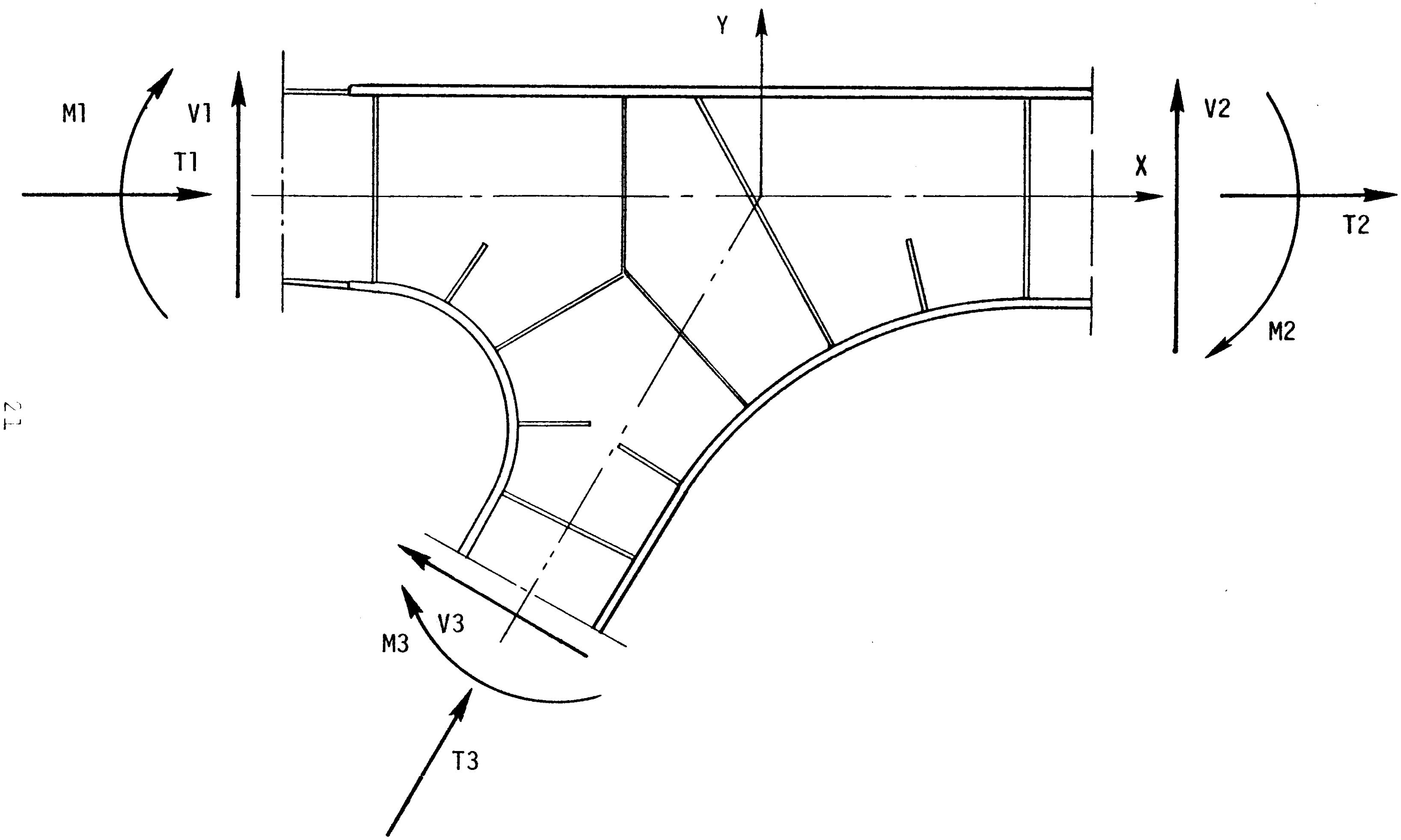

Figure 12. Sign Convention For Forces About Haunch Region .

Shown in Figures 13 and 14 are stress contours of minimum principal stress for the vehicle at location 28% for the haunch region with and without stiffeners, respectively. In Figure 13, note the concentration of stresses at the end and junction of the stiffeners; in Figure 14, for the haunch without stiffeners, the stress contours are much smoother, with only a few areas of stress concentration about the curved lower flange. The largest compressive stress for the haunch with stiffeners was 3,126 psi (21.55 MPa) while the largest stress for the haunch without stiffeners was 3,259 psi (22.47 MPa), an increase of only approximately 4%.

Shown in Figures 15 and 16 are the stress contours of principal stress for vehicle location 35% for the haunch region with and without stiffeners, respectively. Again, there are stress concentrations at the end and junction of the stiffeners, Figure 15, while only a few areas of high stress concentrations are present for the haunch region without stiffeners, Figure 16. The largest compressive stress present in the haunch with stiffeners was 3,147 psi (21.70 MPa), while the largest stress for the haunch without stiffeners was 3,516 psi (29.24 MPa), an increase of about 12%.

Exhibited in Figures 17 and 18 are the stress contours of minimum principal stress for vehicle location 42%. The stress levels within the haunch for vehicle location 42% are low as compared to those for the other vehicle locations. The effect of stiffeners on the stress levels at 42% is almost negligible, as can be seen from Figures 17 and 18. In both figures there appears to be only one area of stress concentration, with the minimum compressive stress for the haunch with stiffeners being 1,637 psi (11.29 MPa) and that for the haunch region without stiffeners being 2,029 psi (13.99 MPa), an increase of almost 24%. This appears to be a significant increase; but, since these stresses are much lower in magnitude than the stresses obtained in the haunch region at locations 28% and 35%, one may conclude that the effect of stiffeners on the stress levels within the haunch is significant only with the vehicle in the immediate vicinity of the haunch region.

Referring to Figures 13 through 18, it appears that including stiffeners in the haunch region of the bridge may lead to areas of high stress concentrations at the end and junction of the stiffeners. Values in these figures also indicate that the deletion of stiffeners altogether within the haunch appears to produce fewer areas of high stress concentrations and smoother flow patterns of stress contours. If stiffeners are used, such areas of high stress concentration may be particularly susceptible to fatigue type failures in the welds joining the stiffeners to the web. Thus, the use of stiffeners may lead to detrimental results in the web portion of the haunch, and care should be exercised in the design of the haunch with regard to the use and placement of the stiffeners.

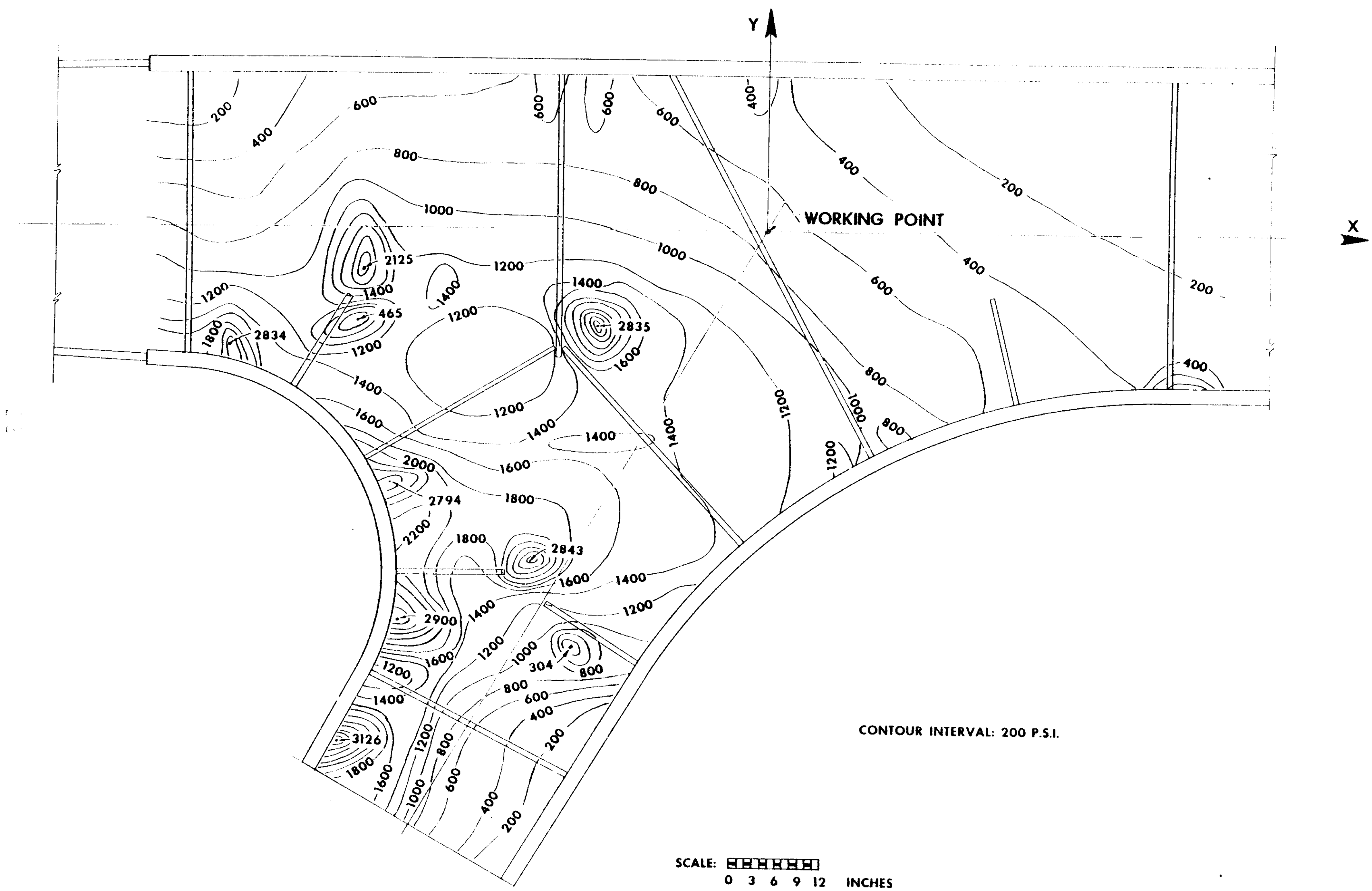

Figure 13. Stress Contours in Haunch Region With Stiffeners For Vehicle at 28% .

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

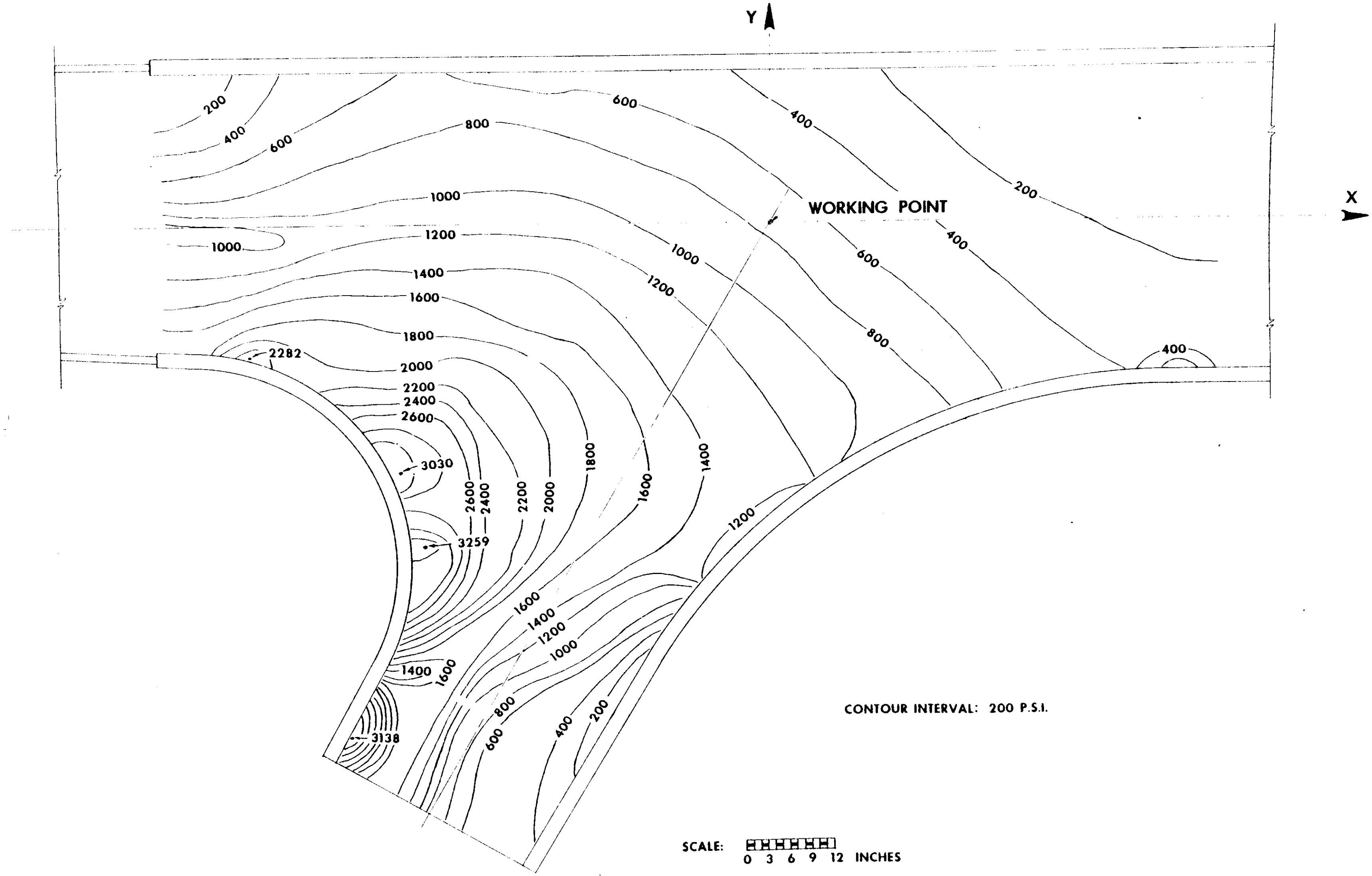

Figure 14. Stress Contours in Haunch Region Without Stiffeners For Vehicle at 28%.

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

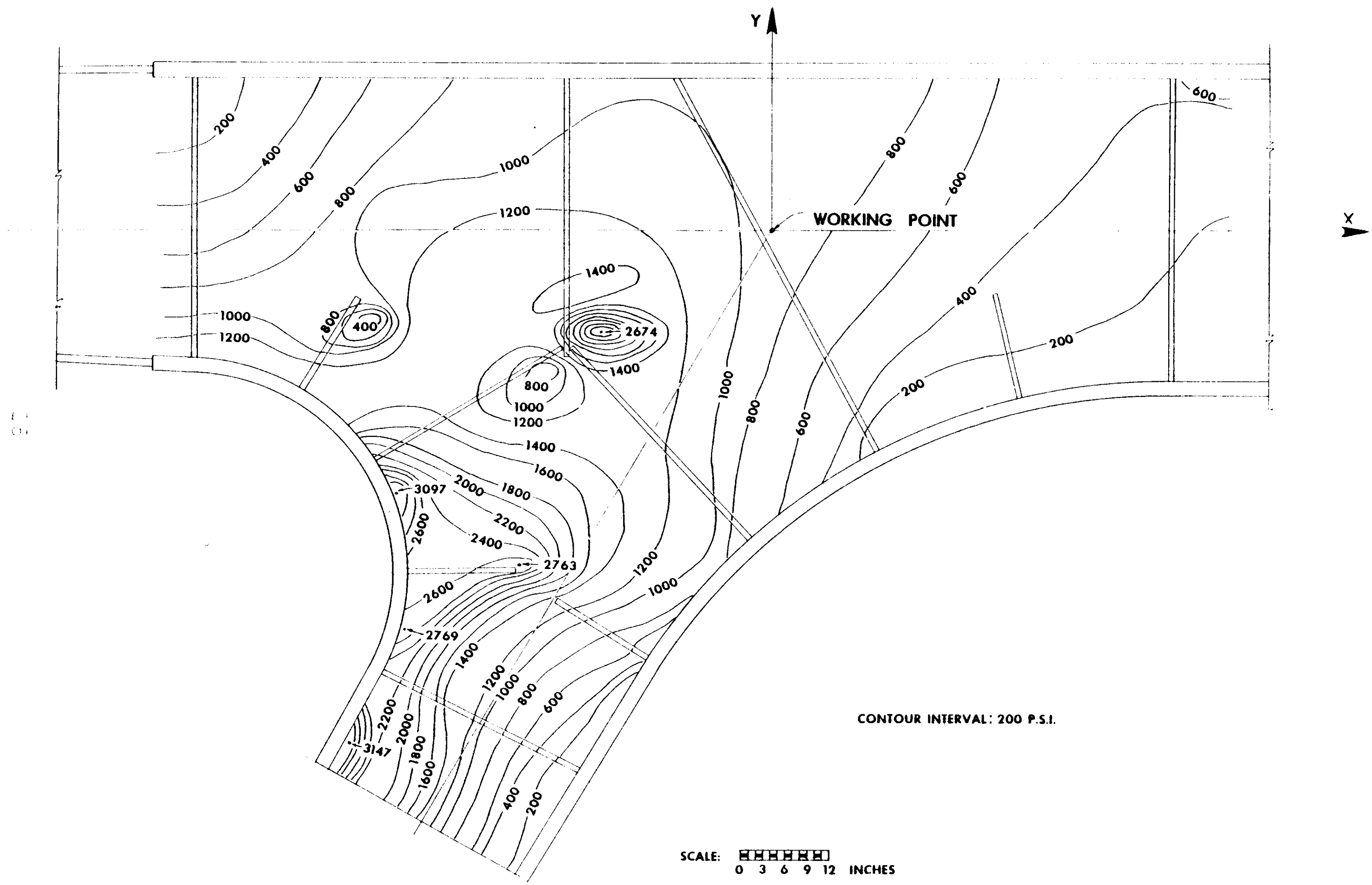

Figure 15. Stress Contours in Haunch Region With Stiffeners For Vehicle at 35% .

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

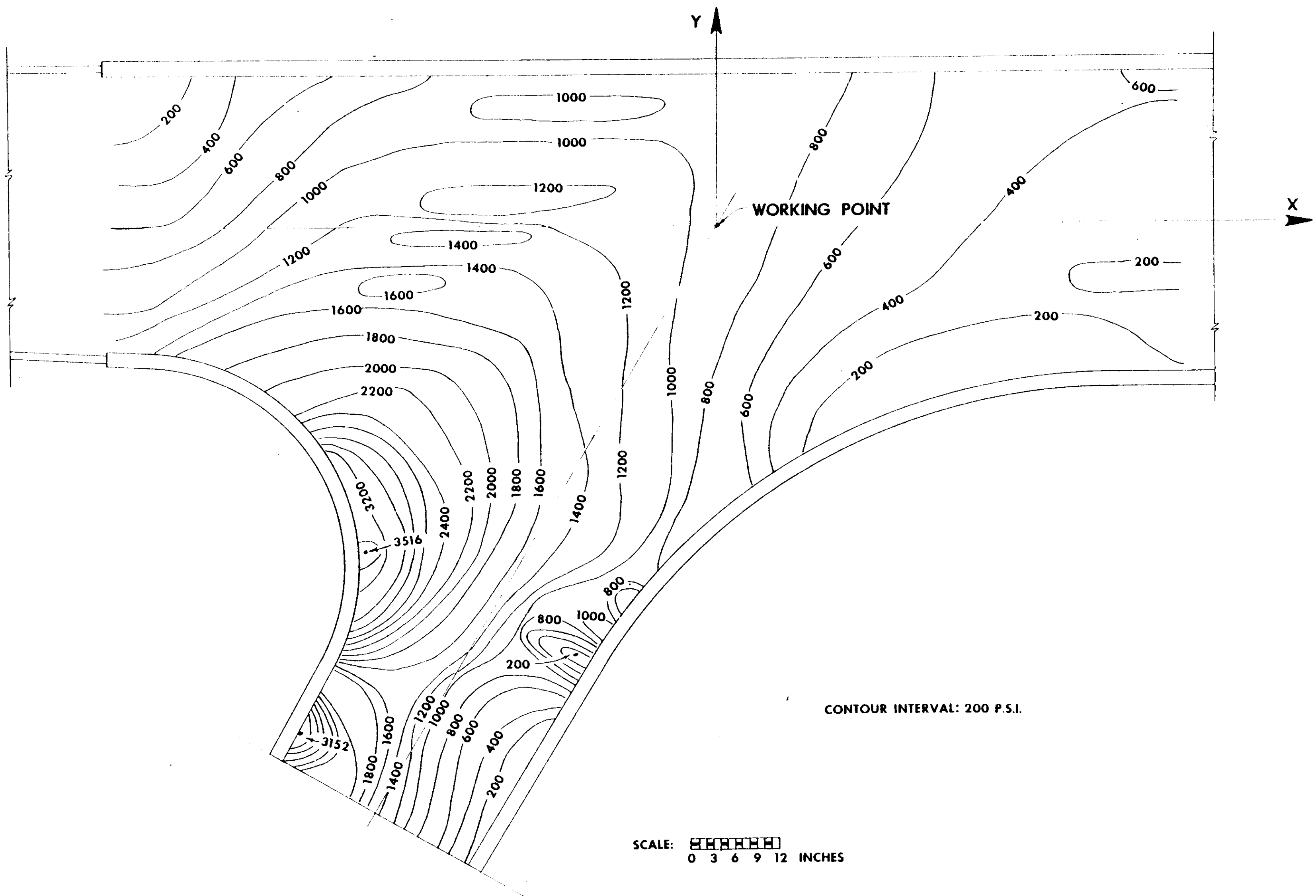

Figure 16. Stress Contours in Haunch Region Without Stiffeners For Vehicle at 35%.

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

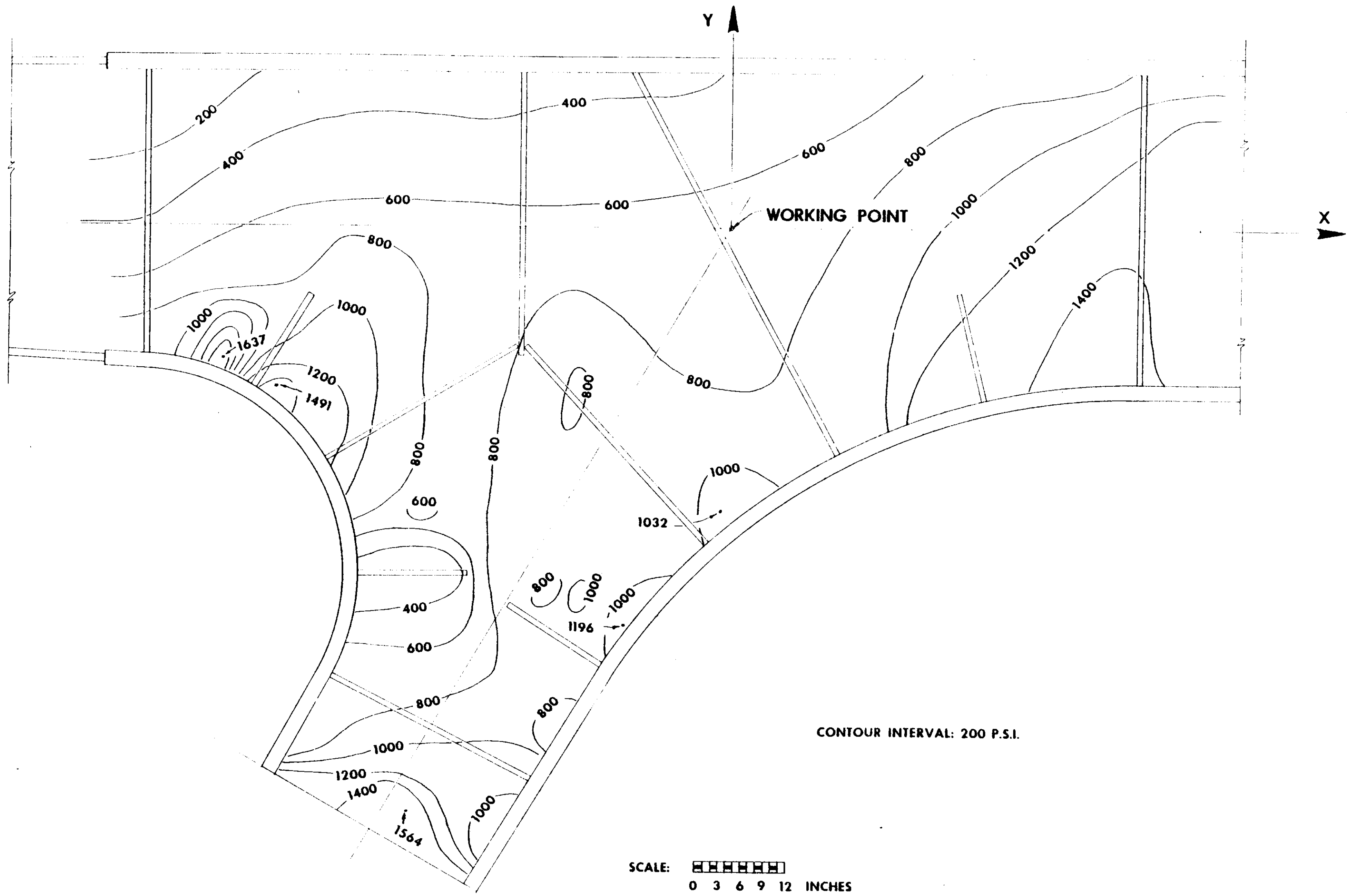

Figure 17. Stress Contours in Haunch Region With Stiffeners For Vehicle at 42%.

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

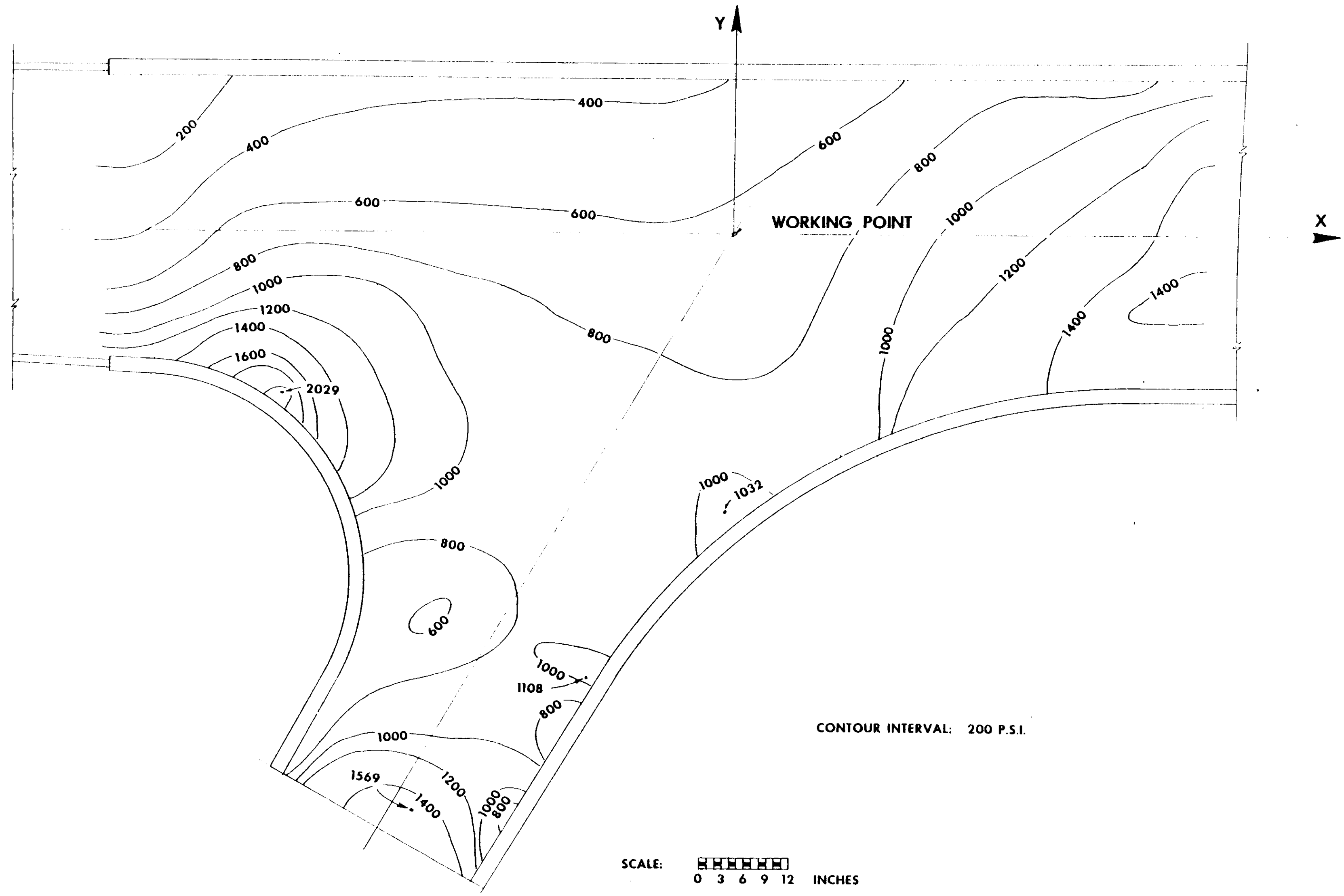

Figure 18. Stress Contours in Haunch Region Without Stiffeners For Vehicle at 42% .

(1 in. = 0.0254 m; 1 psi = 6895 Pascals)

Effect of Web and Flange Thickness on Stress Levels

Displayed in Figure 19 are eight critical regions in which minimum compessive stresses will most probably occur. These eight were chosen based on the areas of high stress concentrations illustrated in Figures 13 through 18.

A number of analyses were made of the finite element model of the haunch with stiffeners for various web and flange thicknesses with the vehicle at location 28%. In Table 2 is a summary of the results; flange thicknesses ranged from 1.0 in. (0.0254 m) to 2.0 in. (0.0508 m), while web thickness varied from 3/8 in. (0.0095 m) to 1/2 in. (0.0127 m). Note that there appears to be no recognizable pattern of stress magnitudes. The highest stress occurs in different regions for the various web and flange thicknesses. Generally, the stresses in almost all of the eight regions for a constant flange thickness increased as the web became thinner – which is an expected result. A noteworthy observation is that for a constant web thickness, the stress levels within all eight regions of the haunch do not necessarily increase as the flange thickness decreases. One possible explanation of this phenomenon could be the redistribution of stresses due to the reduction in flange thickness.

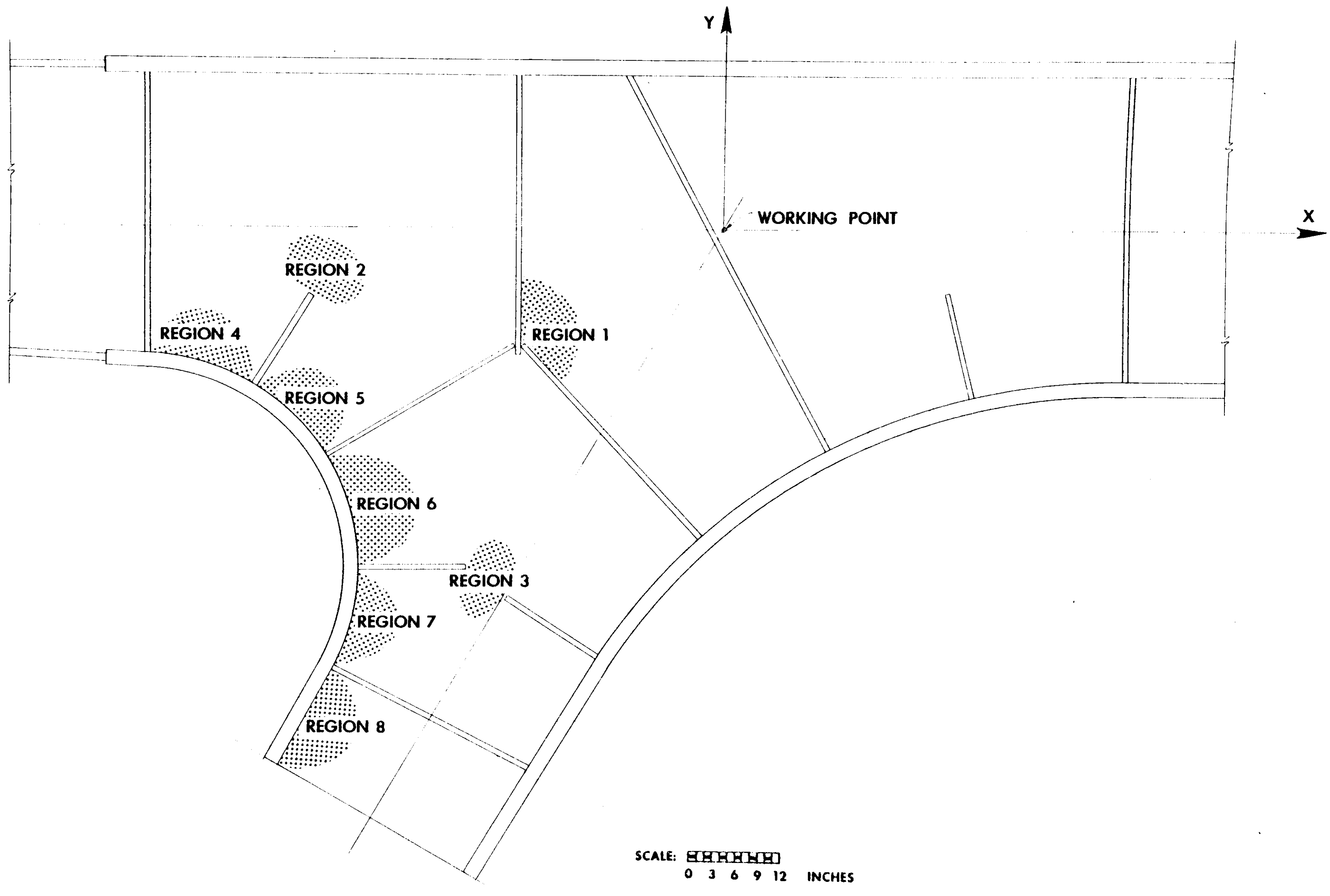

Figure 19. Critical Stress Regions in Haunch.

(1 in. = 0.0254 m)

Table 2

ANALYTICAL MINIMUM PRINCIPAL STRESS (psi) IN CRITICAL REGIONS OF HAUNCH FOR VARIOUS WEB AND FLANGE THICKNESS FOR LIVE LOAD
(1 psi = 6895 Pascals)
(1 inch = 0.0254 m)

Critical* Region	Flange thickness (inch)/Web thickness (inch)				
	2/1/2	2/3/8	1 1/4/1/2	1/1/2	1/3/8
1	-2835	-3905	-2921	-2983	-4050
2	-2125	-2842	-2115	-2128	-2825
3	-2858	-3890	-2720	-2664	-3619
4	-2235	-2837	-2463	-2592	-3268
5	-1867	-1630	-2167	-2622	-2943
6	-2794	-3391	-3162	-3815	-5001
7	-2900	-3429	-4007	-4687	-5116
8	-3126	-3374	-2301	-5076	-5537

NOTE: Bridges on roller supports at end abutments.
*For critical regions in haunch see Figure 19.

Stresses Due to Dead Load

The analysis of haunch stresses due to live load indicated regions of stress concentration which might possibly be stressed beyond the allowable limit when the total dead load plus live load was considered. Hence, a limited study was undertaken to determine approximate peak stresses in the haunch when the dead load was included.

Summarized in Table 3 are the levels of minimum compressive stresses (extracted from SPAR computer output of the analytical haunch model) due to the dead load of the bridge in the eight critical regions of the haunch (see Figure 19), with and without stiffeners. Note, as demonstrated from the contour plots of the live loads, that the absence of stiffeners does not necessarily increase the minimum compressive stress in all eight regions. Another worthwhile observation is that the dead load stresses obtained from computer runs based on the assumption of the bridge being pin supported at the end abutments are all lower in magnitude than the results based on the assumption of a roller support. Thus, it is obvious that the internal forces about the haunch extremities are changed significantly whenever horizontal movement of the bridge is restricted.

From Table 3, the minimum principal compressive stress due to dead load for the haunch with stiffeners was 9,784 psi (67.46 MPa) for the bridge on roller supports at the end abutments, while the largest stress for the haunch without stiffeners was 11,127 psi (76.72 MPa). Correspondingly, the largest live load stresses were 3,147 psi (21.70 MPa) and 3,516 psi (29.24 MPa) for the haunch with and without stiffeners, respectively. The worst possible combination of dead load plus live load for these two cases would simply be the sum of the two — say, approximately 13,000 psi (89.64 MPa) and 14,700 psi (101.36 MPa) in compression — which are well below the maximum allowable stress of 20,000 psi (137.90 MPa).

TABLE 3

ANALYTICAL MINIMUM PRINCIPAL STRESS (psi) IN CRITICAL REGIONS OF HAUNCH FOR DEAD LOAD (1 psi = 6895 Pascals)

CRITICAL* REGION	WITH STIFFENERS SUPPORT CONDITION** ROLLER	WITH STIFFENERS SUPPORT CONDITION** PINNED	WITHOUT STIFFENERS SUPPORT CONDITION** ROLLER	WITHOUT STIFFENERS SUPPORT CONDITION** PINNED
1	-4294	-3824	-5887	-4952
2	-6035	-5050	-5756	-4754
3	-4298	-3892	-3461	-3178
4	-8186	-7790	-8450	-8012
5	-9784	-9013	-11127	-10254
6	-8512	-7734	-8660	-7764
7	-1581	-1394	-1988	-1159
8	-1452	-1446	-854	-846

*For critical regions in haunch see Figure 19.

**Refers to support condition at end abutments.

COMPARISON AND DISCUSSION OF RESULTS

The determination of stresses in the haunch region due to live load and dead load, presented in the previous section of this report, provides a basis for comparison with experimentally measured stresses and for evaluation of existing design concepts and procedures. This section describes this aspect of the study. The comparison of analytical and experimental data also provided a means of validating the haunch model developed. Thus, assuming the model capable of predicting accurate stresses, the reliability of more approximate design procedures could be evaluated and appropriate design criteria discussed.

Comparison of Experimental and Analytical Stresses

Shown in Table 4 is a comparison of the principal stresses in the web obtained from the experimental study and those from the finite element analysis of the haunch. Both support conditions of the bridge at the end abutments are considered. Experimental results for vehicle location 28%, unfortunately, were not available.

As can be noted from Table 4, most of the analytical results compare favorably with the experimental results. The experimental results appear to be bounded by the analytical results for pinned and roller supports at the end abutment. The experimental stresses do, however, appear to agree more closely with the analytical stresses for the pinned support rather than for the roller support. For the live load considered, it appears that perhaps there is some restriction of horizontal movement of the bridge at the end abutments. This may be due to either of two reasons: (1) frictional resistance of the girder against the bearing plate at the end abutments may be restricting horizontal movement, or (2) the relative magnitude of the live load may be so small, in comparison to the dead load, that most of the lateral movement of the bridge is taken up by the rotation of the legs at the interior supports.

Shown in Table 5 is a comparison of stresses for the experimental and analytical studies for the gages on the upper and lower flanges and on the stiffeners. At vehicle location 42%, the experimental and analytical results are in reasonably good agreement, with most of the experimental values being bounded by the analytical results for pinned and roller supports at the end abutments. However, at locations 28% and 35%, the results of the analytical study compare satisfactorily with the experimental results for some gages but not favorably for others. The discrepancy of the live load stresses comparing well at vehicle location 42% and not too favorably at 28% and 35% may be the result of a number of factors. For example, the experimental results were measured manually from oscillograph tapes and some discrepancies in peak values of stresses may have been introduced. Still, it is likely that these experimental values represent a good approximation to the actual stress values. Also, the stresses obtained at vehicle locations 28% and 35% may be more sensitive to the degree of restraint of the legs of the bridge than those of vehicle location 42%. The bridge used for the analytical results modeled the legs as being pinned at their base. The actual degree of restraint of the legs is not known, although the pinned support appears to be the more rational assumption.

TABLE 4

EXPERIMENTAL AND ANALYTICAL LIVE LOAD STRESSES (psi) IN HAUNCH WEB AT GAGE LOCATIONS (1 psi = 6895 Pascals)

Rosette Gage	Principal Stress	Vehicle Location: 28% Experimental Stress	28% Analytical Stress, Support Condition*: Roller	28% Analytical Stress, Support Condition*: Pinned	35% Experimental Stress	35% Analytical Stress, Support Condition*: Roller	35% Analytical Stress, Support Condition*: Pinned	42% Experimental Stress	42% Analytical Stress, Support Condition*: Roller	42% Analytical Stress, Support Condition*: Pinned
R1 & R2***	σ MIN	**	-983	-821	-433	-1000	-447	-460	-662	-380
	σ MAX		179	95	259	352	117	-86	57	284
	τ MAX		581	458	346	676	282	187	360	332
R3	σ MIN		-2125	-1883	-1095	-1943	-1089	-868	-1415	-806
	σ MAX		-176	-234	-457	70	-95	4	-42	374
	τ MAX		974	824	319	1006	497	436	687	590
R4	σ MIN		-1504	-1395	-1454	-1328	-895	-926	-848	-1058
	σ MAX		1109	931	166	843	192	278	-73	-141
	τ MAX		1307	1163	810	1085	544	602	387	458
R5	σ MIN		-2006	-1821	-1418	-1904	-1230	-828	-920	-650
	σ MAX		144	84	-123	2	-224	-534	-304	71
	τ MAX		1075	952	648	952	503	147	308	361
R6	σ MIN		-1068	-972	-1141	-1125	-678	-817	-704	-701
	σ MAX		601	493	141	548	89	-401	-23	-7
	τ MAX		835	733	641	836	384	207	341	346
R7	σ MIN		-302	-308	-699	-569	-531	-1369	-922	-839
	σ MAX		110	42	-238	201	-37	445	379	79
	τ MAX		206	175	231	385	247	907	651	459
R8	σ MIN		-411	-478	-893	-396	-512	-1808	-1120	-1347
	σ MAX		192	80	-440	334	-116	-388	276	-223
	τ MAX		302	279	227	365	198	710	698	433

* Refers to support condition of bridge at end abutments
** Experimental data unavailable
***Average value of stress at Rosettes 1 and 2 taken

TABLE 5

EXPERIMENTAL AND ANALYTICAL LIVE LOAD STRESSES (psi) IN HAUNCH AT GAGE LOCATIONS (1 psi = 6895 Pascals)

Gage	Vehicle Location								
	28%			35%			42%		
	Experimental Stress	Analytical Stress Support Condition*		Experimental Stress	Analytical Stress Support Condition*		Experimental Stress	Analytical Stress Support Condition*	
		Roller	Pinned		Roller	Pinned		Roller	Pinned
L2	75	-828	-779	-414	-638	-452	-546	-655	-376
L3	-1653	-1067	-959	-1025	-912	-527	-33	-412	139
L4	-2316	-1318	-1135	-1317	-1265	-636	179	-260	620
L5	-1430	-608	-511	-896	-608	-275	-82	-105	400
L6	567	537	421	-427	447	49	-1388	-179	-731
L7	466	1100	835	-499	868	-37	-1453	-555	-1812
L8	-130	497	288	-658	400	-324	-951	-748	-1731
L9	-528	16	-158	-862	144	-422	-983	-805	-1581
L10	-571	-222	-400	-755	159	-409	-848	-650	-1410
L11	-576	-83	-533	-451	60	-98	-593	-104	-317
U2	67	63	111	0	36	74	0	107	172
U3	-48	-52	-33	0	-130	-54	0	-21	94
U4	-17	58	66	-50	180	219	266	-100	-31
U5	0	61	93	0	-55	57	36	7	167
S1	-815	-856	-744	-492	-812	-425	-34	-261	285
S2	-694	-746	-653	-425	-735	-408	-90	-325	141
S3	-53	265	210	-212	255	67	-362	-30	-290
S4	-119	238	158	-170	304	36	-238	-94	-460
S5	-93	132	79	-168	174	-9	-317	-52	-305

* Refers to support condition of bridge at end abutments

In spite of the variations observed between certain of the experimental and analytical stresses, the agreement for the most part was surprisingly good. Factors that could not adequately be incorporated in the analytical model, such as residual stresses from fabrication and induced loads transmitted through the diaphragms, may have caused some of the variation. Nevertheless, the calculated stresses were of the same order of magnitude as the experimental values, and when stresses were also calculated using slightly different assumptions for the model, the analytical stresses frequently bounded the experimental values.

Thus, it is felt that the haunch model developed in this study, as well as the haunch loading used, accurately represents the actual structure and may be used with confidence for design purposes in future similar structures.

Comparison of Stresses from Analytical Model and from Original Design Procedure

One of the primary objectives of this study was to develop an analytical model capable of predicting accurate stress levels within the haunch portion of the bridge. With the model developed, a limited evaluation of current design procedures was carried out.

The procedure for determining web stresses used in the design of the haunch of the studied bridge was based on an approximate analysis of wedge-shaped beams proposed by Olander.(1) Olander's analysis is an approximation to the exact solution of wedge-shaped beams first introduced by Osgood.(4)

The technique proposed by Olander was originally developed for computing the stresses in the corners of rigid frames where the extreme fibers were not parallel, as typified by the corner in Figure 20(a). It has subsequently been extended to predict stresses in haunches of rigid frame bridges.

Olander's method consists of taking a circular section, such as section A-B in Figure 20(a), that cuts the extreme fibers at right angles, and developing the section, as shown in Figure 20(b), to obtain an area A and a moment of inertia I. Forces to the right of section A-B are resolved into the components P_o, V_o, and M_o about the point 0, the center of the arc. The force P_o passes through the center of gravity of section A-B, V_o is normal to P_o, and M_o is the moment of the forces about point 0.

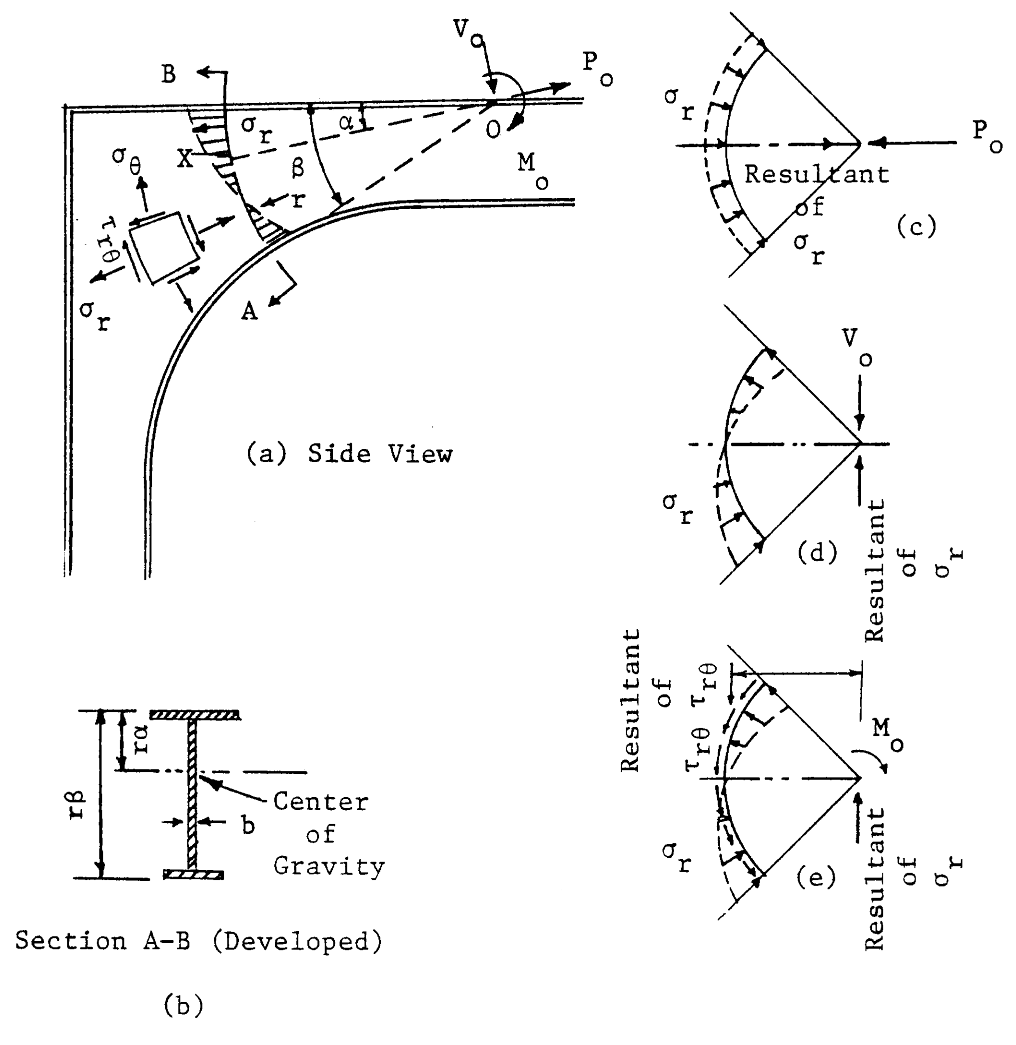

Figure 20. Olander's Procedure For Rigid Frame Corners. (From Ref. 1)

On section A-B, where r is the radius of the arc, the total shear is

$$V = M_o/r, \quad (1)$$

and the resultant moment is given by

$$M = M_o + V_o\, r. \quad (2)$$

The corresponding stresses on the section are thus calculated as

$$\tau_{r\theta} = VQ/Ib, \quad (3)$$

and

$$\sigma_r = \frac{P_o}{A} \pm \frac{Mc}{I}. \quad (4)$$

For the wedge angle < 45°, the stresses given by equations (3) and (4) are within 5% of those given by Osgood's solution.[4]

To provide an evaluation of the feasibility of using Olander's design method for the haunch, a comparison was made of minimum principal live load stresses obtained from the finite element model and those from Olander's method. Stresses were calculated and compared at three arc locations as shown in Figure 21.

The location of the vehicle for arcs A-A and B-B was at 28%, while that for arc C-C was at 42%. These locations were chosen so as to avoid external loading within the wedge section.

Table 6 presents stresses calculated by Olander's procedure and those determined analytically from the finite element model for five points along each arc. The five points selected were approximately equally spaced as shown in Figure 21.

At the points along each arc section, the minimum principal stress was derived from the finite element model by interpolation between contour lines. Only the stress contours for live load on the haunch region without stiffeners were considered. At the same points of each arc section, radial and shear stresses were computed by Olander's method of analysis, and corresponding principal stresses were calculated. As with previous stress calculations, composite behavior was assumed in both cases.

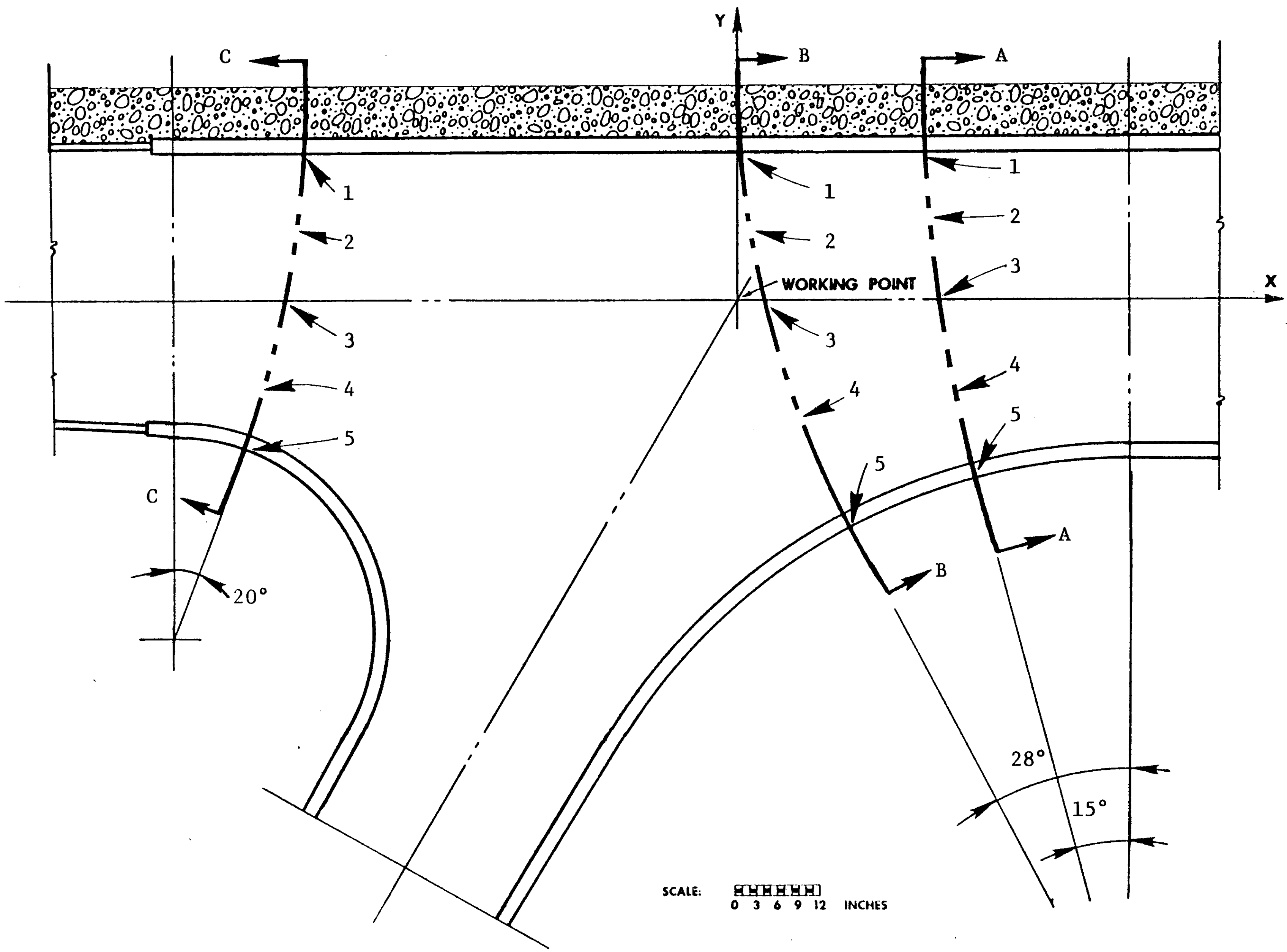

Figure 21. Locations For Stress Calculations By Analytical and Olander's Methods.

(1 in. = 0.0254 m)

TABLE 6

COMPARISON OF MINIMUM PRINCIPAL LIVE LOAD STRESSES (psi)
(1 psi = 6895 Pascals)

Arc Points	Section*					
	A-A**		B-B**		C-C**	
	σ min		σ min		σ min	
	Analytical	Olander	Analytical	Olander	Analytical	Olander
1	-100	-50	-360	-50	-300	-110
2	-180	-110	-500	-150	-450	-315
3	-320	-150	-680	-160	-700	-390
4	-420	-225	-850	-190	-1100	-535
5	-590	-330	-900	-275	-1700	-790

*Refer to Figure 21.

**Sections A-A & B-B for vehicle location 28%; Section C-C for vehicle location 42%.

As can be seen from the results in Table 6, analysis using the finite element model predicts stresses that are generally two to three times larger than those obtained from Olander's analysis. One explanation for this difference may be that Olander's original procedure considered only a two-member haunch whereas a three-member haunch was considered here. It should be kept in mind that the results in Table 6 were calculated for only one haunch geometry without stiffeners and only one vehicle loading. Nevertheless, the results from this limited study clearly indicate that stresses predicted from the procedure suggested by Olander may be significantly lower than those actually produced in the haunch. This conclusion, of course, assumes that the results from the analytical model are a reasonable approximation to the true stresses. Based on this evidence, designers should exercise extreme care in using Olander's method to predict the need for web stiffeners in the haunch.

Since the haunch in the actual structure was designed based on Olander's procedure, it might seem that actual service loads should overstress the haunch, whereas experimental measurements and analytical results indicate service stresses well below the allowable. However, it should be recalled that in the design of the structure, several factors served to increase the factor of safety. For example, the design analysis assumed noncomposite moment of inertia in the negative moment areas, whereas the finite element analysis assumed composite moment of inertia throughout. In many cases the live load used in the design as prescribed by AASHTO produced more severe stress conditions in the haunch than the single vehicle used in the finite element analysis. Also, the original design utilized a distribution factor of 0.834, while a factor of 0.3887, based on experimental data, was used in this stress analysis.

Factors other than dead load plus live load must be considered in the design of the haunch. Impact, temperature stresses, camber, wind stresses, residual stresses, and settlement are just a few items that must be taken into account.

Nevertheless, if haunch stresses had been calculated using the analytical model developed herein, but with the live load and distribution factors used in the actual design, the stress magnitudes may have been well above the allowable. Fortunately, these design conditions rarely occur in service. Additional research into this topic would certainly seem warranted.

SUMMARY AND CONCLUSIONS

The primary purpose of this study was to develop an analytical model of the haunch region of a rigid frame bridge, including the associated methodology for stress analysis, and to determine the magnitude and distribution of stress in a typical haunch configuration. A finite element model was adopted for the haunch configuration and loadings determined from an analysis of the entire frame. For various live load locations, stresses and displacements throughout the haunch region were calculated. Stresses in the web and flanges were compared with experimentally measured stresses available from a field test of the structure on which the analytical model was based. The comparison tended to validate the reliability of the model and method of analysis.

Subsequent stress analyses of the haunch with and without stiffeners were conducted. The effect of web and flange thickness on the stress distribution in the haunch was also examined. Results from the analytical study were used to evaluate analysis procedures used in the design of haunches.

Stresses determined from the analytical study were found to compare favorably with corresponding stresses measured experimentally both in the web and flanges of the haunch. Based on these results, it was concluded that the haunch model developed would yield reliable stresses for the geometry considered and could be used with confidence to study the effect of various parameters on haunch stresses. Other conclusions based on this investigation are given below.

1. Placement of stiffeners within the haunch region may induce areas of high stress concentration at the end and junctions of stiffeners. This possibility may be a factor in designing against fatigue, and extreme care should be exercised in placing and locating stiffeners in the haunch region of the rigid frame bridge.

2. Variations in web thickness had a significant effect on stress levels within the haunch, while variations in flange thickness had a relatively small effect. By decreasing the web thickness with constant flange thickness, stress levels were generally increased in the web of the haunch. No recognizable pattern of stress distribution resulted from varying the flange thickness with the web thickness being kept constant.

3. Current analysis procedures of the haunch region are generally based on a method proposed by Olander. A comparison between stresses calculated in this study and those determined using Olander's method indicated that the use of Olander's procedure may be on the non-conservative side, and that designers should be extremely cautious in the use of this procedure.

While considerable valuable information was obtained during the course of this investigation, additional research certainly seems warranted. This is particularly true with regard to optimum stiffener location and more exact design procedures for the haunch region.

ACKNOWLEDGMENTS

The research reported here was conducted under the general supervision of Jack H. Dillard, head, Virginia Highway and Transportation Research Council. W. T. McKeel, Jr., principal research engineer, and H. L. Kinnier, faculty research engineer, offered valuable suggestions during the analytical study and the preparation of the report.

The authors acknowledge the assistance of the Research Council and the Federal Highway Administration in making available experimental results from the field testing of the rigid frame bridge modeled in the study. In addition, the assistance of personnel in the data section of the Council and the computer center of the University is gratefully acknowledged. Finally, special appreciation is expressed to those many individuals who assisted in the preparation of the report, in particular to Paul Hughes and Allen Baker for the graphics.

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