THE INVESTIGATION OF THE TIMBER CONCRETE COMPOSITE BRIDGE DECKS DESIGN

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Yu Chi Lin

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THESIS

This is to certify that the

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THE INVESTIGATION OF THE TIMBER CONCRETE
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THE INVESTIGATION OF THE TIMBER CONCRETE COMPOSITE BRIDGE DECKS DESIGN

BY

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Contents

Introduction

The purpose of this investigation of the timber-concrete composite design is to take advantages in the locations where the timber is at low cost and the cement is rather easy to obtain. In other words, this investigation aimed specifically at a low cost construction with a comparatively long service.

The timber-concrete composite construction presented herein is aimed at reducing construction costs, through primarily reduction of the steel cost and secondly of on-the-job labor.

In this connection, some of the features by which this design will accomplish construction at lower costs are presented in the following:

1. The strength properties of the concrete wearing surface in combination with the timber has been utilized to support the load.
2. There is no need for falsework or form-work
3. Details are such that the most available grades and sizes of lumber may be used; this is the most economical way to buy lumber.
4. Steel or hardware is held to a minimum.
5. Span length is variable, permitting a selection for an economic balance between deck and substructure
6. Loading and roadway width may be selected to suit local conditions.

7. Panel units of the laminated deck may be shop fabricated and assembled.

Labor costs at present constitute nearly 60 per cent of the total cost against 40 per cent for materials. A reduction in labor costs of one-third would thus effect a saving of 20 per cent of total cost not to mention the reducing costs of materials.

General Treatment of the Composite Timber-Concrete Construction.

The timber-concrete composite construction can be briefly described as a laminated wood slab of treated plank rigidly interlocked with a heavy concrete mat. This assembly is transformed into a unit adequate for bridge decks designed for any standard highway loading, or for piers, docks, warehouse, ramps or other structures requiring heavy duty floor.

The wood slab or timber base is usually of 2 inch plank on edge, 4 to 12 inches in width, depending on span and load. The alternate plank are of different width, so to have a raise of 1-3/8 inch to 2 inches to form the longitudinal grooves. The same effect may be obtained by using plank of two widths and alternating them in the assembly. Small trapezoidal steel plates-shear-developers are driven into pre-cut transverse slots in these grooves, forming the shear connection between wood and concrete. Uplift spikes or dowels are driven into the raised laminations on from 2 to 4 foot centers to restrain the tendency of concrete to curl. These are not taken into account in computing shear reinforcement, but do add to it.

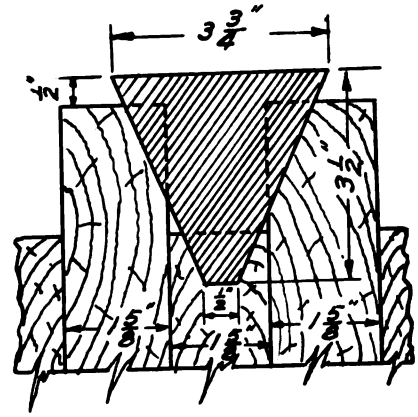
The principal problem in the design of slabs, of whatever nature, is that pertaining to lateral distribution of a concentrated load. Obviously, one must know what load is to be carried on the particular width of slab used as the unit on which the calculations are based. The elements that enter into the problem are numerous and difficult to evaluate, so dependence has been put very largely on empirical formulas.

Laminated wood pieces spiked together in accordance with a given pattern and securely bonded to a concrete mat function effectively in carrying transverse stresses. Where large scale installations are involved it will be advantageous to employ prefabricated panels of uniform widths made up of wood laminations rigidly fastened together with spiral dowels and accurately bored for driving of additional dowels in the field, to join the separate panels in one whole slab assembly. The dowels are effectively bonded in the wood because their spirally grooved ridges lie outside the diameter of the lead hole in which the dowel is entered. Consequently, they provide effective transverse reinforcement for the composite slab.

For ordinarily, the span lengths used in trestle construction all plank should be full panel length, laid immediately on the support in a direction parallel to the roadway center line. Where in multiple span construction, a continuous, unbroken deck is often desirable, one-third of the laminations are butt jointed over the support centers and one-third at or near each quarter-span point, the joints being made in regular rotation. The theory back of this is no matter what kind of arrangements are possible, it is necessary that at least two-thirds of the strips should be continuous across splice joints. In the above arrangement there are two-thirds of the strips extend across and are effective at any of these points, and a full timber section extends throughout the midspan reach between quarter points.

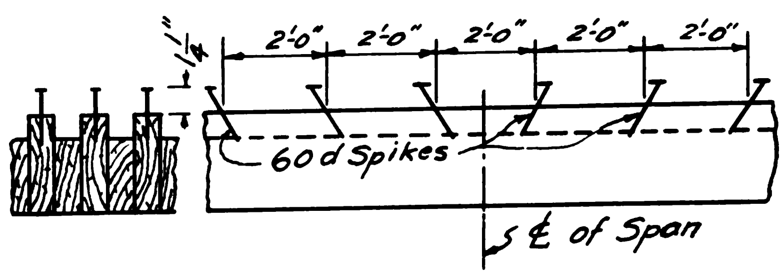
Shear Developers and Uplift Spikes.

The shear developers are made of steel plate in trapesoidal shapes. The standard dimensions are 3-3/4 in. on the top and 1/2 in. at the bottom. A height or altitude of 3-1/2 in. with a thickness of 3/32 in. They are driven in the pre-cut slots of the timber base, their tops protrude 1/2 in. above the timber so as to engage the concrete mat.



The spacing of the shear developers is from the plans and should be followed accurately. However, a slight shifts in spacing to avoid any knots in the timber which would prevent their effective seating.

The uplift spikes are also driven into the raised timber and their protruding heads are embedded well in the concrete mat to preclude any tendency to vertical separation between the concrete and the timber due to their curling action of concrete induced by differential temperatures on its top and bottom surfaces.



Concrete Mat.

The concrete to be poured on top of the laminated timber base form a mat over the base and is only reinforced for shrinkage and temperature stresses to prevent cracking. Generally the mat reinforcement consists of a mesh made up of 3/8-in. or 1/2-in. round bars placed from 9 to 12 inches on centers both longitudinally and transversely. However, in continuous spans, where negative moment occurs over the supports, sufficient steel is added to take care of the stresses calculated. For reinforcing these stresses generally the bars of about half span length are staggered between the longitudinal bars. In case the longitudinal bars should call for splicing, they are preferably to be spliced at mid-span and not over a support. The alternate bars should extend across the joints in the timber base near the quarter points.

Experimental Treatment of the Composite Timber-Concrete Construction.

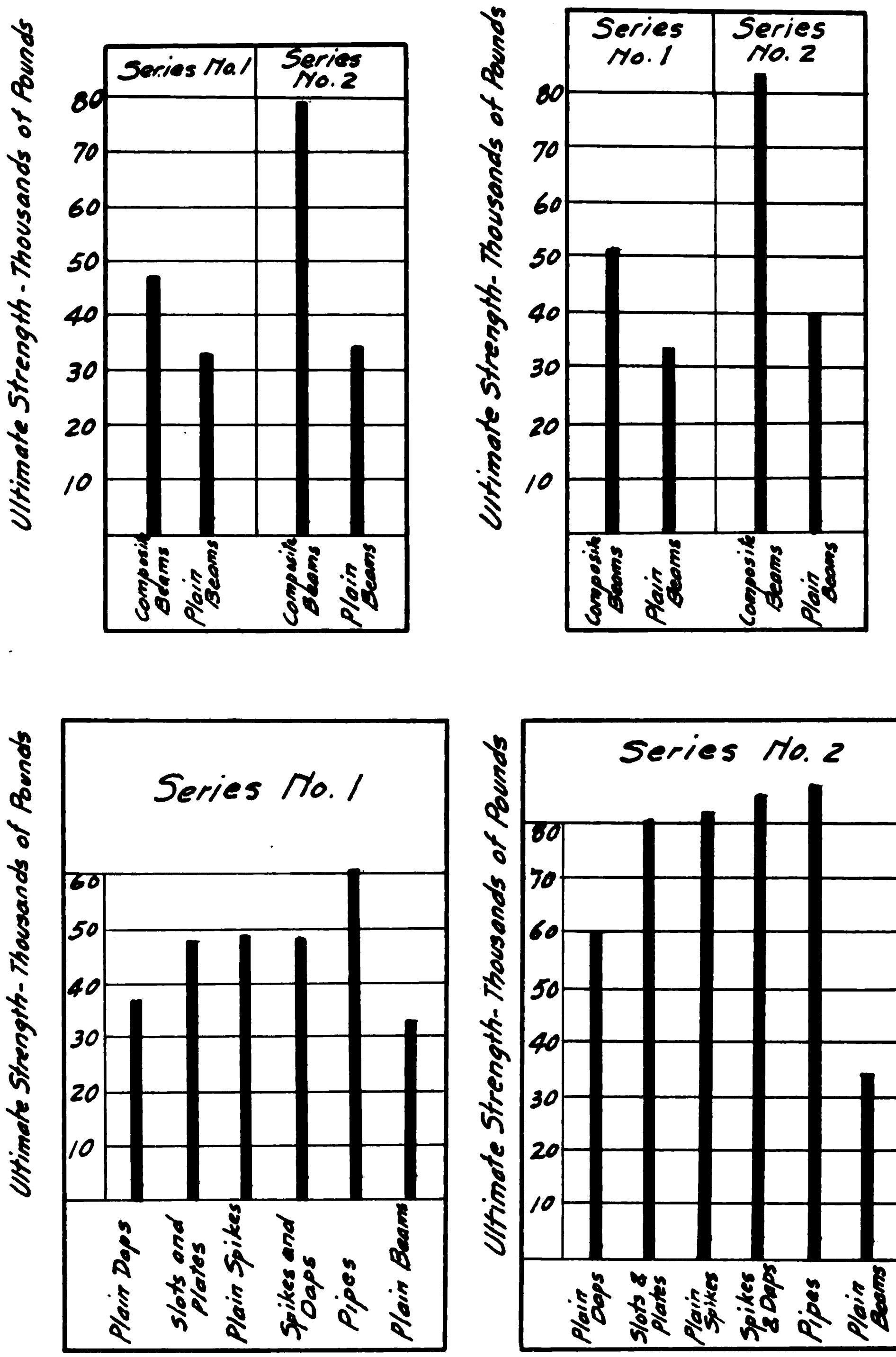
Comparison of Ultimate Strength:-

It is essential, in the first place, to see if there is any benefit resulting from the use of composite construction. To start with we first compare the ultimate strength of the composite beams with that of plain timber beams of corresponding stem dimension. At once, we see that this is a fair comparison; for the reason that if the concrete deck were placed upon the timber base without any connection between them, there would be no increase in strength because of the deck. In fact, if the timber base were removed from underneath the deck it would not be able to withstand its own weight for an average span of, say, 20 feet. From the above, we see, actually the deck would be a burden on the timber base or stringers. However, with proper connection or adequate shear reinforcing, as tested by Oregon State Highway Department as in this connection, it shows that the strength of the composite beam is nearly double that of the corresponding plain beams. Therefore it concludes that "If a most suitable types of shears reinforcings were employed that the use of the composite type of construction will produce structural members having an ultimate strength at least twice that for the same materials and sizes used independently".*

*Technical Bulletin No.1, Oregon State Highway Department, pp. 71

Comparison of Ultimate Strengths of Composite

Beams and Plain Beams

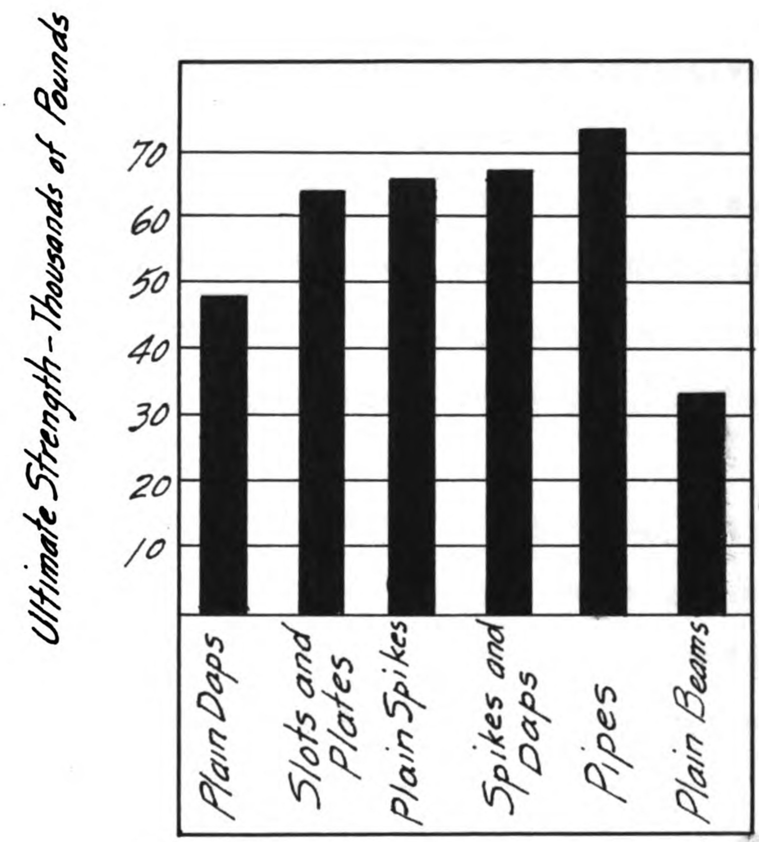


Note: These figures are taken from Technical Bulletin No. 1, Oregon State Highway Department, pp. 72

Comparison of Ultimate Strength for

Each Type of Shear Connection Average for

Series No. 1 and No. 2



Note: This figure is taken from Technical Bulletin No. 1, Oregon Highway Department, pp. 73

Comparison of the Different Types of Shear Connections.

Since the composite design is the combination in a structural member of two elements having different mechanical properties, therefore, it requires a definite study about the behaviors of the horizontal and vertical shears as well as the end slip at the junction of the two different elements under different loadings including the alternated load. With the aid of this study a better and more effective shear connections can be provided so as to produce the best results.

In this connection, I would like to quote some the work carried by Mr. R. H. Baldock and Mr. C.B. McCullough of Oregon State Highway Department on Loading Tests on a New Composite-Type Short-span Highway Bridge Combining Concrete and Timber in Flexure.

The tests have been carried on by using two sizes of T-beams. Series No. 1 includes all the beams of smaller size; that is, those with 6" x 15" concrete flange and 4" x 14" timber webs. Series No. 2 includes all of the beams of the larger size; that is, those with 6" x 24" concrete flange and 6" x 16" timber web. All of the timber used in the webs or stems of the beams graded somewhat under structural Douglas fir, the sticks being thus selected in order to represent the lowest quality likely encountered under actual construction conditions. All of the concrete was of Class "D" mix as defined in the "Specifications for State Highway Bridge Construction" for the State of Oregon. This is a nominal 1:2:3 mix designed to produce concrete which will withstand not less than 2,800 pounds per square inch at 28 days and is the class generally used by the Oregon State Highway Department for bridge decks.

Five types of shear connections were used, and two beams in each series for each type of shear connection were tested.

The first type of shear connection - type 1 - consisted of 3/8-in. by 8-in. round spikes, driven five inches into the timber web. Holes slightly smaller than the spikes were bored before the spikes were driven in order to prevent splitting of the timber. The spikes were spaced every two inches between the supports and the load points and every five inches between the load points. Furthermore the spikes were staggered in three rows so as to further reduce the tendency toward splitting of the timber. After the spikes were driven, the top three inches extended up above into the concrete when it was poured around them.

Type 2 consisted of daps in the wood, together with the keys formed in them by the concrete. Between the supports and the load points, the daps were five inches long with five inches between the daps. Between the load points, the daps were six inches long with twelve inches clear between daps.

Type 3 was a combination of Type 1 and Type 2 with slight modification. The spacing of the daps was the same as Type 2, but the spikes were spaced two in each dap, one between daps between the supports and the load points, and none between the load points. The daps were sawed out, and the spikes were placed before the timber was set up for pouring the concrete.

Type 4 made use of pipes in place of spikes and in much the same manner as the spikes in Type 1. Two and one-half-inch pipes (outside diameter), four and one-half inches long, were driven two and one-half inches into the timber and allowed to extend up into the concrete flange a distance of two inches. The pipes were spaced at six and one-half-inch centers between the supports and the load points and twelve and one-fourth-inch centers between the load points. Holes one-sixteenth inch smaller than the pipe diameter were bored before the pipes were driven.

Type 5 consisted of 4" x 3/4" x 4" steel plates driven into slots in the timber one inch deep and allowed to extend three inches into the concrete. They were spaced eight and one-half inches apart between the supports and the load points and fourteen inches between the load points.

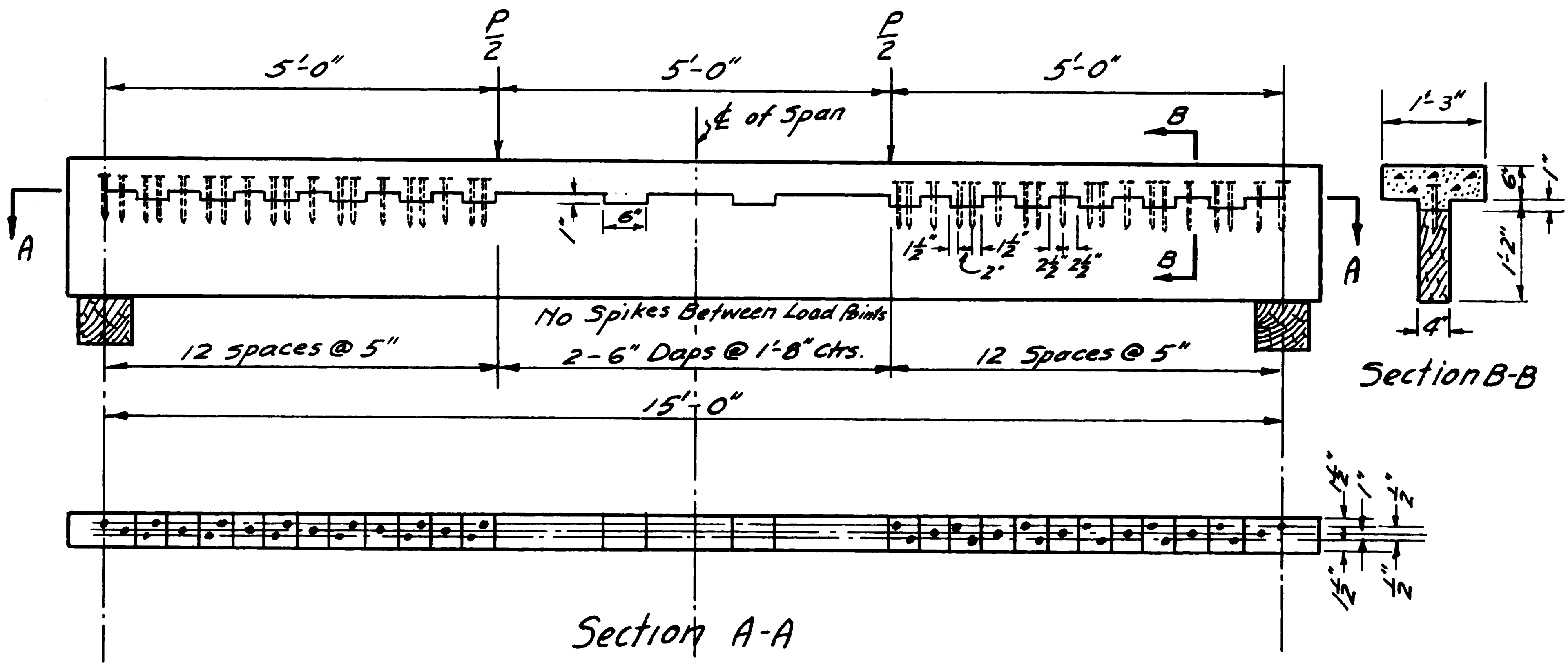
From the above arrangement of shear connections, the results were summing up by Mr. Baldock and Mr. McCullough as follows:

"First, the plain dap type of connection is wholly inadquate to develop the strength of the rest of the beam and should not be used.

Second, slots and plates form a good connection from the standpoint of rigidly, but for the sizes used in these tests, permit the web to fail at low loads by horizontal shear below the bottom of the plates. Since it is not feasible to extend the plates far enough into the timber stem to reinforce it against this type of failure, the slots and plates do not appear to be a suitable type of connection.

Third, the plain spikes form a fairly satisfactory method of reinforcing, but they permit an excessive amount of end slip and, consequently, too much deflection and too much independent action, thereby reducing the strength of the beam.

This leaves (1) the pipes and (2) the combined spikes and daps as the most effective types with little room for choice between them. However, there is still one factor influencing the selection which has not yet been considered; namely, that the combined spikes and daps are somewhat cheaper than the pipes, especially in cases where the connection must be extended well into the web to reinforce it against horizontal shearing stress. All in all, therefore, the combined spike and dap connection - Type 3 - should be accorded first place with Type 4-pipes - a close second".



Recommended Shear Connection

Type 3

Spikes and Daps

Comparison of Deflection.

The matter of deflection will now be considered. It appeared in the above that the use of a rigid connection between deck and timber base would add considerably to the stiffness of structures of this type. At a glance of the following tables will suffice to draw the conclusion that the deflection of a suitable-connected composite beam under a given loading will not be more than 25 per cent of the corresponding deflection for the same sizes and materials used independently.

Comparison of Deflections for Each Pair of Beams

Load	Plain Spikes	Plain Daps	Spikes & Daps	Pipes	Slots & Plates	Plain Beams	Theo-retical
			Types of Shear Connection				
			Series No. 1				
5,000	0.07	0.06	0.05	0.06	0.05	0.26	----
10,000	0.15	0.13	0.11	0.12	0.11	0.59	0.10
15,000	0.23	0.19	0.17	0.19	0.17	0.88	-----
20,000	0.32	0.24	0.24	0.26	0.23	1.18	0.21
25,000	0.43	0.32	0.31	0.33	0.30	1.53	----
30,000	0.55	----	0.37	0.41	0.37	----	0.32
35,000	0.71	----	0.44	0.48	0.43	----	----
40,000	----	----	0.52	0.56	0.51	----	----

All deflections given in inches.

Note:

This table is taken from Technical Bulletin No. 1, Oregon State Highway Department, pp. 76

Comparison of Deflections for Each Pair of Beams

Types of Shear Connection

Series No. 2

Load	Plain Spikes	Plain Daps	Spikes & Daps	Pipes	Slots & Plates	Plain Beams	Theo-rectical
5,000	0.03	0.04	0.03	0.05	0.02	0.02	----
10,000	0.06	0.08	0.06	0.08	0.06	0.41	0.06
15,000	0.08	0.13	0.10	0.11	0.10	0.55	----
20,000	0.11	0.17	0.13	0.16	0.13	0.73	0.12
25,000	0.16	0.21	0.16	0.19	0.16	0.92	----
30,000	0.20	0.25	0.20	0.22	0.20	1.16	0.18
35,000	0.25	0.30	0.25	0.28	0.24	----	----
40,000	0.30	0.34	0.34	0.34	0.28	----	----
45,000	0.35	0.38	0.34	0.37	0.32	----	----
50,000	0.41	0.43	0.39	0.41	0.36	----	----
55,000	0.47	----	0.43	0.45	0.40	----	----
60,000	0.53	----	0.48	0.51	0.45	----	----
65,000	0.62	----	----	0.57	0.51	----	----
70,000	0.73	----	----	0.62	0.57	----	-----

All deflections given in inches.

Note:
This table is taken from Bulletin No.1, Oregon State Highway Department, pp. 76

Comparison of End Slips.

The comparing of the amount of end slip for each different types of shear connections will be next considered. The results of the comparison as tested by Oregon State Highway Department giving the following statement with tables to show the end slips for each type of shear connection:

Comparison of End Slips for Each Pair of Beams

Types of Shear Connection

Load	Plain Spikes	Plain Daps	Spikes & Daps	Pipes	Slots & Plates
			Series No. 1		
5,000	1.6	9.4	0.1	2.3	5.5
10,000	3.1	15.4	0.9	4.6	7.9
15,000	8.1	20.2	2.2	5.8	11.8
20,000	18.6	24.8	3.5	7.1	17.6
25,000	35.4	29.3	5.9	8.8	21.2
30,000	61.0	----	8.5	10.5	31.9
35,000	101.0	----	11.6	11.9	40.2
40,000	----	----	14.8	13.3	48.1

All end slips given in thousands of an inch.

Comparison of End Slips for Each Pair of Beams

Types of Shear Connection

Load	Plain Spikes	Plain Daps	Spikes & Daps	Pipes	Slots & Plates
		Series No. 2			
5,000	----	4.8	----	0.1	-----
10,000	----	9.3	----	0.1	-----
15,000	----	12.1	----	0.3	0.2
20,000	----	15.8	----	1.1	0.2
25,000	----	17.8	----	1.9	0.2
30,000	4.4	20.4	----	3.3	0.3
35,000	12.3	23.5	----	6.0	0.3
40,000	22.0	26.5	1.8	9.2	0.6
45,000	30.8	32.0	4.6	12.0	1.3
50,000	44.0	37.4	14.3	15.1	1.8
55,000	56.8	----	19.0	19.5	3.0
60,000	76.6	----	26.9	24.3	4.0
65,000	101.2	----	----	30.0	8.6
70,000	130.6	----	----	36.2	12.5

All end slips given in thousandths of an inch.

The results of this comparison for beams in series No. 1 as shown in the above tables, indicates that combined spikes and daps give the least slip with pipes next. The other three types show a much greater amount of slip than these two, probably due to the inherent difficulty in getting a tight fit in the cases of the plain daps and the slots and plates and to the tendency to spring and bend in the case of the plain spikes. The same information for the beams in Series No. 2. Slots and plates gave the least amount of slip in this test, with spikes and daps second, and pipes third. The plain spikes hold well for the lower loads, but as soon as this type of connection starts to slip it gives rapidly. The plain daps slip at a fairly high rate from the start and continue on to the point of failure.

It is obvious that the deflections of these beams and the amount of end slip are closely related since the more positive the shear connection the more nearly is complete integral action attained, and, hence the less will be the deflection.

Effects of Alternated Loads

With the composite beam when tested under the alternated load, a certain amount of plastic displacement in connection with the end slip as it does in the static loading. However, there is a partial elastic recovery. This plastic shear displacement is undoubtedly the result of tightening up or seating of the shear connections accompanied by a local crushing of the timber fibers, or of the concrete, or of both. As has been observed by the Oregon State Highway Department that as the loading is alternately applied and released, further successive increments of plastic displacement become smaller as the number of loading cycles is increased. The total cumulative plastic slippage approaches a fixed and definite value as the load alternations are increased indefinitely.

"Tests to destruction of the beams subjected to the application of alternated loads indicated no appreciable loss in ultimate strength as a result of such alternated loading. Even the initial plastic displacements were of comparatively small magnitude, and the additional plastic increments very rapidly decreased to a nearly negligible value. In view of these facts, there appears to be no danger of any material alternation in the manner of stress distribution throughout the cross-section of a well-designed composite beam as a result of slippage at the shear connection".*

Location of Neutral Axis.

If there were no connection between the concrete flange and the timber stem and each section were perfectly free to slip past the other, the top of the flange and the top of the stem would be under compressive stress and the bottom of each section would be under tensile stress with a neutral axis at about the middle depth of each section. On the other hand, if the connection between the two sections could be made absolutely tight and rigid with no spring or slip of any kind, entire integral action could be obtained and there would be but one neutral plane, the same as in any homogeneous beam. However, there was a very small slip between the concrete and timber sections, as previously noted, showing that the true condition lies somewhere between the above two extremes, so it would be expected that there would be two neutral axes

* Bulletin No. 1, Oregon State Highway Department, pp. 81

quite close to each other and to the theoretical axis, determined by assuming an entirely rigid shear connection.

As the conclusion drawn previously by the Oregon State Highway Department indicates that the Type 3 and Type 4 shear connections between the junction of the concrete and timber composite beam is more preferable than the other types of connections. Therefore a study of the neutral axis of the composite beams will accordingly restricted to these two types of shear connections.

A discussion on the shear slip effects the neutral axis of the composite beams in the research by Oregon State Highway Department for the composite design reveals that if the shear slippage were evenly distributed throughout the outer third sectors there would be no slip at the load points; but, proceeding from the load point outwardly, the slip would be cumulative, each increment being added to the preceding increment so that the total end slip, as measured, would be the sum of the slip increments from each load point to the end of the beam. Were this condition to obtain, there would be no slippage between load points, and the two neutral axes would coincide with each other and with the theoretical position as obtained by applying the ordinary transformed section formulae*. This means that if the more positive types of shear connection could be secured the slippage was not sufficient to materially modify the general stress distribution and that within the range of accuracy demanded

* Technical Bulletin No. 1, Oregon State Highway Department, pp. 80

in designs of this character theoretical formulae may be safely employed.

From what has been said, it is permissible to use the ordinary transformed section formulas for the neutral axis in the composite design if the shear connections are of the adequate and rigid connections such as the connection consisting of combined spikes and daps.

Temperature Effect

Before any discussion of the effect of alternated temperatures upon composite design of this type, it appears necessary to arrive, if possible, at some definite understanding concerning the coefficient of thermal expansion for Douglas fir timber parallel to the grain.

All in all, the data in reference to this property of timber are very meager and perhaps somewhat unreliable. Koehler in his "Properties and Uses of Wood" makes the statement that "different investigators are not in close agreement in their results for the thermal expansion of wood". Studies by Hendershot of Syracuse University indicate that the density of wood has little influence on thermal expansion, but that the presence of moisture may increase the thermal coefficient to a considerable extent.

Hendershot's experiments did not include fir timber, and their only applicability to the case in point is by analogy. Furthermore, the data do not include temperatures below freezing, and it is felt that the expansion of the moiture in fairly damp timber as the temperatures drop below freezing may be sufficient to counteract, in part, at least, the contraction due to normal thermal movement in the timber fibers proper.

Mr. J. Elton Lodewick, in charge of the section on Forest Products, United States Department of Agriculture, makes the following significant comment:

"One point, however, should be borne in mind in this connection. Increase in temperature will cause expansion directly but, at the same time, the increased temperature tends to decrease the moisture content of the wood and the decrease is always accompanied by a decrease in size. From the meager investigations available it would seem logical to disregard thermal expansion in view of the larger shrinkage occuring during drying".

Again the Smithsonian Physical Tables for 1929 give a value of 0.0000021 inch per inch per degree Fahrenheit for the coefficient of thermal expansion for wood, but no data are at hand regarding the physical condition or the moisture content of the specimens.

In view of all of the above facts, it appears the part of wisdom to investigate the phenomena developed in these composite designs in the light of two hypotheses: (1) the Smithsonian coefficient (0.0000021 inch per inch per degree Fahrenheit) and (2) neglecting the thermal expansion and contraction of the timber stem entirely.

In the tests of Oregon State Highway Commission on composite beams, an elaborate studied on the effect of temperature change on stress in the beam. The study was carried out to find the effect of temperature change. Temperature change operates to modify stress distribution in two ways as follows: First, by restraining the normal movement of concrete and timber at the junction of these two materials, additional shearing stresses are induced which must obviously be provided for by increasing the strength of the shear connections. Second, by virtue of the secondary bending, temperature stresses are set up which become a maximum at the extreme fibers of both concrete and timber, in other words, at the top of the concrete flange and at the bottom of the timber stem.

The conclusion drawn from the above study is as follow: "It would appear that the only temperature stresses which need to be investigated in connection with the design of composite beams of the type included in this investigation are the junction point shears and that these should be determined by the following formulae:

$$f_c = \frac{2 E_c C_c t}{\left[1 + \frac{A_c E_c}{A_t E_t}\right]}$$

and $F_c = \frac{1}{2} A_c f_c$; $\quad n = \frac{2F_c}{s}$

where E_c = the modulus of elasticity for the concrete flange

E_t = the modulus of elasticity for the timber web

C_c = the thermal coefficient for concrete

A_c = area of the cross-section of concrete flange

A_t = area of the cross-section of timber stem

f_c = fiber stress at the bottom of the concrete flange at the junction point at the center of the beam.

F_c = the total stress in concrete flange

In the above formulae the term "n" represents the number of shear connections which must be provided for temperature stress (these, of course, to be in addition to the connections necessary for the transference of load stress). The term "s" represents the value of each shear connection, and the term "t" represents the maximum range in temperature from the assumed mean or construction temperature."

I would like to quote the conclusions drawn up by this study as follows:

"Variations in temperature produced stresses of sufficient magnitude to warrant a definite provision for the same in design. This feature, however, can be adequately taken care of through a proper design of the shear connection. The secondary bending resulting from thermal effects does not appear to induce stress of sufficient magnitude to cause concern; however, the development of tension due to restraint in the lower portions of the concrete slab points to the advisability of an adequate percentage of longitudinal temperature reinforcement".*

* Technical Bulletin No. 1, Oregon State Highway Department, pp. 104

Evolution of Theory.

From what has been said before, it seems, at this point, it is necessary to develope the formula whereby the composite members of this kind can be designed without resorting to physical experimentation.

In the design of this type of member, it is obviously first to assume, as in the case of all T-beam girders, the thickness of the deck slab and then the transverse spacing of timber stems. For short spans, the thickness of the concrete flange will probably be determined by the permissible minimum deck thickness (not less than 6 inches for modern highway loadings), and the spacing of timber stem or stringers will be determined by the transverse carrying capacity of the deck slab exactly as in the case of a homogeneous T-beam design. When the length of span increases, a point will be reached wherein this method is no longer applicable, and it becomes necessary to increase the thickness of the concrete deck so as to provide sufficient T-beam flange.

A reviewing of a certain empirical restrictions is necessary at this moment. For homogeneous concrete beams, the standard specifications of the American Association of State Highway Officials provide that the assumed effective T-beam flange width shall not exceed the following:

(a) One-fourth of the span length of the beam.

(b) The distance center to center of beams.

(c) Six times the width of the beam.

(d) Eight times the least thickness of the slab plus the width of the girder stem.

Now assuming the adequate bond and shear resistance have been provided at the junction of stem and flange, the effective flange width assumed in the design calculation must be arbitrarily selected in accordance with the above empirical restrictions. However, the composite beams tested in connection with this research project carried by Oregon State Highway Department restricted the flange width to four times the width of the stem. It further states that unless more experimental work is done in this connection the restriction regarding the effective flange width (that is four times the stem width) should be adhered to.

After the approximate dimensions of the composite members having obtained in the manner as mentioned, it remains necessary to find the method for the determination of stress. It is very evident that the stress analysis involves two distinct phases, that is:

(1) The evaluation of the extreme fiber bending stress

(2) The determination of the shearing stresses in the junction point and the design of proper connection details.

Each one of the two above will be considered in order.

1. The evaluation of the extreme fiber bending stress.

If an adequate shear connection has been assumed to give a rigid and truly elastic junction plane, then the composite beam may be designed by transforming the composite section into an equivalent homogeneous section in the ordinary manner which obviously consists simply in multiplying the assumed flange width by the ratio of the elastic moduli of concrete and timber, E_c/E_t, and thereafter treating the entirely beam as a purely homogeneous timber beam of the transformed dimension.

It should, at this moment, to be borne in mind that the above assumption holds only for a rigid and truly elastic shear connection. However, as stated previously (see comparison of End Slips) that a certain amount of plastic shear slippage and the consequent formation of two distinct neutral axes, neither of which is coincident with the computed position for an integral composite structure. The first phase of this problem, therefore, is the determination of the degree to which such shear slippage operates to modify theoretical results.

The figures in the next page were taken from Technical Bulletin No. 1, Oregon State Highway Department, show distinctly the relationship of each stress curve to the others. Each figure contains six curves (three for tensile stress and three for compressive stress). The pair designated as "measured stresses" was obtained from the deformations actually measured during the tests and the moduli of elasticity as determined experimentally for each beam. The curves marked "theoretical stresses" was obtained by transforming the composite beam into an equivalent timber section and using the actual values for the moduli of elasticity which were experimentally determined for the beam in question. It will be necessary to point out that the"measured stress" curves and the "theoretical stress" curves are based upon the same moduli of elasticity, so that the variation of these two curves shows the discrepancy introduced by shear slippage at the connections with small errors in experiment as have been introduced. Practically in each case, the two curves follow each other very closely. Especially in the range within the working limits of the material.

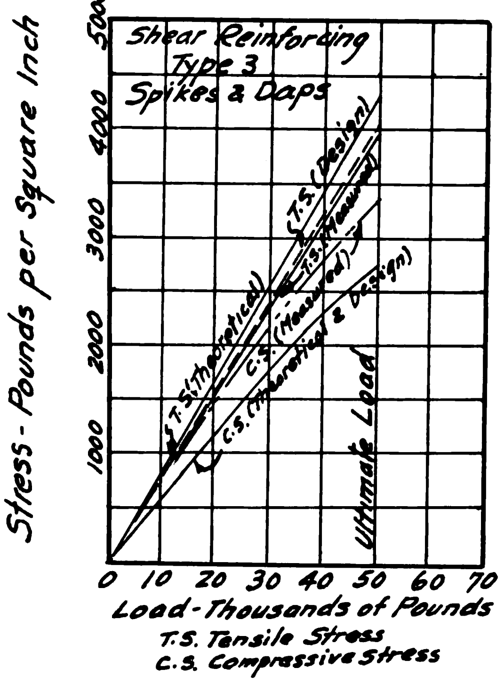
The remained set of curves designated as "design stress" have been produced in exactly the same manner as the theoretical stress curves except that, an assumed modulus of elasticity (3,000,000 pounds per square inch for concrete and 1,500,000 pounds per square inch for timber) instead of the actual measured one have been used. In this way, the difference between the "design stress" and "measured stress" curves not only shows the discrepancy arising from shear slippage and instrumental errors but also the errors introduced from the variation in elastic moduli of the materials from the assumed values usually employed.

As it points out from the test that "In extreme cases the variation between "design stress" and "measured Stress" amounts to as much as 25 per cent, but that for the average condition the measured tensile stress was 6.6 per cent below the design stress, and the measured compressive stress was 5.8 per cent above the design stress. It should be remembered that all discrepancies or errors arising from a variation between assumed and actual elastic moduli are not peculiar to the

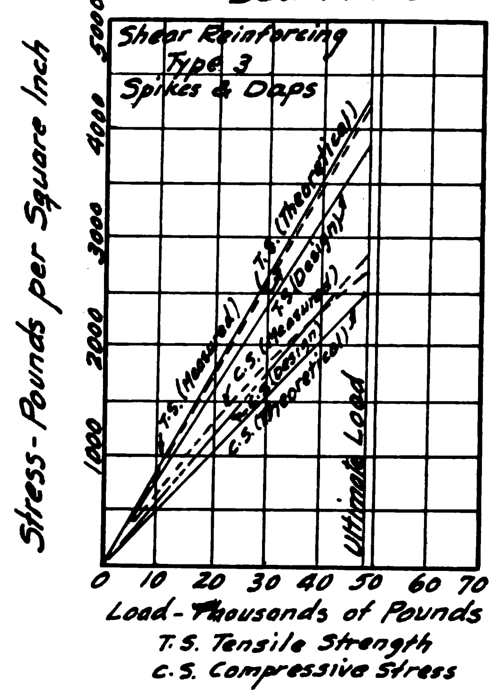
Composite Beams Test

Series No. 1

Beam No. 5

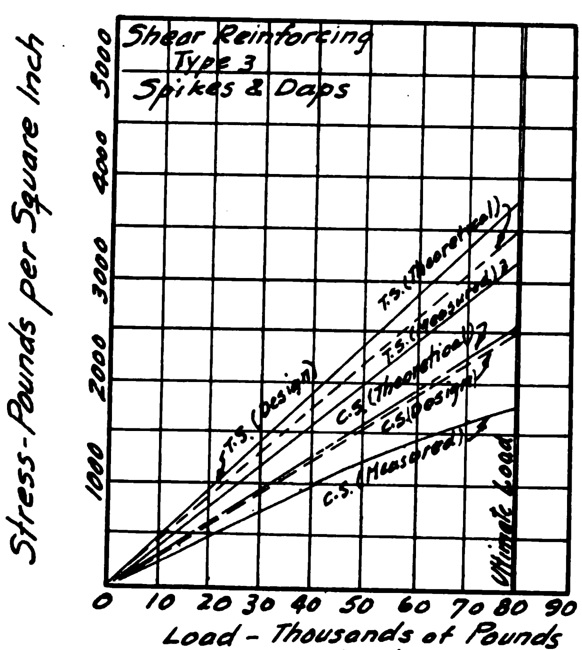


Beam No. 6

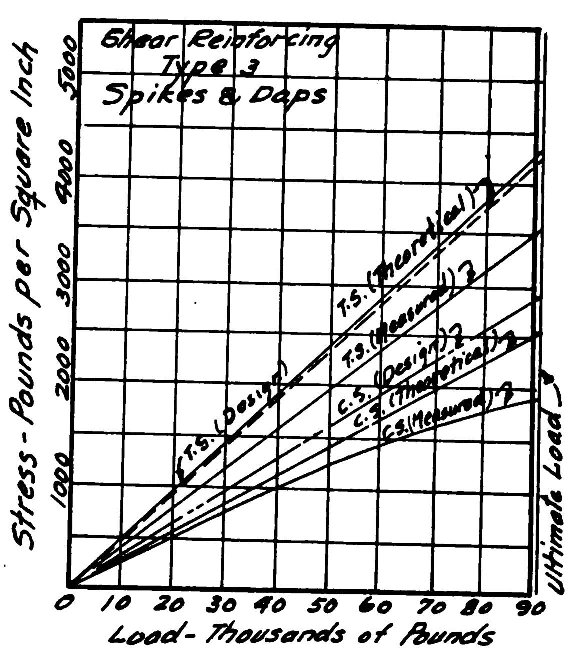


Series No. 2

Beam No. 17



Beam No. 18



composite type of construction but are inherent in all structures built of these materials whether homogeneous or composite."

From what have been said above, it may conclude that, while there is some discrepancy between the measured stresses and those determined by theory, the variations are not sufficient in magnitude to cause any appreciable error for materials of this character. Therefore the theoretical design formulae using average values for elastic moduli may be safely used.

There is one fact that the errors arising by a variation in the elastic properties of the individual materials are considerably greater than those introduced by shear slippage points to the advisability of determing the elastic modulus of both timber and concrete, whenever possible.

2. The determination of junction point shearing stresses.

The shearing forces along the junction plane between timber and concrete are the horizontal shears due to dead load, live load, impact and the shearing force set up from temperature variations. These shearing forces must obviously be resisted by the connections between the junction of the two materials. The provision for these shearing forces employs no new principle. It simply involves the designing of connections which are adequate in shear and bearing.

Since the most suitable shear connection as recommended previously was the combination of spikes and daps - type 3 -

For the present the discussion of connection will adhere to this type of shear connection. However, in the practical design the shear developers will be used instead of spikes and daps, because of its simplicity and economical, and, give just as adequate and rigid connection as spikes and daps.

For all practical purposes, it may be assumed that the entire bearing stress taken by each spike is concentrated in that one inch of length on each side of the junction plane. Such an assumption will be safe and will give reasonable results, and, for ordinary commercial sizes, it will be found that the factor of bearing against timber or concrete will be the limiting one.

In connection with the use of spikes (and it should be remembered that this type of connection is recommended only in combination with dapped joints), a maximum spacing of 6 inches should be employed regardless of the theoretical number required to take care of the shear. A very important point in connection with the use of spikes and daps is the necessity of driving the spikes far enough below the junction plane to reinforced the timber stem against failure by longitudinal shear in the timber fibers proper.

In the mechanics it has been shown that the shear on any cross-section of a rectangular beam is distributed as the ordinates to a parabola in accordance with the following formula:

$$s_y = \frac{S}{I}\left[\frac{c^2 - y^2}{2}\right]$$

where s_y = the shear at the point in question.

S = the total shear over the cross-section.

I = the moment of inertia of the cross-section about its neutral axis.

c = distance from the neutral axis to the extreme fiber

y = the distance from the neutral axis to the plane in question.

The shear is, of course, a maximum at the neutral axis, diminishing in accordance with the above law to zero at the extreme fibers. It is apparent that there will be a plane somewhat below the neutral axis, at and below which the residual shear can be carried by the timber itself at safe working stresses. The spikes used in the junction plane connection should obviously be driven to this point. They should also extend upward into the concrete about half the thickness of the slab and preferably not less than 3 inches in any ordinary design.

The Practical Design.

Before taking up the actual design of the timber-concrete composite deck a brief summaries of what have been said is necessary. The dead load and live load stresses must be segregated and applied to the proper elements. In continuous beam or slab construction the distribution of bending moment between midspan and support will depend upon the elastic properties of the assembly. In order to determine the position of the neutral axis, moment of inertia, and section modulus, it is necessary to transform the assembly into a hypothetical wood section, which is done by multiplying the steel area by the ratio of the modulus of elasticity of steel to that of wood as explained previously (which is usually taken as 18.75 for southern pine and Douglas fir).

It should be noted that the critical condition for which the member should be designed exists when the slab is first placed in service, before the concrete has attained its ultimate modulus of elasticity. Consequently, it is necessary and be conservative to treat the assembly as a homogeneous timber beam throughout the reaches where the concrete will be in compression, that is $E_c = E_t$. As in continuous spans over supports where the timber section will be in compression, the negative bending moment is resisted by the steel in tension and the timber in compression. The concrete is considered to have no value in tension.

The entire dead load of timber and concrete falls to the timber section alone, and not to the composite assembly (centering for support of composite decks may be required in exceptional cases).

The calculation of stresses for dead load in a continuous span construction will depend upon the relative magnitude of the positive and negative bending moments at critical points. However, because of the joints in the timber subbase, there is no known theoretical basis on which to calculate these moments in accordance with the action of a continuous beam of variable moment of inertia without resorting to actual loading tests in order to determine these factors. Such tests have shown that for a continuous timber subdeck with joints as above described the positive and negative bending moments for uniform dead load in interior spans are very nearly equal. For all practical purposes, each can be taken as 50 per cent of the simple span dead load moment and they occur simultaneously. Positive moment in end span will be slightly greater, usually assumed at 60 per cent of the simple span moment.

The live load stresses are resisted by the composite section. In order to determine the forces acting on a given width of slab it is necessary to fix the lateral distribution of a concentrated load. Tests by the Maryland Sate Roads Commission show a distribution of 15.1 ft. for a single live load placed at the center of an ordinary trestle.* When the effect of passing wheels placed in accordance with standard clearance diagrams is taken into consideration, the lateral distribution for a critical concentrated load is 5.1 feet, (by critical concentrated load is one of the interior heavy wheels of two passing load units in accordance with standard clearance diagrams).

It has been customary, however, to assume a somewhat narrower load distribution for shear than for moment calculations, since the critical position of the load for maximum shear will be closer to the support. Thus the distribution for shear calculations is taken at 4.0 feet and for moment at 5.0 ft.*

For live load, the positive moment is taken as 70 per cent and negative moment as 50 per cent of the simple span live load moment in interior spans; and 85 per cent and 40 per cent, respectively, of the simple span moment in end spans and 2-span structures. Maximum positive and negative moments do not occur simultaneously.

Working stresses:

Timber: According to grades specified, viz., 1200f, 1400f, or 1600f.

Concrete: 800 lbs. per square inch in compression (some designers use 900 lbs., which, of course, give a somewhat higher resisting moment for the midspan composite section.

* Proc., American Wood-Preserver's Association, 1939, pp. 272

Steel: 18,000 lbs. per square inch.

Shear Developers: 1750 lbs. each for 3/32 in thick steel plate in groove 2 inches or less in width.

The following is the detailed calculations for a 3-span timber-concrete composite decks highway bridge with 24 ft. roadway. The two end spans at 18 ft. from back face of abutment cap to center line of first pier (17 ft. 6 in. center to center of bents). The intermediate span is also at 18 ft. The total length of bridge is 54 ft. back to back of abutment caps. The bridge is designed for the standard H-15 loading.

The timber sub-base consist of alternating 2 x 4 in. and 2 x 6 in. plank laid on edge, spiked together with a concrete surface, increasing from $3\frac{1}{2}$ in. at the curb to $4\frac{1}{2}$ in. at the roadway center. Crown is not to be less than 1 in. All plank are dressed to a standard thickness of 1-5/8 in. The top and bottom surfaces of the wood lamination lie in true horizontal planes in order to maintain a uniform depth of groove.

The shear developers are used for the shear connection between the junction of concrete and timber base.

A one-foot width of slab is assumed in the calculations as follows:

Dead Load Moment.

Timber: Average thickness .... 4-5/8 in.

Weight .... 60 lbs. per cu. ft.

Concrete: Average thickness .... 5 in.

Weight .... 150 lbs. per cu. ft.

Total dead load is:

Timber: $\frac{4.625}{12} \times 60 = 23.1$ lbs. per sq. ft.

Concrete: $\frac{5}{12} \times 150 = 62.5$ lbs. per sq. ft.

Total 85.6 lbs. per sq. ft.

Simple span dead load moment for 18 ft. span is:

$$\frac{85.6 \times 18 \times 18 \times 12}{8} = 41,600 \text{ in.-lbs.}$$

For interior span this moment is distributed:

Positive moment = 41,600 x 0.5 = 20,800 in.-lbs.

Negative moment = 41,600 x 0.5 = 20,800 in.-lbs.

Beam elements for 12 in. of slab:

Wood section at mid-span:

There are 6 in. of width for each height of laminations. then let y is the distance from base of the lamination to center of gravity of the section, we have,

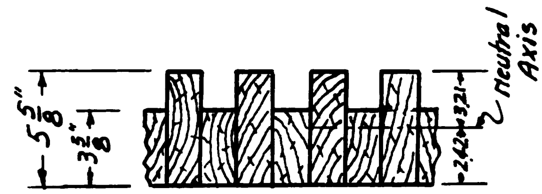
$$y = \frac{6(5.625)(2.81) + 6(3.625)(1.81)}{6(5.625) + 6(3.625)} = 2.42 \text{ in.}$$

$$I_1 = \frac{6(3.21)^3}{3} = 66.2$$

$$\frac{6(1.21)^3}{3} = 3.5$$

$$\frac{12(2.42)^3}{3} = 56.8$$

$$\overline{126.5} \text{ in}^4$$ is the moment of inertia of wood only



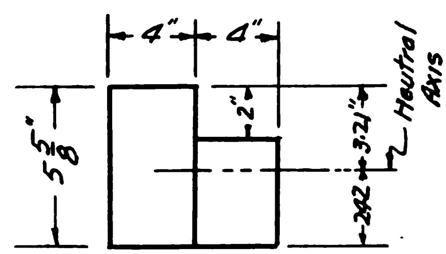
Wood Section At Midspan

Wood section at Support:

Since only two-thirds of the plank strips are continuous over the support. The moment of inertia is therefore 2/3 of the mid-span section, or

$$I_2 = \frac{2}{3} \times 126.5 = 84.4 \text{ in}^4$$

where y = 2.42 in.



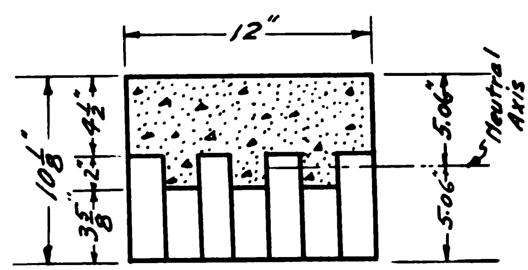
Wood Section at Support

Composite section at mid-span:

This section is considered homogeneous, therefore y is equal to 5.06 in.

$$I_3 = \frac{12(10.12)^3}{12}$$

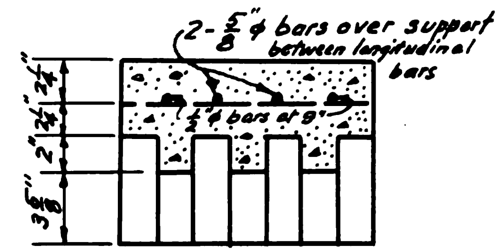
$$= 1024 \text{ in}^4$$



Composite section at support:

Same as before, because of joints in the plank strips at supports, only two-thirds of the laminations are effective.

Each height of lamination in a 12 in. width of slab has therefore a width of 4 in. It is further assumed to use two 5/8 in. round bars over support between longitudinal bars to take care of the negative moment.



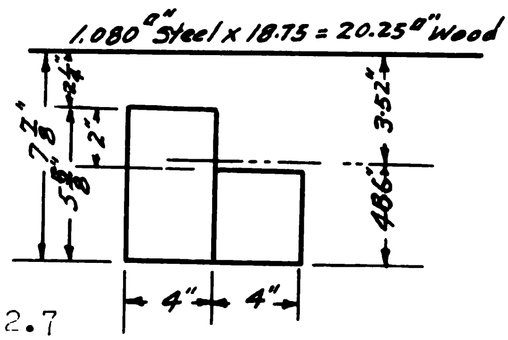
Let y_s be the distance from neutral axis to steel.

y_t be the distance from neutral axis to the bottom of wood.

then,

$$y_s = \frac{22.5(5.06) + 14.5(6.06) + 20.25 \times 0}{22.5 + 14.5 + 20.25}$$

$= 3.52$ in.



$y_t = 7.88 - 3.52$

$= 4.36$ in.

$$I_4 = \frac{4(1.27)^3}{3} \;\ldots\ldots\ldots\ldots\; 2.7$$

$$\frac{4(4.36)^3}{3} \;\ldots\ldots\ldots\; 110.0$$

$$\frac{4(3.63)^3}{12}$$

$$4(3.63)(2.55)^2 \ldots\; 111.0$$

$$\frac{4(0.73)^3}{3} \;\ldots\ldots\ldots\ldots\; 5.3$$

$$17.33\,(3.52)^2 \ldots\ldots\; 216.0$$

445.0 in.^{4}

Dead Load Stresses.

Tension at mid-span: $$f_t = \frac{20,800 \times 2.42}{126.5}$$

$$= 398 \text{ lbs. per sq. in.}$$

Compression at support: $$f_t = \frac{20,800 \times 2.42}{84.4}$$

$$= 596 \text{ lbs. per sq. in.}$$

Live Load Stresses.

For the Standard H-15 loading of the A.A.S.H.O. a 15-ton truck with 0.4 load on each rear wheel

$$15 \times 2,000 \times 0.4 = 12,000 \text{ lbs.}$$

This load on a 1-ft. width of slab is

$$\frac{12,000}{5} = 2,400 \text{ lbs. per foot width of slab}$$

For simple span moment

$$\frac{1}{4} \times 2,400 \times 18 \times 12 = 129,600 \text{ in.-lbs.}$$

Positive moment (interior span)

$$129,600 \times 0.7 = 90,720 \text{ in.-lbs.}$$

Negative moment (interior supports)

$$129,600 \times 0.5 = 64,800 \text{ in.-lbs.}$$

Mid-span:

Tension in wood:

$$f_t = \frac{90,720 \times 5.06}{1024} = 448 \text{ lbs. per sq. in.}$$

Compression in concrete:

$$f_c = \frac{90,720 \times 5.06}{1024} \times 1.3 = 582 \text{ lbs. per sq. in.}$$

where 1.3 is the 30 per cent impact allowance.

Support

Compression in wood:

$$f_t = \frac{64,800 \times 4.36}{445} = 636 \text{ lbs. per sq. in.}$$

Tension in steel:

$$f_s = \frac{64,800 \times 3.52}{445} \times 18.75 \times 1.3 = 12,500 \text{ lbs. per sq. in.}$$

where $18.75 = E_s/E_t$

Maximum Combined Stresses

Timber.

Mid-span: 398 + 448 = 846 lbs. per sq. in.

Support: 596 + 636 = 1,232 lbs. per sq. in.

Concrete.

Mid-span: Total stress is 582 lbs. per sq. in.

Steel.

Support: Total stress is 12,500 lbs. per sq. in.

Spacing of Shear Developers.

As has been said before the shear connection is designed for live load only. Lateral distribution is taken as 4 ft. The critical position of load unit for computing shear will be at three times the depth of slab from support as specified by Service Bureau of American Wood Preservers' Association. In this case it is about 2 ft. 6 in. Therefore;

Effective load per ft. width: $\frac{12,000}{4} = 3,000$ lbs.

Adding 30 per cent impact...........	900 lbs.
Total	3,900 lbs.

End Reaction is $3900 \times \frac{15.5}{18} = 3,360$ lbs.

Net Section: $(12 \times 10.12) - 2(5.625) - (2 \times 3.625)$

$= 102.5$ sq. in.

(Note: Splices eliminate two laminations from the section)

Average unit shear: $\frac{3,360}{102.5} = 33$ lbs. per sq. in.

Maximum Shear at Neutral Axis:

$33 \times 1.5 = 49.5$ lbs. per sq. in.

Total Horizontal Shear on 1 sq.-ft. is, therefore,

$49.5 \times 144 = 7130$ lbs.

Shear Developers required per sq. ft. of contact area:

$$\frac{7130}{1750} = 4.1$$

Since there are $\frac{12}{1.625} \times \frac{1}{2} = 3.69$ grooves per foot of width (with 1-5/8 in. in strips). The number of shear developers required per linear foot in each groove is:

$$\frac{4.1}{3.69} = 1.11$$

The spacing in each groove is, therefore,

$$\frac{12}{1.11} = 10.8 \text{ in., or say } 10 \text{ in.}$$

The spacing is maintained near the support where the shear is a maximum, and increased to 18 to 20 in. at mid-span.

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