

3. For maximum load on a cross girder the average load on the two adjoining panels must be a maximum, and of equal value.

4. Maximum moment in a beam occurs under a wheel load, which is as much to one side of the centre as the centre of gravity of all the loads is to the other side.

5. Maximum shear at any point in a beam occurs generally with longer segment loaded, and leading wheel load at the point. If, however, this wheel load divided by the distance to the second wheel is less than the average load on the span, then the second wheel must be placed at the point for a maximum.

The mathematical demonstration of these rules is to be found in several textbooks on the subject, which need not here be further pursued. In the diagram, since the loads are plotted as ordinates and spacings as abscissæ, any sloped line between the axes represents the co-relation of these factors, the tangent of the slope angle being an expression of the average unit load. This briefly explains the application of the diagram.

For example, let the maximum central moment for the 120 ft. truss be required. The panel points are first ticked off on a strip of paper to the scale of the diagram. It is probable that one of the driving wheels will be at the centre. Try No. 11, which is about a half-span distance from the head of the train. Place centre of span at this point, and note that wheels Nos. 2 to 20 are on the span. Run a line through the load ordinates at the extreme ends of the span, by means of string or straight-edge, and note that it cuts the load step over wheel No. 11. This is an indication that the arrangement of the wheel loads fulfils the requirements of the first rule. Had the line fallen below the step, it would have been necessary to move the panel point to the next wheel load on the right, and vice versa, if above.

The value of the moment will be, using the figures:—

$$(7,955 + 120 \times 1'0 - 3 \cdot 25 \times 128'0) \div 120 \times 60 = 3,829 \cdot 5$$

$$-(2,388 \cdot 5 - 3 \cdot 25 \times 68'0) \qquad \qquad \qquad = 2,167 \cdot 5$$

moment in foot tons = 1,662·0

If a line be run between the end moment ordinates, then the vertical intercept between this line and the general line of moments at the panel point will give the above value. Since wheel No. 1 is off the span, its moment line must be used as a datum for scaling moments and loads for abutment reactions or shears at any point.

The process for finding maximum shear in a panel is somewhat similar. Take the panel just to the right of the centre in the same truss. Some wheel near the head of the train will

give the desired result. Obviously wheel No. 1 will not comply with the second rule. Try wheel No. 2, and run line as before through end load ordinates. Through right hand end of panel run another line parallel to the first. If this cuts the load step above wheel No. 2 the condition of the second rule is fulfilled, and the position is a maximum. It will be noted that wheel No. 11 is at left hand support, and its load, therefore, may optionally be considered on or off the span. In the latter case only is the condition just fulfilled. In view of this try wheel No. 3, and proceed as before. Wheel No. 12 is now at the end, and its load may be included or otherwise. In either case, however, we find the required condition fulfilled, so that this arrangement is the correct one for maximum shear. The value is very simply calculated by dividing the left hand end moment by the span, which gives the abutment reaction at the right hand end, and subtracting from this the right hand panel reaction, which is the left hand panel moment divided by the panel length. In the latter arrangement of wheel loads this value is found to be—

$$2716 \div 120 - 78.5 \div 20 = 18.7 \text{ tons.}$$

The value for wheel 2 at the panel point is 18.6 tons.

The diagram possesses other useful properties in addition to those indicated above. The position of the centre of gravity for any group of wheel loads may be found by producing the end lines of the portion of the moment polygon, included between the load ordinates, to an intersection. The moment polygon is in fact an equilibrium polygon for the system of loads. Care must be exercised in using the diagram to ensure that a final and not a temporary maximum is obtained.

MATERIALS.

Under the contract which the Government of this State made in the first place with Wm. Sandford, Ltd., and later with G. and C. Hoskins, Ltd., all material which is scheduled and can be supplied by the Lithgow Iron and Steel Works, must be obtained thence by the bridge manufacturer. The specified tests, therefore, for steel and iron, conform to the tests prescribed in that contract. Originally these tests were as follows:—Structural Steel: Ultimate strength, 27-31 tons; elastic limit, 50 per cent. of above; elongation, 20 per cent. Rivet Steel: Ultimate strength, 26-30 tons; elastic limit, 50 per cent. of above; elongation, 30 per cent. Cast Steel—Ultimate strength, 26 tons. Bending tests were also specified to ensure ductility.

Recently these tests have, by agreement with Messrs. G. and C. Hoskins, been revised to bring them more into accordance with the values adopted by the Engineering Standards Com-

mittee of Great Britain, and the full text of the tests, quality and workmanship, as now specified, will be found appended. Roughly, about 15 per cent. of the material required for the bridges is supplied from Lithgow, and the balance imported.

DETAILS OF DESIGNS.

PLATE WEB GIRDERS.—As far as possible flange plates have been dispensed with to reduce the amount of shop work and to avoid troublesome notching of transoms. The girders are made as deep as is consistent with economy, and an allowance is made for the resistance of the web to bending (except where spliced) by adding one-eighth of its total area to each flange. In the case of stringers and cross girders, the deep floor has the further advantage of increasing the general stiffness of the whole structure, and thereby minimising the secondary stresses. Stiffeners are placed at intervals in conformity with the formula previously stated, but no attempt has been made to design their cross section. The experiments of Mr. C. A. P. Turner in America in 1897, and those of Professor Lilly, Trinity College, Dublin, in 1907, seem to show that the stiffeners act to a large extent as struts, thus relieving the web of a considerable amount of diagonal compressive stresses, and increasing the diagonal tensile stresses, and also the rivet stresses at top and bottom of the stiffeners. This action is probably more marked where there is initial wave or buckle in the web plates, a not uncommon occurrence where the girders are deep or the stiffeners far apart. The experiments of Professor Turneure in 1907 also point to a bending stress on the stiffeners, due no doubt to the deflection of the cross girders, and to the flexure of the top chord under compression. These facts and others want collating and there is still room for rigid experiment, such as would be well within the scope of the Engineering School.

The flange angles and web are cut and spliced on the same line to avoid overhanging of bars and to keep the joint compact. Shelf angles are provided for stringers to facilitate erection, but the main attachment is through the webs.

The 66ft. girders, both deck and through types, are made up in three sections and riveted together at the site. The 32ft. girder leaves the shop in one length.

Expansion is provided for by planing the underside of the bearing plate, which is attached to the girder, and covering the surface of the base plate with a sheet of gunmetal or rolled brass, $\frac{1}{4}$ inch thick. Clip washers under the nuts of the holding down bolts prevent upward movement.

TRESTLE PIERS.—The columns are of solid box section, set to a batter of 1 in 5, spread out at top and bottom to form cap, and base by means of gus-

sets. The bracing was originally $1\frac{1}{4}$ in. diameter round rods, with forged eyes, and turn buckles for ties, and T bars for struts. After studying the action of these in the shop and field, the author came to the conclusion that more rigid members and connections were desirable on several grounds, including the greater liability to damage from flood debris, which the former method incurred. Angle bars have now been adopted for the diagonals with riveted connections. In a long series of trestled spans, longitudinal stability is given by building concrete piers every sixth span in lieu of the steel trestles.

TRUSSES. — These are of the through Pratt type, with riveted connections. The ratio of depth to span has not been kept constant, as it would lead to either an unnecessarily great depth in the 200ft. span, or too little in the 120ft. span, to enable efficient sway and portal bracing to be provided. Uniformity, however, has been secured by making the depth a constant ratio to the panel width, namely, six to five, so that the bracing in all spans is at the same angle, which was selected as being the best on practical and aesthetic grounds. The tension members in the web are made of stiff section, similar to the compression members, and the lower chord is likewise similar to the upper, with the exception of the cover plate. The advantages claimed for this arrangement are that it ensures an even stress on the section, adds to the general rigidity of the structure, does away with the counterbrace, except in the case of an odd number of panels, and enables erection to be carried out on the building-out method, if the contractor should so desire.

Neutral axes in the vertical and horizontal framing intersect at common points, but the planes containing the axes do not intersect on common lines. This leads to some eccentricity of loading, but not to any serious extent, as the lateral stresses are small compared with those in the main part of the structure. It greatly simplifies the details, on the other hand, a result which goes to justify a departure from strict theoretic principles.

Column lacing has been put in on conventional lines, following the best American practice. This is a matter which has been rather shelved in the past, but the recent failure of the Quebec bridge, which was primarily due to weakness in this respect, has turned a strong light on the subject, and led to much theorising and some experiments. It has been shown, however, that for small columns, such as we have under consideration, the conventional practice is quite safe.

No adjustable members are provided, but initial stress is produced where desirable by the use of draw holes. The joints are drawn up tight by the same method.

Camber is obtained in the truss spans by increasing the

length of the top over the bottom chord to the extent of one-eighth of an inch in every ten feet. The vertical offsets at each post are computed and shown on the setting-out diagram for the shop.

The bearings are of cast steel of a mild character, similar in design for all three spans, the rollers for the expansion end being of constant diameter, but of different lengths, and varying in number according to the span. The movement of the rollers is controlled by double side bars, and a limit to the movement ensured by projections on the ends of the rollers between the side bars, which are machined so as to come to an even bearing on the latter in either direction.

All bearing parts of rollers, saddles, and plates are accurately machined, and the diameters of rollers are carefully gauged at the finish.

The allowance for expansion is, for a total range of 120 degrees F., one inch in every hundred feet of span.

DETAILS OF MANUFACTURE.

The bulk of the material comes from English and Scottish firms, but Carnegie has been drawn on in America, and Krupp's of Essen have supplied for several bridges the cast steel bearings. While waiting for the material, full size drawings of the half-truss span and other framing are generally in progress on a prepared floor, and are carefully checked by the Inspector. When the material arrives it is sorted out and straightened by means of rolls, or special machines. Test sheets are at the same time forwarded to the Department by the State's Consulting Engineer in London, and identified with the material. Templates are cut from the floor diagram, often on the actual bridge members themselves, rivet lines and pitches being carefully marked off. Bars and plates are then cold-sawn to exact lengths, or sheared and planed. Stiffeners are left with a margin to allow of juggling over flange angles and trimming ends to form a good butt. Drilling is the next stage, now generally done with single high speed drills, which have displaced the old multiple drillers. No punching is allowed in any part of the actual structure. After drilling all burrs are removed from the holes. Gusset plates of any size are trimmed to shape by punching a series of holes clear of the finishing lines, breaking off, and then planing the edges to requirements. The members are then built up with service bolts and pass on to the hydraulic riveters. After being tested on the floor diagram, they are assembled for temporary erection in the yard, which is carried out by travelling gantries. Supports, now generally of concrete or brickwork, are provided at each post for the truss spans, the levels of which are carefully taken so that the camber may be properly blocked up in the lower chords. When erection is complete, the final inspection is made, and the whole of

the metalwork is then painted and marked off for re-erection in the field. The truss span, centre to centre of bearings, is measured on a steel wire, the temperature and pull being noted, and marked by nicks on brass tags attached, and this is forwarded when required to the Engineer-in-Charge in the field, to enable the holding down bolts and bearings to be properly centred. After the span is dismantled, all parts are weighed, and invoices giving the actual detailed weights are forwarded to the field.

EVOLUTION IN BRIDGE DESIGN.

In the mechanical, as in the organic world, change, gradual yet persistent, is an increasing feature. Here, too, there is a struggle between types and final survival of the fittest. Experience and experiment keep on pointing out new ways. Sometimes, no doubt, intuition guides, and the idea, newly conceived, is matured and adapted to the service of man.

If we make a comparative examination of the bridges of this State, from the earliest to the most recent, we have an epitome on a small scale of the development that has been going on all over the world during the last half century. In Table C, appended, will be found a list of all the bridges over 100ft. span constructed in this State prior to the North Coast Railway, together with such information as was considered essential to a general comparison.

The first railway bridges were of British origin, both as to design, materials, and manufacture, and they afford good examples of early British practice. They are characterised by shallow floors with solid decking, close spacing of cross girders, shallow, through main girders, either of the double plate web type with bi-cellular chords, or of the multiple lattice web type, invariably continuous over two or three spans. Upper wind bracing is absent, and the only provision for sway bracing is by segmental built-up members, forming arches between the top chords. They were constructed of wrought-iron, with cast-iron bearings. Expansion rollers were 4in. diameter. These bridges are carrying the heavy traffic of to-day with safety, which speaks volumes for their manufacture and the quality of the material.

At the end of this series we have the Hawkesbury River Bridge, forming a class by itself, notable and unique. The world was given a chance for the contract, with the result that it was designed in America, manufactured in Scotland, and erected by an American firm. Steel was for the first time introduced, and the pin replaced the rivet group. The American influence now made itself felt, and we find in the early departmental designs simple triangular trusses of the Pratt type, with considerable depth and width of panel. Portal upper wind bracing

was provided, and the rollers increased to 9in. diameter. The bridges were still manufactured abroad.

Later the panels were still further widened and deepened, and top wind and sway bracing of an effective character provided. The expansion rollers were increased to 10in. diameter, and all bearings made of cast steel. Two more bridges of American design and manufacture were imported at this period, that over the Murrumbidgee at Gundagai being a pin-connected polygonal truss, and that over the Wollondilly at Goulburn a Warren-braced riveted truss, with stiff members. Some of the bridges of this period were locally manufactured, all material being imported.

In the present designs, the best features of the foregoing examples have been preserved, the floor has been further deepened, rigid sections used throughout, end cross girders provided, and counter-bracing practically eliminated. Various minor modifications have been made in the joint and other details. Flange plates in the girders have been largely dispensed with.

Mention must be made here of the high-class bridges constructed by the Railway Commissioners' staff over the Nepean River and Glennie's Creek, to replace existing structures which were no longer safe. The author gained many valuable hints from the study of these structures.

The construction of the North Coast Railway, including the bridgework, is in the hands of Mr. Hutchinson, M.C.E. (Melbourne), M. Inst. C.E., Chief Engineer for Railway and Tramway Construction. The standard designs have been prepared, and the inspection of the steel work during manufacture carried out by Mr. Dare, M.E., Assoc. M. Inst. C.E., Assistant Engineer, and his staff of officers. The calculations, estimates, and details of the designs have been worked out by the author under Mr. Dare. The drawings for the bridge-work and railway generally have been made under the supervision of Mr. Bradfield, M.E. Assoc. M. Inst. C.E., Assistant Engineer. The erection of the bridges is being carried out by contract, under the supervision of Messrs. Bode, Wickham and Stawell, Assoc. M.M. Inst. C.E., Engineers-in-Charge of the first, second, and third sections respectively, acting under Mr. Hutchinson.

The thanks of the Author are due to the Under-Secretary for Public Works, Mr. Hanna, and the Chief Engineer, for permission to use Departmental plans and information. He takes this opportunity also of thanking Mr. Shellshear, Chief Assistant Engineer for Existing Lines, and Mr. Small, Principal Assistant Engineer for Railway Construction, for photographs of bridges showing erection methods, and Mr. Quodling, of the Locomotive Branch, for negatives of locomotives and bridges from which, and the departmental collection, the lantern slides have been prepared by Mr. Degotardi, of the Public Works.

The subjoined paper contains some very interesting notes on location and erection of steel railway bridges by Mr. H. F. T. Bode, Assoc. M. Inst. C.E., Engineer-in-Charge of the first section of the North Coast Railway, which extends from West Maitland to Dungog, and includes the crossing of the Hunter and Paterson Rivers. They cover briefly the initial and final stages in the matter of bridge building, and have been contributed, at the Author's request, at a very busy time, for the which thanks are now extended.

APPENDIX.

ABSTRACT FROM SPECIFICATION.

9.—Quality of Materials and Workmanship.

The whole of the wrought-iron, steel, cast-steel, and gun-metal used in this work must be of approved quality; and the Engineer reserves the power of testing and rejecting, from time to time, any portion of the wrought-iron, steel, cast-steel, or gun-metal used in the construction of this work. It must be distinctly understood that each bridge and every part thereof is to be of first-class workmanship.

The wrought-iron and steel must be of uniform quality, free from scales, blisters, laminations, and all other defects, and before leaving the rolling-mill works, is to be scraped and cleaned, and well coated with boiled linseed oil.

The steel for castings to be made by the open-hearth or Bessemer process and thoroughly annealed. Castings to be accurately moulded, clean, sharp, and perfectly free from all flaws, cracks, sand-holes, air-bubbles, scoria, or other defects of any kind, and to be of uniform thickness and quality; no cement stopping will be permitted, and due allowance is to be made for shrinkage. Castings to be machined where shown or specified, sufficient allowance being made in all cases for turning or facing up true.

The gun-metal is to be of approved hard composition. Rolled brass of approved quality may be used in lieu thereof.

Allowable variation.—Any variation in sectional area, cross section dimensions, or weight of more than $2\frac{1}{2}$ per cent. from that specified, may be sufficient cause for rejection.

Workmanship.—All plates and sections to be of uniform thickness, with regular edges. Plates to be free from buckle and out of winding. Edges to be planed if not truly cut or rolled. The sizes shown or specified to be the finished sizes. Joints to be made only at such positions as shown upon the Plans. All butt joints of web plates and bars to be milled or planed true and square, so as to ensure perfect contact. The greatest care to be taken in making all welds, so as to ensure perfect soundness. Covering plates and wrappers to be fitted accurately to the sections which they have to cover. Stiffeners to be a neat

fit under flanges. Forged work to be carefully annealed and neatly finished with a surface of reasonable smoothness, to the approval of the Engineer.

10.—Testing of Materials.

Before proceeding with the manufacture, the Contractor will be required to submit samples of the material he proposes to use to the Engineer for testing. The samples must be of the dimensions indicated, and must be cut as directed by the Engineer from the material to be used in any part of the structure.

The tests for Structural Steel shall be as follows:—

Test specimens, of the same thickness as the plates or bars from which they were taken, to give the following results:—

Tensile strength in tons, per square inch . . . 27 to 32

Elastic limit not less than 50 per cent. of ultimate strength from test.

Elongation in 8 inches, not less than, per cent. . . 20

Test bars to have a sectional area of 1 square inch, or as near thereto as possible, with a maximum width of 2 inches, and to be of sufficient length to ensure 8 inches, at least, being in actual tension under test.

This steel must, in all cases, stand bending hot, cold, or quenched, round a bar of diameter not more than one and a-half times the thickness of the test piece.

The tests for Rivet Steel shall be as follows:—

Test specimens from the finished material of each melt to give the following results:—

Tensile strength in tons, per square inch 25 to 28

Elastic limit not less than 50 per cent. of ultimate strength from test.

Elongation in 8 inches, not less than, per cent. . . 25

Rivet steel must be of good welding quality, and must not crack or crumble at all when hammered at a welding heat. Sample pieces will be taken for testing the welding qualities of this steel, by welding two pieces together, and bending the bar or plate in the direction of the weld when cold.

Rivet steel must bend when hot, cold, or quenched, through 180 degrees without cracking, and the head to stand flattening out whilst red hot until it is equal to two and a-half times the diameter of shank, without any sign of fracture.

All mild steel of every specified test shall be further subjected to such chemical, hot, and cold forge tests, impact and bending tests, as may be sufficient, in the opinion of the Testing Engineer, to prove the soundness, ductility, and regularity of the material, and fitness for the service required.

The material must not show, on analysis, more than .06 per cent. of sulphur, nor more than .06 per cent. of phosphorus.

The tests for Wrought-iron shall be as follows:—

Specimens cut from any round bar to give the following results:—