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IMPACT STRESSES.

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The many stresses to which the various members of a bridge may be subjected may be divided into two classes, the first consisting of those always to be considered, viz., the dead and live load stresses, and the second being those acting only under certain definite conditions, due to wind, snow, temperature, centrifugal force, traction, etc. Another stress, that due to impact, which might really be termed a part of the live load stress, is now generally considered and usually appears as a definite feature of the stress sheet belonging to the first class. This stress may either be derived as a percentage of the live load stress or of the sum of the dead and live load stresses (which would not seem to be so rational), or it may be allowed for by using different unit stresses in the members according to the function which they perform in the truss.

When the imperfections of construction in the rolling stock (such as imperfect balancing) and track (such as bad joints) in a bridge are considered, together with the vibration always attending train motion, it must be at once apparent that the effect of impact cannot be overlooked. It will have more effect in short spans than on long ones, and on members which receive their full live load almost instantly than on those which receive it more gradually. Consequently, the material in short spans, and such members as hangers, hip verticals, etc., will become fatigued more quickly and to a greater extent than in long spans, and in those members receiving their maximum stresses from a full loading of the span. As the chords all receive their maximum loads from a fully-loaded span, the percentage to be added for impact should be the same for all panels. As the web members, with the exception of those at the ends, receive their maximum stresses from a partial loading of the bridge, those nearest the centre are subjected to their maximum stresses more suddenly than those nearer the ends. The percentage, therefore, to be added to web members for impact will increase from a minimum at the end to a maximum at the centre.

The effect of a train at high speed on a perfect track is supposed to closely resemble that of a suddenly-applied load. Now, it is well known that the effect of a suddenly applied load is double that of a gradually applied one, and that the effect of a moving train on a bridge is intermediate between the effect produced by the same load applied suddenly and the same load applied gradually. Such being the case, we find a number of the formulae used to determine the impact stress in a member are dependent upon the length of span loaded when that member receives its maximum stress. The method of allowing different unit stresses for various members would not seem to be so commendable, since the effect of impact is to increase the stress and not to lower the elastic limit or working stress of the material in use. By considering impact as an increase in the stress its effect is carried into the connections as well as being computed in the main body of the member. This seems a more reasonable assumption than to make allowance for it by a diminished unit stress in the body of the member, and to use the same data for designing a connection in which there is no impact as in one in which there is impact. By increasing the stress, and keeping the unit a constant, the connection will be increased in strength in the same proportion as the member. The unit stress which the material will stand is definitely determined by experiment, whereas the stresses resulting from the dynamic train load are merely the closest approximations which we are able to make with our limited knowledge.

It has been experimentally determined that failure may be brought about by a much smaller load than the breaking load if repeated often enough, that the greater the variation in load the fewer repetitions will be required, and that for the same variation, the effect is greater when the stresses are of opposite kinds than when of the same kind. Allowable unit stresses may be determined from the formulae of Launhardt and Weyrauch, based upon these facts:

If f = working stress per square inch

p = primitive strength; i.e., the resistance to fracture under a given number of repeated stresses of the same kind.

s = statical strength; i.e., the resistance to fracture under a gradually applied load

v = vibration strength; i.e., the resistance to fracture under stresses of equal intensity but of opposite kind.

F = factor of safety

then, for stresses of the same kind, by Launhardt,

$$f = \frac{p}{F}\left\{1 + \frac{s-p}{p} \cdot \frac{\text{min. stress}}{\text{max. stress}}\right\}$$

and, for stresses of opposite kinds, by Weyrauch,

$$f = \frac{p}{F}\left\{1 - \frac{p-v}{p} \cdot \frac{\text{max. small stress}}{\text{max. large stress}}\right\}$$

These, for structural steel, become

$$f = 10{,}000\left(1 + \frac{2}{3} \cdot \frac{\text{min. stress}}{\text{max. stress}}\right)$$

and

$$f = 10{,}000\left(1 - \frac{2}{5} \cdot \frac{\text{max. small stress}}{\text{max. large stress}}\right)$$

On these formulae are based specifications which make use of different units in making allowance for impact.

Theodore Cooper specifies various units for the different members according to their position in the bridge, these members subject to the greatest effect from impact, such as floor beam hangers, having the lowest unit stresses, and for most of the main members, allowing twice as much load per unit for dead load as for live.

The Pennsylvania Railroad increases their maximum calculated stress (M) in a member by a coefficient $(1+K)$, and the resulting stress $M\ (1+K)$ is the stress for which the member is designed using a constant unit stress.

For members with the stress of one kind only, $K = 1 - 2R + R^2$.

For members subject to reversal of stress, $K = 1 + 2R - R^2$.

$$R = \frac{m}{M}$$

m = minimum calculated stress in members subjected to one kind of stress only, or the maximum calculated stress of lesser kind in members subjected to reversal of stress. By minimum stress is meant the absolute minimum, i. e., in a diagonal or post m is the calculated dead-load stress minus the maximum calculated counter stress due to live load.

The Canadian Pacific Railway, Baltimore and Ohio Railway, American Bridge Company, and Pencoyd specifications express the impact stress by the following formula:

$$I = S\left(\frac{300}{L+300}\right)$$

where, I = Impact stress, S = Live load stress,
L = Length of loaded distance in feet when the maximum stress in the member occurs.

Dr. J. A. L. Waddell recommends the formula

$$I = \frac{40,000}{L+500} \text{ for Railway bridges,}$$

and

$$I = \frac{10,000}{L+150} \text{ for Highway bridges,}$$

where L = length of span in feet loaded where maximum stress is produced, and I = percentage to be added to maximum static live load stress.

The Chicago, Rock Island & Pacific Railway Company use certain units which they multiply by $\left(1+\frac{\text{minimum}}{\text{maximum}}\right)$, thus changing the units for the various members rather than adding an additional stress for impact.

The Dominion Government, for spans over 80 feet, and the Osborne Engineering Company use the formula

$$I = \frac{L^2}{L+D}$$

when I = the impact stress, L = the live load stress, and D = the dead load stress.

The Dominion Government, for spans less than 80 feet, and for members of trusses subjected to their maximum stresses by a load covering a shorter length of span than 80 feet, use the formula

$$I = \left(1.40 - \frac{S}{200}\right)L$$

where S = loaded length in feet when member receives its maximum stress, and L = live load stress.

Mr. S. Bouscaren has proposed the formula

$$I = S\left(0.1 + \frac{56.25}{l \times 62.5}\right)$$

where I = impact stress, S = Live load stress, l = length in feet of loaded distance which produces maximum stress in member.

Prof. Melan has proposed the following:

$$I = 14. + 2600 \div (L + 33).$$

I = percentage of live load stress to be added for impact, and L is the length of the span in feet.

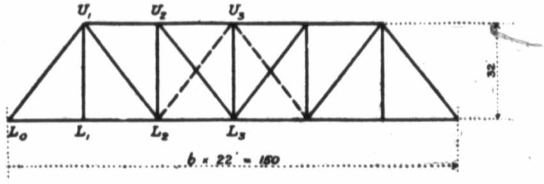
The American Bridge Company add 25% of the live load stress for impact in highway bridges.

In the Transactions of the American Society of Civil Engineers for 1884 and 1885, we find papers setting forth various percentages adopted by different engineers to be added to the different members in a bridge to make allowance for impact.

The two following examples, viz., a 60-ft. deck plate girder and a 150-ft. through span, have been worked out with six different methods for calculating the impact, the dead and live load stresses being taken as the same in all cases. The live load used was Class I. Engine, Dominion Government Specification 1905.

60-FT. PLATE GIRDER (DECK).

Specification	Stress	Shear	Flange Stress	Comparative Weight
	Dead Load. . .	22,200	55,400	. . . .
Dom. Gov., 1905, Cl. I.	Live Load	116,400	255,800	. . . .
Dom. Gov., 1901	Impact	97,000	208,500	
$I=\frac{L^2}{D+L}$	Total	235,600	519,700	.92
	Unit	10,000	16,000	
	Area reqd	23.6 sq.in.	32.5 sq.in.	
Dom. Gov., 1905	Impact	128,000	281,400	
$I=\left(1.40-\frac{200}{S}\right)L$	Total.	266,600	592,600	1.00
	Unit	10,000	16,000	
	Area reqd.	26.7 sq.in.	37.0 sq.in.	
Canadian Pacific Railway and American Bridge Company	Impact	97,000	213,100	
$I=\left(\frac{300}{S+300}\right)L$	Total.	235,600	524,300	.93
	Unit	10,000	16,000	
	Area reqd.	23.6 sq.in.	32.8 sq.in.	
Pennsylvania Railway	Impact	97,900	209,100	
	Total.	236,500	520,300	.92
	Unit	10,000	16,000	
	Area reqd	23.7 sq.in.	32.5 sq.in.	
F. H. Lewis, C. E., in Johnson's "Framed Structures,"	Total.	138,600	311,200	
$9{,}000\left(1+\frac{\text{min.}}{\text{max.}}\right)$ Tension	Unit	6,000	10,600	.87
	Area reqd	23.1 sq.in.	29.3 sq.in.	
Cooper, 1901	Total.	138,600	311,200	
	Unit	. . . . .	10,000	.90
	Area reqd	. . . . .	31.1 sq.in.	



150 ft Through Pratt Truss.

Specification	Stress	L_0L_2	L_2L_3	U_1U_2	U_2U_3	L_0U_1	L_1U_1	L_2U_2	L_3U_3	U_1L_2	U_2L_3	U_3L_2	Comparative Weight
	Dead Load.	– 58,600	– 93,800	+ 93,800	+ 105,500	+ 95,100	– 20,000	+ 25,000	+ 10,000	– 57,200	– 19,100	+ 13,400	{ Trusses only
Dom. Gov. 1905, Cl. I.	Live Load..	– 151,200	– 232,600	+ 232,600	+ 263,400	+ 245,700	– 69,300	+ 75,000	+ 35,300	– 163,200	– 95,300	– 44,800	
Dom. Gov. 1901	Impact.....	– 109,100	– 166,500	+ 166,500	+ 184,500	+ 177,200	– 53,600	+ 56,000	+ 27,700	– 121,200	– 78,900	– 18,500	
$I=\frac{L^2}{D+L}$	Total......	– 318,900	– 492,900	+ 492,900	+ 553,400	+ 518,000	– 142,900	+ 156,000	+ 73,000	– 341,600	– 193,300	– 49,900	.95
	Unit.......	16,000	16,000	14,600	14,600	....	16,000	8,100	8,100	16,000	16,000	16,000	
	Area reqd..	19.9	30.8	33.8	37.9	40.4	8.9	19.3	9.0	21.4	12.1	3.1	
Dom. Gov., 1905	Impact.....	– 109,100	– 166,500	+ 166,500	+ 184,500	+ 177,200	– 79,700	+ 76,900	+ 40,600	– 121,200	– 97,700	– 51,500	
$I=\frac{L^2}{D+L}>80$ ft.	Total......	– 318,900	– 492,900	+ 492,900	+ 553,400	+ 578,000	– 169,000	+ 176,900	+ 85,900	– 341,600	– 212,100	– 109,700	1.00
$I=\left(1.40-\frac{S}{200}\right)L<80$ ft.	Unit.......	16,000	14,600	14,600	14,600	....	16,000	8,100	8,100	16,000	16,000	16,000	
	Area reqd..	19.9	33.8	33.8	37.9	40.4	10.6	21.9	10.7	21.4	13.3	6.9	
Canadian Pacific Railway and American Bridge Company	Impact.....	– 100,800	– 155,100	+ 155,100	+ 175,600	+ 163,800	– 59,400	+ 60,000	+ 30,300	– 122,400	– 76,300	– 38,400	
	Total......	– 310,600	– 481,500	+ 481,500	+ 544,500	+ 504,600	– 148,700	+ 160,000	+ 75,600	– 342,800	– 190,700	– 96,600	.96
$I=\left(\frac{300}{S+300}\right)L$	Unit.......	16,000	16,000	14,600	14,600	....	16,000	8,100	8,100	16,000	16,000	16,000	
	Area reqd..	19.4	30.1	33.0	37.3	39.6	9.3	19.8	9.3	21.4	11.9	6.0	
Pennsylvania Railway	Impact.....	– 108,700	– 164,500	+ 164,500	+ 185,900	+ 176,500	– 54,300	+ 81,000	+ 27,500	– 141,100	– 114,400	– 31,400	
	Total......	– 318,500	– 490,900	+ 490,900	+ 554,800	+ 517,300	– 143,600	+ 181,000	+ 72,800	– 361,500	– 228,800	– 62,800	.98
	Unit.......	16,000	16,000	14,600	14,600	....	16,000	8,100	8,100	16,000	16,000	16,000	
	Area reqd..	19.9	30.7	33.6	38.0	40.4	9.0	22.3	9.0	22.6	14.3	3.9	
F. H. Lewis, C.E., in Johnson's "Framed Structures,"	Total......	– 209,800	– 326,400	+ 326,400	+ 368,900	+ 340,800	– 89,300	+ 100,000	+ 45,300	– 220,400	– 114,400	– 31,400	
$9{,}000\left(1+\frac{\text{min.}}{\text{max.}}\right)$ Tension	Unit.......	11,500	11,600	9,800	9,800	....	11,000	5,350	6,500	10,800	9,000	9,000	.91
$8{,}400\left(1+\frac{\text{min.}}{\text{max.}}\right)$ Compression	Area reqd..	18.3	28.1	33.3	37.6	38.8	8.1	18.7	7.0	20.4	12.7	3.5	
Cooper, 1901	Total......	– 209,800	– 326,400	+ 326,400	+ 368,900	+ 340,800	– 89,300	+ 100,000	+ 45,300	– 220,400	– 114,400	– 31,400	
	Units.. {	10,000	10,000	9,100	9,100	....	10,000	5,400	5,400	10,000	10,000	10,000	.89
		20,000	20,000	18,200	18,200	....	20,000	10,800	10,800	20,000	20,000	20,000	
	Area reqd..	18.1	28.0	30.7	34.7	41.4	7.9	16.2	7.5	19.2	10.5	5.2	

It will be noticed that the Dominion Government Specification 1901, Canadian Pacific Railway, and Pennsylvania Railway Specifications give almost identical weights, while the maximum difference between all six is about 10%.

Some curious literature may be found on the subject of impact, as in *Engineering News*, May 9, 1895. In a paper giving the results of measured bridge stresses, it is stated that "the actual strains agree very closely with the theoretical statically computed strains, even in the hip verticals, under a speed of train of 55 miles per hour."

Also an article on "The measurement of live load strains in bridges," by J. J. Hankenson and H. Ledger, it is stated that "Longitudinal vibration or rapid vibration of stress is much more excessive in the lower chord near the centre than in any other tension member; while the hip vertical shows a stress far less variable than that in the lower chord. The main diagonal is subjected to a less amount of longitudinal vibration than any other of the tension members. The reason for the great amount of longitudinal vibration in the lower chord is that it receives the stresses of all the members on their way to the abutments, consequently every variation of the stresses in the web members caused by a moving train, and shocks from its concentrated truck loads, must cause a variation in stress of the lower chord. This shows that the metal of the lower chord is subjected to a much more fatiguing stress than that of any other member of the truss."

In building, the effect of impact is felt to the greatest extent in the floor joists, to a less extent in the main girders, and still less in the columns. Allowance for it may be made by adding various percentages to the live load stresses, according to the position of the member under consideration.

From all this it appears that we have various methods and formulae for determining impact stress, all of which, however, are empirical and lack confirmation by actual experiment, so that it would seem that here lies a comparatively unexplored field for the research man; with time, instruments, and "the sinews of war," a first-class series of experiments might be carried out, and a formula derived which could be used with the knowledge that its results would be very close to the truth.