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REPORT
ON THE
ACKWELL'S ISLAND BRIDGE
(QUEENSBORO BRIDGE)

★
By **F. C. KUNZ**
Chief Engineer

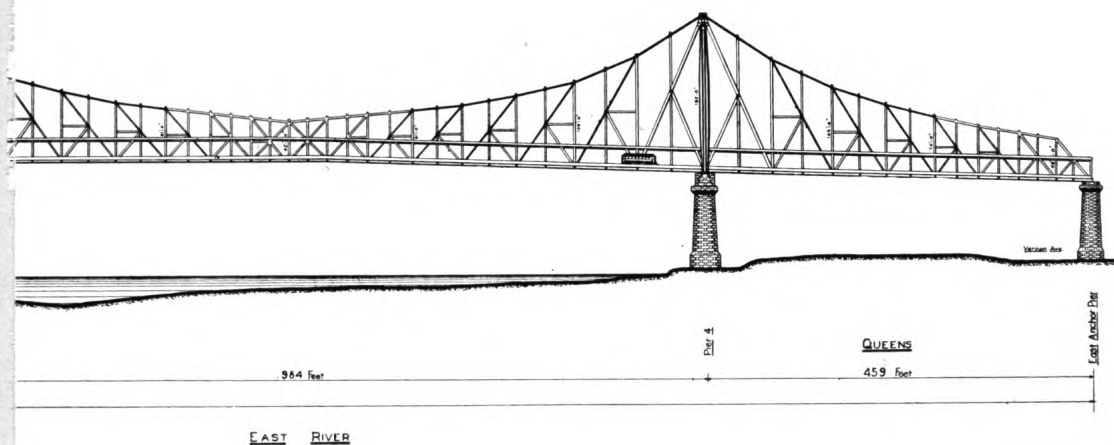
THE PENNSYLVANIA STEEL CO.

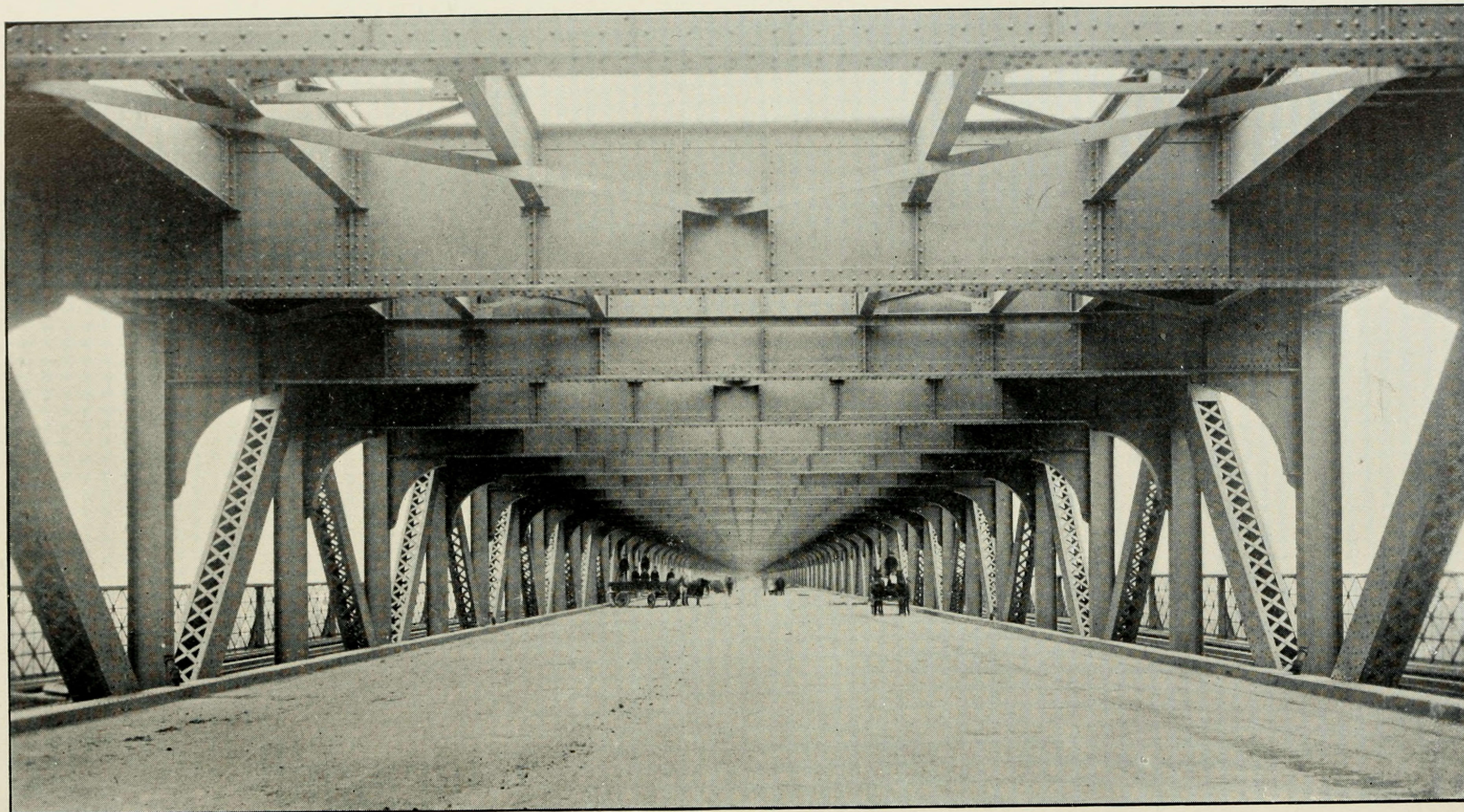
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J. E. GREINER, Consulting Engineer

Steelton, Penna., March 24, 1909





Queens Approach, view looking through bridge showing main roadway

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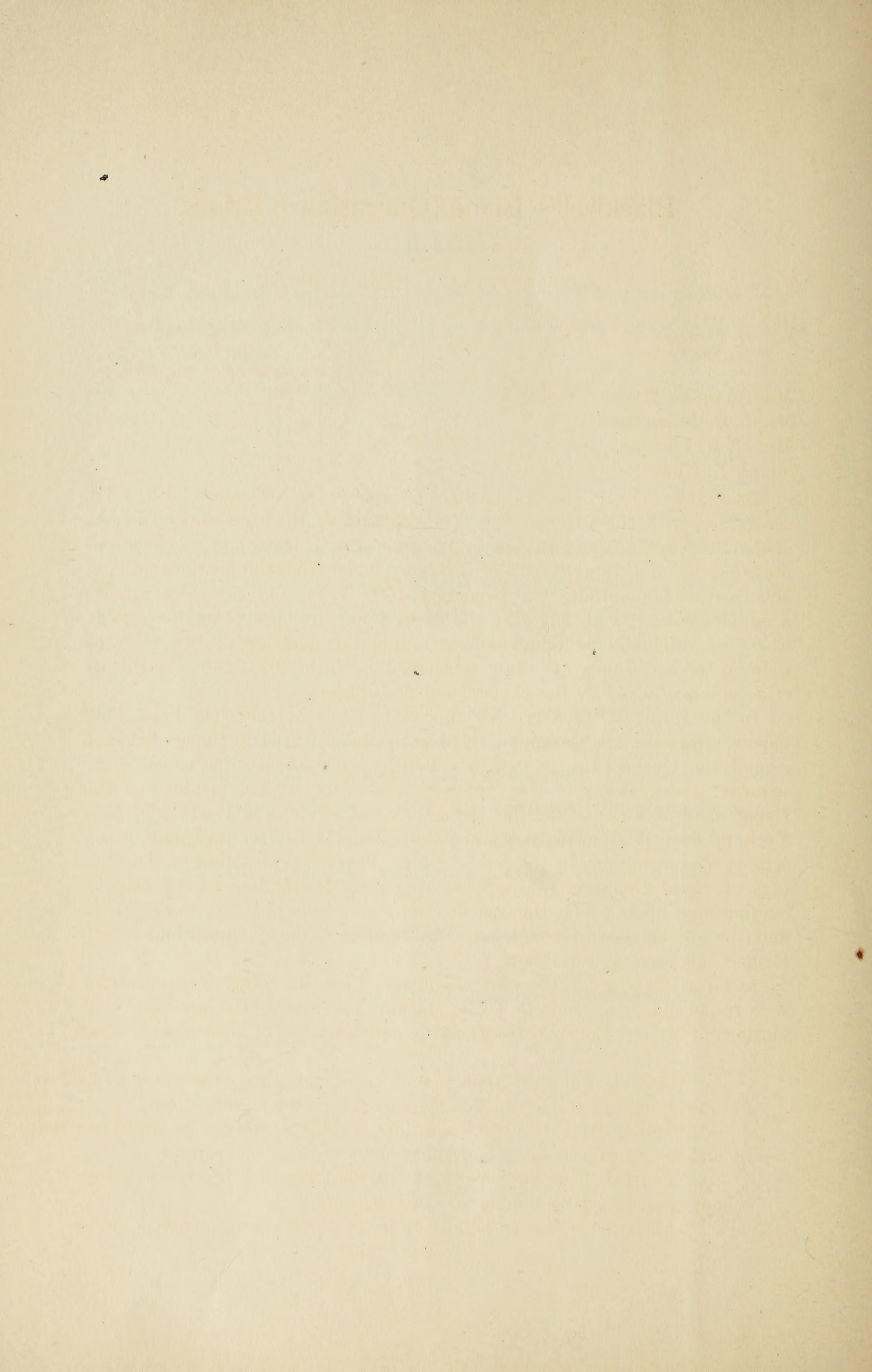
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Blackwell's Island (Queensboro) Bridge

THE PENNSYLVANIA STEEL COMPANY

OFFICE OF THE VICE-PRESIDENT

J. V. W. REYNDERS
Vice-President

Steelton, Pa., December 22d, 1908.

MR. CHARLES MACDONALD,
MR. C. C. SCHNEIDER,
MR. H. R. LEONARD,
MR. J. E. GREINER.

Dear Sirs: The Pennsylvania Steel Company, on November 20th, 1903, contracted with the City of New York to furnish the steel superstructure of the Blackwell's Island Bridge, in accordance with plans and specifications prepared by the Department of Bridges, under the commissionership of Mr. Gustav Lindenthal. On December 15th, 1904, the city entered into a supplementary contract with the Steel Company, providing for certain work not originally contemplated, including the addition of two elevated railroad tracks. The work was completed June 15th, 1908, and a certificate of acceptance issued by the Department of Bridges.

In the spring of 1908 articles appeared in one of the New York daily papers criticizing the design of the bridge and drawing analogies between it and the Quebec Bridge, which collapsed some months previous. Subsequent investigations of the structure, conducted at the instance of the Department of Finance and the Department of Bridges, of the City of New York, by Prof. William H. Burr and Messrs. Boller & Hodge, have led to many serious misconceptions in the public mind. Proper appreciation of the findings of these engineers presupposes a clear understanding of the original assumptions upon which the specifications and general plans were drawn, and the significance of which must be read into the computations which formed the basis of the reports.

As far as that part of the work is concerned for which the steel contractor was responsible, the reports are uniformly favorable, the following conclusions of Messrs. Boller & Hodge being characteristic of both reports, viz.:

“(Second) That the steel manufactured for this structure is first-class bridge material and in accordance with the specifications.

“(Third) That the workmanship of this structure is first-class and in accordance with the requirements of the specifications.

“(Fourth) That the erection and field riveting of the structure appears to have been done in a first-class manner.

“(Fifth) That the actual sections of the various members agree

Conclusions
of Messrs.
Boller &
Hodge

with the sections ordered on the working drawings and shown on our sheets Nos. 8 and 9, and that the shipping weights are correct."

(See Appendix A, page 49, for other extracts.)

While, therefore, from the point of view of a contractor we are not involved in any of the issues that have been raised, it is proper that our knowledge of the situation should be made available both for the information of the engineering profession as well as the general public, whose sense of security in respect to this structure has been unduly disturbed.

From this point of view, we have asked our Chief Engineer, Mr. F. C. Kunz, to prepare a report setting forth all the salient points that have been raised from time to time, with such information as we are able to supply in regard to the same, and we now ask that you, as a Commission, carefully examine this report, advising us whether you agree or disagree with the conclusions as set forth therein, and stating briefly the grounds upon which your opinion is based.

Very truly yours,

J. V. W. REYNDERS,
Vice-President.

Report of Commission

THE PENNSYLVANIA STEEL COMPANY,

J. V. W. REYNDERS, Vice-President, Steelton, Pa.

Dear Sir: Since the failure of the Quebec Bridge, public confidence has been somewhat disturbed as regards the safety of bridges of unusual magnitude. This feeling of distrust has been aggravated by the opinion expressed in the report of the Royal Commission, appointed to inquire into and report on the cause of the failure of the Quebec Bridge; this report was published and has been extensively quoted by the technical journals in this country as well as abroad. The unwarranted remark contained in this report, that "under extreme conditions, the Quebec Bridge stresses are in general harmony with those permitted in the Blackwell's Island Bridge," produced the impression in the minds of the New York public that the Blackwell's Island Bridge might, sooner or later, share the fate of the Quebec Bridge.

For the purpose of obtaining an unbiased opinion as to the true condition of the Blackwell's Island Bridge, the Commissioner of the Department of Bridges of New York City appointed two experts to examine the design and construction of this bridge.

Owing to the technical nature of their reports and a lack of clear understanding of the significance of the *assumptions* upon which the computations were based, the statements and conclusions contained therein have led to many serious misconceptions in the public mind. They have been misunderstood and misinterpreted by engineers who are not experts in bridge design, have been used by a small section of the local engineering press as a basis for the unjust assumption that the early designers of the bridge, as well as those who followed, blundered seriously, and foreign technical journals have taken the opportunity for the abuse and wholesale condemnation of American practice in general, and the judgment of American engineers in particular.

The public confidence, which was disturbed by the Quebec failure and by the unwarranted comparison of that bridge with the Blackwell's Island Bridge structure, has certainly not been restored by the reports of the city's experts on the latter bridge. In fact, the strength of the Blackwell's Island Bridge has now become a question of such serious and far-reaching importance, affecting not only the confidence of the public in engineering works, but the professional standing of American engineers, that it is assuredly proper and advisable for the contractors to make available their knowledge of the situation; and the undersigned, at the request of The Pennsylvania Steel Co., as conveyed in your letter of December 22nd, 1908, have reviewed the report of your Chief Engineer covering this subject.

We have carefully examined *this report*, dated November 27th, 1908, together with all data and information furnished in connection therewith, including the specifications, contract, strain sheets, reports of experts, etc., etc., and substantially endorse the arguments, conclusions and recommendations therein set forth.

Our report should, in our judgment, be an answer to the unjust and disturbing criticisms that have appeared in the public prints rather than a résumé of the entire subject; the structure has been so overloaded mathematically that the confidence of the public has been shaken, and it is only by an appeal to common sense, rather than to technicalities, that this lost confidence can be regained.

No question has been raised as to the sincerity of the contractors in executing the work; and the quality of the material, character of workmanship and adherence to approved plans have been endorsed by the city's experts. It is our understanding, after careful study, that the contract gave free powers to the Bridge Department in general design and engineering changes, and does not presuppose engineering knowledge on the part of the contractor further than is necessary for the proper execution in detail of the general orders of the Department.

The entire argument can well be based on the consideration of this question: Were the *original specifications*, and the *subsequent modifications* made by the Bridge Department, of such a nature that there can be any engineering doubt as to the *safety* and *usefulness* of this structure as a public highway?

A bridge is a highway and is not designed *primarily* to carry so many pounds per linear foot, but to accommodate so much traffic, and from an estimated weight of such *traffic* the design is perfected. For a railroad bridge such weights are easily determined, but for a roadway structure, the determination of *live load* weights is largely a matter of good judgment.

No material difference of opinion can arise as to the proper loading to be assumed in the designing of minor parts, i. e., hangers, floor-beams, stringers, etc.; such loading would cover but small areas and a *maximum* density of traffic could easily be conceived, but for the main trusses in a structure of any magnitude, the application of live loads of maximum density ("congested") over any extended areas would preclude all possibility of motion, a condition that would destroy the usefulness of the structure as a vehicle of traffic, a condition so *absurd* that it would not be tolerated in a city street.

The measure of usefulness of any public highway is the amount of traffic that can be safely and expeditiously handled over same, and any police regulation made for the proper handling of such traffic is a *necessity* for the *benefit* of the traveling public and does not, in itself, lessen confidence in the structure.

Traffic rules, requiring the surface cars to keep to certain clear distances apart are no hardship; quite the reverse, in allowing for greater freedom of motion, they are in the interest of increased traffic and consequently increased capacity. On elevated railroad tracks, proper spacing of trains is

a measure of safety, and the greater the interval of space, the greater could be the permissible speed; block signal and automatic devices now in use absolutely prevent electric trains from encroaching on the space deemed necessary for their prompt and safe handling.

Live Load. Professor Burr, in his report on this bridge, states:

“Proper provision for various classes of loading for a structure of such magnitude, designed to carry an extraordinary volume of traffic, with the corresponding working stresses, is largely a matter of judgment.”

In this connection, we wish to call attention to the investigations of a Commission appointed in 1903 by the Mayor of New York City to examine and pass upon the plans of the Manhattan Bridge, which was then designed as a suspension bridge with stiffened eye-bar cables. While the report of this Commission refers wholly to the Manhattan Bridge, the recommendations as to the live loads for which it should be designed have been applied to the Blackwell's Island structure, which was intended to carry the same kind of traffic. This Commission, consisting of Messrs. George S. Morison, C. C. Schneider, Henry W. Hodge, Mansfield Merriman and Theodore Cooper, made a very thorough investigation of the subject and recommended that the Manhattan Bridge should be designed, in so far as the main members are concerned, to carry a “maximum working” load of 8,000 pounds per linear foot, and that a so-called “congested” load of 16,000 pounds be used in proportioning the hangers. Quoting from this report, in regard to further use of the “congested” load,

“We consider that the bridge should be so proportioned that with a congested load of 16,000 pounds per linear foot, covering the whole bridge, combined with dead load and wind pressure, no stresses should be produced anywhere reaching the elastic limit of the material or impairing the stability of the anchorages. We consider that the *working load* of 8,000 pounds per linear foot should be used in designing the main members of the structure. The congested load should be used in proportioning the hangers.”

Any fair interpretation of this report would indicate that the intention of the Commission was to recommend the use of the “congested” load for proportioning the hangers (using a working-unit stress) and as an extreme test of the cables and anchorages; and that the “working” load be used in proportioning the main members of the structure. The above quotation appears (in part) in the report of one of the city's experts, but the omission of the words “or impairing the stability of the anchorages” and the omission of the last two sentences quoted above, admits of an entirely different construction being placed on the recommendation.

Independent investigations which we have made confirm the estimate of the Manhattan Bridge Commission as well as that of the city's experts, fixing the maximum load on any extended area of roadway or footwalk at

50 pounds per square foot. With the increased weights of elevated and surface cars, cited in the experts' reports, we would obtain as a "congested" load, 15,955 pounds per linear foot, made up as follows:

4 elevated 8-car trains, at 1,810 pounds.....	7,240 per lineal foot
4 trolley tracks, at 1,460 pounds.....	5,840 per lineal foot
35.5 feet roadway, at 50 pounds per square foot.....	1,775 per lineal foot
22 feet footwalk, at 50 pounds per square foot.....	1,100 per lineal foot
	15,955

Such a load, with roadway and footwalk crowded, trolley cars bumper to bumper, and elevated tracks completely covered, is an *impossibility* unless *special* and *extraordinary* means were taken to produce it, and the term "congested" applied to such a loading in connection with the computations of stresses in main members is misleading; a "theoretical test load" or "extraordinary load" would be terms more applicable.

A *maximum working load* is much more complex of analysis than a "congested" load, and is a matter on which the judgment of engineers may be expected to differ. With a "congested" load provided for, the possibility of failure from collapse is eliminated, but to provide for such a load at ordinary working units would be an unwarranted extravagance, hence the economical necessity for determining a *working load* and providing for such loading with *working units*. The working load of 8,000 pounds per linear foot of bridge recommended by the Manhattan Bridge Commission and used in the computations of the Blackwell's Island structure by the Bridge Department could be analyzed as made up of 20 pounds per square foot on the footwalks (an uncomfortable walking crowd), 30 pounds per square foot on the roadway (equivalent to a semi-congestion of *average* vehicles), 1,945 pounds per linear foot on four trolley tracks (equal to heaviest loaded cars spaced two car-lengths apart) and 4,550 pounds per linear foot as an equivalent load for the elevated trains on four tracks, trains spaced about 1,000 feet apart; such loading can be conceived, but it is doubtful whether these weights would be reached once in a period of years.

Dead Load. Among the first criticisms appearing in the public prints was the assertion that the dead load had been enormously increased without a corresponding increase in size of the main carrying members. This assertion,—one that would appeal directly to the fears of the public, for whose benefit this bridge has been constructed, appeared in one of the leading daily papers, and was answered at the time by the Bridge Department, but it can be better answered now by a frank discussion of the entire dead-weight problem.

The weight of steel now in place, plus an estimated weight of the additional steel required to complete the overhanging footwalks, amounts to 106,650,000 pounds. The same items were assumed, in the calculations of 1904, at 103,100,000 pounds. The assumptions as to these items were therefore exceeded but $3\frac{1}{2}$ per cent.

The uniform loading (other than steel) covering the weight of track

for both elevated and surface cars, hand-rails, paving for roadway and footwalks, pipes, etc., assumed in the 1904 calculations, was 5,109 pounds per foot, aggregating 19,030,000 pounds. This weight was estimated from such plans of the structure as were perfected at that time; but the Bridge Department in 1907 changed the plans as to roadway, increasing the weight of paving, inside trolley rails and hand-rails, making the uniform load (other than steel) 6,968 pounds per linear foot, or an increase of 1,859 pounds for these items. This was an admitted error, and has been partially remedied by reducing the weight of paving, etc., on the river spans to the extent of 1,168 pounds per foot; no change being made on the anchor and island spans, as the added dead weight on these spans relieved the maximum stresses to some extent.

The total dead weight of the structure, with four elevated tracks and overhanging footwalks, would then be about 130,700,000 pounds against an assumed (1904) weight of about 122,130,000 pounds,—an increase of $6\frac{1}{2}$ per cent. The dead load stresses, however, are not increased to the above extent, as the distribution of increased weights is not uniform, a large proportion of increase in steel having gone to the towers and anchorages.

The design of this structure, a cantilever bridge without a suspended connecting span, gives a continuity not found in the ordinary cantilever, inasmuch as a load on any part of the bridge affects the stresses in the entire structure from end to end. A strict interpretation of the *specifications* requires the loads to be placed in such positions as to give the greatest stress on any member of the structure. The Bridge Department, in preparing the strain sheets from which the bridge was built, did not strictly follow this clause, and applied the loads both “working” and “congested” in one continuous stretch, this stretch of *any* length covering one or more of the subdivisions of the bridge or the entire length of the structure, but with no unloaded gaps. The city’s experts, in making their analysis of the structure, interpreted the specifications literally, and obtained stresses (the mathematical accuracy of which we do not question) alarmingly high, when considering the “congested” load together with the increased weight of pavement.

A reasonable and proper distribution of assumed live load on a structure of this type and magnitude is again a matter of engineering judgment.

The adopted method of the Bridge Department was well within their rights, especially as regards the so-called “congested” load, and it is our judgment that such placing of the loads would cover all possible contingencies liable to arise.

The alarmingly high stresses obtained by the experts, as stated before, were arrived at by a strict interpretation of the printed specifications by placing the “congested” load of 16,000 pounds per linear foot on certain fixed portions of the bridge with fixed lengths of gaps in which there could be no load whatever; a method that might well be described as the placing of impossible loads in an impossible manner.

Professor Burr’s method of calculation of stresses produced by the elevated railroad trains, spacing eight car trains in position to give maximum stress, but not less than 1,000 feet apart, is rational, and we fully endorse

his method; but, when more than one track is treated in this manner, some concession should be made either in unit stresses or weight of trains for economic reasons; the possibility that two or more trains of 410 feet length (eight cars) fully loaded, occupying exact spaces on one track, should be provided for; that this same loading and spacing could occur on a second track at the same instant of time is only conceivable, but that all four tracks should be loaded in this exact manner is well nigh impossible, and places such loading immediately in the category of "congested" loads to be provided for by a higher unit.

According to Professor Burr's conclusions, "a controlled traffic on the four trolley lines of the lower deck and on two elevated railways of the upper deck, carrying the heaviest cars of their classes now in use in the City of New York, together with a vehicular traffic on the roadway, and two loaded sidewalks, may be permitted without exceeding the specified unit stresses for the regular live load and dead load and without exceeding the safe limits of stresses for such a structure." This conclusion was based on reducing the dead load by a considerable amount.

We have made an investigation, using Professor Burr's method of distribution of train load on *four* elevated tracks, together with 8,000 pounds per linear foot of bridge, and find that the stresses produced by this extreme load practically agree with those specified for the congested load, and are, therefore, well within the limits of safety.

Considering the character of the structure and assumed loads, the unit stresses specified and used in the computations were conservative; a distinction should be made, however, between unit stresses for "working" loads and "congested" loads. The City's experts recommend, with one exception, the unit stresses for working loads fixed by the specifications, the exception being a slightly *higher* unit for steel in compression, due to change in reduction formula. One of the experts, after listing "working" unit stresses substantially in accord with the specifications, stated that these stresses "are the *limit of safety* for the direct stresses from the sum of the live and dead loads, as the secondary and snow load stresses heretofore referred to will add to these stresses." The secondary stresses are small, especially in the tension members, where the higher units are specified, and we believe that a snow load may be safely neglected when considering working or "congested" loads; it would, therefore, seem that the term "limit of safety," as applied to such working stresses, was unfortunate and tending to cause unnecessary alarm. The "limit of safety" would, in a theoretically perfect structure, be just under the elastic limit of the material; secondary stresses and imperfect distribution of stresses should be allowed for, and we believe that sufficient allowance was made for such factors in the specifications, in fixing on the unit stresses to be used in connection with the "congested" load.

CONCLUSIONS

(1) We are of the opinion that the live loads provided for in the original specifications, with the subsequent modifications made by the Bridge Department, both as to weights and distribution of same, are sufficient for the traffic the bridge is intended to carry, and cover all possible contingencies.

(2) That the unit working stresses specified are in accordance with good practice, and the limiting stresses for extreme conditions of loading are well within safe limits.

(3) That the actual weight of steel superstructure practically agrees with the estimated weight used in calculating the stresses, within the usual allowance permitted in bridge work.

(4) That the superstructure, as built, conforms to the specifications and designs approved by the Bridge Department.

(5) That the bridge, as now constructed, with provision for two elevated tracks is entirely safe to carry all traffic which can possibly come upon it under present conditions, without any other restrictions than those necessary to regulate such traffic (see cut, page 58).

(6) That, for conditions of traffic, i. e., the weight of vehicles, surface and elevated cars, as now existing, the bridge would also be safe to carry all the lines of traffic contemplated in the final design of the bridge subject to ordinary traffic regulations (see cut, page 59).

Respectfully submitted,

CHARLES MACDONALD,
C. C. SCHNEIDER,
H. R. LEONARD,
J. E. GREINER.

March 8th, 1909

Report on Blackwell's Island Bridge

THE PENNSYLVANIA STEEL COMPANY

BRIDGE AND CONSTRUCTION DEPARTMENT

THOMAS EARLE
Superintendent

F. C. KUNZ
Chief Engineer

Steelton, Pa., November 27th, 1908

MR. J. V. W. REYNDERS, Vice-President.

Dear Sir: The Reports of the Consulting Engineers, Prof. William H. Burr and Messrs. Boller & Hodge, appointed June 9th by the Commissioner of the Department of Bridges of the City of New York, in accordance with the resolution of the Board of Estimate and Apportionment, dated June 5th, to examine "the design and structure" of the Queensboro Bridge, formerly called Blackwell's Island Bridge, are now public property. Coming from such an eminent source, the conclusions of the reports have been accepted without question, and much unfavorable criticism of the design of the bridge followed. It would, no doubt, interest the profession to know that there are engineers, not responsible for the design, who do not agree with all the conclusions of the reports. It would seem that the experts were not in possession of all data concerning the design, which would not be surprising, considering the fact that the construction of this bridge was directed by three different administrations of the Department of Bridges. Furthermore, while the reports are fair, they are based on assumptions different from those originally made.

Professor Burr remarks at the beginning of his criticism:

"Proper provision for various classes of loading for a structure of such magnitude, designed to carry an extraordinary volume of traffic with corresponding working stresses, is largely a matter of judgment."

Permissible
Unit Stresses
Depend on
Assumed
Live Load

Here lies the whole difficulty. After the necessary assumptions have been made, the rest is a mechanical procedure, and the bridge will be declared "unsafe in some members and wastefully proportioned in others," or "safe and uniformly proportioned," depending on these assumptions; the moment they are changed, the verdict may be totally reversed. It is safe to state that, the assumptions being a *matter of judgment*, in other words, one on which engineers may honestly hold different opinions, no two engineers, working independently, would be likely to recommend the same maximum live loads and corresponding unit stresses for a structure of the magnitude of the Blackwell's Island Bridge, with its two independent floors, accommodating fourteen lines of traffic of four different kinds, viz.: four trolley tracks, four elevated railroad tracks, four rows of wagons on the roadway and two

promenades for pedestrians. It is true that there were specifications; but all specifications, even for ordinary bridge work, can be interpreted in different ways by different engineers.

The Specified Live Loads The original specifications for the Blackwell's Island Bridge, written in 1903, prescribed a "regular" live load of 6,300 pounds and a "congested" live load of 12,600 pounds per linear foot of bridge, which in April, 1904, were changed to 8,000 pounds and 16,000 pounds respectively, when the Department of Bridges decided to add two elevated railroad tracks, making the traffic facilities equal to those of the Manhattan Bridge. These live loads were adopted from a report submitted to the Department of Bridges early in 1903, by a commission of bridge experts appointed to examine and pass upon a design of the Manhattan Bridge with eye-bar chains. The specifications for both bridges having been written at the same time, it would have been surprising had the specified loads not been the same.

The following is the paragraph of the report on the Manhattan Bridge concerning the live loads for truss members:

"The maximum congested load which could possibly be brought upon this bridge would consist of a continuous train of rapid transit cars on each of the four elevated tracks, of a continuous line of trolley cars on each of the four trolley tracks; of a crowd of heavy teams on the roadway and of a crowd of people on the footway. The heaviest rapid transit train which may run over this bridge is that adopted by the Interborough Company for the subway; such a train, in which two-thirds of the cars are motor cars (corresponding to the practice of the Manhattan Railway), with 120 passengers on each car, is estimated to have a possible weight of 1,700 pounds per linear foot and an extreme axle load of 26,000 pounds. The estimated maximum weight of a continuous line of trolley cars is 1,000 pounds per linear foot. The estimated greatest possible congested weight per linear foot on this bridge would then be as follows:

4 rapid transit trains, at 1,700 pounds	6,800 pounds
4 lines of trolley cars, at 1,000 pounds	4,000 pounds
35.5 feet of roadway, at 100 pounds per square foot	3,550 pounds
22 feet of footway, at 75 pounds per square foot	1,650 pounds
Total	16,000 pounds

"This is a possible load which could never occur unless special pains were taken to produce it. The conditions which would block the tracks with continuous lines of cars in one direction would probably prevent cars entering the bridge from the other direction. The weight on the sidewalks would correspond to about twelve people per linear foot, or something like 35,000 people on the bridge. The congestion of the roadway would be such that teams could not move. Under these conditions we consider that one-half of this amount may be taken as a

maximum working load; *this would make the working load 8,000 pounds per linear foot, which is three times that provided in the plans of the Brooklyn Bridge and 40 per cent. greater than that taken for the Williamsburg Bridge.**

"The provision of 2,000 pounds per linear foot for wind pressure proposed by the Commissioner is ample.

"In calculating the effect of temperature, provision has been made for an extreme variation of 110 degrees Fahrenheit, which we consider sufficient.

"We consider that the bridge should be so proportioned that with the congested load of 16,000 pounds per linear foot, *covering the whole bridge*, combined with dead load and wind pressure, no stresses would be produced anywhere reaching the elastic limit of the material or impairing the stability of the anchorages. *In other words, it should not be possible for such extraordinary congested load to do any permanent injury to the bridge.*

"*We consider that the working load of 8,000 pounds per linear foot should be used in designing the main members of the structure.*"

[The italics are ours.—F. C. K.]

The Commission on the Manhattan Bridge did not specify any unit stresses under the action of the two kinds of live load except that "such extraordinary congested load should not do any permanent injury to the bridge." The specifications for the Blackwell's Island Bridge, prescribed for tension of truss members 20,000 pounds per square inch for "*regular*" or *working*, and 24,000 pounds per square inch for "*congested*" or *extraordinary* live load, with more than the usual reduction for compression, viz., 20,000–90 *l/r* and 24,000–100 *l/r* respectively. [Messrs. Boller & Hodge and Professor Burr, in their reports, reduce their permissible unit stress for compression according to formula 20,000–50 *l/r*; many engineers allow for compression up to "*l over r*" equal 40 and even 50, nearly or exactly the same unit stress as for tension.] Professor Burr calls these stresses "safe and satisfactory in the light of knowledge and precedent available when they were drawn" (in 1903) and, since the amount of the "*regular*" and the "*congested*" live load was also specified and actually used in the calculations, it would only remain to explain how this live load was placed to obtain the greatest stresses.

The Specified
Unit Stresses

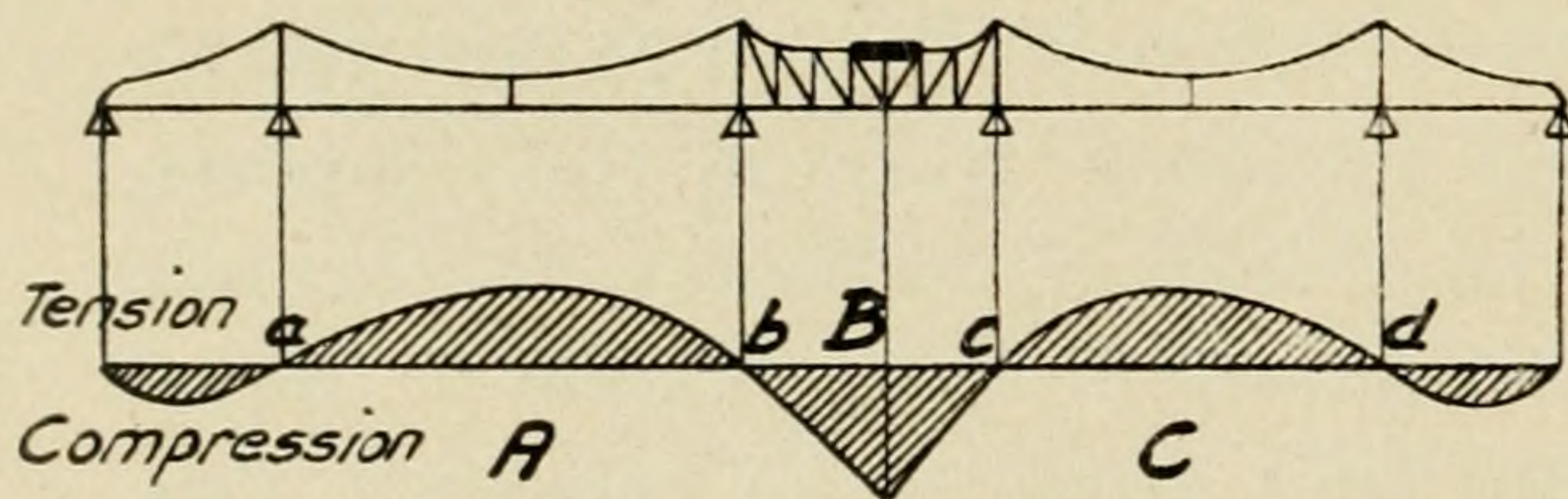
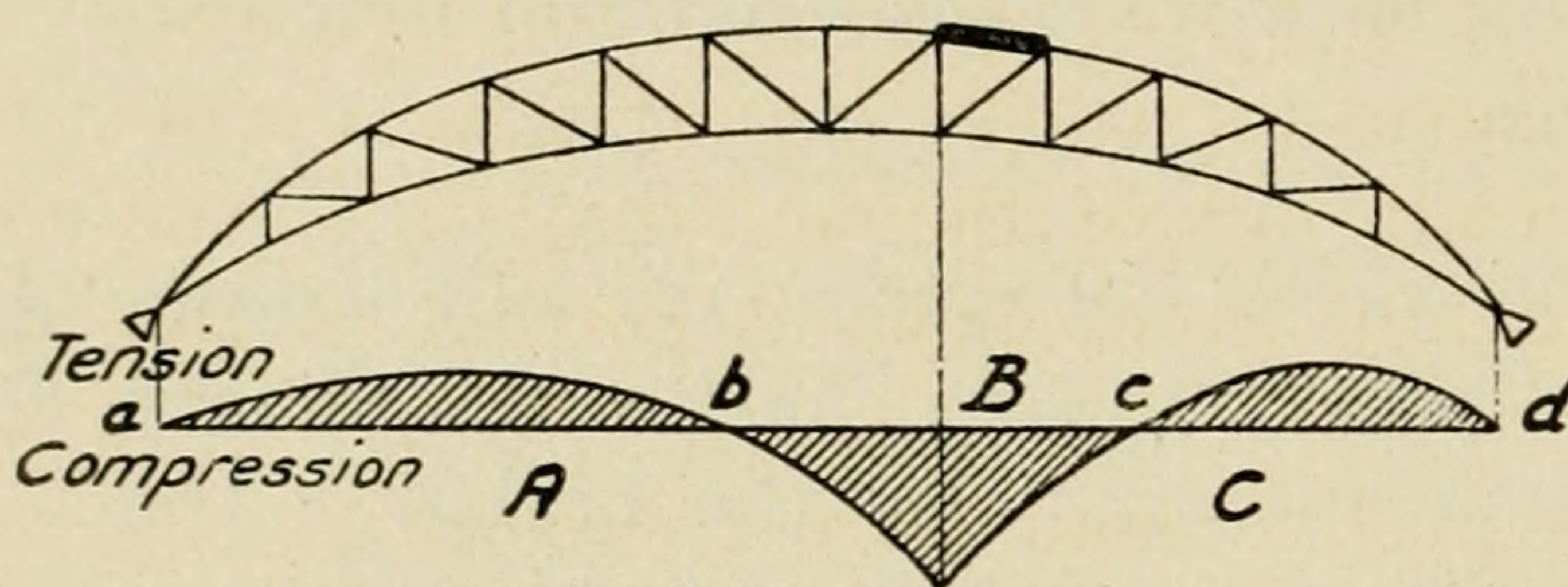
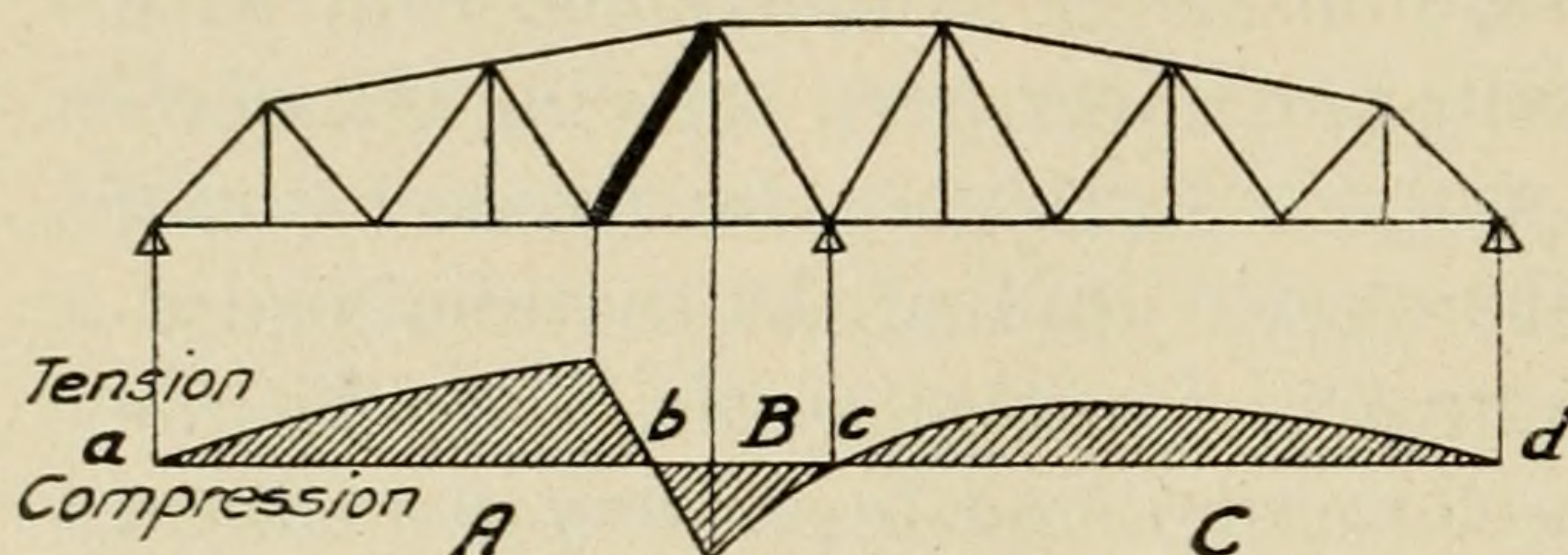
The specifications stated that the live load should be "placed so as to give the greatest strain in each part of the structure." This is the usual wording used in specifications for bridge work. For ordinary trusses, the live load causing the greatest strain or stress in each truss member covers one *continuous*, i. e., *uninterrupted* stretch. However, in arches, most types of suspension bridges, like the Brooklyn and the Manhattan Bridges, swing bridges or trusses with several supports, like the Blackwell's Island Bridge, the live load for the greatest theoretical stress in some members would have

Distribution
of Live Load
"Continuous"
or "Discon-
tinuous"

*Brooklyn Bridge 2,600 pounds, Williamsburg Bridge 5,700 pounds for the cables and 4,500 pounds for the stiffening trusses, all figures per linear foot of bridge.

to cover certain *discontinuous*, i. e., *isolated* stretches, different for different groups of members, with absolutely no live load between these stretches and on the other parts of the bridge. Any live load outside of these isolated stretches would reduce the greatest stress in the truss members.

To illustrate this, three influence lines are shown in the accompanying figure; the first for the stress in a diagonal of a swing bridge on three supports, the second for the stress in a top chord member near the center of an arch with two hinges, and the third for the stress in a top chord member of the island span of the Blackwell's Island Bridge. A live load in the stretch



ab or *cd* will cause tension in the diagonal or the top-chord members respectively, and a live load in the stretch *bc* will cause compression in these members. The absolute greatest tension, therefore, would be caused by *ab* and *cd* loaded and *bc* unloaded.

The usual method, however, is to assume a *continuous*, i. e., *uninterrupted* stretch, loaded as follows:

If area B is smaller than area C, then load from *a* to *d*; if area B is greater than area C, then load from *a* to *b*; in other words, choose that *uninterrupted* loaded stretch (*ab* or *ad*) which ever gives the greatest stress. This method was followed in the design of the Blackwell's Island Bridge. It is the usual method, even in single-track railroad bridges, where one train may cover *cd* and another *ab*. The stresses

in the arch bridge of 840 feet span at the Niagara Falls were calculated in a similar manner. For the Washington arch bridge in New York City, the stresses were determined for a live load covering the whole span (510 feet) and a live load covering the stretch from abutment to the center of span only, and the greater stresses considered.

Live Load Stresses

In March, 1904, the contractor was furnished with the "Loading Key" (dated January 26th, 1904), prepared by the Department of Bridges. (See paper by Mr. R. C. Strachan, Assistant Engineer of the Department of Bridges, published in the Proceedings of the Engineers' Club of Brooklyn, 1905; also in "Engineering News," February 16th, 1905, in which the analytical method of computation is described; also paper by Mr. F. H. Cilley, in the Transactions of the American Society of Civil Engineers, 1904, describing the graphical method used for checking the live load stresses.) This "Loading Key" gives the influence on the stresses in the different truss members

of a live load of 1,000 pounds per foot per truss, covering different adjoining stretches of the bridge.

The following is quoted from Mr. Strachan's paper:

"The calculations were made by the writer for the Department of Bridges of the City of New York during the years 1902 and 1903, and have more recently been revised to conform to altered data.

"The subsequent addition of two elevated railroad tracks, giving a total "congested" live load of 8,000 pounds, or "working load" of 4,000 pounds per foot per truss, made necessary a revision of both dead and live load stresses; and the results herein given are based upon the latter load.

"The connection of the cantilever arms at mid-channel causes a load on any subdivision of the bridge to produce stresses in all the others; the effect, however, upon the east truss of a load on the west truss, or vice versa, is generally small, as will appear when the results are examined. Moreover, the simultaneous and complete stoppage of all the many lines of traffic in such a way as to give full live load on two widely separated subdivisions (divisions 2 and 7, for example), and nowhere else, is practically out of the question, as is the assumption of live load covering any short isolated portion of the bridge.

"Stresses for seven positions of live load were therefore calculated, the load in each case covering one of the seven subdivisions; and the maxima, both tensile and compressive, obtained by combining these for a continuous load extending over one or more subdivisions, are the governing stresses for the main members of the truss.

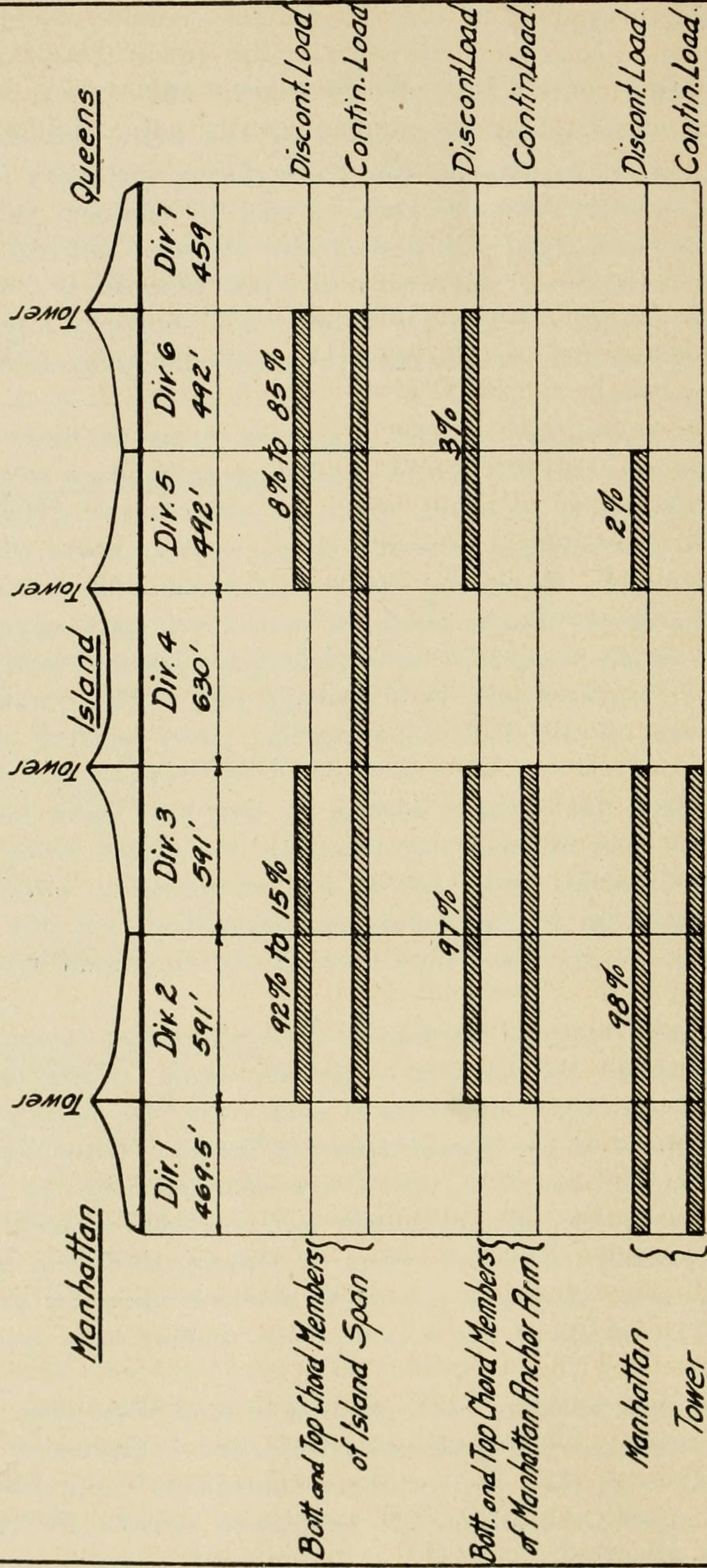
"In the absence of any similar truss with which to compare, weights were obtained by successive approximation, after the study of a sufficient number of members of each class had made possible a reliable estimate of the percentages for details. Those stresses, due to either dead or live load, which are unaffected by the action of the rockers, were obtained graphically and checked by analytical methods, more especially to detect errors in scaling, since the proper closing of the graphic diagrams constitute a check upon the graphical work itself. The results of the action of the rockers were ascertained analytically.

"We thus have for each position of load what may be called the partial stresses. The greatest tension and compression from a load extending over one, two or more continuous subdivisions, obtained by the algebraic addition of the partial stresses for each member, are then tabulated."

The following diagram explains the *continuous* and *discontinuous* loading for a few members.

Diagram Showing Continuous and Discontinuous Loading for Maximum

Stresses in various Members.



The above % figures show the part of stress contributed to the maximum stress by the load below the figure.

It may be of interest to mention that, in the discussion which followed the reading of this paper, and in which Messrs. Nichols and R. S. Buck participated, and the correspondence sent in by Messrs. Duryea, Mayer, Hodge and Moisseiff, no mention was made of the difference of the effect of a live load in *continuous* stretches as compared with that of a live load in *discontinuous* stretches.

Mr. Henry W. Hodge writes in his discussion:

"I have taken great pleasure in reading Mr. Strachan's paper on the "Computation of Stresses in the Blackwell's Island Bridge," and his method of arriving at these stresses in the rocker members connecting the cantilever arms is as simple and accurate as any method can be which is based on deflection of framed structures.

"As far as the computation of stresses in this structure is concerned, there is little discussion possible; but, as to the wider question of the advisability of constructing a cantilever structure on this semi-continuous design, engineers' opinions vary considerably."

Mr. Joseph Mayer writes:

"The method given, which is based on that used in the accurate calculation of drawbridge stresses is, I believe, the only accurate method available. I believe the stresses and sections required in the absence of rockers will give good preliminary data for calculating the deflections produced by given forces at the rockers. Your paper is so faultless that it does not lend itself to much discussion."

Mr. Edwin Duryea, Jr., referring in his letter to the assumption as to "probable" maximum live load, mentions that a "more rational method" of estimating probable maximum load had been used by Mr. Joseph Mayer in designing a railway bridge across the North River. In looking this up in the Transactions of the American Society of Civil Engineers, Vol. 48, 1902, p. 375, we find:

"For the purpose of comparing the merits of various types for a bridge across the Hudson, the writer, therefore, will use a bridge having twelve tracks, of which two are for freight trains, two for long-distance passenger trains, six for trains of the elevated and underground roads of New York City and two for surface cars.

"Such a bridge has sufficient capacity for the business it can secure in the near future. The distant future may bring other competing bridges or tunnels, therefore it need not be considered.

"The moving load would consist of two freight trains, each 1,000 feet long, weighing 3,000 pounds per linear foot; two long-distance passenger trains, each 1,000 feet long and weighing 1,500 pounds per linear foot; six rapid transit electric trains, each 500 feet long and weighing 1,200 pounds per linear foot.

"The surface cars, on the two tracks provided for them, should

run at a speed of at least 15 miles per hour. They would, therefore, be twice as far apart as on the street approach; a distance of 100 feet from center to center of cars would be closer than practicable. This distance, with cars weighing 40,000 pounds, gives a load of 400 pounds per linear foot of track.

"For the calculation of the cables, anchorages and the towers above the floor level, this load is equivalent to 8,421 pounds per linear foot of bridge, covering the whole length of the main span. The writer, therefore, will assume a moving load of 8,500 pounds per linear foot of bridge for these calculations.

"For the calculation of the stiffening trusses, the *loads on the surface-car tracks may be neglected, as they are nearly uniformly distributed over the length of the bridge.* The stresses produced by the trains 1,000 feet long and those produced by the trains 500 feet long would have to be calculated separately and then added, if the exact stresses corresponding to these loads are wanted. No equivalent load of one length will give the same stresses in every member of the stiffening trusses as the two loads of different lengths."

[The italics are ours.—F. C. K.]

The working live load of 8,500 pounds per linear foot, *representing ten railway tracks, two of which for heavy freight and two trolley lines,* is therefore about the same as the 8,000 pounds working or "regular" specified in 1903 for the Blackwell's Island Bridge, based on a weight of rapid transit trains of 1,700 pounds per linear foot, in place of Mr. Mayer's 1,200, and of trolley cars weighing 43,000 pounds each, in place of Mr. Mayer's 40,000. (The reports of Messrs. Boller & Hodge and Professor Burr give 1,810 pounds per linear foot and 62,000 pounds respectively, which is an *increase of 50 per cent. in seven years.*) It is also interesting to note that Mr. Mayer, for a bridge of 2,800 feet span, assumes on each of the ten railway tracks *only one train* of 1,000 feet or 500 feet length respectively, and disregards the possibility of partial "bunching" of trolley cars on the two tracks provided for them.

The method of probabilities mentioned by Mr. Duryea as having been used by Mr. Mayer to determine the probable frequency of the concurrence of trains with the maximum load on two, three, four, five, six, etc., tracks, assuming the trains running on schedule time with certain constant velocities, could not be employed here, as even the most elaborate assumptions as regards speed and time of the rapid transit trains on the four tracks will be often overthrown by delays, accidents, etc., and the traffic on the trolley tracks, roadway and promenades is beyond the reach of a mathematical expression of its probability.

The following is quoted from "Engineering News" concerning Mr. O. F. Nichols' (at that time Chief Engineer of the Department of Bridges) discussion of Mr. Strachan's paper:

"Speaking then in some detail of the width and traffic capacity of the structure, Mr. Nichols expressed the opinion that the roadway as described is far too narrow to accommodate the street traffic which will come upon it. He expects that the roadway will be overcrowded as soon as the bridge is opened. The provision of two lines of elevated railway as originally made would also have proved a serious error, but the four tracks now provided will suffice for some years. One or two details of the structural design were also discussed by the speaker." ("Engineering News," 1905, Vol. 53, p. 178.)

In this connection, it may also be of interest to quote from Mr. G. S. Morison's paper (see Transactions of the American Society of Civil Engineers, Vol. 36, 1896) regarding the assumed live load for a suspension bridge of 3,100 feet span over the Hudson River in the City of New York, even if the paper was written twelve years ago:

"The bridge has been designed to carry a total load of 25 tons, or 50,000 pounds, per lineal foot. The design has then been developed and the dead weight calculated, and the result is a balance for the live load of 11,000 pounds per foot over the entire structure. As the width of the structure is 92 feet between the stiffening trusses, this corresponds to about 120 pounds per square foot of floor. If this space were to be occupied by eight railroad tracks, it would amount to *1,375 pounds per lineal foot per track, which exceeds the weight of any passenger train*. It would amount in the aggregate on a length of 3,100 feet to 34,110,000 pounds—equivalent to eight freight trains 1,400 feet long, each weighing 3,000 pounds per lineal foot. It is probable that the requirements of any location where a bridge of this magnitude would be considered would be satisfied by four railroad tracks adapted to a heavy class of traffic and *four rapid transit tracks to be operated by electric cars or short trains of a character which would require only a floor stiffener to secure the necessary rigidity*. Therefore, in proportioning the stiffening truss, the variable load has been taken on the basis of 12,000 pounds per lineal foot, corresponding to a load of 3,000 pounds per foot on each of the railroad tracks, *with no provision for unequal weight on the rapid transit tracks*, or to 1,500 pounds per lineal foot on all eight of the tracks. These provisions correspond to four maximum freight trains or eight maximum passenger trains."

[The italics are ours.—F. C. K.]

The original estimate of the contract of the Blackwell's Island Bridge, with two elevated railroad tracks, made in 1903, was about 86,000,000 pounds; the addition of two elevated railroad tracks to the capacity of the bridge, with changes and additions made by the Department of Bridges in 1904, as well as more specific knowledge in regard to the probable weight of details, indicated that, with the live load in *continuous* stretches, the

total steel weight would run up to about 100,000,000 pounds, with a greatest cross-section of the bottom chord of the trusses equal to about 1,100 square inches, while, for the live load in *discontinuous* stretches, the total steel weight would be about 10 per cent. and the chords about 25 per cent. heavier, with all the accompanying difficulties of excessive thickness of material, inadequate space for the lacing of bottom chord and for the packing on the pins, etc.

The question of *continuity* or *discontinuity* of the live load, therefore, involved a question of an increase of the steel weight of the structure of about 10,000,000 pounds, at a corresponding additional cost of approximately \$700,000.

The adoption of the *continuous* live load by the Department of Bridges was based on the unlikelihood that the fourteen lines of live load, consisting of four kinds of traffic, i. e., rapid transit trains, trolley cars, wagons and pedestrians, on two independent floors, would be distributed simultaneously in the following manner, viz.:

First—Each kind of traffic up to its assumed “regular” or “congested” maximum per linear foot;

Second—Each separate kind of traffic in two or three isolated stretches only, these stretches being in certain exact distances from each other and from the ends of the bridge;

Third—These stretches and distances to be alike, i. e., in the same location for all the fourteen lines of traffic on the two floors;

Fourth—Absolutely no other live load on the bridge.

Such conditions of loading could scarcely be produced, even as an experiment, since trains, cars, wagons and horses each cover certain definite spaces.

The Department of Bridges, therefore, adopted in 1904 the following conditions for designing the truss members, Nos. 1 and 2 providing for *continuous* live loads, while Nos. 3 and 4 take account of extraordinary conditions that might arise from *discontinuous* loading, viz.:

(1) The “regular” live load of 8,000 pounds per linear foot of bridge in one *continuous* stretch for each member, over such a portion of the bridge as to cause maximum stress, and using 20,000 pounds per square inch in tension (reduced for compression) for structural and 30,000 pounds per square inch in tension for nickel-steel as permissible *without wind acting*, and increased 20 per cent. *with wind acting*.

(2) The “congested” live load of 16,000 pounds per linear foot of bridge in one *continuous* stretch for each member, over such a portion of the bridge as to cause maximum stress without wind, using 24,000 pounds per square inch in tension (reduced for compression) for structural and 39,000 pounds per square inch in tension for nickel-steel as permissible.

Of these two conditions, the one causing the greater cross section in each truss member should be taken.

(3) If the "congested" live load of 16,000 pounds per linear foot of bridge in *discontinuous* stretches (without wind acting) causes an absolute maximum compression greater than the dead load tension in a member, such member to be made of a section able to withstand the resulting compression.

(4) Furthermore, in order to be sure that no overstressed condition could result from the "regular" live load of 8,000 pounds per linear foot of bridge, in *discontinuous* stretches causing absolute maximum stress without wind, the unit stresses are to be analyzed for this condition.

For this last condition, the greatest unit stress in tension, with two exceptions, would not be greater than 20,700 pounds, and in compression 18,800 pounds for $l/r=27$ for structural steel and 34,100 pounds in tension for nickel-steel, the exceptions being the verticals U59-L59 and U73-L73, which are stressed in tension to 22,900 (see plate No. 5).

The combination of live load with the *greatest wind pressure* was considered for condition (1) only, and the unit stresses increased by the usual amount of 20 per cent., since it is hardly possible that the bridge will be crowded on the roadway and the promenades in stormy weather. A concurrence of a great wind pressure with conditions (2), (3) or (4) is practically impossible.

The *wind pressure* was assumed at 2,000 pounds per linear foot of bridge, representing about 35 to 40 pounds per square foot, which would be caused by a velocity of wind of 100 miles per hour; and it is clear that in the same degree as the wind pressure increases, the live load on the roadway, promenades, etc., naturally decreases, which is distinctive of highway bridges as compared with railroad bridges, since, in the latter, the maximum live loads will pass irrespective of the velocity of the wind.

Attached plates Nos. 1 and 2 show the position of the live load in *continuous stretches* as assumed by the *Department of Bridges for the design*, and plates Nos. 3 and 4 the live load in *discontinuous stretches* as assumed by the *experts for the report*. Heavy full lines indicate loading for greatest tension; dotted lines, loading for greatest compression in each member.

To illustrate: For the top and bottom chord, in fact for most truss members of the *Island span*, where, according to the reports of the experts, the highest stresses occur, the *continuous* "congested" loading covers the distance between piers Nos. 1 and 4; i. e., both river spans and the *Island span*, a total length of 2,796 feet, while for the *discontinuous* loading the "congested" load would cover the distance of 1,182 feet between piers Nos. 1 and 2, no live load whatever for a distance of 630 feet between piers Nos. 2 and 3, and again, the "congested" live load for a distance of 984 feet between piers Nos. 3 and 4; whether loading is continuous or discontinuous, both anchor spans have to be absolutely free of any live load. The possibility that the "congested" live load of 16,000 pounds per linear foot of bridge, representing fourteen lines of traffic of four different kinds on two inde-

Distribution
of Live Load
for the Great-
est Stresses in
the Island
Span (Com-
pare Table
No. 7)

pendent floors of uniform construction from end to end of the bridge, will cover 1,182 feet, is indeed very small; that another bunching of traffic on all fourteen lines on the two floors precisely in the same location for a length of 984 feet will occur, is still smaller; but what can be the possibility that these two loaded stretches will be 630 feet apart with no live load on the two floors between them, a distribution which even for the working live load of a *single-track* railroad bridge would be exceptional?

It would not be surprising if some engineers, differing honestly in their judgment from that of the designers, would reverse the question and claim that the "congested" live load of 16,000 pounds per linear foot of bridge, even in a *continuous* stretch of 2,796 feet, representing a distribution of cars, wagons, pedestrians, as shown on plate No. 7, from piers Nos. 1 to 4, and no other live load on the two floors from these piers to the abutments, is a practical impossibility; that therefore the Island span has been wastefully designed by the Department of Bridges and that the engineers of the contractor must have known this and should have protested even if the bridge was paid for by the pound.

In order to represent on plate No. 7 the "congested" load of 16,000 pounds per linear foot of bridge, it was necessary to place:

First—On each of the four trolley tracks an uninterrupted line of trolley cars of 1,000 pounds per linear foot of track;

Second—On each of the four rapid transit tracks, an uninterrupted line of trains of 1,700 pounds per linear foot of track;

Third—On the two promenades, a crowd of people weighing 75 pounds per square foot, and

Fourth—On the roadway ($35\frac{1}{2}$ feet wide) four uninterrupted lines of the heaviest electric trucks known to us, of 7 feet by 18 feet net area and weighing loaded 18,000 pounds, all assumed carrying their maximum load and placed two feet apart in the clear. It is questionable whether two feet clear space is sufficient, but this would be the only possibility to reach a live load of 100 pounds per square foot on the roadway. The heaviest electric trucks used in New York City (20,000 pounds, with 8 feet by 20 feet net area) and the heaviest coal wagons (19,000 pounds, drawn by three horses of 1,800 pounds each, and 9 feet 3 inches by 25 feet total net area) would give considerably smaller load per linear foot of bridge, being not only lighter per square foot, but also wider, so that the roadway between the trolley tracks could accommodate only three lines. Considering further that in the great lengths of live load necessary to cause maximum stresses in the main members of the trusses, there will be different kinds of vehicles weighing considerably less (the heaviest touring car weighs 4,800 pounds, with a net area of 5 feet 6 inches by 13 feet, which gives 43 pounds per square foot, assuming two feet clear space), especially since the bridge is, for its greater part, on a grade of $3\frac{1}{2}$ per cent., and that there will be many large floor spaces empty on account of irregular dimensions of the vehicles, it would seem that a live load of even 50 pounds per square foot of roadway could hardly be reached.

The "regular" live load of the specifications (8,000 pounds per linear foot of bridge, represented by 37.5 pounds per square foot on the promenades, 50 pounds per square foot on the roadway, $4 \times 500 = 2,000$ pounds per linear foot of bridge for the trolley tracks, and $4 \times 850 = 3,400$ pounds per linear foot of bridge for the rapid transit tracks) takes into account only one-half of the density of the "congested" live load (shown for most members of the Island span on plate No. 7), simply assuming a greater space between the small units represented by pedestrians, wagons, trolley cars and the trains. This requires no further explanation except for the rapid transit trains.

In the "congested" live load the average weight of a train is assumed at 1,700 pounds per linear foot; since of this amount about three-fourths is for the cars and one-fourth for the passengers, it is clear that a smaller number of passengers reduces the weight of the trains only slightly, so that their share of 6,800 pounds in the "congested" live load of 16,000 pounds per linear foot of bridge cannot be reduced materially for their share in the "regular" live load of 8,000 pounds per linear foot of bridge. However, there are other more important considerations which make it permissible to assume that the stresses due to the trains in the "congested" live load of 16,000 pounds per linear foot of bridge can be reduced one-half to represent those due to the trains in the "regular" live load, viz.:

Influence of
Rapid Transit
Trains on
Stresses

First—The trains are limited to a certain length (express trains about 400 feet long, with eight cars, and local trains about 250 feet long, with five cars), so that they produce in most members only part of the stress which would be produced by the lengths required for the maximum stress, these lengths being, as a rule, considerably greater.

Second—The trains following on the same track have to maintain, on account of safety of operation, a clear interval between them of not less than 2,400 feet (for the stresses in the Island span, for instance, no two trains can therefore be at the same time on the same track between piers Nos. 1 and 4).

Third—The probability that all the trains on the four tracks will pass simultaneously through the danger zones where they contribute their maximum share to the stress in a truss member is very small, as a rule they will be more uniformly distributed over the bridge.

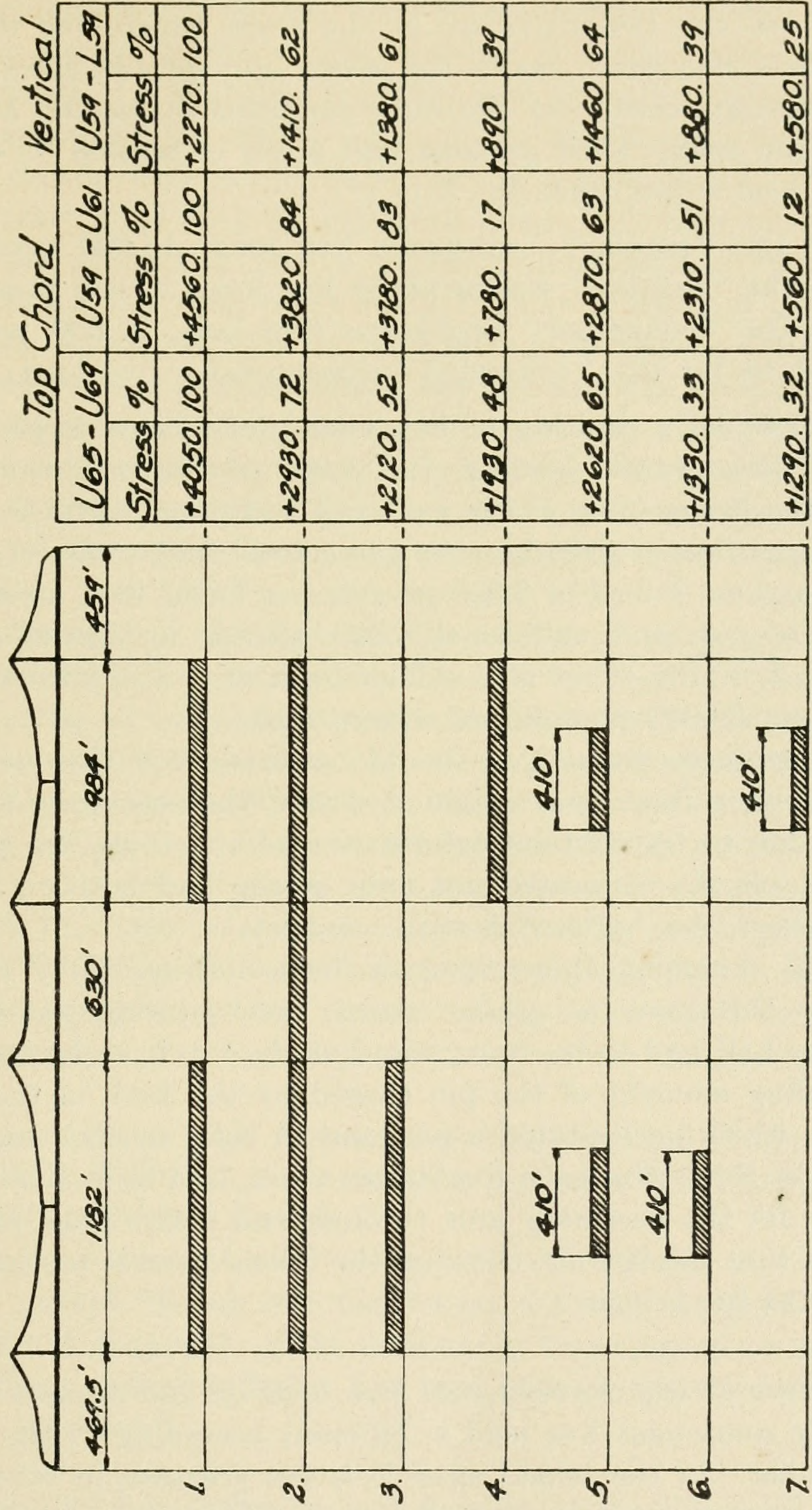
Should it, however, happen that, through some accident, two trains on the same track would follow each other in a shorter clear distance than 2,400 feet (for the greatest stress in the Island span about 1,300 feet clear), the probability that each of the other three tracks will be loaded by two trains following each other in the same clear distance, that these two groups, each of four trains, will pass at the same time through their respective danger zones (for the Island span, four trains in the middle of the one river span,

and four trains in the middle of the other river span), and at the same time that the other three kinds of traffic of "regular" density weighing 8,000 minus 3,400=4,600 pounds per linear foot of bridge will cover even the *continuous* lengths necessary to produce the maximum stresses, is so remote that this combination of the train loads with the other loading should be considered an *extreme* condition, and unit stresses allowed somewhere between the permissible unit stress for the "regular" and for the "congested" live load. As a matter of fact, this condition would cause a unit stress in tension in the vertical U59-L59 of approximately 21,200 pounds and in the top chord U65-U69 (nickel-steel) of approximately 33,500 pounds.

The examples in the following table show the influence on the stresses due to a load of $4 \times 1,700 = 6,800$ pounds per linear foot of bridge of various length and location. (6,800 pounds is the assumed train load on the four tracks per foot of bridge.)

Table Showing Maximum Stresses in Some Members
of the Island Span Produced by Train Load on
the 4 Elevated Tracks of 6800 lbs. per lin.ft. of Bridge

Diagram Showing Length and Location of Train Load



Stresses are given in units of 1000/lbs.
Figures in the second column give the stress in % of the
stress from Case I

Top Chord			Vertical	
U65-U69	U59-U61	U59-L59		
Stress %	Stress %	Stress %		
+4050.	+4560.	+2270.	100	
+2930.	+3820.	+1410.	62	
+2120.	+3780.	+1380.	61	
+1930.	+780.	+890.	39	
+2620.	+2870.	+1460.	64	
+1330.	+2310.	+880.	39	
+1290.	+560.	+580.	25	

The stresses from the 410-ft. trains are only approximate; the influence lines were not given us by the Department of Bridges, the "Loading Key" indicating only the totals for certain stretches of the live load.

Stresses in
Verticals
U59-L59 and
U73-L73

These are the two members in the truss which, for the "congested" *discontinuous* live load, make the most unfavorable showing, since they get the greatest tension for the same condition of loading as the top chord in the Island span. Their greatest unit stress in tension for dead and maximum live load is (see table No. 5):

for the "regular" *continuous* live load, 18,600 pounds;
for the "regular" *discontinuous* live load, 22,900 pounds;
for the "congested" *continuous* live load, 25,600 pounds.;
and would be for the "congested" *discontinuous* live load, 34,200 pounds.

If we assume a position of live load which is more probable than the "regular" *discontinuous* loading, viz., 4,000 pounds per linear foot of bridge covering the bridge *from end to end*, and additional 4,000 pounds per linear foot in *discontinuous* stretches, we get a total unit stress of 20,300 pounds.

If we assume a load of 8,000 pounds per linear foot, covering the bridge *from end to end*, and additional 8,000 pounds in *discontinuous* stretches, merely to show the effect of a milder form of a *discontinuous* "congested" load, we get 29,000 pounds per square inch.

To judge these stresses, it should be considered that verticals have no bending stresses from own weight and that the secondary stresses of these verticals, due to the vertical deformation of the truss, are practically zero, since they are pin connected not only at top and bottom, but also at the middle, where the horizontal strut connects.

Even in designing these verticals for condition (2) ("congested" *continuous*) as this gave the greater stress, their packing was so difficult that their upper half had to be made flared at the top in order to minimize the great bending moment of the pin caused by the kink in the outline of the top chord, which necessitated a pin joint in their center; and their bottom, consisting of three ribs, each five inches thick, had to be made of nickel-steel to provide for the necessary area through and back of the pin hole, in spite of the fact that the bottom chord in the Island span is ten inches wider than in the rest of the bridge.

Difference in
Effect of Dis-
continuous
and Continu-
ous Live Load
Is not
Uniform

The effect of the *discontinuous* live load on the stresses compared with that of the *continuous* live load is different according to the location of the truss member. In the Island span it is the greatest, in other members the difference is zero or approximately zero, as for instance in nearly all the members of the cantilever arms, in the bottom chord L58-L59 of the Island span, etc. This, and the fact that in every bridge, especially of long span, there are certain members which cannot be reduced in their section in the same proportion as the stresses decrease, account for the non-uniformity of the unit stresses, and, therefore, apparent waste of material, deduced

by a certain Engineering paper from the reports of the experts, who considered a *discontinuous* "congested" live load.

As mentioned before, the condition (3), page 25, that is, the "congested" *discontinuous* live load, has been considered for reversal of tensile stresses. If in a riveted member or an eye-bar in tension the elastic limit or yield point should be slightly exceeded, nothing more serious than an imperceptible permanent set would occur which would even decrease (not disappear) if a certain time would elapse before a repetition of this stress, since the yield point rises with an excessive stress. [It may be of interest to mention in this connection that a well-known mechanical engineer proposes to stress above the yield point every eye-bar in full size before using it in the structure in order to raise its yield point.] In making a full-size eye-bar test, the yield point is reached within the first minute, while it may take half an hour of increasing pulling to reach the ultimate strength and break the bar. If, however, an eye-bar were only slightly compressed, the bridge might fall. For this reason, some top chords in the middle of the river spans and at the ends of the anchor arms and some diagonals were made riveted members; and since they had to have a certain "*l over r*" to be stiff, their unit stress is naturally low and their section, therefore, seemingly excessive.

Provision
for Reversal
of Stress in
Tension
Members

Taking a practically *impossible* load, like the "congested" *discontinuous* live load for reversal of tensile stresses, but not for determining the cross section of truss members, is in line with modern specifications for railroad bridges, one of the foremost railroads in the country using for a *working* load of Cooper's E50 a unit stress in tension of 16,000 pounds (considering impact), but also providing for an *extraordinary* live load of Cooper's E100, allowing in this case twice the unit stress for the working load, therefore, for tension, 32,000 pounds per square inch (elastic limit), in order to reach those truss members whose stresses could be reversed by a considerable future increase of live load and other unforeseen circumstances.

In the Transactions of the American Society of Civil Engineers, Vol. 42, p. 547, Mr. H. S. Prichard, discussing the provision for a future increase in live load of 100 per cent., states:

"In a good modern specification the unit stresses allowed are so low that, except for the counterstresses, a bridge designed and built in accordance therewith could reasonably be expected to be able to carry without serious injury a live load at least twice as great as the live load specified."

In the Blackwell's Island Bridge, the impact of the live load need not be considered; coming from three different kinds of traffic (rapid transit, trolleys and wagons) it cannot accumulate, and what little may result is absorbed or diffused by the solid floor and the heavy floor construction before reaching the heavy trusses; and, furthermore, the more the bridge is crowded, the less motion of the live load is possible.

Impact
Negligible

But, assuming, for the sake of argument, that the Blackwell's Island Bridge would carry eight tracks of a *steam* railway with *wooden ties* instead of with a heavy, solid floor, then the impact, according to the specifications of the American Railway Engineering and Maintenance of Way Association, would be for the top and bottom chords of the Island span (also for the previously mentioned verticals U59-L59 and U73-L73):

$$\text{with continuous load } \frac{300}{8 \times 2,796 + 300} = \frac{300}{22,668} = 0.013 = 1.3 \text{ per cent,}$$

$$\text{with discontinuous load } \frac{300}{8 \times 2,166 + 300} = \frac{300}{17,628} = 0.017 = 1.7 \text{ per cent,}$$

since the loaded length has to be taken equal to the total length of single track; in fact, the percentage could even be reduced, since the number of tracks should be increased to twelve, counting also the four lines of wagons on the roadway.

Mr. Prichard, in his paper "Proportioning of Steel Railway Bridge Members" (Proceedings of the Engineers' Society of Western Pennsylvania, 1907, also "Engineering News," September 19th, 1907), reduces the impact to zero for spans of 1,000 feet for one track, of 700 feet for four tracks, and of 300 feet for eight tracks.

**Snow Load
Negligible**

To consider a snow load did not seem necessary. Assuming a thickness of compact snow of 12 inches, or of ice and slush of four inches, representing about 15 pounds per square foot, a larger amount may safely be deducted from the "congested" live load of 100 pounds per square foot (representing a solid mass of people unable to move) or the "regular" live load of 50 pounds per square foot (representing a crowd hardly able to move) of the roadway or the promenades. To combine the absolute maxima of stresses caused by a *discontinuous* "congested" live load representing 14 lines of traffic with those caused by a *discontinuous* greatest wind load covering the same *discontinuous* stretches and no others and do probably the same thing with a snow load would be a *reductio ad absurdum*.

**Temperature
Stresses
Negligible**

From a uniform change of temperature the trusses of the Blackwell's Island Bridge receive stresses only on account of the action of the rocker posts in the middle of the river spans. When the two adjoining cantilever arms expand or contract, these rockers assume an inclined position, which results in the lowering of the one and the lifting of the other cantilever arm, causing insignificant stresses.

By order of the Department of Bridges, early in 1904, the originally intended pin at the bottom of each tower post was replaced by a flat bearing; consequently, the towers receive bending stresses from any horizontal deflection of their top. From a change of temperature, the top of tower U75-L75 only is deflected, since this is the only tower where the trusses are able to slide, being rigidly connected to the piers at the other three towers. For a change of $\pm 60^\circ$ the deflection at the top is $3\frac{1}{4}$ inches and the stress at the bottom of tower 3,200 pounds per square inch in the extreme fiber

of the section, while the actual stress for the dead and the "regular" *continuous* live load is 15,200 pounds per square inch. The total extreme fiber stress, therefore, is only 18,400 pounds per square inch, which is well below 20,000 pounds per square inch permissible for a uniformly distributed direct stress.

To illustrate the effect of the "regular" and the "congested" live load of the Blackwell's Island Bridge on an actually used bridge of similar magnitude, the writer derived some figures from a publication by Professor Melan concerning the stiffening trusses of the Williamsburg Bridge. (Handbuch der Ingenieur-wissenschaften, Arch and Suspension Bridges.) This bridge has four trolley tracks, only two rapid transit tracks (similarly to the original plans for the Blackwell's Island Bridge), but has two roadways 20 feet each, two foot-walks 10 feet 6 inches each and two "bicycle" paths 7 feet each; it has, therefore, 4 feet 6 inches wider roadways, and 13 feet wider promenades than the Blackwell's Island Bridge. This would give a "congested" load of $16,000 - 3,400 + 450 + 950 = 14,000$ pounds per linear foot of bridge and half of this, that is, 7,000 pounds per linear foot of bridge, as "regular" load; but, assuming only those originally specified for the Blackwell's Island Bridge (with two rapid transit tracks), namely, 12,600 and 6,300 respectively, which is 10 per cent. less, we get for the unit stresses in the stiffening trusses the following maxima in pounds:

Effect of
"Regular"
and "Con-
gested" Live
Load on
Trusses of
Williamsburg
Bridge

TOP CHORD		
	<i>Tension</i>	<i>Compression</i>
For "regular" live load alone.....	21,600	20,800
For "regular" live load and temperature	24,600	23,600
For "regular" live load, temperature and wind	32,200	30,200
For "congested" live load alone	43,200	41,600
For "congested" live load and temperature	46,200	44,400
For "congested" live load, temperature and wind	52,000	50,000
BOTTOM CHORD		
For "regular" live load alone	25,600	19,800
For "regular" live load and temperature	28,200	21,400
For "regular" live load, temperature and wind.....	36,000	27,400
For "congested" live load alone	51,200	39,600
For "congested" live load and temperature	53,800	40,700
For "congested" live load, temperature and wind.....	59,400	46,600

To cause these stresses, the live load has to extend for only about 1,100 feet in a *continuous* stretch from the pier, and may be of any length beyond the pier, a condition which is certainly far more possible than even the *continuous* live load mentioned above for the island span of the Blackwell's Island Bridge. These figures do not take into account any snow load whatever.

To treat the Williamsburg Bridge as rigorously as the Blackwell's Island Bridge, it should be considered that a suspension bridge has to be so adjusted that, for no live load on it, the ends of the stiffening trusses have no reaction and that it is very probable that this condition is not fulfilled at

the present time in the Williamsburg Bridge, which would add some more thousand pounds to the chord stresses in the stiffening trusses.

To compare these figures with those for the Blackwell's Island Bridge it should also be kept in mind that in a stiffening truss temperature stresses are positively sure, like dead load stresses in ordinary trusses, since the condition of no reaction of the stiffening trusses can be accomplished only for a certain temperature, the stiffening trusses being deflected and, therefore, stressed for any other temperature.

Up to September 1908, the two elevated tracks on the Williamsburg Bridge were not in use, but the bridge had been open to the other traffic since February 1905; taking a "regular" live load of only $6,300-1,700=4,600$ and a "congested" load of 9,200, the above-given unit stresses would be reduced by only about one-fourth, would therefore be in maximum:

for the "regular" live load, 29,000 pounds in tension, and 23,000 pounds in compression, and

for the "congested" live load, 44,000 pounds in tension, and 35,000 pounds in compression,

considerably greater, therefore, than the corresponding values even for the whole traffic of 8,000 pounds per linear foot of bridge as "regular" live load, or of 16,000 pounds per linear foot of bridge as "congested" live load in *discontinuous* stretches for the Blackwell's Island Bridge.

As a matter of fact, the stiffening trusses of the Williamsburg Bridge were proportioned for a load of 4,500 pounds per linear foot of bridge, this to represent the whole traffic including the two elevated railroad tracks; no "congested" or extraordinary load was considered.

Dead Loads
Used in 1904
Compared
with Subse-
quent
Assumptions

The statement has been made that the dead load stresses of the Blackwell's Island Bridge were considerably underestimated. This is only partly correct. The designers realized from the beginning that the dead load of a highway bridge, especially of one of this magnitude, is of the greatest importance. For this reason, the floor-beams were curved in their top chord, to follow the crowning of the paving in order to reduce its weight to a minimum. There are city bridges in existence for which the weight of the paving was assumed for the design with 75 pounds per square foot, while it actually amounts to 150 pounds, and even more, eating up all allowance for live load. The Department estimated the weight of the paving material for the lower floor, concrete at 40 pounds, wood blocks at 20 pounds, and for the promenades at 20 pounds per square foot.

The original estimate of the steel weight, made by the Department of Bridges, was 86,000,000 pounds. Beginning early in 1904, the Department ordered changes and additions made, such as changing the tracks of the upper floor from longitudinal ties with bulb angles to wooden cross ties, addition of two elevated railroad tracks, reduction of pressure of the tower bases on the masonry, replacing the pin-bearing of the bottom of the towers by a flat bearing, additional sway bracing, bottom and top laterals, top struts, bracing between stringers, secondary verticals from the middle pins

to the top chord, etc. (See "Report of the Commissioner of the Department of Bridges, 1904," printed in 1905, pages 31, 32, 33, 34), all this increasing the steel weight to about 95,000,000 pounds, with a total dead load of 116,000,000 pounds. New dead load stress sheets were made by the Department, and at their suggestion, also, by the contractor subject to their approval, in order to hasten the time of the ordering of the material. To these dead load stresses, the live load stresses calculated by the Department for the new live load (with four elevated railroad tracks) were added, the sections of the truss members determined and all section sheets which the contractor prepared for use in his own drawing-room were submitted to and finally approved by the Department with certain changes.

Shop drawings for the Island span were now made by the contractor and approved by the Department with changes, and the material ordered. In the meantime, new dead load concentrations and new stress sheets were worked out, finished about November, 1904, based on a new estimate of the steel weight of 100,750,000 pounds, giving with the originally prescribed weight of paving, pipes, etc., a total dead load of 122,130,000 pounds. As far as possible, the old stress and section sheets were then revised. To change the material for the Island span, at that time in the process of manufacture in the Bridge Shop, was impossible, and it is here that some dead load stresses figured with these new dead load concentrations slightly overrun the older figures. The actual shipping weight of the bridge proper (deducting test eye-bars and other test material) is about 105,150,000 pounds, which, of course, includes all changes and additions made after November, 1904, as reinforcing certain trolley stringers for elevated railroad trains, strengthening of the anchorages, etc., giving a total dead load of 125,680,000 pounds. It should be stated in this connection that certain changes in the design, which caused a considerable addition to the steel weight, did not affect the dead load stresses, as strengthening of the anchorages, increase in weight of tower bases and towers, in the weight of the box girders between trusses at the piers, in the weight of the shoes on top of the towers, etc.

About a year ago, after the whole bridge material had been fabricated, the writer, for his own satisfaction, had new stress sheets prepared, using the dead load concentrations of November, 1904, and the stresses from the continuous "regular" live load given us by the Department of Bridges, considering also the fiber stresses due to the own weight of the truss members, and satisfied his own mind that the bridge was able to carry the specified traffic. The dead load concentrations of November, 1904 were considered sufficiently accurate, as they were based on a total dead load of 122,130,000 pounds, while the actual dead load using the shipped steel weight was only 3 per cent. higher, that is, 125,680,000.

In the meantime, the Department changed the plans for the paving. As mentioned above, the original paving, consisting of concrete and wooden blocks, was intended to be as light as possible and to extend only between the curbs for 35½ feet; the tee-rails for the inside trolley tracks were to be 5 inches high and their top about 6 inches above top of stringer; the buckle

plates between the rails merely covered with asphalt and the rails free. The Department now decided to extend the paving over the whole roadway between the trusses (53 feet) with heavy cast-iron curbs and of much greater thickness, using 7-inch grooved rails with their tops nearly 12 inches above the top of the stringer, and imbedded entirely in concrete. If ordinary concrete and not cinder concrete be used, this makes the paving about 1,341 pounds per linear foot heavier than originally intended and assumed in the calculations, increasing the total dead load to 132,300,000 pounds, *which figure was used by the experts in their calculations of the dead load stresses.* The change in the paving was an after-thought, not concerning the contractor, since at that time the bridge was completely manufactured, and, in its greatest part, erected. This, and the fact that the additional paving, being of uniform weight over the whole bridge, forms a larger percentage of the dead load near the ends of the cantilever arms, where its influence on the stresses is the greatest, accounts for the great difference in the dead load stresses as used for the design of the bridge and as calculated by Professor Burr and Messrs. Boller & Hodge.

The writer understands that the Department of Bridges recalculated the dead load stresses after arranging for heavier paving, and that they got the same excessive figures as the experts, but that they had not come to any conclusion pending a recalculation of the live load stresses when the experts were invited to make a report.

Although it does not seem within the scope of this report, it may nevertheless be useful to mention, for the information of those critics of the Blackwell's Island Bridge who are not experts in bridge engineering, that the excellence of the design of different bridges cannot be judged by the proportion of the steel weight necessary to carry a pound of live load. This depends for bridges of similar character almost entirely on the length of the span. For a very short span, one pound of steel may suffice to carry one hundred pounds of live load, and for a very long span the reverse may be the case. The proportion of steel weight to the assumed working live load is for the following bridges approximately:

	<i>Greatest Span</i>	<i>Proportion</i>
Thebes bridge.....	671'	1.0
Monongahela bridge	812'	1.0
Memphis bridge (one track only)	790'	1.7
Blackwell's Island bridge (heavy solid floor)	1,182'	3.4
Quebec bridge	1,800'	4.3
Firth of Forth bridge	1,710'	4.7

The Blackwell's Island Bridge carries a heavy solid floor, the others only wooden ties. If the former had had a light wooden floor, possibly 2.5 pounds of steel instead of 3.4 pounds would have been sufficient to carry a pound of live load.

The report of Messrs. Boller & Hodge states:

"We have made no additions for reverse stresses, as the specifications state that the sections are to be computed for the stress requiring the greatest area, so that the unit stresses we have shown are the direct stresses from dead and live loads without any additions for reverse stresses, snow, wind, impact, or secondary stresses."

Reverse and
Secondary
Stresses

The effect of snow, wind and impact has been mentioned before; there remains now to discuss reverse and secondary stresses.

In most specifications for railroad bridges written in the last ten years, the effect of a reversal of stress is only considered if the reversal occurs in immediate succession, as "counter stresses in web members or chords in continuous trusses" (this is the usual wording) under the passage of the same train; in highway bridges, it is as a rule disregarded. It is hard to conceive how a reversal of stress could happen in the Blackwell's Island Bridge in immediate succession with the different kinds of live load.

The report of Messrs. Boller & Hodge states:

"The secondary stresses due to distortion of the true figures of the trusses by the live load are quite considerable, as the vertical deflection of the point L37 is $18\frac{5}{10}$ inches, and of the point L91, $14\frac{2}{10}$ inches for a live load of 3,000 pounds per 1 linear foot of truss.

"We have made a careful analytical computation of the horizontal movement of the point U17 (and other similar points over the main piers) caused by this distortion."

To avoid possible misunderstanding, it may be well to point out that the horizontal movement of point U17 (top of tower) is not caused by the vertical deflection of point U37 (where the cantilever arms join), but merely by one contributing cause to this vertical deflection, namely the deformation of the anchor arm, the other more important cause contributing to the vertical deflection of point U37 being the deformation of the cantilever itself which does not affect the horizontal movement of the top of the tower.

The effect of secondary stresses, which lately has been given its deserved prominence, should, however, not be over-estimated. In the case of rolled or riveted girders, in bending, we allow the same extreme fiber stresses as for axial stresses, since the chords are concentrated as in an articulated truss, being kept apart by the web. In bending of a compact section, however, like a pin, we allow up to 50 per cent. more stress, which is not only based on experience with pins but also on theory, or rather the non-fulfilment of one assumption of the theory of bending, viz., that the fibers deform independently. The fiber stresses not being uniformly distributed over the whole section, only the outer rows having the maximum stress, the less-stressed adjoining fibers must hold in position, that is, *relieve*, the overstressed fibers. The assumption that a cross section remains a plane after bending is, in reality, not strictly fulfilled, even for stresses within the elastic limit, and

the beam is actually stronger than would seem from the theory of bending. The allowable bending stress should be derived from bending tests, but the results would vary with the shape of the cross section.

The secondary stresses caused by the own weight of the members and by the distortion of the truss are bending stresses, and as such could be allowed for the compact sections of the truss members of the Blackwell's Island Bridge at least 20 per cent. higher units than for axial tension. Cooper's standard specifications (and others) take this into account, specifying "if the fiber strain resulting from the weight only of any member exceeds 10 per cent. of the allowed unit strain on such member, such excess must be considered in proportioning the area." Most specifications disregard these stresses entirely.

The original specifications (1903) did not contain a clause to consider these fiber stresses. In April, 1904, the Department decided to take them into account, this to the writer's recollection having been done in order to be on the safe side in proportioning the truss members liberally, as it was realized at that time that the actual dead load stresses may easily overrun.

The Quebec
Bridge Report
and the
Blackwell's
Island Bridge

The writer is not aware that the failure of the Quebec Bridge caused any "suspicion" (as stated in a technical paper a few weeks ago) of the safety of the Blackwell's Island Bridge. He does not think that such was the case until the Royal Canadian Commission, which was appointed "to inquire into the cause of the collapse of the Quebec Bridge," made the following published statements:

"By reference to the table, it will be seen that the specified stresses for the Quebec Bridge, under working conditions, are in advance of current practice, and we believe they are without precedent in the history of bridge-engineering. Under extreme conditions, the Quebec Bridge stresses are in general harmony with those permitted in the Blackwell's Island Bridge." (Page 148 of their report.)

And, comparing the bottom chords of the two bridges (page 140):

"The development of the detail plans of the Blackwell's Island Bridge was contemporaneous with that of the Quebec Bridge plans; the Quebec designers had not access to the Blackwell's Island plans. In fairness to the Quebec Bridge designers, however, it should be pointed out that in the Blackwell's Island Bridge the proportions of many of the details are much more nearly in accord with Quebec Bridge practice than are those of the earlier bridges, although the principles of the designs are very different."

Now, the Blackwell's Island Bridge was not on trial and, being unfinished, a comparison of unit stresses allowed for the sum of the dead and imaginary live loads did not prove anything as far as the Quebec failure was concerned, merely creating a false impression. It would not even have proven anything if the Blackwell's Island Bridge had been in use, since the Quebec Bridge was a steam railway bridge, designed to carry two tracks of comparatively

light loading, and neglecting entirely any live load on the two roadways of 17 feet width each, while the Blackwell's Island Bridge was designed to carry eight tracks for electric traffic, which were *assumed* loaded with a heavy load in addition to a heavy load on the roadways and promenades, a total live load unprecedented for any highway bridge, even in New York City.

The Commission compares the six largest cantilever bridges as to the live loads and permissible unit stresses assumed when they were designed. It would have been more interesting to have indicated the greatest live loads actually used at the present time on the four which are in use.

The Forth Bridge (designer Sir Benjamin Baker) is designed for two tracks, each loaded with approximately Cooper's E22 loading, and a compression allowed of 17,000 pounds per square inch, with no reduction for buckling, and 16,350 pounds per square inch for tension; no impact is considered. The actual live load on this bridge has certainly increased well beyond the original assumptions, and may have to be doubled and tripled in the near future. (In this country, the Atchison, Topeka & Santa Fe use Cooper's E66, while the New York, New Haven, the Carolina, Clinchfield & Ohio, the Lake Erie & Western, the Crescent line, the Tidewater R. R., specify a working live load of Cooper's E60.)

The Monongahela Bridge (designers Messrs. Boller & Hodge) is designed for two tracks, each for E45 working load, and considering 100 per cent. impact to the live load, allows 21,000 pounds per square inch compression, with no reduction for l/r smaller than 40, and 22,000 pounds per square inch for tension.

The Thebes Bridge (designer Mr. R. Modjeski) is designed for two tracks, each for a working load of a little less than E50, and, considering 100 per cent. impact to the live load, allows 21,000 pounds per square inch compression, with no reduction for l/r smaller than about 45, and a tension of 20,000 pounds per square inch.

The Memphis Bridge (designer Mr. G. S. Morison) is designed for one track for a loading of E40, allowing a compression of 14,000 pounds per square inch, with no reduction for l/r smaller than 45, and no consideration of impact; and a tension of 20,000 pounds per square inch, adding an impact of 100 per cent. to the live load.

The Quebec Bridge was designed for two tracks, carrying

(1) A *working* load for certain members hardly equal to E30, with a permissible unit tension and compression reaching 21,200 pounds per square inch (f. i. in the compression chords) with impact considered by means of a "minimum over maximum" formula, but no reduction for buckling made for l/r smaller than 50 (no reduction, therefore, for the compression chords); or,

(2) An *extraordinary* load of 50 per cent. more, therefore, about E45, with a permissible unit stress of 24,000 pounds per square inch in tension, and also in compression for the chords and main diagonals, and of 24,000—100 l/r in compression for the posts. (A snow load of 1,600 pounds per linear foot of bridge was added to the specifications in June, 1905, after the stress sheets and some shop drawings were approved.)

In order to compare the "safety" of these five bridges, we would not only have to compare the unit stresses, the assumed live loads, the loaded length causing maximum stresses, the assumed impacts, etc., but also investigate how closely the assumed live loads approach the present everyday working loads, and the possibility of a future increase in the latter for which, with the exception of the Quebec Bridge, no provision was made, no *extraordinary* load having been specified with corresponding increase in unit stresses.

The comparison of the design of these structures, all of them being railroad bridges, is therefore, not a simple matter; how much more complex to compare them with the Blackwell's Island Bridge, which was designed for a highway traffic of unprecedented complexity.

In a double-track railroad bridge, the trusses are stressed close to the permissible maximum every time two trains meet on the bridge, and yet most engineers allow higher unit stresses than for single-track bridges, or reduce the live load per track.

Sir Benjamin Baker, in a paper on "Working Stress of Iron and Steel," read before the American Society of Mechanical Engineers in 1886, remarks justly (see Railroad Gazette, February 18th, 1887): "A machine or bridge can only be well proportioned by carefully considering the special conditions of the case, in the light of experimental data and past experience. A string of formulæ will not make an engineer."

In the report of the Royal Commission on the Quebec Bridge, page 161, are given the specifications revised by Mr. Theodore Cooper in 1904 as follows:

First case.—"The maximum strains produced by the following live loads and wind shall be used for proportioning all members of the trusses or towers:

"(1) A continuous train of any length, weighing 3,000 pounds per foot of track, moving in either direction on each track.

"(2) A train 900 feet long, consisting of two E33 engines, followed by a load of 3,300 pounds per linear foot, upon each railroad track and moving in either direction.

"(3) A train 550 feet long, consisting of one E40 engine, followed by 4,000 pounds per linear foot of track, on each track.

"(4) For the suspended span a lateral wind force of 700 pounds per linear foot of the top chord and 1,700 pounds per linear foot of the lower chord, one-half of which shall be used for lateral and diagonal bracing.

"For the cantilever and anchor arms, a lateral force of 500 pounds on the top chord and 1,000 pounds on the lower chord, per linear foot, in addition to the wind force on the suspended span, shall be considered.

"Only one-third of this maximum wind force need be considered in proportioning the chords. It shall be considered as a live load.

Unless this increases the strains due to the live and dead loads only more than 25 per cent. the sections need not be increased."

Second case.—On page 162, provision for a *future increase* of 50 per cent. in the train loads is called for.

Mr. C. C. Schneider, who was appointed on behalf of the Canadian Government "to inquire into and pass upon the sufficiency of the present design of the Quebec Bridge," independently of the Royal Commissioners, states as follows:

"The first case will be called hereafter the working load, and the second case the extreme load. The strains produced by the working load, which is by no means excessive, should leave a reasonable margin for safety. The strains produced by the extreme loads should remain within the elastic limit of the material."

On page 156, Mr. Schneider reports as follows:

"The extreme unit strains within which in the writer's judgment the structure may be considered to be able to sustain the loads provided for in the specifications, are:

(1) For the dead and live loads combined with the snow load: For tension, 21,000 pounds per square inch of net section; for compression, 21,000—90 *l/r* per square inch of gross section.

"(2) For the extreme provisions of one and one-half times the live load, dead and snow loads, combined with one-third of the wind strains: For tension, 24,000 pounds per square inch of net section, for compression, 24,000—100 *l/r* per square inch of gross section."

From this follows that Mr. Schneider considered for the Quebec Bridge a live load (from less than E30 to less than E40 for the different truss members) which in all probability would have occurred every time two trains had met on the center span as "working load," and a live load 50 per cent. higher as "extreme load," and *for these loads* together with the dead and specified snow loads he assumes, but "does not advocate," unit stresses not exceeding 21,000 and 24,000 pounds respectively. These live loads cannot be compared with those of the Blackwell's Island Bridge, that is 8,000 pounds per linear foot of bridge as "regular" and 16,000 pounds per linear foot of bridge as "congested."

It should also be remembered that the Quebec Bridge was built to carry besides the double-track steam railway also "a roadway on each side 17 feet wide in the clear, suitable for ordinary highway traffic with one electric railway track on each roadway," and that for the roadways absolutely no allowance was made in the live load for the trusses, originally not even for a snow load, so that at least the effect of a snow load should be added to the originally specified unit stresses before a comparison is made. In the Blackwell's Island Bridge a snow load can safely be deducted from the assumed uniformly distributed live load on the roadways and footwalks, since it is only reasonable to assume that the live load will be reduced by that amount.

Considering the probabilities of the different live loads specified for the Quebec Bridge and the Blackwell's Island Bridge, it would seem more correct to compare the effect of the "extreme" load plus the snow load on the Quebec Bridge with the effect of the "regular" load of 8,000 pounds per linear foot of bridge, in *continuous* stretches on the Blackwell's Island Bridge. This would give, for instance, for the bottom chords a greatest unit stress in compression of 30,100 pounds for the Quebec Bridge, to be compared with 17,000 pounds for the Blackwell's Island Bridge.

The following greatest unit stresses in compression for the bottom chords may be of interest:

Quebec Bridge

For "extreme" condition, after completion	30,100 (24,000 specified)
For "working" condition, after completion	24,600 (20,500 specified)
For dead load alone after completion	17,500

Blackwell's Island Bridge

With a live load of 16,000 pounds per linear foot after completion	22,100 (21,300 specified)
With a live load of 8,000 pounds per linear foot after completion	17,000 (17,600 specified)
For dead load alone after completion	11,900

To illustrate further Mr. Schneider's ideas of the difference between "working" and "extreme" load, the following taken from his closing discussion of his paper on "The Structural Design of Buildings" (Transactions of the American Society of Civil Engineers, 1905) may be of interest:

"Some of the discussors have discovered that it is possible to obtain greater loads than those specified. The writer wishes to state that he was fully aware of all these possibilities of loading, and, also, has carefully studied all the literature on the subject of experiments on the weights of crowds of people, . . . and has given all these facts due consideration in determining the live loads to be specified for buildings. . . . The writer, in determining upon the live loads of buildings of various kinds, as stated before, has been guided by what is now considered the best practice in bridge-building. It is not a question of how much load of any kind it is possible to pile on a square foot of floor area. The question is to make the buildings absolutely safe without wasting large quantities of material in places where it is not needed. A structure should be proportioned for a working load with the ordinary unit strains, and provision made for a congested load with unit strains well within the elastic limit, so that the structure may yet be safe under such extraordinary conditions of loading. The working load should be the probable maximum load which may be reasonably expected to occur, while the congested load is a load which is improbable, but within the reach of possibility."—

"Railroad bridges are now generally designed for a possible future increase in the weight of locomotives and rolling stock, but no engineer would think of designing a long-span, double-track bridge under the assumption that both tracks will be entirely covered with the heaviest type of locomotives, or cars carrying heavy ordnance and using the ordinary unit strain for that condition.

"The writer desires to emphasize the fact that such unusual and extraordinary conditions of loading have been considered in his specifications, in assuming that it is rational and unquestionably good practice to allow a unit strain of 25 per cent. in excess of the ordinary working strain, or 20,000 pounds per square inch for congested loads and 50 per cent. in excess, or 24,000 pounds per square inch for very extreme cases.

"There are railroad bridges in existence now, some members of which are strained to 24,000 pounds per square inch, including impact, almost every time a train passes."

The statement has been made that, in the last years, the permissible unit stresses used in the design of bridges have been increased; and "high" permissible unit stresses and our "ignorance" in regard to the proper design of compression members were widely criticized ever since the Quebec Bridge failed.

High Unit
Stresses and
Design of
Compression
Members

The writer, based on his own investigations, is of the opinion that if the design of some of the older bridges as described in the Transactions of the American Society of Civil Engineers and other publications would be analyzed, especially with the *actual* and not the assumed dead loads and the assumed light live loads properly considered, the statement concerning unit stresses would appear in a somewhat different light. The impression was probably caused by a few specifications allowing high unit stresses for a practically *impossible excessive* live load in order to provide amply for counterstresses in tension members which might be caused by a *possible* increase of the ordinary working load and other unforeseen circumstances, without increasing unduly the costs of the bridge. This precaution may be of great value (as mentioned before) while the additional weight is only small.

Sir Benjamin Baker in the above-named paper (in 1886) states as follows:

"The writer has availed himself of the opportunity afforded by the large use of special plant and machinery at the Forth Bridge works to note the influence of varying stresses on full-sized riveted steel girders. These observations are still in progress and can be but very briefly referred to herein. In one instance the lever of a large plate-bending press is of box-girder section, built up of eight 4 x 4 x $\frac{5}{8}$ inch angle bars, 13 x $\frac{3}{4}$ inch web plates, and two 17 x $\frac{1}{2}$ x $\frac{5}{8}$ inch flanges. The span is 15 feet 8 inches, and the ordinary daily working stress on the metal is 43,000 pounds, and occasionally 57,000 pounds per

square inch. Many thousand applications of this stress have been made, and the beam has taken a permanent set of seven-eighths of an inch, but so far is otherwise intact."—

"As regards the important question of the proper working stress on iron and steel, the writer's experience leads him to believe that both the old-fashioned government regulations, giving the same limiting stress for all kinds of loading, and the modern formula, based chiefly on Wöhler's experiments, fail to meet the just requirements of the practical engineer. It is, in many cases, a great economical advantage and convenience to have reference not merely to the variation of stress, but also to the probable number of applications. For example, the writer knows the bending press box girder lever, previously referred to, will last its time, although the working stress is about two-thirds of the ultimate strength of the material; and it would have been a mere waste of money to make it four times as strong, and so give it the factor of safety of six, usual and proper enough for a structure such as the Elevated Railway of New York, where a practical infinite number of repetitions of stress have to be provided for."—

"The Conway Tubular Bridge, which has carried the heavy traffic of the London & Northwestern Railway for the past thirty-six years, is 412 feet in span, and under its own weight the tensile stress is 13,000 pounds per square inch. With ordinary trains, the stress is 17,000 pounds, and, if covered with the heaviest engines in use on the line, 20,000 pounds per square inch. The ultimate strength of the riveted structure is about 42,000 pounds per square inch. No indications of weakness have developed during the thirty-six years' working nor anything to suggest that the factor of safety of, say 2 to $2\frac{1}{2}$, is unduly low."—

"Twenty years ago, being uncontrolled by government regulations, the writer adopted a working stress of 16,000 pounds per square inch* on many large iron girders carrying a heavy dead load, although at that time a departure from the usual 11,200 pounds per square inch was regarded with suspicion. The results of modern research have, however, now given the engineer a free hand, and the British five tons per square inch and the Continental six kilos per square millimeter have ceased to be regarded with superstitious reverence."

Professor Engesser, in his well-known book on secondary stresses, published in 1893, writes as follows:

"For repeated stresses, a rupture of the material takes place for stresses below the ultimate strength, depending on the range of the stress. This ultimate for repeated stresses is the lower, the greater the range, and has its lowest value for alternate tension and

* This refers to working live load without impact and to iron, not steel. The writer was unable to find any heavier bridge built by Mr. Baker at that time than the Mersey bridge, built 1869, near Liverpool, a span of 305 feet with a weight of steel probably only three-fourths of the assumed live load.

equal compression. The ultimate for repeated stresses is for stresses of the same sign greater than the elastic limit, for stresses of opposite signs smaller than the elastic limit. It should be remembered, however, that the tests made by Wöhler and Bauschinger were made with stresses repeated rapidly in immediate succession, while the conditions in a bridge member are entirely different.—The permissible unit stress should remain therefore for the ordinary working load below the ultimate for repeated stresses, while for extraordinary conditions it may exceed the ultimate for repeated stresses without danger to the structure”—

in other words, for extraordinary conditions, occurring only a few times, if ever, during the lifetime of the bridge, it could for stresses of the same sign without danger even exceed the elastic limit.

The statement concerning our ignorance in regard to the design of compression members has to be taken with reservation. Bridge Engineering is not an exact science. Any engineer who prepares a design and follows it through the drawing-room and shop will admit that there are a hundred and one questions where his “string of formulæ” proves insufficient and he has to rely on his judgment, consciously and unconsciously derived from past experience. The lacing of compression members is a detail and, like other details, can only partly be analyzed theoretically. The connection of floor-beams to the posts, particularly of floor-beams cut out for the pins, in fact, all riveted connections, including splices, especially such with several ribs and many rivets, are more complex matters than the lacing of a column. But, after all, is our knowledge of tension members as complete as pretended? We have tested eye-bars and single wires or small ropes in full size, but these are not tension members, merely parts of a tension member, and about the distribution of stress over the cross-section of a cable, for instance, we have only approximate ideas.

Table No. 5 gives unit stresses of those main members of the trusses which, according to the reports of the experts, show most unfavorably. The unit stresses due to the dead load were taken not from the stress sheets used in the design, but, to avoid criticism, were derived from the dead load stresses of the experts, reducing the stresses by an amount due to the reduction of the paving, pipes, railings, to their originally assumed weight. Six sets of unit stresses due to three different conditions of the “congested” and of the “regular” live load were compiled from the “Loading Key” and combined with the stresses from dead load, as follows:

Appended
Stress Sheet
Table No. 5

- (1) “Congested” live load in *continuous* stretches;
- (2) “Congested” live load in *discontinuous* stretches;
- (3) One-half of the “congested” live load *over the whole bridge* (from end to end) and one-half in *discontinuous* stretches;
- (4) “Regular” live load in *continuous* stretches;
- (5) “Regular” live load in *discontinuous* stretches;
- (6) One-half of “regular” live load *over the whole bridge* (from end to end) and one-half in *discontinuous* stretches.

Recommendations

Based on these figures and the foregoing discussion on the probability of the different live loads, the writer recommends the following:

(1) The paving should be reduced to its originally intended weight on the whole bridge, or, which may be even more effective for its final purpose of reducing certain stresses, reduce the paving only on the river spans, leaving on the Island span and the two anchor arms the heavy paving, as the writer suggested to you about three months ago, before the reports of the experts were known.*

(2) With the paving altered in this way, the bridge is safe for the intended traffic, viz.: two promenades of 11 feet each, a roadway of $35\frac{1}{2}$ feet, four trolley tracks of 1,000 pounds per linear foot, and four elevated railroad tracks of 1,700 pounds per linear foot.

(3) The experts state that, since the bridge was designed, the weight of trolleys and rapid transit trains increased, the former from 1,000 to 1,460 and the latter from 1,700 to 1,810 pounds per linear foot. This may or may not be only a passing phase in the development of the rolling stock of electric traffic. In the next few years, before any rapid transit traffic will cross the Blackwell's Island Bridge, many changes in the rolling stock may occur, the cars may get longer, or the character of the traffic may change (moving seat platforms, etc.), and the calculations will have to be revised for the new conditions.

(4) To remove the stringers designed to carry two elevated railroad tracks is not necessary, and not advisable, as they now support the footwalks and new ones would have to be provided to replace them, and, as nobody can tell whether they may not be of use within the next ten years, should the weight or character of rapid transit traffic change.

(5) A thorough investigation of the actual traffic on the existing bridges in New York City should be made by the Engineering staff of the Department of Bridges (not by laymen) in order to prove conclusively that the traffic needs no police regulations, beyond those customary in the case of ordinary city streets on which "congestion" or "bunching" of traffic to the extent of 50 pounds per square foot over any great area would not be tolerated by the police, and to establish again that sense of proportion which, in this whole controversy, seems to have been lost.

In taking instantaneous photographs of the traffic of the Williamsburg, the Brooklyn and other bridges, possibly also on crowded streets (from an upper story) and approximating from them the live load over a certain length, a fair estimate of the weight per square foot at different hours and its maximum could be established, while the usual assumptions of 50, 75 or 100 pounds per square foot, derived from experiments in buildings with stationary live loads, are for floors of long bridges hardly better than guesswork.

Very truly yours,

F. C. KUNZ, Chief Engineer.

*See Supplement, page 47

Supplement to the Report

Steelton, Pa., December 28th, 1908.

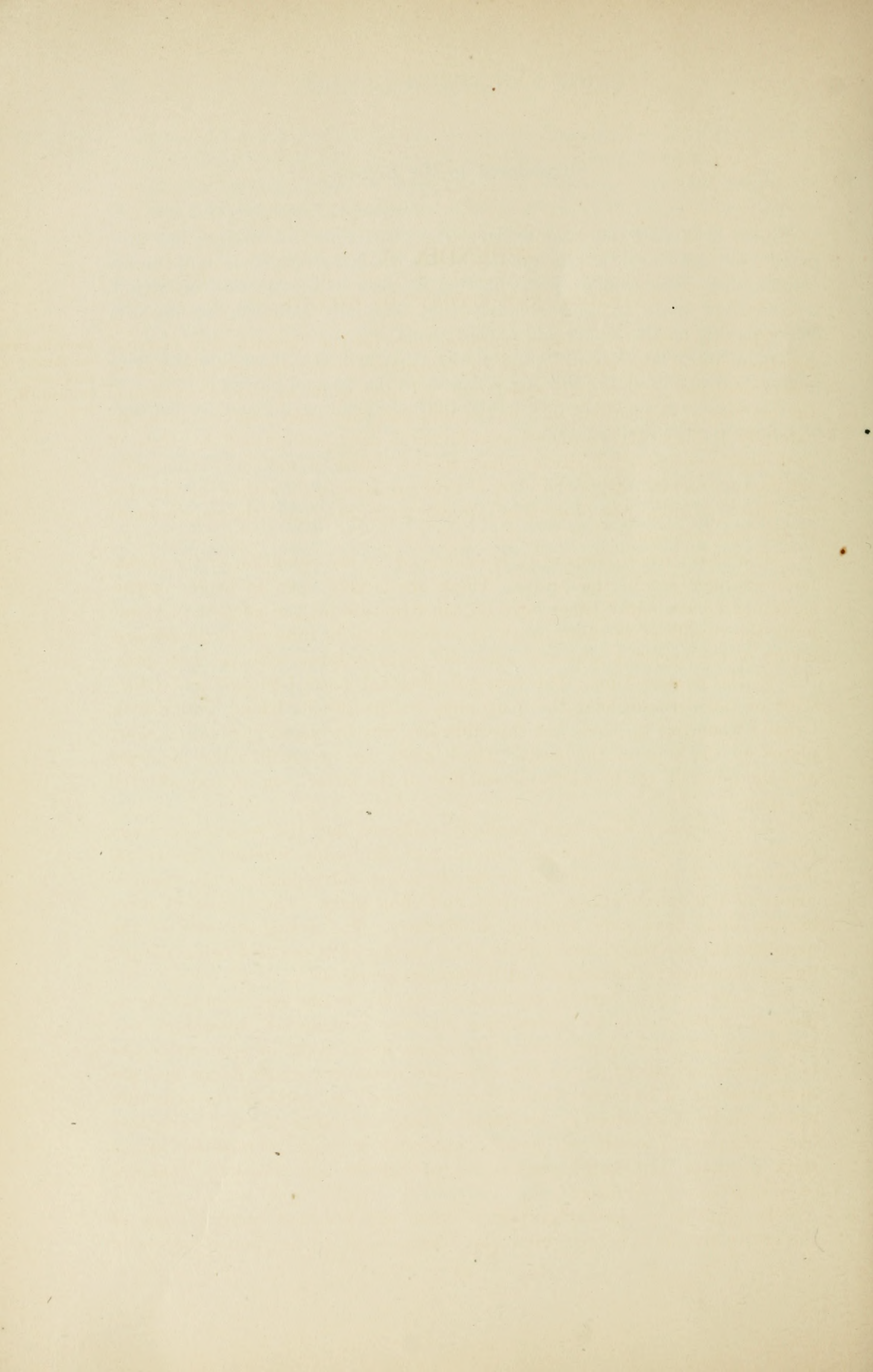
Since this report has been written, the Department of Bridges has prepared new drawings for the paving of the roadway, which is now under construction. The change in the paving consists in a reduction of weight on the river spans to approximately that originally assumed, leaving the heavy paving on the Island and anchor spans.

Table No. 6 shows the stresses of the truss members, based on the *final paving*, corresponding to table No. 5, based on the original paving.

The stresses given in the report refer to the *original* paving and are changed as follows for the *final* paving:

	<i>Stress for Original Paving</i>	<i>Stress for Final Paving</i>
Page 25, twelfth line from top	20,700	21,100
Page 25, twelfth line from top	18,800	18,900
Page 25, thirteenth line from top	34,100	34,200
Page 25, fifteenth line from top	22,900	22,700
Page 28, sixth line from bottom	21,200	21,000
Page 28, fifth line from bottom	33,500	33,600
Page 30, ninth line from top	18,600	18,400
Page 30, tenth line from top	22,900	22,700
Page 30, eleventh line from top	25,600	25,400
Page 30, twelfth line from top	34,200	34,000
Page 30, sixteenth line from top	20,300	20,100
Page 30, twentieth line from top	29,000	28,800
Page 42, seventeenth line from top	22,100	22,200
Page 42, nineteenth line from top	17,000	17,100
Page 42, twentieth line from top	11,900	12,100

F. C. K.



APPENDIX A

EXTRACTS FROM REPORTS OF EXPERTS

"The provisions of the specifications, both for the chemical and physical requirements of the material, are in accordance with the best practice of the present day, and entirely satisfactory. I have as far as possible examined the manner of inspection, both in the mill and shop, and I have scrutinized carefully a mass of records of experimental data, established by tests of both specimens and full-size eye-bars in the course of mill inspection, with satisfactory results. All of this class of evidence goes to show that the material put into the bridge was of excellent quality and fully (met) the requirements of the specifications."

**Extracts from
the Report of
Professor
William H.
Burr**

"The character of shop work is evidenced by the condition of the manufactured members in the bridge. These are largely open to ocular inspection, and I have many times been on the structure for the purpose of examining the results of the shop work. I believe it to be fully up to the requirements of the specifications and generally in accordance with the best practice of the present time. The four sub-diagonal posts C56-L57 and L107-C108 on both trusses near the main piers on Blackwell's Island were a little twisted when put in place, but this difficulty was corrected by riveting cover plates on the tops of the posts. These posts are not main truss members and the question of the safety or stability of the latter is in no way affected by them."

"Many sections have been carefully calipered and the weights of many members have been computed in order to determine whether the actual dimensions of pieces as placed in the structure correspond to the requirements of the specifications, contract and shop plans. The results of these examinations have been entirely satisfactory. The actual sections of the members are generally found a little full, a margin of $2\frac{1}{2}$ per cent being allowed by specifications in accordance with common practice."

"I have made a careful examination of the entire structure as far as possible, with a view to determining whether evidence of distress of any members exist, such as permanent distortion, loose rivets, or other evidences of overloading, misfitting, or any other circumstances which might indicate over-stressing. I have not found such evidence. All parts of the structure appear to be in satisfactory condition. There are minor or small variations from alignment, which are usually observed in large, completed bridge work in place, but nothing whatever to indicate any inherent weakness or unsatisfactory condition."

"In addition to this inspection, I have had accurate surveys made of the entire structure to determine the alignment of trusses and elevation of

lower chord points in July and as late as October 26, the past month. These surveys indicate that the alignment of the trusses is entirely satisfactory, and that the deflections vertically are only those caused by the dead weight of the structure at the two dates stated, the dead weight at the latter date being considerably more than at the former in consequence of the large amount of floor material put in place between the periods named."

"Adverse comment has been made on the heavy compression lower chords of this bridge and their design therefore has been scrutinized with great care. . . . No criticisms of these lower chord compression members would probably have been made except for the failure of chord sections of somewhat similar general shape of section in the Quebec Bridge. The similarity, however, lies only in the general form of section of the component parts. The spacing details of the Blackwell's Island chord sections, consisting of heavy lattice bars, batten and tie plates, and transverse diaphragms, are relatively far heavier, stiffer and stronger than corresponding details in the Quebec trusses; indeed, in the latter, there were no transverse diaphragms such as are found in the Blackwell's Island Bridge. There is, therefore, little or no similarity as to the unit-carrying capacity of the compression chord members in the two bridges."

Professor
Burr's
Conclusions

"(Second) Both the shop and mill inspection were efficiently performed, resulting in securing excellent quality of material and the fabrication of truss members of good quality and accurate dimensions.

"(Third) The various members of the structure possess the full sections required by the unit stresses and the working plans, and the shipping weights correspond correctly to those sections as well as to the computed weights.

"(Fourth) The erection was successfully and satisfactorily performed, leaving the trusses in correct alignment and elevation."

Extracts from
the Report of
Messrs. Boller
& Hodge

"We have examined the detailed reports of the mill inspectors on this material, and find that they show the metal fulfilled the above specifications."

"In addition to figuring the stresses on all members, we have had a large number of the actual bridge members measured and calipered in the field, and we find that they agree with the sections we took from the shop drawings and used in these calculations, which sections we show in detail on sheets 8 and 9."

"We have also computed the weight of a number of members from the shop drawings and find that such weights agree with the shipping weights on the invoices, showing that the scale weights used for the dead load are correct."

"We have not carefully examined all the details of this structure, but we have checked the end connections of such members as are most heavily stressed, and find them equal in strength to those members."

"We have carefully considered the form and details of the lower chord, as this feature has been criticized in the public journals, and the impression

has been given that the lower chords in this structure are weaker than those of the Quebec Bridge, which, in our opinion, is not the case."

"We have made a careful examination of the bridge as now completed, and find no evidence of loose rivets or buckling of members, or other indications of overloading, but there are four sub-diagonal posts, C56-L57 on both trusses, and L107-C108 on both trusses, which had a "wind" in them during erection, and this was corrected by riveting a cover plate on the top of each post. It will be noticed that both of these members are "sub-diagonals," which in no way affect the main stresses and are only for the support of one local panel load, and the stress sheet shows that they will never be subject to heavy stresses, so they are evidently safe and the "wind" was probably due to a bend in the shop or during erection, or to a slight over-run in length."

"(Second) That the steel manufactured for this structure is first-class bridge material and in accordance with the specifications.

Messrs.
Boller &
Hodge's
Conclusions

"(Third) That the workmanship of this structure is first-class and in accordance with the requirements of the specifications.

"(Fourth) That the erection and field riveting of the structure appears to have been done in a first-class manner.

"(Fifth) That the actual sections of the various members agree with the sections ordered on the working drawings and shown on our sheets Nos. 8 and 9, and that the shipping weights are correct."

APPENDIX B

EXTRACT OF SPECIFICATIONS FOR THE STEEL SUPERSTRUCTURE
OF THE BLACKWELL'S ISLAND BRIDGE

Manufacture and Requirements All steel shall be made by the open-hearth process, and shall fulfil the following requirements:

CHEMICAL REQUIREMENTS

NICKEL STEEL

	Phosphorus P C. Max		Sulphur P C Max	Nickel P C Min
	Basic	Acid		
Eye-bars and Pins	.04	06	05	3 25

STRUCTURAL STEEL

Plate Shapes Bars and Pins.....	.04	.08	05	
Rivet Steel	04	.04	.04	
Steel Castings	05	.08	05	

PHYSICAL REQUIREMENTS

SPECIMEN TESTS
NICKEL STEEL

	Ultimate Tensile Strength	Elastic Limit	Elongation		Reduction of area	Character of Fracture	Cold Bend Without Fracture	
			Min. % in 8"	Min. % in 2"			Material less than 1 inch in thickness	Material 1 inch or more in thickness
Eye-bars (annealed)	100,000 min.	55,000 min. {	$\frac{1,600,000}{\text{Ultimate Tensile}}$	}	{ To be recorded	{ Mostly silky and free from coarse crystals	180° around a pin with diameter=3t	180° around a pin with diameter=3t
Eye-bars (annealed)	85,000 min.	48,000 min. {	$\frac{1,600,000}{\text{Ultimate Tensile}}$	}				
53 Pins (annealed)	90,000 min.	50,000 min.		20	{ To be recorded	{ "	180° around pin with diameter=2t.	Pieces of bar not less than 4" wide. 180° around pin with diameter=2t.
53 Pins (annealed)								

STRUCTURAL STEEL

Plates, Shapes and Bars for riveted work	60,000 desired	30,000 min. {	$\frac{1,500,000}{\text{Ultimate Tensile}}$	}		Silky	180° Flat	180° around pin with diameter =2t. "
	66,000 desired	$\frac{1}{2}$ of ultimate	"	22				
Eye-bars and Pins	50,000 desired	$\frac{1}{2}$ of ultimate	"		{ Fine, silky, gradual break — uniform fracture	"	180° Flat	"
Rivet Steel								
Rivet Steel, nicked and bent around bar of same diameter as rivet rod					{ Silky or fine granular	"	90° around pin with diameter=3t	"
Steel Castings (annealed)	65,000 min.	$\frac{1}{2}$ of ultimate		18				

t is the thickness of the specimen

PHYSICAL REQUIREMENTS

FULL SIZE TESTS
NICKEL STEEL

	Ultimate Tensile Strength	Elastic Limit	Elongation	Reduction of Area	Character of Fracture	Cold Bent Without Fracture
	Pounds per sq. inch	Pounds per sq. inch		%		
Eye-bars				To be recorded	{ Mostly silky and free from coarse crystals	180° around pin with diameter =3t. In the neck of the bar, 90° around pin with diame- ter = 2½t.
Eye-bars (annealed).....	85,000 min.	48,000 min. {	9% in 18 feet, in- cluding fracture			

STRUCTURAL STEEL

Eye-bars						{	180° around pin with diameter =2t.
Eye-bars (annealed)	56,000 min.	½ of Ultimate	10% in body of bar			{ Fine, silky	Open flat, under blows of a ham- mer
Angles ¾" or less in thickness						{	Bent shut, under blows of a ham- mer
Angles ½" or less in thickness						{	

t is the thickness of the piece.

Tensile tests of structural steel, showing an ultimate strength within 4,000 pounds of that desired will be considered satisfactory.

Allowable
Variation in
Strength

LOADS

The bridge shall be proportioned to carry in addition to its own weight and that of the floor a live load either uniform or concentrated, or both as specified below, placed so as to give the greatest strain in each part of the structure.

Live Loads

For the main members of the trusses and the towers:

(a) A load of 8,000 pounds per linear foot of bridge as "regular," or

(b) 16,000 pounds per linear foot of bridge as "congested" traffic.

For the secondary members of the trusses, the floor-beams and the floor system:

(c) On each elevated railroad track a load of 52 tons on four axles, 6+10+6 feet apart (the motor ends of two motor cars of the Interborough Rapid Transit Co.).

(d) On each street-car track, either a load of 26 tons on two axles 10 feet apart, or a load of 1,800 pounds per linear foot of track.

(e) On any part of the roadway a load of 24 tons on two axles 10 feet apart and 5 feet gauge (assumed to occupy a width of 12 feet and a length of 30 feet), and upon the remaining portion of the floor a load of 100 pounds per square foot.

(f) On the footwalks a load of 100 pounds per square foot.

The wind pressure shall be assumed as a moving load acting in either direction horizontally with 2,000 pounds per linear foot.

Wind
Pressure

PROPORTIONING OF PARTS

For the main sections of members of the trusses and towers, no material shall be used less than one-half an inch thick; all other material shall not be less than three-eighths of an an inch thick.

Least
Thickness
of Steel

All steel work shall be so proportioned that the maximum strains from dead load and live load, or dead load and wind, shall not cause greater unit strains than the following:

Permissible
Unit Strains

	For Dead Load and Regular Live Load or for Dead Load and Wind	For Dead Load and Congested Live Load
	Pounds per square inch	
(a) <i>For Nickel Steel in Eye-bars and Pins—</i>		
Tension.....	30,000	39,000
Shear on pins.....	20,000	24,000
Bearing on diameter of pins.....	40,000	48,000
Bending on outer fibre of pins.....	40,000	48,000
(b) <i>For Structural Steel in Main Members of Trusses, Towers and Bracing—</i>		
Tension.....	20,000	24,000
Compression.....	20,000-90 $\frac{l}{r}$ *	24,000-100 $\frac{l}{r}$ *
Shear on shop rivets, bolts and pins.....	13,000	16,000
Bearing on diameter of shop rivets, bolts and pins.....	25,000	30,000
Bending on outer fiber of pins.....	25,000	30,000
	Pounds per square inch	
(c) <i>For Structural Steel in Secondary Members of Trusses—</i>		
Tension in subverticals (hangers).....	18,000	
Compression in subdiagonals.....	18,000-80 $\frac{l}{r}$ *	
Shear on shop rivets and bolts.....	12,000	
Bearing on diameter of shop rivets and bolts.....	24,000	
(d) <i>For Structural Steel in Floor System of Roadway and Footway and in All Floor Beams—</i>		
Tension chords.....	15,000	
Shear on shop rivets, bolts and web-plate net section.....	10,000	
Bearings on shop rivets and bolts.....	20,000	
(e) <i>For Structural Steel in Floor System (including brackets) for Railroad and Trolley Tracks—</i>		
Tension chords.....	10,000	
Shear on shop rivets, bolts and web-plate net section.....	7,000	
Bearing on shop rivets and bolts.....	14,000	

* Where l = length and r = least radius of gyration, both in inches.

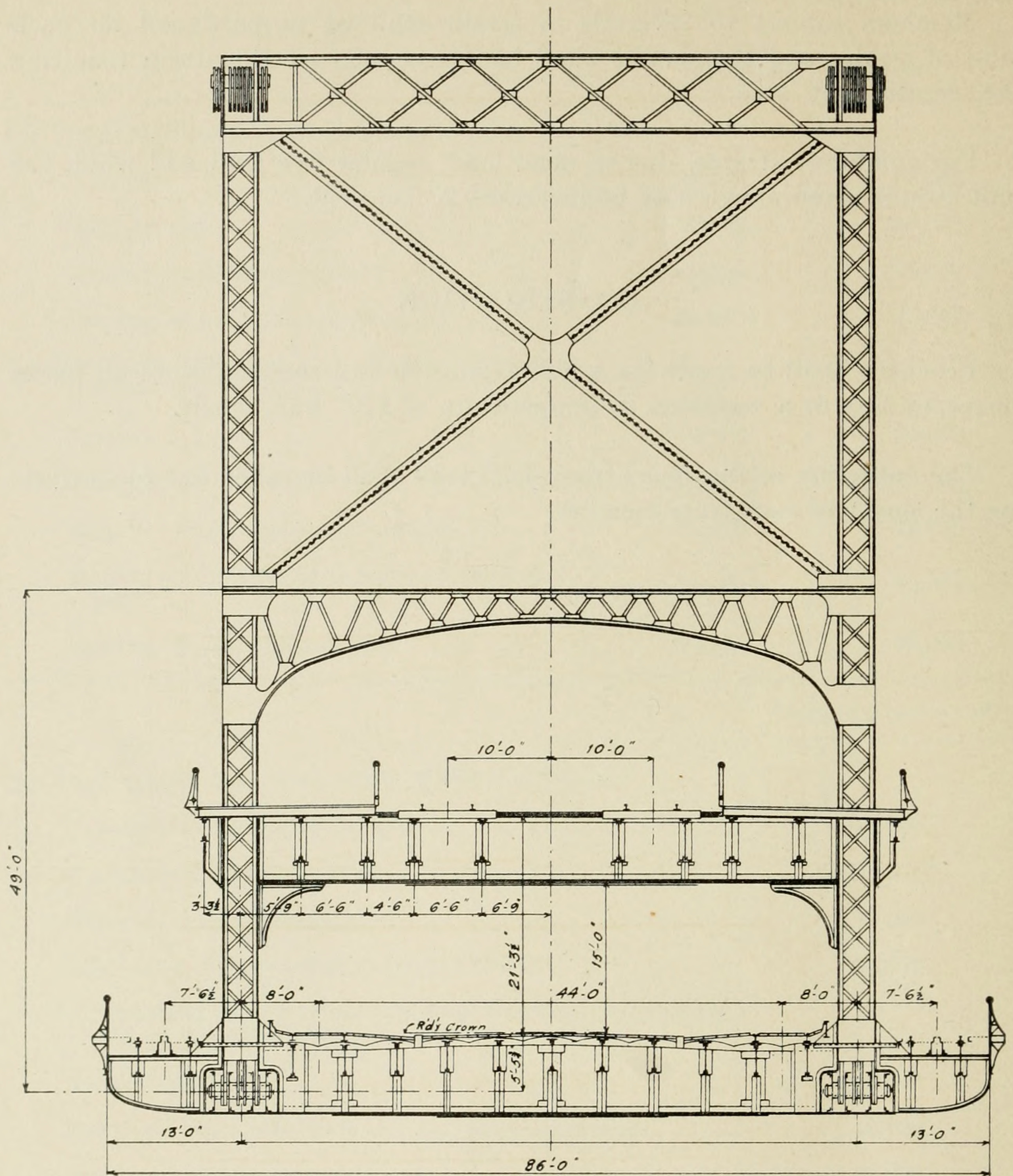
Members subject to reversals of strain shall be proportioned for each kind of strain, and the section shall be determined by the strain requiring the greater net area. **Reversals of Strain**

For combined strains, due to dead load, regular live load and wind, the unit strains given above may be increased 20 per cent. **Combined Strains**

CONSTRUCTION

Provision shall be made for a free expansion and contraction of all parts, corresponding to a variation in temperature of 110° Fahrenheit. **Temperature**

The deflection of the spans from dead load shall be taken out by correcting the length of each truss member. **Camber**

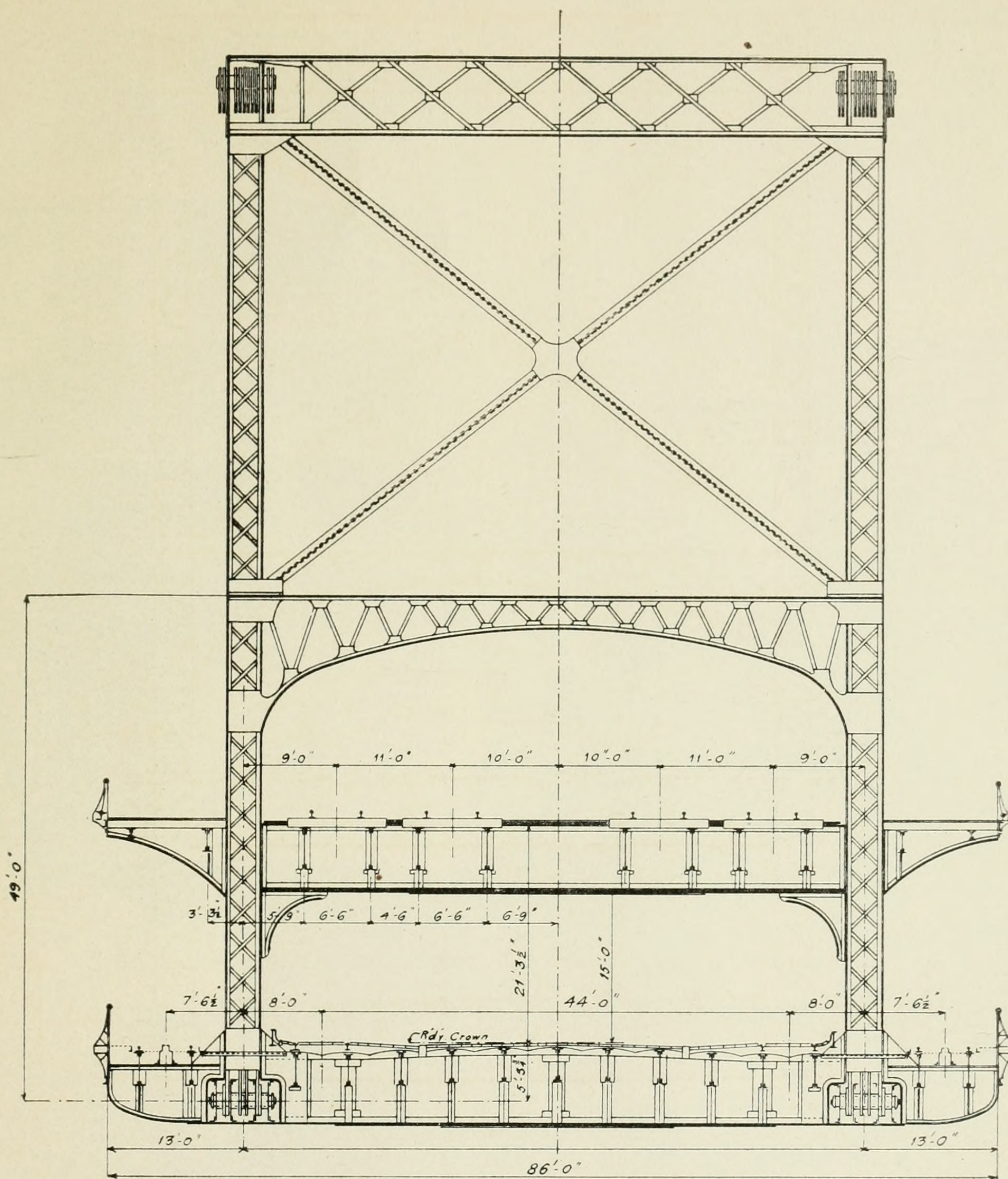


Cross Section

BLACKWELL'S ISLAND BRIDGE

with

Two Rapid Transit R.R. Tracks

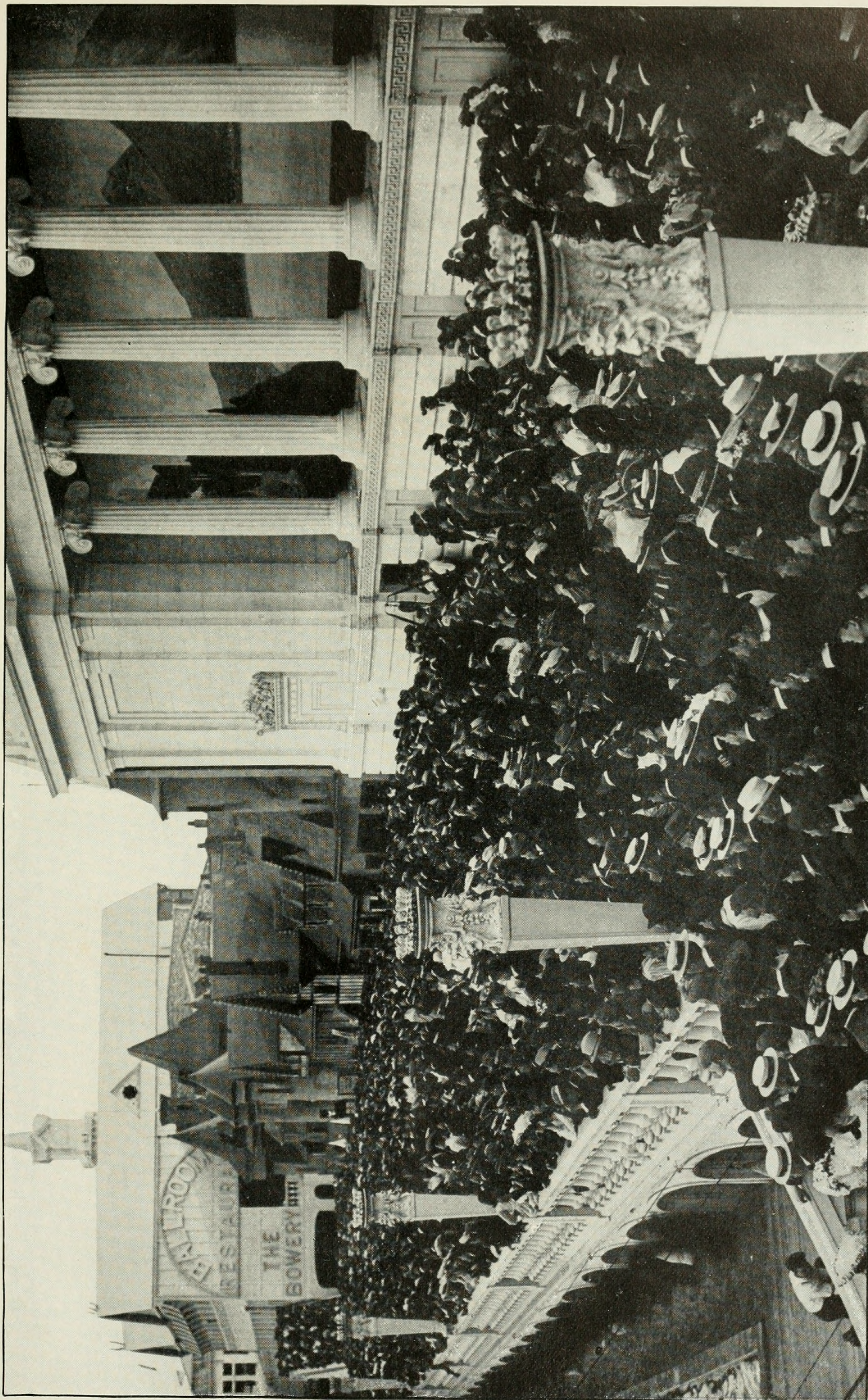


Cross Section

BLACKWELL'S ISLAND BRIDGE

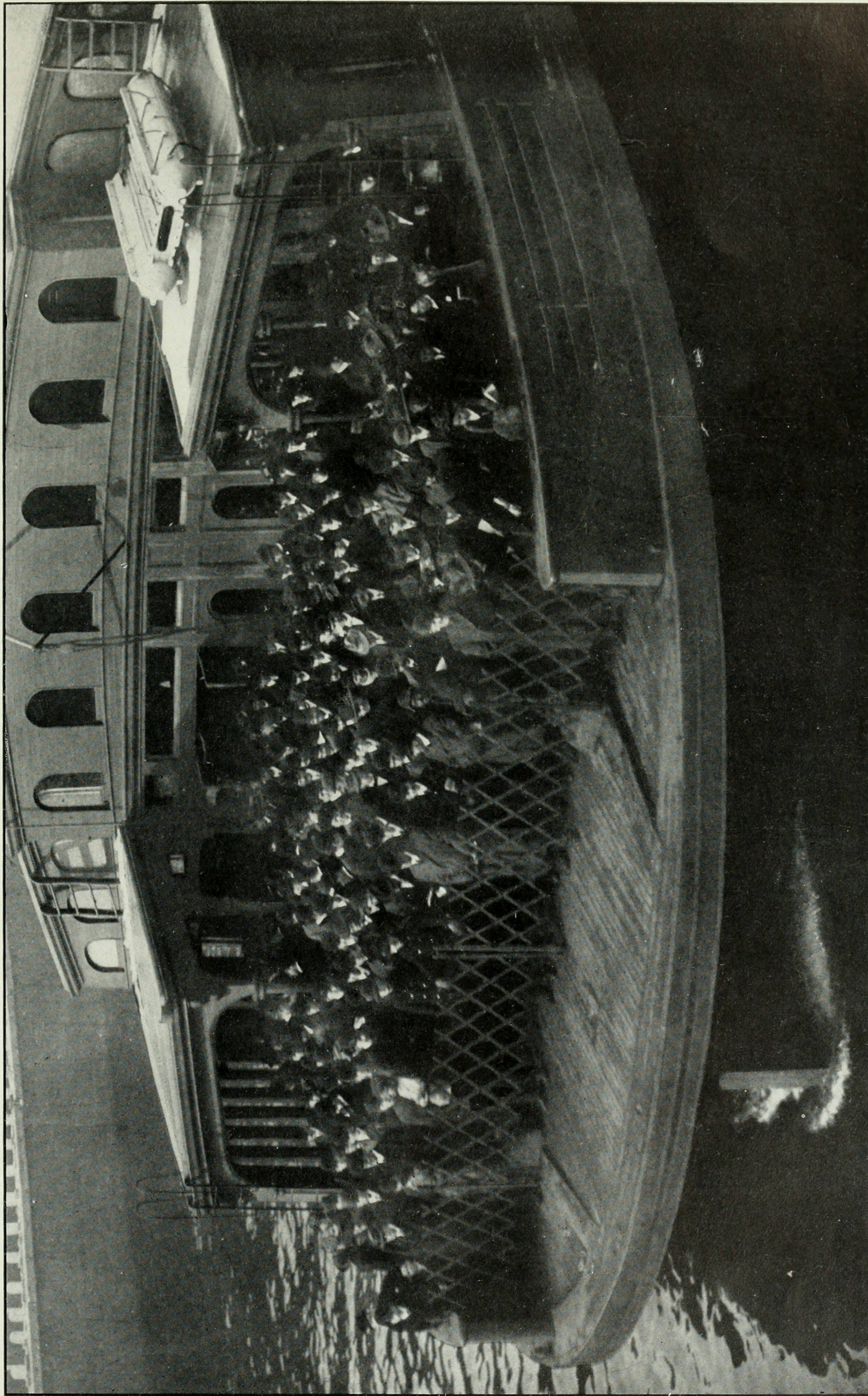
with

Four Rapid Transit R.R. Tracks



Crowd in Coney Island, weighing 13.3 pounds per square foot

(61)



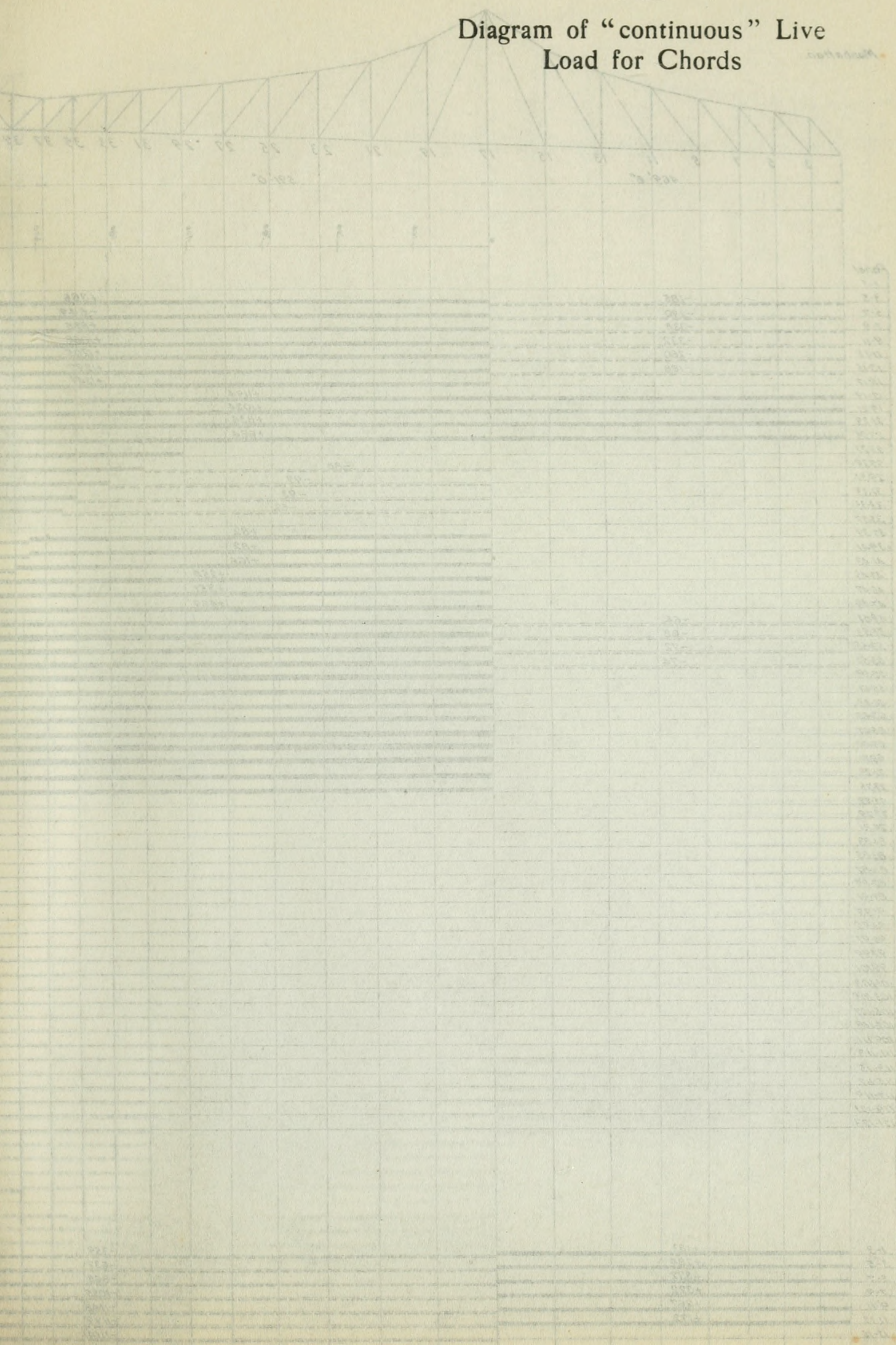
Crowd on forward deck of Erie Railroad Company's Ferry-boat "Susquehanna" in New York just before landing.
Weight of people, 26 pounds per square foot



Gathering of teams outside of market house at Newark, N. J. Weight about 15 pounds per square foot

TABLE 1

Diagram of "continuous" Live Load for Chords



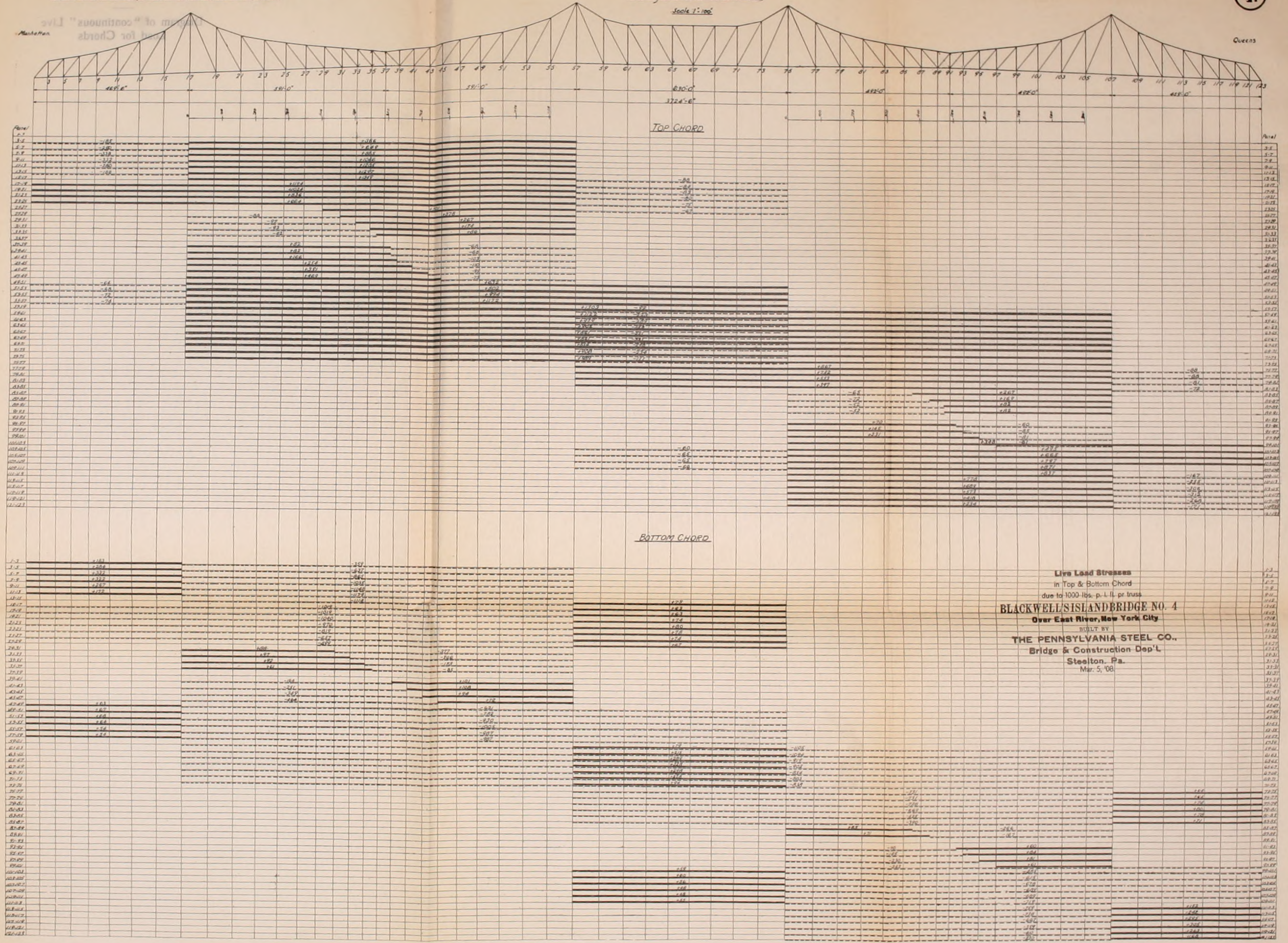
Heavy full lines denote loaded lengths for maximum tension.
 Dotted lines denote contributing stresses due to a live load of 1000 lbs. ft. of truss.
 Stresses in units of 1000 pounds (+ denotes tension - compression)

Diagram No. 1
 Loading in Continuous Stretches

Scale 1"=100'

1.

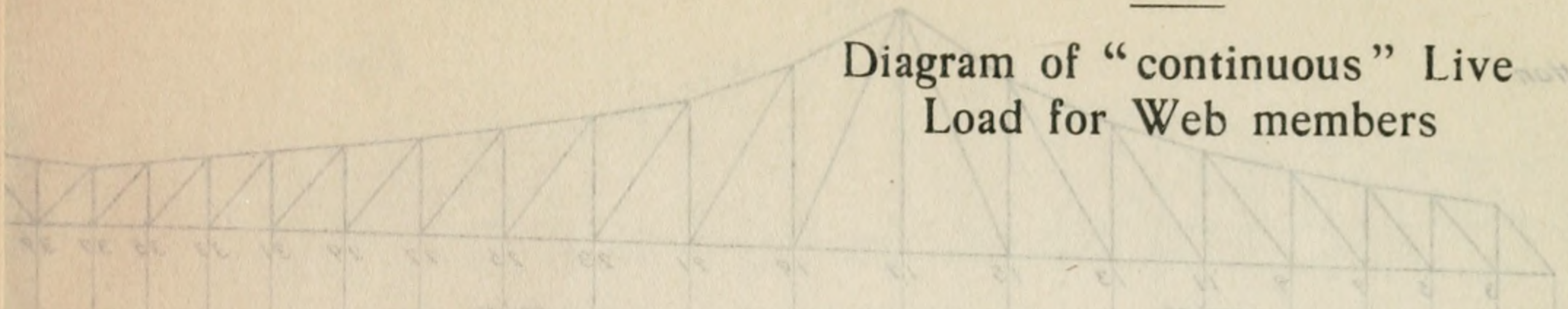
Queens



Live Load Stresses
 in Top & Bottom Chord
 due to 1000 lbs. p. l. ft. pr. truss
BLACKWELL'S ISLAND BRIDGE NO. 4
 Over East River, New York City
 DESIGNED BY
THE PENNSYLVANIA STEEL CO.,
 Bridge & Construction Dept.
 Steelton, Pa.
 Mar. 5, '08

TABLE 2

Diagram of "continuous" Live Load for Web members



Heavy full lines denote loads lengths for maximum tension
Dotted lines denote loads lengths for maximum compression
Figures denote contributing stresses due to a live load of 1000 per lin ft of truss
Stresses in units of 1000 pounds (+ denotes tension - compression)

Diagram No II
Loading in Continuous Stretches

Scale 1/1000

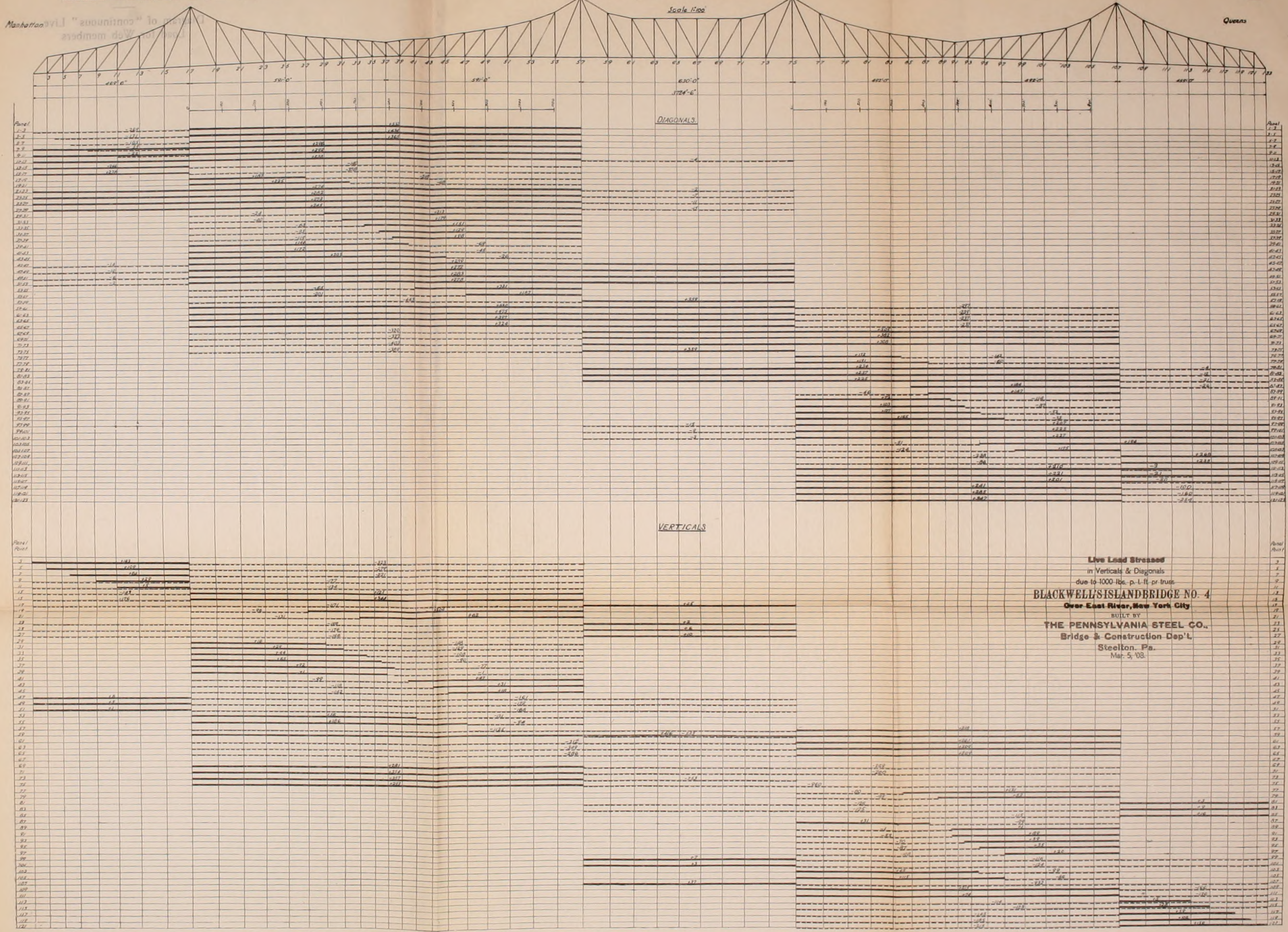
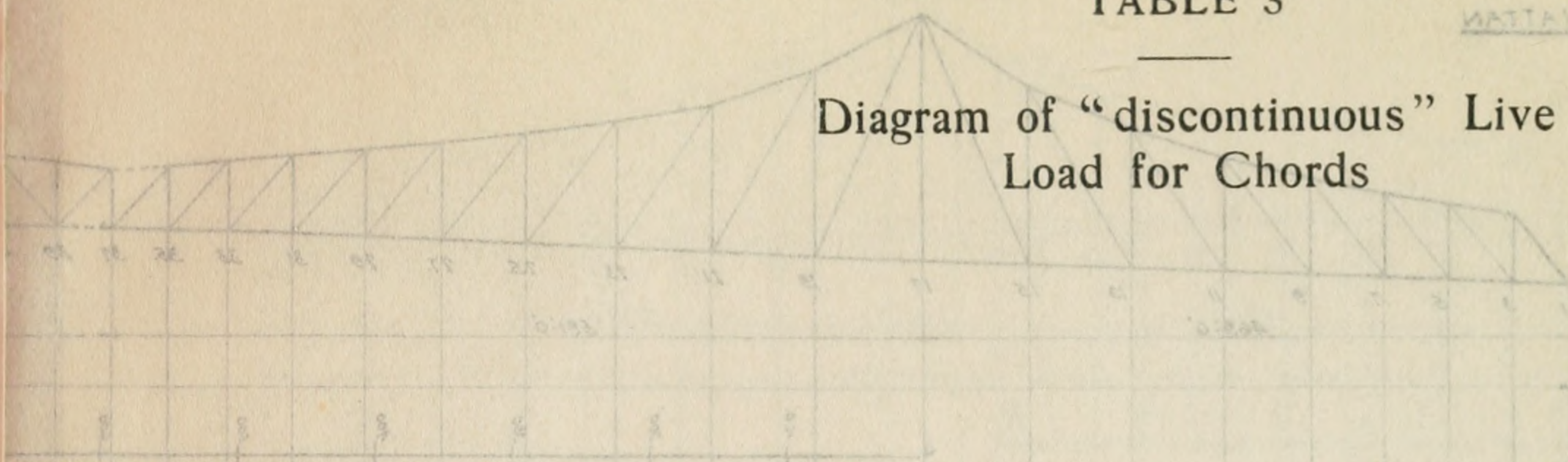


TABLE 3
Diagram of "discontinuous" Live Load for Chords

WATKINSON

Diagram of "discontinuous" Live Load for Chords



Heavy full lines denote loaded length for maximum tension.
Dotted lines denote contributing stress due to a live load of 100 lbs. per ft. of truss.
Figures denote contributing stress due to a live load of 100 lbs. per ft. of truss.
Stress in units of 1000 lbs. (+ denotes tension - compression)

Diagram No. III.
Loading in Discontinuous Stretches

Scale 1"=100'

MANHATTAN

QUEENS

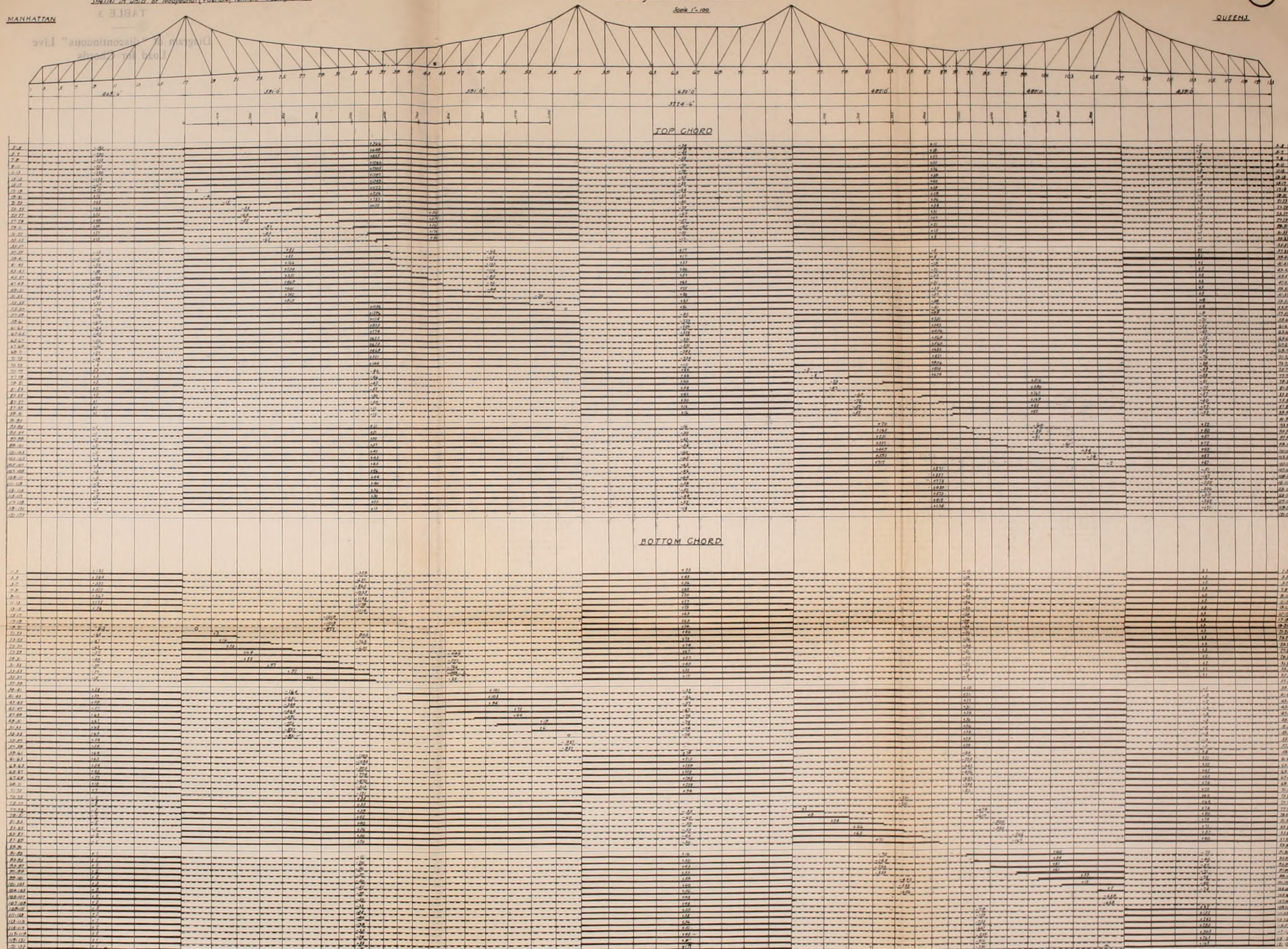


TABLE 4

Heavy full lines denote loaded length for maximum tension
Dotted lines denote contributing stress due to a live load of 1000 per lin. ft. of span
Figures denote contributing stress due to a live load of 1000 per lin. ft. of span
(stress in units of 1000 pounds) (+denotes tension - compression)

Diagram No. IV
Loading in Discontinuous Stretches

Scale 1"=100'

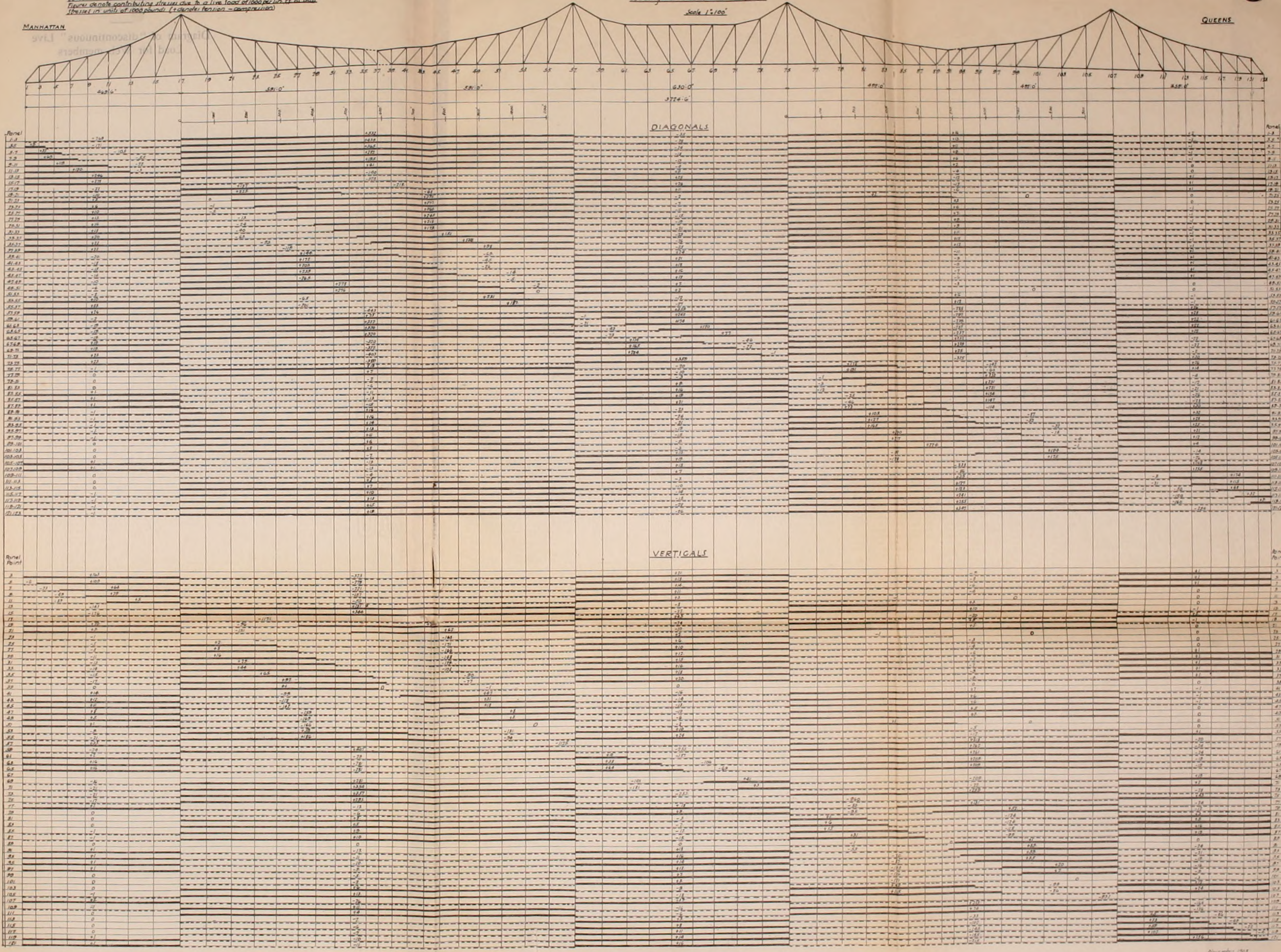
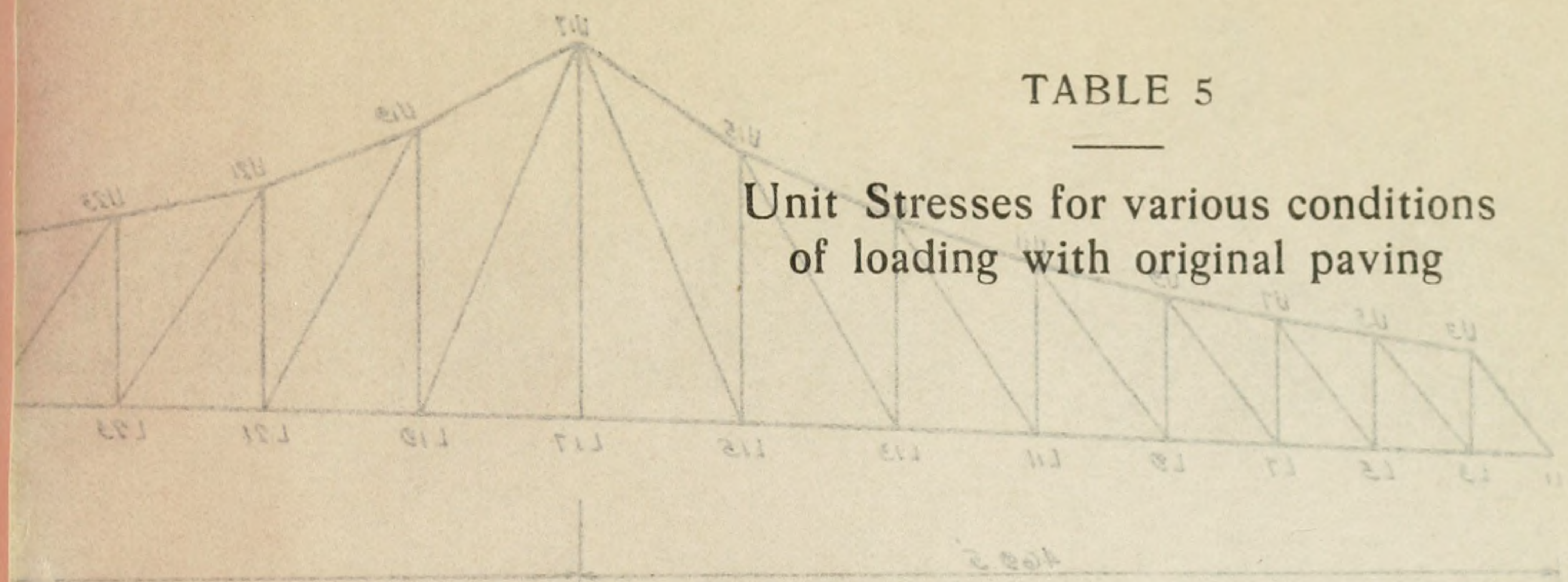


TABLE 5

Unit Stresses for various conditions
of loading with original paving

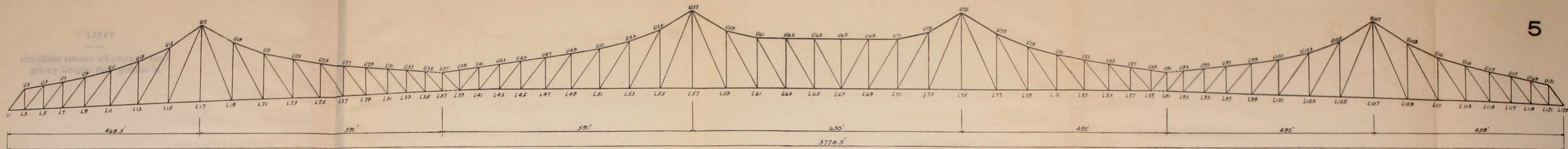


MANHATTAN ANCHOR ARM

Membr.	Area		Dead Load	Live Load				Total Load			
	Gross	Net		Con- tinuous timbers	Discon- tinuous	Total	Con- tinuous timbers	Discon- tinuous	Total		
U 7-U 9				23.8	18.4	42.2	38.3	30.0	68.3	26.0	27.3
U 9-U 11				21.1	17.7	38.8	38.1	28.7	66.8	27.5	27.9
U 11-U 13				19.4	17.1	36.5	38.0	28.0	66.0	28.3	28.6
U 13-U 15				17.8	16.5	34.3	38.0	28.0	66.0	28.1	29.4
U 15-U 17				16.9	16.4	33.3	36.9	27.3	64.2	28.9	29.2
L 5-L 7	885.8		8.5	12.9	13.3	26.2	21.4	21.8	43.2	14.9	15.2
L 7-L 9	691.6		9.4	12.0	12.3	24.3	21.4	21.7	43.1	15.1	15.6
L 9-L 11	853.2		10.1	10.8	11.0	21.9	20.9	21.1	42.0	15.5	15.6
L 11-L 13	973.0		10.8	9.7	10.0	20.5	20.8	19.8	40.3	15.7	15.8
L 13-L 15	1029.2		11.3	8.6	8.9	19.5	19.9	20.2	40.7	15.6	15.8
C 3-C 4			9.3	13.0	11.7	24.7	23.9	24.3	48.0	16.1	16.8
C 4-U 5			9.2	13.1	11.5	24.3	22.9	24.3	47.2	16.4	17.0
L 5-C 6			11.8	11.8	11.4	23.6	23.4	23.0	46.6	17.5	18.4
C 6-U 7			11.3	11.3	11.2	23.6	23.4	23.3	46.3	17.8	18.6
L 7-C 8			9.9	12.6	11.4	23.3	23.0	24.8	47.1	18.3	19.7
C 8-U 9			9.7	12.2	11.0	23.3	23.8	24.6	47.1	18.5	19.7
L 9-C 10			9.1	10.0	9.4	23.0	23.9	23.3	46.3	18.5	18.9
C 10-U 11			9.0	9.0	9.4	23.0	23.1	24.0	46.1	18.6	19.1
U 8-L 8	192.0		6.8	13.5	13.8	20.3	20.3	20.8	41.1	13.5	13.7
U 5-L 5	192.0		9.0	11.6	12.3	20.6	21.2	18.6	39.8	14.8	15.1
U 7-L 7	181.5		6.8	11.0	11.2	20.3	21.1	19.7	41.0	14.8	14.9
U 9-L 9	181.5		7.8	9.8	10.4	20.0	20.0	20.0	40.0	15.5	16.2
U 11-L 11	163.1		6.8	6.8	6.7	13.8	13.8	13.9	27.5	15.5	15.6
U 13-L 13	130.5		8.1	8.7	9.2	17.8	17.3	18.2	36.1	15.1	15.7
U 15-L 15	147.5		5.5	15.0	13.8	28.3	24.4	25.0	53.3	15.1	15.4
U 17-L 17	1366.2		12.0	6.8	6.7	12.8	12.8	12.7	25.5	15.1	15.1

ISLAND CANTILEVER ARM, EAST

TOP CHORD											
U 7-U 7	400.0		21.3	17.3	16.0	34.6	38.6	37.3	73.9	30.0	30.0
U 7-U 9	324.0		20.8	18.0	16.0	34.8	38.1	37.1	71.9	29.8	30.0
U 9-U 11	240.0		19.0	18.4	16.5	34.4	38.7	36.7	71.1	29.1	29.5
U 11-U 13	162.0		19.4	18.5	16.0	34.9	41.8	37.4	72.2	29.2	30.4
U 13-U 15	92.0		17.7	21.8	19.3	40.0	47.1	37.0	87.0	28.6	30.1
C 8-U 7	22.5		2.5	15.3	14.3	29.8	30.0	30.7	60.5	16.1	17.8
U 17-U 17	36.0		4.0	18.2	17.0	31.2	33.1	33.9	65.1	14.0	15.9
BOTTOM CHORD											
L 7-L 7	630.2		11.0	9.1	9.3	20.1	20.1	20.3	40.4	15.6	15.6
L 9-L 9	544.8		11.0	9.6	9.8	20.6	20.6	20.0	41.2	15.8	16.0



MANHATTAN ANCHOR ARM																	
Memb.	Area		Dead Load	Live Load						Total Load						Permissible Live Stresses	
	Gross	Net		Congested			Regular			Congested			Regular			Cong. Cont.	Reg. Cont.
				Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.		
TOP CHORD																	
U 7-U 9	310.0	+15.5		+22.8	+23.5	+18.4	+11.4	+11.8	+9.2	+38.3	+39.0	+33.9	+26.9	+27.3	+24.7	+35.0	+30.0
U 9-U11	404.0	+17.0		+21.1	+21.7	+17.7	+10.5	+10.9	+8.9	+38.1	+38.7	+34.7	+27.5	+27.9	+25.9	+36.0	+30.0
U11-U13	496.0	+18.6		+19.4	+20.0	+17.1	+9.7	+10.0	+8.5	+38.0	+38.6	+35.7	+28.3	+28.6	+27.1	+39.0	+30.0
U13-U15	580.0	+20.2		+17.8	+18.4	+16.5	+8.9	+9.2	+8.2	+38.0	+38.6	+36.7	+29.1	+29.4	+28.4	+39.0	+30.0
U15-U17	680.0	+21.0		+15.9	+16.3	+15.4	+7.9	+8.2	+7.7	+36.9	+37.3	+36.4	+28.9	+29.2	+28.7	+39.0	+30.0
BOTTOM CHORD																	
L 5-L 7	535.6		-8.5	-12.9	-13.3	-10.4	-6.4	-6.7	-5.2	-21.4	-21.8	-18.9	-14.9	-15.2	-13.7	-21.9	-18.1
L 7-L 9	691.0		-9.4	-12.0	-12.3	-10.0	-6.0	-6.2	-5.0	-21.4	-21.7	-19.4	-15.4	-15.6	-14.4	-21.7	-17.9
L 9-L11	853.2		-10.1	-10.8	-11.0	-9.4	-5.4	-5.5	-4.7	-20.9	-21.1	-19.5	-15.5	-15.6	-14.8	-21.4	-17.7
L11-L13	972.0		-10.8	-9.7	-10.0	-9.0	-4.9	-5.0	-4.5	-20.5	-20.8	-19.8	-15.7	-15.8	-15.3	-21.2	-17.5
L13-L15	1029.2		-11.3	-8.6	-8.9	-8.4	-4.3	-4.5	-4.2	-19.9	-20.2	-19.7	-15.6	-15.8	-15.5	-20.9	-17.2

MAIN DIAGONALS																
L-3-C-4	244.0	+9.3	+13.0	+15.0	+11.7	+6.8	+7.5	+5.9	+22.0	+24.3	+21.0	+16.1	+16.8	+15.2	+24.0	+20.0
C-4-U-5	252.0	+8	+13.1	+14.5	+11.5	+6.0	+7.2	+8	+22.9	+24.3	+21.3	+16.4	+17.0	+16.5	+24.0	+20.0
L-5-C-6	242.0	+11.6	+11.8	+13.5	+11.4	+5.9	+6.8	+5.7	+23.4	+25.1	+23.0	+17.5	+18.4	+17.3	+24.0	+20.0
C-6-U-7	252.0	+12.1	+11.3	+13.0	+11.2	+5.7	+6.5	+5.6	+23.4	+25.1	+23.3	+17.8	+18.6	+17.7	+24.0	+20.0
L-7-C-8	228.0	+13.4	+9.5	+12.6	+11.4	+4.9	+6.3	+6.7	+23.5	+24.0	+24.8	+18.3	+19.7	+19.1	+24.0	+20.0
C-8-U-9	244.0	+15.0	+12.2	+11.0	+4.9	+6.1	+5.5	+6.3	+23.5	+24.0	+24.8	+18.5	+19.1	+19.1	+24.0	+20.0
L-9-C10	240.0	+13.9	+9.1	+10.0	+9.4	+4.0	+5.0	+4.7	+23.0	+23.9	+23.3	+18.5	+18.9	+18.6	+24.0	+20.0
C10-U11	258.0	+14.1	+9.0	+9.9	+9.4	+4.5	+5.0	+4.7	+23.1	+24.0	+23.6	+18.6	+19.3	+18.8	+24.0	+20.0

		MAIN POINTS															
U-3-L 3	192.0	-6.8	-13.5	-13.8	-10.0	-6.7	-6.9	-5.0	-20.3	-20.6	-16.8	-13.5	-13.7	-11.8	-20.7	-17.0	3.3
U-5-L 5	192.0	-9.0	-11.6	-12.2	-9.0	-5.8	-6.1	4.8	-20.0	-21.2	-18.6	-14.8	-15.1	-13.8	-20.3	-16.7	3.7
U-7-L 7	181.5	-10.4	-9.8	-11.0	9.3	-4.9	-5.5	-4.7	-20.2	-21.4	-19.7	-15.3	-15.9	-15.1	-19.9	-16.3	4.1
U-9-L 9	181.5	-11.0	-7.8	-9.3	8.4	-3.0	-4.0	-4.2	-19.4	-20.0	-19.0	-15.0	-15.2	-15.8	-19.2	-15.7	4.8
U-11-L 11	181.5	-12.2	-6.0	-6.8	-6.7	-3.3	-3.4	-3.3	-19.8	-19.0	-18.9	-15.5	-15.0	-15.5	-18.6	-15.2	5.4
U-13-L 13	181.5	-8.1	-9.2	-7.4	4.3	-4.0	-4.3	-4.0	-17.3	-13.5	-12.4	-12.8	-17.4	-14.1	-16.6	-15.1	5.6
U-15-L 15	174.5	-5.8	-18.8	-19.2	-13.8	+3.3	+6.0	+6.9	+24.4	+25.0	+19.0	+15.1	+15.4	+15.7	+24.0	+20.0	5.8
U-17-L 17	1396.3	-12.0	-6.8	-19.8	-6.7	-3.3	-3.4	-3.4	-18.8	-18.8	-18.7	-15.4	-15.4	-15.4	-18.6	-15.2	5.4

	ISLAND CANTILEVER ARM, EAST													
	TOP CHORD													
U75-U77	400.0 + 21.3	+17.3	+17.4	+10.0	+7.7	+8.7	+8.0	+38.0	+38.7	+37.3	+30.0	+30.0	+29.3	+39.0 + 30.0
U77-U79	324.0 + 20.8	+18.0	+18.3	+10.6	+9.0	+9.2	+8.3	+38.8	+39.1	+37.4	+29.8	+30.0	+29.1	+30.0 + 30.0
U79-U81	240.0 + 19.9	+18.4	+19.1	+10.8	+9.2	+9.6	+8.4	+38.3	+39.2	+36.7	+20.1	+29.5	+28.3	+30.0 + 30.0
U81-U83	162.8 + 19.4	+19.5	+19.5	+18.0	+9.8	+11.0	9.0	+38.9	+41.3	+37.4	+29.2	+30.4	+28.4	+30.0 + 30.0
U83-U85	98.0 + 17.7	+21.8	+25.4	+19.3	+10.9	+12.7	+9.7	+39.5	+43.1	+37.9	+28.0	+30.4	+27.4	+39.0 + 30.0

[illegible]

C76-L77	91.5	+4.8	+8.1	+10.6	+9.4	+6.0	+7.5	+4.8	+22.2	+25.0	+19.2	+16.1	+17.5	+14.6	+24.0	+20.0
C78-L79	136.0	+12.3	+10.0	+11.2	+9.2	+5.0	+5.0	+5.0	+4.0	+23.2	+25.0	+17.8	+15.6	+16.6	+13.1	+24.0
C79-L80	198.0	+13.8	+9.4	+9.5	+9.4	+4.7	+4.8	+4.7	+23.2	+23.3	+23.2	+18.2	+18.0	+18.5	+24.0	+20.0
C80-L81	180.0	+13.6	+6.5	+9.5	+9.4	+4.7	+4.8	+4.7	+23.1	+23.1	+23.0	+18.3	+18.4	+18.3	+24.0	+20.0
C82-L83	168.0	+13.1	+6.1	+9.5	+9.4	+5.1	+4.9	+5.1	+23.1	+23.3	+22.8	+18.1	+18.2	+18.0	+24.0	+20.0
C82-L83	173.1	+12.9	+10.0	+10.2	+9.3	+5.0	+5.0	+5.0	+23.2	+23.2	+23.1	+18.0	+17.9	+18.0	+17.8	+24.0
C83-L84	168.0	+12.5	+10.7	+11.4	+10.3	+5.3	+5.7	+5.2	+23.2	+23.2	+23.1	+17.8	+18.0	+17.9	+24.0	+20.0
C84-L85	152.4	+12.4	+10.8	+12.2	+10.3	+5.4	+6.1	+5.2	+23.2	+24.6	+22.7	+17.8	+18.5	+17.6	+24.0	+20.0
C85-L86	126.0	+11.8	+11.7	+13.0	+10.9	+5.9	+6.5	+5.5	+23.5	+24.8	+22.7	+17.7	+18.3	+17.3	+24.0	+20.0

[illegible]

U87-L87	78.8	-8.0	10.8	-11.6	-8.6	-5.4	-5.8	-4.3	-18.5	-20.1	-18.5	-14.4	-15.0	-14.2	-19.7	-16.1	4.3	
									-4.3	-18.5	-19.0	-16.0	-13.4	-13.8	-12.3	-19.9	-16.3	4.3
ROCKER ARM																		
U91-L91	{ 44.1	31.2	+0.4	+22.8	+27.7	+15.9	+11.4	+13.9	+7.9	+23.2	+28.1	+16.3	+11.8	+14.3	+8.3	+24.0	+20.0	
			+0.3	-10.0	-16.7	-0.9	-5.0	-8.4	-3.5	-10.3	-17.0	-7.2	-5.3	-8.7	-3.8	-17.7	-14.3	0.6

MANHATTAN CANTILEVER ARM																			
Memb.	Area		Live Load									Total Load						Permissible Unit Stresses for	
	Gross	Net	Congested			Regular			Congested			Regular			Uwg. Cont.	Reg. Cont.			
			Con-tinuous	Discon-tinuous	Total Disc.	Con-tinuous	Discon-tinuous	Total Disc.	Con-tinuous	Discon-tinuous	Total Disc.	Con-tinuous	Discon-tinuous	Total Disc.					
TOP CHORD																			
U19-U21	518.0	+21.1	+15.8	+16.5	+15.8	+ 7.9	+ 8.2	+ 7.9	+36.9	+37.6	+36.9	+29.0	+29.3	+29.0	+39.0	+30.0			
U21-U23	404.0	+20.8	+16.6	+17.6	+16.6	+ 8.3	+ 8.8	+ 8.3	+37.4	+38.4	+37.4	+29.1	+29.6	+29.1	+39.0	+30.0			
U23-U25	302.0	+21.0	+17.6	+19.4	+18.0	+ 8.8	+ 9.7	+ 9.0	+36.6	+40.4	+39.0	+29.8	+30.7	+30.0	+39.0	+30.0			
U25-U27	218.7	+20.1	+18.3	+21.6	+19.1	+ 9.2	+10.8	+ 9.6	+38.5	+41.7	+39.2	+29.3	+30.9	+29.7	+39.0	+30.0			
U27-U29	152.2	+18.4	+19.8	+23.8	+20.0	+ 9.9	+11.9	+10.0	+34.2	+42.7	+38.4	+28.3	+30.3	+28.4	+39.0	+30.0			
U29-U31	98.0	+15.8	+21.8	+26.7	+23.8	+10.9	+13.3	+10.4	+37.6	+42.5	+36.0	+26.7	+29.1	+26.2	+30.0	+30.0			
U31-U33	90.9	+ 7.5	+15.4	+19.0	+13.3	+ 7.7	+ 9.5	+ 7.7	+22.9	+26.5	+20.9	+15.2	+17.0	+14.2	+24.0	+20.0			
U33-U35	30.0	+ 4.4	+19.1	+24.2	+14.2	+ 9.6	+12.1	+ 7.1	+23.5	+28.6	+18.0	+14.0	+16.5	+11.5	+24.0	+20.0			
BOTTOM CHORD																			
L19-L21	994.7	-11.5	- 8.3	- 8.6	- 8.3	- 4.2	- 4.3	- 4.2	-19.8	-20.1	-19.8	-15.7	-15.8	-15.7	-20.8	-17.1			
L21-L23	877.8	-11.5	- 8.7	- 9.2	- 8.8	- 4.3	- 4.6	- 4.4	-20.2	-20.7	-20.3	-15.8	-16.1	-15.9	-21.0	-17.3			

L23-L25	714.8	-11.4	9.0	-9.7	-9.2	4.5	4.4	-4.6	-20.4	-21.1	-20.6	-15.9	-16.2	-16.0	-21.2	-17.5
L25-L27	535.6	-11.4	9.0	-10.9	-10.0	4.8	5.5	5.0	-20.0	-22.3	-21.4	-16.2	-16.9	-16.4	-21.4	17.7
L27-L29	379.5	-11.2	10.5	-12.4	-11.2	5.0	6.7	6.0	-20.0	-22.3	-21.6	-17.4	-16.7	-16.0	-21.6	17.8
L29-L31	271.6	-9.8	-11.1	-13.4	-11.2	5.0	6.7	5.6	-19.9	-23.2	-21.0	-15.4	-16.5	-15.4	-21.8	18.0
L33-L35	105.0	-6.6	-13.2	-16.4	-11.6	-6.8	8.2	5.8	-18.8	-22.0	-17.2	-12.2	-13.8	-11.4	-22.1	18.3
MAIN DIAGONALS																
L17-C18	106.7	+7.8	+14.0	+16.0	+6.6	+7.0	+8.0	+3.3	+21.8	+23.8	+14.4	+14.8	+15.8	+11.1	+24.0	+20.0
U23-C24	250.0	+14.0	+9.0	+9.2	+9.0	4.5	+6	+4.5	+23.0	+23.2	+23.0	+18.5	+18.6	+18.5	+24.0	+20.0
L24-L25	232.0	+13.8	+9.1	+9.2	+9.1	+4.6	+6	+4.6	+22.9	+23.0	+22.9	+18.4	+18.4	+18.4	+24.0	+20.0

C25-C26	229.2	+13.5	9.6	+9	+9	+6	+4.8	+5.0	+4.8	+23.1	+23.4	+23.1	+18.3	+18.5	+18.3	+24.0	+20.0
C26-C27	211.7	+13.4	+6	+10.0	+9.7	+4.8	+5.0	+4.8	+23.0	+23.4	+23.1	+18.2	+18.4	+18.2	+24.0	+20.0	
C27-C28	190.0	+13.0	+6	+10.8	+10.4	+5.0	+5.4	+5.1	+23.3	+23.6	+23.5	+18.7	+18.9	+18.7	+24.0	+20.0	
C28-C29	178.1	+13.1	+10.2	+12.1	+11.0	+5.0	+5.5	+5.3	+23.3	+24.2	+23.5	+18.2	+18.7	+18.3	+24.0	+20.0	
U29-C30	157.5	+12.5	+10.8	+12.1	+11.4	+5.4	+6.1	+5.3	+23.3	+24.6	+23.5	+17.9	+18.9	+18.0	+24.0	+20.0	
C30-C31	145.2	+12.2	+10.8	+12.2	+11.0	+5.4	+6.1	+5.5	+23.0	+24.4	+23.2	+17.6	+18.3	+17.7	+24.0	+20.0	
C31-C32	126.0	+10.7	+11.4	+13.1	+11.1	+5.7	+6.0	+5.6	+22.1	+23.8	+21.8	+16.4	+17.3	+16.3	+24.0	+20.0	
C32-C33	108.5	+10.9	+12.0	+14.0	+11.8	+6.0	+7.0	+5.9	+22.9	+24.0	+22.7	+16.0	+17.9	+16.8	+24.0	+20.0	
C33-C34	92.4	+10.8	+12.4	+14.4	+11.8	+6.4	+7.4	+6.3	+22.8	+24.0	+22.6	+16.0	+17.9	+16.8	+24.0	+20.0	
C34-C35	79.8	+8.4	+14.3	+17.8	+13.2	+7.2	+8.8	+6.9	+22.8	+24.0	+21.9	+15.6	+17.2	+15.9	+24.0	+20.0	

U35-C36	68.6	+3.9	17.0	+2.2	+1.2	+8.8	+11.1	+7.1	+25.5	+26.1	+18.0	+12.7	+15.0	+11.0	+24.0	+20.0
U36-L37	58.5	+0.3	+20.6	+16.2	+16.3	+10.3	+13.3	+8.1	+20.9	+26.7	+16.5	+10.5	+13.0	+8.4	+24.0	+20.0
	MAIN POSTS															
U19-L19	86.2	+5.1	+18.9	+21.6	+16.0	+9.5	+10.8	+8.0	+24.0	+26.7	+21.1	+14.6	+15.9	+13.1	+24.0	+20.0
U21-L21	136.2	-10.7	-7.7	-8.3	-6.0	-3.9	-4.2	-3.0	-18.4	-19.0	-16.7	-14.6	-14.9	-13.7	-17.9	-14.5
U23-L23	190.7	-12.3	-7.1	-7.1	-7.1	-3.5	-3.5	-3.5	-19.4	-19.4	-19.4	-15.8	-15.8	-15.8	-18.8	-15.3
U25-L25	182.1	-11.8	-7.6	-7.8	-7.7	-3.8	-3.9	-3.8	-19.4	-19.4	-19.6	-15.5	-15.7	-15.0	-18.5	-15.0
U27-L27	175.2	-10.3	-7.2	-7.2	-7.2	-4.1	-4.1	-4.1	-3.8	-19.0	-18.9	-15.0	-15.0	-15.0	-19.3	-15.5
U29-L29	171.1	-10.2	-8.8	-8.8	-8.8	-4.2	-4.2	-4.2	-3.8	-19.0	-18.9	-15.7	-14.9	-14.1	-14.1	-14.1

	U31-L31	109.2	-9.4	-10.0	-10.9	-9.3	-5.0	-5.5	-4.7	-19.4	-20.3	-18.7	-14.4	-14.9	-14.1	-19.9	-16.3
	U33-L33	78.8	-7.8	-11.0	-13.1	-10.0	-5.5	-6.0	-5.0	-18.8	-20.9	-17.8	-13.3	-14.4	-12.8	-20.0	-16.4
	U35-L35	50.1	-3.6	-14.4	-18.2	-11.5	-7.2	-9.1	-5.8	-18.0	-21.8	-15.1	-10.8	-12.7	-9.4	-20.4	-16.8
QUEENS CANTILEVER ARM																	
TOP ORBD																	
U 93-U 95		36.0	+ 4.8	+15.6	+22.8	+14.3	+ 7.8	+11.4	+ 7.2	+20.4	+27.6	+19.1	+12.6	+16.2	+12.0	+24.0	+20.0
U 95-U 97		81.6	+ 9.2	+14.2	+20.2	+14.5	+ 7.1	+10.1	+ 7.3	+23.4	+24.9	+23.7	+16.3	+19.3	+19.5	+24.0	+20.0

U 97-U 99	94.5	+18.1	+19.5	+26.9	+20.6	+9.8	+13.1	+10.8	+37.6	+45.0	+39.7	+27.9	+31.5	+28.9	+3.0	+30.0
U 99-U 101	152.2	+20.5	+18.3	+23.4	+21.4	+9.2	+13.7	+10.2	+38.8	+43.9	+40.9	+27.9	+32.2	+30.7	+3.0	+30.0
U 101-U 103	226.0	+19.7	+20.0	+20.2	+18.5	+8.8	+10.1	+9.2	+38.2	+40.9	+39.2	+29.5	+30.8	+29.9	+3.0	+30.0
U 103-U 105	320.0	+20.7	+16.6	+18.1	+17.0	+5.3	+9.1	+8.3	+37.3	+38.8	+37.3	+29.1	+29.5	+29.2	+3.0	+30.0
U 105-U 107	396.0	+21.0	+16.1	+17.0	+16.3	+8.1	+8.5	+8.2	+43.1	+38.0	+37.3	+29.1	+29.5	+29.2	+3.0	+30.0
BOTTOM CHORD																
I 93-L 95	105.0	-6.2	-11.0	-15.6	-11.2	-5.5	-7.8	-5.6	-17.2	-18.8	-17.4	-11.7	-14.0	-11.8	-21.0	-18.1
I 95-L 97	210.0	-7.5	-8.8	-11.6	-9.7	-10.0	-10.0	-10.0	-19.5	-17.5	-11.9	-14.5	-12.4	-21.7	-17.7	-15.0
I 97-L 99	271.6	-10.7	-10.9	-13.9	-12.2	-6.0	-6.5	-4.9	-20.3	-23.1	-21.9	-14.7	-13.3	-12.1	-21.0	-18.1

L 99-L101	391.6	-11.3	-9.6	-11.4	-10.4	-4.8	-5.7	-5.2	-20.9	-22.7	-21.7	-16.1	-17.0	-16.5	-21.3	-17.6
L 101-L103	523.6	-11.3	-9.2	-10.2	-10.6	-4.0	-5.1	-4.8	-20.5	-21.5	-20.9	-15.9	-16.4	-16.1	-21.0	-17.3
L 103-L103	606.8	-11.9	-8.8	-9.5	-9.1	-4.4	-4.8	-4.0	-20.3	-21.0	-20.6	-15.9	-16.3	-16.1	-20.8	-17.1
MAIN DIAGONALS																
L 91-U 93	381.0	+4.4	+10.3	+23.2	+14.6	+9.7	+14.6	+8.8	+19.7	+29.6	+18.0	+10.1	+15.0	+9.2	+2.40	+20.0
C 92-U 92	55.5	+0.8	+6.2	+23.7	+17.9	+8.1	+11.9	+7.5	+21.0	+28.5	+19.7	+12.9	+16.7	+12.3	+2.40	+20.0
L 93-C 94	38.3	+0.8	+13.2	+17.6	+13.0	+0.6	+8.9	+6.9	+23.0	+27.6	+23.6	+16.8	+18.7	+16.7	+2.40	+20.0
C 94-U 95	81.3	+0.6	+12.4	+16.6	+13.0	+0.6	+8.9	+6.9	+23.0	+27.6	+23.6	+16.8	+18.7	+16.7	+2.40	+20.0
L 95-C 96	105.0	+11.9	+11.2	+14.2	+12.2	+5.9	+7.3	+6.3	+23.1	+21.1	+24.1	+16.4	+18.9	+17.1	+2.40	+20.0

C 96-U 97	119.0	+12.0	+11.1	+13.6	+11.9	+ 5.6	+ 6.8	+ 5.9	+ 23.1	+25.6	+23.9	+17.6	+18.8	+17.9	+24.0	+20.0
L 97-U 98	147.0	+12.6	+10.2	+11.6	+10.8	+ 5.1	+ 5.8	+ 5.4	+22.8	+24.2	+23.4	+17.7	+18.4	+18.0	+24.0	+20.0
C 98-U 99	128.7	+12.9	+10.2	+11.4	+10.6	+ 5.1	+ 5.7	+ 5.3	+22.8	+24.0	+23.2	+17.7	+18.3	+17.9	+24.0	+20.0
L 99-U 100	168.0	+12.9	+ 9.8	+10.2	+10.0	+ 5.1	+ 5.6	+ 5.0	+22.8	+24.0	+23.2	+17.7	+18.3	+17.9	+24.0	+20.0
U100-U101	180.2	+13.2	+ 9.9	+10.3	+10.0	+ 4.9	+ 5.2	+ 5.0	+23.1	+23.5	+23.2	+17.8	+18.0	+17.9	+24.0	+20.0
L105-U106	91.9	+ 8.2	+13.4	+15.0	+ 8.0	+ 6.7	+ 7.5	+ 4.0	+19.6	+23.2	+16.2	+14.9	+15.7	+12.2	+24.0	+20.0
C106-U107	129.7	+ 8.5	+13.8	+12.0	+ 7.0	+ 5.4	+ 6.0	+ 3.5	+19.3	+20.5	+15.5	+13.9	+14.5	+12.0	+24.0	+20.0
MAIN POINTS																
U 93-L 93	44.1	- 4.2	-12.7	-18.5	-11.6	- 6.3	- 9.3	- 5.8	-16.9	-22.7	-15.8	-10.5	-13.5	-10.0	-20.4	-16.0

U 95-L 95	70.5	- 8.1	9.9	-13.2	-10.4	- 4.9	- 6.6	- 5.2	-18.8	-22.1	-19.3	-13.8	-15.5	-14.1	-20.0	-16.4	
U 95-L 97	99.6	-10.1	- 9.3	-10.7	- 9.4	- 4.7	- 5.4	- 4.7	-19.4	-20.8	-19.5	-14.8	-15.5	-14.8	-19.8	-16.2	
U 99-L 99	122.7	-10.2	- 7.5	- 8.5	- 8.0	- 3.8	- 4.3	- 4.0	-17.7	-18.7	-18.2	-14.0	-14.5	-14.2	-19.3	-15.8	
U101-L101	131.7	-11.2	- 7.0	- 7.6	- 7.6	- 3.5	- 3.8	- 3.8	-18.2	-18.8	-18.2	-14.7	-15.0	-15.0	-18.4	-15.0	
U105-L105		51.5	- 0.7	+17.8	+23.6	+15.4	+ 8.9	+11.8	+ 7.7	+18.5	+24.3	+10.1	+ 9.6	+12.5	+ 8.4	+24.0	+20.0

ABOVE STRESSES ARE GIVEN

ISLAND CANTILEVER ARM WEST																	
Memb.	Area		Dead Load	Live Load						Total Load						Permissible Unit Stresses or	
	Gross	Net		Congested			Regular			Congested			Regular				
				Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.	Con- tinuous	Discon- tinuous	Total Disc.		
TOP CHORD																	
U39-U41		36.0 + 4.7	+18.2 +22.2 + 4.6	+ 9.1	+11.1 + 2.3	+22.9 +20.9 + 9.3	+13.8 +15.8 + 7.0	+24.0 +20.0									
U41-U43		87.6 + 8.1	+15.2 +18.3 +11.6	+ 7.6	+ 9.1 + 5.8	+23.3 +24.4 +19.7	+15.7 +17.2 +13.9 +24.0	+20.0									
U43-U45		94.5 +10.8	+21.5 +23.5 +18.4	+10.8	+12.7 + 0.2	+38.3 +42.3 +35.2	+27.0 +29.5 +20.0 +39.0	+30.0									
U45-U47		141.8 +19.8	+19.1 +23.2 +18.4	+ 9.9	+11.6 + 0.2	+39.6 +43.0 +38.2	+29.7 +31.4 +29.0 +39.0	+30.0									
U47-U49		210.0 +21.0	+17.8 +20.5 +17.4	+ 8.9	+10.2 + 8.7	+38.8 +41.5 +38.4	+29.0 +31.2 +29.7 +39.0	+30.0									
U49-U51		290.0 +21.8	+17.4 +18.8 +16.8	+ 8.7	+ 8.4 +39.2 +40.6 +35.6	+30.5 +31.2 +30.2 +39.0	+30.0										
U51-U53		384.0 +22.0	+16.8 +17.2 +15.8	+ 8.4	+ 8.6 + 7.0	+38.8 +39.2 +37.8	+30.4 +30.6 +29.9 +39.0	+30.0									
U53-U55		406.0 +22.0	+16.0 +16.2 +15.2	+ 8.0	+ 8.1 + 7.0	+38.0 +38.2 +37.2	+30.0 +30.1 +29.6 +39.0	+30.0									
U55-U57		612.0 +22.0	+15.3 +15.4 +14.6	+ 7.6	+ 7.7 + 7.3	+37.3 +37.4 +36.6	+29.6 +29.7 +29.3 +39.0	+30.0									
BOTTOM CHORD																	
L43-L45	247.6	-10.8 -11.2	-13.2 -10.4	- 6.6	- 6.6 -11.2	-22.0 -24.0 -21.2	-16.4 -17.4 -10.0	-21.8 -18.8									

L45-L47	355.6	-11.1	-10.4	-12.0	-10.2	-6.2	6.0	-5.1	-22.3	-23.0	-22.1	-17.1	-17.9	-16.5	-21.4	-17.7
L48-L49	51.0	-11.1	-10.4	-12.0	-10.2	-6.2	6.0	-5.1	-22.3	-23.0	-22.1	-17.1	-17.9	-16.5	-21.4	-17.7
L49-L51	606.8	-12.1	9.3	-9.6	8.9	-4.6	5.2	4.7	-21.4	-23.9	-21.7	-16.6	-17.0	-16.0	-21.6	-17.7
L51-L53	834.8	-12.0	8.8	-9.0	8.5	-4.4	5.5	2.2	-20.8	-21.0	-20.5	-16.4	-16.5	-14.2	-21.0	-17.7
L53-L55	949.2	-11.9	-8.4	-8.5	-8.1	-4.2	-4.2	-4.0	-20.3	-20.4	-20.0	-16.1	-16.1	-15.9	-20.8	-17.7
MAIN DIAGONALS																
U37-C38	57.2	-0.4	-15.2	-18.8	-9.2	-7.6	-9.4	-4.0	-15.0	-19.2	-9.6	-8.0	-9.8	-5.0	-19.6	-16.6
L39-C40	76.5	+9.2	+14.4	+17.0	+11.7	+7.2	+8.5	+5.0	+23.6	+26.2	+20.9	+16.4	+17.7	+15.1	+24.0	+20.0
C40-U41	83.2	+10.2	+13.8	+17.2	+11.4	+6.7	+8.1	+5.7	+24.0	+26.4	+21.6	+17.1	+18.3	+15.9	+24.0	+20.0

L41-C42	105.5 ± 11.1	+11.9	+13.4	+10.8	+ 5.9	+ 6.5	+5.4	+23.5	+25.2	+22.4	+17.5	+18.4	+17.0	+24.0	+20.0
L41-C43	136.7 ± 11.7	+11.9	+13.4	+10.8	+ 5.8	+ 6.0	+5.5	+23.8	+25.3	+22.7	+17.8	+18.6	+17.4	+24.0	+20.0
L41-C44	135.6 ± 13.9	+11.9	+13.4	+10.6	+ 5.5	+ 6.8	+5.0	+23.5	+25.2	+22.4	+17.5	+18.6	+17.0	+24.0	+20.0
L44-C45	152.2 ± 12.8	+10.7	+11.6	+10.4	+ 5.3	+ 5.8	+ 5.2	+23.5	+24.4	+23.2	+18.1	+18.6	+18.0	+24.0	+20.0
C44-C46	173.3 ± 13.6	+10.3	+11.6	+10.2	+ 5.1	+ 5.8	+ 5.1	+23.9	+25.2	+23.8	+18.7	+19.4	+18.7	+24.0	+20.0
C46-U7	189.0 ± 13.6	+10.2	+10.8	+10.1	+ 5.1	+ 5.4	+ 5.1	+23.8	+24.4	+23.7	+18.7	+19.0	+18.7	+24.0	+20.0
L47-U8	210.0 ± 13.6	+ 9.6	+ 9.9	+ 9.5	+ 4.8	+ 4.9	+ 4.8	+23.2	+23.5	+23.1	+18.4	+18.4	+18.4	+24.0	+20.0
L48-C49	227.3 ± 13.8	+ 9.6	+ 9.9	+ 9.5	+ 4.9	+ 4.9	+ 4.9	+23.2	+23.5	+23.1	+18.4	+18.4	+18.4	+24.0	+20.0
L49-C50	228.0 ± 14.0	+ 9.2	+ 9.3	+ 9.1	+ 4.6	+ 4.6	+ 4.5	+23.2	+23.3	+23.1	+18.6	+18.6	+18.5	+24.0	+20.0
C50-U1	248.0 ± 14.0	+ 9.1	+ 9.2	+ 9.0	+ 4.6	+ 4.6	+ 4.5	+23.1	+23.2	+23.0	+18.6	+18.6	+18.5	+24.0	+20.0

		MAIN POSTS														
U41-L41	74.7	-8.3	-10.6	-12.4	-8.8	-5.3	-6.2	-4.4	-18.9	-20.7	-17.3	-13.0	-14.5	-12.7	-10.9	-16.1
U43-L43	103.0	-9.8	-9.7	-10.3	-8.5	-4.8	5.1	-4.2	19.5	-20.1	-18.3	-14.6	-14.9	-14.0	-19.8	-16.3
U45-L45	130.4	-10.9	-9.1	-9.6	8.5	-4.6	-4.8	4.3	-20.0	-20.5	-19.4	-15.5	-15.7	-15.2	-19.4	-15.3
U47-L47	156.2	-11.5	-8.2	-8.6	-8.1	-4.1	-4.3	4.1	-19.7	-20.1	-19.6	-15.6	-15.8	-15.6	-19.2	-15.2
U49-L49	178.6	-11.8	7.7	-7.8	-7.0	-3.8	-3.9	3.8	-19.5	-20.0	-19.4	-15.6	-15.7	-15.6	-18.6	-15.1
U51-L51	186.4	-12.0	-7.2	-7.2	-7.3	-3.0	-3.6	-3.6	-19.6	-20.0	-19.6	-15.6	-16.0	-15.0	-18.9	-15.3
U53-L53	196.2	-10.4	-7.7	-7.5	6.5	-3.8	-4.1	-3.8	-18.9	-17.1	-14.2	-16.0	-15.8	-18.1	-14.0	-15.1
U55-L55		76.5	5.8	14.9	22.0	+15.4	+3.0	+3.7	+25.2	+27.8	+21.2	+15.0	+16.8	+13.5	+23.0	+15.1

ROCKER ARM																
U37-L37 {	44.1	+ 0.2	-14.0	-18.6	- 8.4	- 7.0	- 9.3	- 4.2	-13.8	-18.4	- 8.2	- 6.8	- 9.1	- 4.0	-17.6	-14.0
		31.2 + 0.3	+23.6	+29.0	+15.8	+11.8	+14.5	+ 7.9	+23.9	+29.3	+16.1	+12.1	+14.8	+ 8.2	+24.0	+20.0
QUEENS ANCHOR ARM																
TOP CHORD																
U107-U109	408.0	+20.6	+17.0	+18.0	+16.8	+ 8.5	+ 0.0	+ 8.4	+37.6	+38.0	+37.4	+29.1	+29.6	+29.0	+39.0	+30.0

U109-U111	348.0 + 18.1	+19.2	+2.0	+17.6	+ 9.6	+10.1	+ 8.8	+37.6	+38.0	+36.0	+28.0	+28.5	+27.2	+39.0	+30.0	
U111-U113	290.0 + 15.9	+21.5	+22.6	+18.2	+10.8	+11.3	+ 9.1	+37.4	+38.5	+34.1	+26.7	+27.2	+25.0	+39.0	+30.0	
U113-U115	292.0 + 13.1	+23.8	+25.0	+18.8	+11.9	+12.5	+ 9.4	+36.9	+38.1	+31.9	+25.0	+25.6	+22.9	+39.0	+30.0	
BOTTOM CHORD																
L109-L111	619.6	-11.0	- 9.3	- 9.8	- 9.1	- 4.6	- 4.6	- 4.0	-20.3	-20.8	-20.1	-15.0	-15.9	-15.6	-20.9	-17.0
L111-L113	571.6	- 9.9	-10.0	-11.2	-10.0	5.3	- 5.9	- 5.0	-20.5	-21.1	-19.9	-15.2	-15.5	-14.9	-21.3	-17.7
L113-L115	475.6	- 9.6	- 9.4	-13.0	-10.5	6.2	- 6.5	- 5.8	-21.4	-22.0	-19.5	-13.2	-15.5	-14.3	-21.5	-17.7
L109-L111	583.9	- 7.4	-13.9	-14.0	-11.1	-11.1	- 7.1	- 9.8	-21.8	-22.5	-19.1	-13.1	-13.8	-13.0	-21.6	-17.8
L111-L113	283.9	- 5.5	-15.3	-14.8	-11.9	- 7.7	- 8.3	- 5.8	-20.8	-22.2	-17.3	-13.2	-13.8	-11.1	-21.9	-17.8

MAIN DIAGONALS																
U111-C112	152.0	+14.2	+ 8.8	+ 9.0	+ 8.8	+ 4.4	+ 5.4	+ 4.4	+23.0	+23.2	+23.1	+18.6	+18.7	+18.6	+24.0	+20.0
U112-C113	182.0	+14.2	+ 8.7	+ 9.0	+ 8.8	+ 4.4	+ 5.4	+ 4.4	+23.0	+23.2	+23.1	+18.6	+18.7	+18.6	+24.0	+20.0
U113-C114	196.0	+13.7	+ 9.5	+10.7	+10.0	+4.8	+ 5.4	+5.0	+23.2	+24.4	+23.7	+18.5	+19.1	+18.7	+24.0	+20.0
U114-C115	168.0	+13.8	+ 9.7	+11.0	+10.2	+4.9	+ 5.5	+5.1	+23.5	+24.8	+24.0	+18.7	+19.3	+18.9	+24.0	+20.0
U115-C116	167.5	+13.8	+10.2	+13.3	+11.6	+ 5.1	+ 6.7	+ 5.8	+24.0	+27.1	+25.4	+18.9	+20.5	+19.9	+24.0	+20.0
U116-C117	147.1	+13.8	+10.4	+13.8	+12.0	+ 5.2	+ 6.9	+ 6.0	+24.2	+26.7	+25.8	+19.0	+20.7	+19.7	+24.0	+20.0
U117-C118	147.0	+10.4	+ 8.7	+15.5	+12.3	+ 6.3	+ 7.8	+ 6.9	+25.0	+27.7	+26.7	+18.7	+19.7	+19.0	+24.0	+20.0
U118-C119	141.7	+ 9.5	+13.2	+16.0	+12.3	+ 6.5	+ 8.1	+ 6.2	+23.7	+25.7	+25.9	+19.3	+17.9	+16.7	+24.0	+20.0

U119-C120	137.6	+ 5.6	+16.0	+18.0	+12.6	+ 8.0	+ 9.0	+ 6.3	+22.7	+24.7	+19.3	+14.7	+15.7	+13.0	+24.0	+20.0
U120-L121	137.6	+ 5.6	+16.0	+18.0	+12.2	+ 8.0	+ 9.0	+ 6.1	+21.6	+23.6	+17.8	+13.3	+14.6	+11.7	+24.0	+20.0
MAIN POSTS																
U107-L107 1047.4	-11.6	- 7.2	- 7.3	- 7.2	- 3.6	- 3.7	- 3.0	-18.8	-18.9	-18.8	-15.2	-15.3	-15.2	-18.6	-15.0	-15.0
U109-L109 88.5	- 3.0	-15.0	-16.6	- 6.3	- 7.5	- 8.3	- 3.2	-18.0	-19.6	- 9.3	-10.5	-11.3	- 6.2	-18.0	-14.0	-14.0
U111-L111 1317.7	- 8.6	- 8.3	- 8.6	- 6.3	- 4.2	- 4.3	- 3.1	-16.9	-17.2	-19.4	-12.8	-12.9	-11.7	-17.3	-14.0	-14.0
U113-L113 128.2	-12.1	- 7.1	- 7.6	- 7.3	- 3.6	- 3.8	- 3.6	-19.2	-19.7	-19.4	-15.7	-15.9	-15.7	-18.6	-15.0	-15.0
U115-L115 133.3	- 4.4	- 3.3	- 3.4	- 3.3	- 4.2	- 4.7	- 3.6	-17.7	-19.8	-18.8	-13.7	-13.8	-13.7	-18.7	-15.0	-15.0
U117-L117 119.7	-10.2	-19.1	-19.1	-19.1	- 9.8	-10.2	- 7.0	-19.1	-19.1	-19.1	-14.7	-14.7	-14.7	-18.7	-15.0	-15.0

U119-L119	110.7	-6	-13.4	-14.4	-10.2	-6.7	-7.2	-5.1	-19.6	-20.6	-16.4	-12.9	-13.4	-11.3	-20.0	-16.0
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UNITS OF 1,000 POUNDS PER SQUARE INCH

ISLAND SPAN																							
Memb.	Area		Dead Load	Live Load									Total Load									Permissible Unit Stresses for T R	
				Congested			Regular			Congested			Regular										
	Gross	Net		Cen- troids	Discon- tinuities	Total Disc.	Cen- troids	Discon- tinuities	Total Disc.	Cen- troids	Discon- tinuities	Total Disc.	Cen- troids	Discon- tinuities	Total Disc.	Cen- troids	Discon- tinuities	Total Disc.	Comp. Cont.	Tens. Cont.			
TOP CHORD																							
U57-U59		660.0	+22.2	+15.8	+16.0	+15.8	+ 7.9	+8.5	+10.9	+38.0	+39.1	+38.0	+30.1	+30.7	+30.1	+39.0	+30.0						
U59-U61		516.0	+22.4	+17.4	+20.9	+18.5	+ 8.7	+10.5	+ 9.2	+39.8	+43.3	+40.9	+31.1	+32.9	+31.6	+39.0	+30.0						
U61-U63		426.0	+21.8	+18.4	+23.8	+20.2	+ 9.2	+11.9	+10.1	+4.0	+45.6	+42.0	+31.0	+32.7	+31.9	+39.0	+30.0						
U63-U65		384.0	+21.1	+18.8	+25.7	+21.3	+ 9.4	+12.0	+10.6	+3.9	+46.8	+42.4	+30.5	+34.0	+31.7	+39.0	+30.0						
U65-U67		360.0	+20.9	+19.2	+26.5	+21.8	+ 9.6	+13.2	+10.9	+4.1	+47.4	+42.7	+30.5	+34.1	+31.8	+39.0	+30.0						
U67-U69		360.0	+20.9	+19.2	+26.5	+21.8	+ 9.6	+13.2	+10.9	+4.1	+47.4	+42.7	+30.5	+34.1	+31.8	+39.0	+30.0						
U69-U71		360.0	+20.8	+19.0	+25.6	+21.2	+ 9.5	+12.8	+10.6	+3.8	+46.4	+42.0	+30.3	+33.6	+31.4	+39.0	+30.0						
U71-U73		392.0	+21.1	+18.5	+23.4	+19.9	+ 9.2	+11.7	+10.0	+3.6	+44.5	+41.7	+30.3	+32.8	+31.1	+39.0	+30.0						
U73-U75		448.0	+21.6	+17.6	+19.7	+17.8	+ 8.8	+ 9.9	+ 8.9	+39.2	+41.3	+39.4	+30.4	+31.5	+30.5	+39.0	+30.0						
BOTTOM CHORD																							
L59-L61	1032.8		-11.8	- 8.6	- 9.2	- 8.6	- 4.3	- 4.6	- 4.3	-20.4	-21.0	-20.4	-16.1	-16.4	-16.1	-20.9	-17.3	31					

L61-163	925.5	-11.9	-9.4	-11.3	-10.0	-4.7	-5.7	-5.0	-21.3	-23.2	-21.9	-16.6	-17.6	-16.9	-21.2	-17.5/28
163-165	770.0	-11.9	-10.2	-13.2	-13.2	6.1	6.6	5.6	-22.1	-25.1	-23.1	-17.0	-18.5	-17.5	-21.3	-17.0/27
165-167	680.7	-11.9	-10.2	-14.4	-11.9	-5.2	-5.2	-5.2	-21.8	-24.8	-22.8	-16.8	-18.3	-17.3	-21.1	-17.0/27
167-169	651.3	-11.3	-10.5	-14.2	-11.8	-5.2	7.1	5.9	-21.8	-25.5	-23.1	-16.5	-18.4	-17.2	-21.3	-17.0/27
L69-171	635.3	-11.6	-10.3	-13.0	-11.1	6.1	6.5	5.6	-21.9	-24.6	-22.7	-16.7	-18.1	-17.2	-21.2	-17.5/28
L71-173	717.0	-11.2	-9.4	-10.4	-9.4	-4.7	-5.2	-4.7	-20.6	-21.6	-20.6	-15.9	-16.1	-15.9	-20.9	-17.3/31
MAIN DIAGONALS																
U57-C58	365.0	-3.9	-0.7	-15.9	-11.4	-4.9	8.0	5.7	-13.6	-19.8	-15.3	-8.8	-11.0	-9.6	-14.6	-12.5/94
C58-L59	408.0	-4.7	-8.9	-14.6	-10.6	-4.4	7.3	-5.3	-13.6	-19.3	-15.3	-9.1	-12.0	-10.0	-14.6	-12.5/94

O2-163	305.3	+1.1	+1.8	+13.0	+0.4	+5.9	+6.5	+4.7	+23.6	+24.8	+21.2	+17.7	+18.3	+16.5	+24.0	+20.0
C83-064	243.1	+10.3	+13.1	+15.2	+10.3	+0.5	-7.6	-5.2	+23.4	+25.5	+20.0	+16.8	+17.9	+15.5	+24.0	+20.0
O2-164	222.1	+9.0	+12.7	+16.0	+10.4	+0.5	-7.6	-5.2	+23.4	+25.5	+20.0	+16.8	+17.9	+15.5	+24.0	+20.0
U65-066	164.7	+5.5	+15.8	+20.4	+12.4	+7.9	+10.2	+6.2	+23.3	+27.9	+19.9	+15.4	+17.7	+13.7	+24.0	+20.0
O66-167	151.7	+6.2	+10.9	+22.0	+13.2	+8.4	+11.0	+6.6	+23.1	+28.2	+19.4	+14.6	+17.2	+12.8	+24.0	+20.0
L67-085	211.9	-1.7	-12.0	-14.6	-8.0	-0.0	-7.3	-4.0	-13.7	-16.3	-9.7	-7.7	-9.0	-5.7	-15.1	-12.0
C83-069	177.5	-0.3	-14.4	-17.6	-9.2	-7.2	-8.8	-4.0	-14.7	-17.9	-9.5	-7.5	-9.1	-4.9	-10.0	-12.8
O66-168	165.3	-5.0	-15.2	-18.5	-10.8	-5.5	-2.8	-10.5	-13.7	-16.3	-9.7	-7.7	-9.0	-5.7	-15.1	-12.0
L70-071	181.8	+0.0	+10.8	+18.6	+10.6	-8.4	+0.3	+5.3	+23.1	+24.9	+16.9	+14.7	+15.6	+11.6	+24.0	+20.0
C73-074	388.7	-5.2	-8.0	-14.7	-10.6	-4.0	-7.3	-5.3	-13.2	-19.9	-15.8	-9.2	-12.5	-10.5	-14.6	-11.5

C74-U75	353.0	-4.4	-8.8	-16.2	-11.5	-4.4	-8.1	-5.7	-13.4	-20.8	-16.1	-9.0	-12.7	10.3	-14.6	-11.594
MAIN POSTS																
U57-157	1250.6	-12.0	-7.2	-7.3	-6.5	-3.6	-3.7	-3.3	-19.2	-19.3	-18.5	-15.6	-15.7	-15.3	-15.6	-15.254
E99-L59	236.9	+11.6	+14.0	+22.6	+17.4	+7.0	+11.3	+8.7	+20.0	+34.2	+18.0	+8.6	+22.9	+20.3	+4.6	+20.254
U63-L63	281.0	-8.7	-9.0	-11.5	-7.8	-4.9	-5.8	-3.9	-18.6	-20.2	-16.5	-13.6	-14.5	-12.6	-17.8	+14.662
U65-L65	191.0	-6.7	-12.0	-15.5	-9.4	-6.0	-7.7	-4.7	-18.7	-22.2	-16.1	-12.7	-14.4	-11.4	-18.3	-14.957
E99-L69	141.8	-0.9	-15.3	-18.4	-12.3	-7.5	-9.6	-7.6	-16.3	-13.7	-9.5	-7.6	-10.5	-8.7	-15.2	+15.254
U71-L71	148.7	+4.9	+9.0	+19.2	+13.2	+8.5	+9.0	+6.0	+33.2	+34.9	+28.7	+14.5	+14.6	+11.1	+24.0	+20.9
U73-L73	230.9	+1.7	+13.6	+22.2	+17.0	+8.6	+11.1	+8.5	+25.3	+33.7	+28.7	+15.5	+22.8	+20.2	+24.0	+20.9

[illegible][illegible]

MAIN POSTS.— $\left\{ \begin{array}{l} \text{U 59-1, 59} \\ \text{U 59-1, 59} \end{array} \right. 25.5 \text{ U 59-1, 59} + 34.2 \text{ U 59-1, 59} + 29.0 \left\{ \begin{array}{l} \text{U 59-1, 59} \\ \text{U 59-1, 59} \end{array} \right. + 18.6 \text{ U 59-1, 59} + 22.9 \text{ U 59-1, 59} + 20.3$

LIVE LOAD.—Congested=16,000 lbs. per ft. of bridge.
Regular=8,000 lbs. per ft. of bridge.
Continuous means a load in a single continuous stretch causing maximum stress.
Discontinuous means a load in two or more continuous stretches causing absolute maximum stress.
 $\frac{1}{2}$ Total means a load over the entire length of the bridge of $\left\{ \begin{array}{l} 8,000 \text{ lbs. per ft. of bridge, congested.} \\ 4,000 \text{ lbs. per ft. of bridge, regular.} \end{array} \right.$

DEAD LOAD.—Steel now in structure=105,150,000 lbs.
Steel in new structure=5,400,000 lbs. per ft. of bridge.

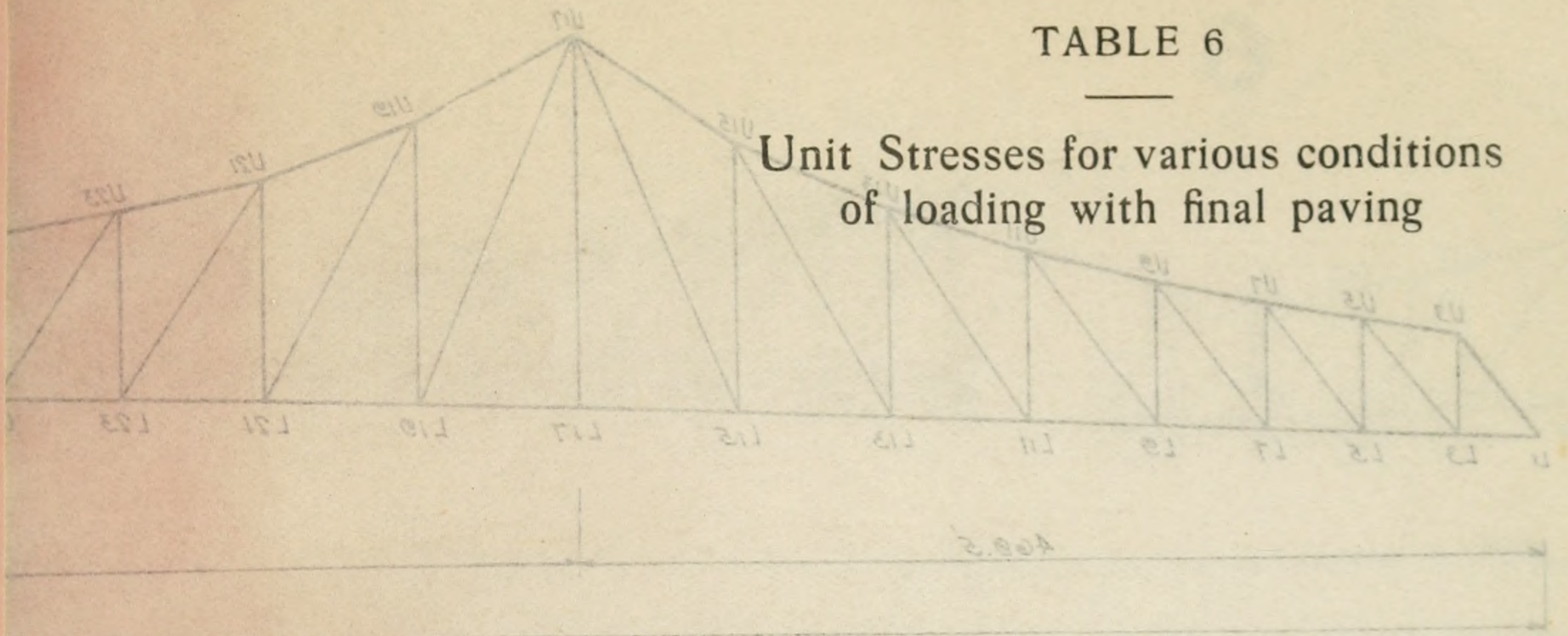
Made Up as Follows:		UNIT STRESSES FOR VARIOUS CONDITIONS OF LOADING INCLUDING DEAD LOAD FOR ORIGINAL PAVING
Rails for 4 Lower Tracks	280 lbs.	BLACKWELL'S ISLAND BRIDGE NO. 4 OVER EAST RIVER, NEW YORK CITY
Pipes, etc.	200 lbs.	
Hand-rails	240 lbs.	
Ties and rails for 4 elevated railroad tracks	1,370 lbs.	
Additional steel	3,400 lbs.	
Paving	3,400 lbs.	

BUILT BY
THE PENNSYLVANIA STEEL CO.
BRIDGE & CONSTRUCTION DEPT.
STEELTON, PA.
DEC., 1908

ABOVE STRESSES ARE GIVEN IN UNITS OF 1000 POUNDS PER SQUARE INCH

TABLE 6

Unit Stresses for various conditions of loading with final paving



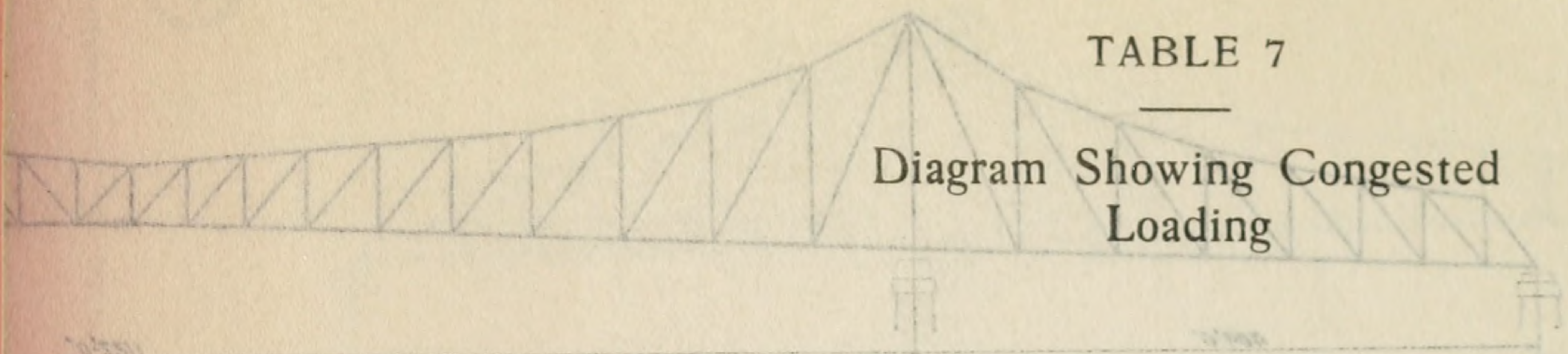
MANHATTAN ANCHOR ARM

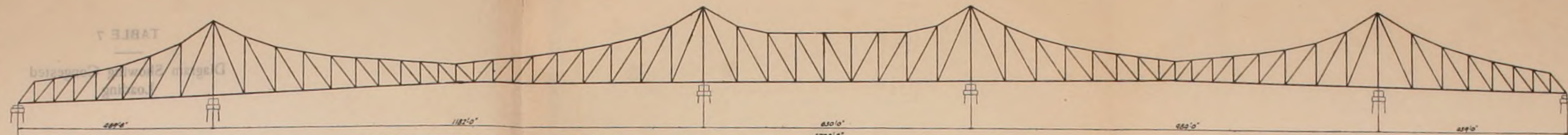
Membr.	Area		Dead Load	Live Load			Total Load							
	Gross	Net		Con- tinuous Disc.	Con- tinuous Disc.	Total Disc.	Con- tinuous Disc.	Con- tinuous Disc.	Total Disc.					
U 7-U 9	310.0	+15.5	+22.8	+23.5	+18.4	+11.4	+11.8	+9.2	+32.3	+39.0	+33.9	+26.9	+27.3	+24
U 9-U 11	404.0	+17.2	+21.1	+21.7	+17.7	+10.5	+10.9	+8.9	+38.3	+38.9	+34.9	+27.7	+28.1	+25
U 11-U 13	496.0	+18.9	+19.4	+20.0	+17.1	+9.7	+10.0	+8.5	+38.3	+38.9	+36.0	+28.6	+28.9	+27
U 13-U 15	580.0	+20.6	+17.8	+18.4	+16.5	+8.9	+9.2	+8.9	+38.4	+39.0	+37.1	+29.5	+29.8	+28
U 15-U 17	650.0	+21.5	+15.9	+16.3	+15.4	+7.9	+8.2	+7.7	+37.4	+37.8	+36.9	+29.4	+29.7	+29
TOP CHORD														
L 5-L 7	555.0	-8.5	-12.9	-13.3	-10.4	-6.4	-6.7	-5.2	-21.4	-21.8	-18.9	-14.9	-15.2	-13
L 7-L 9	601.6	-9.5	-12.9	-12.3	-10.0	-6.0	-6.2	-5.0	-21.5	-21.8	-19.5	-15.5	-15.7	-14
L 9-L 11	633.3	-10.2	-10.8	-11.0	-9.4	-5.4	-5.5	-4.7	-21.0	-21.2	-19.6	-15.6	-15.7	-14
L 11-L 13	672.0	-11.0	-9.7	-10.0	-9.0	-4.9	-5.0	-4.5	-20.7	-21.0	-20.0	-15.9	-16.0	-15
L 13-L 15	1029.2	-11.6	-8.6	-8.9	-8.4	-4.3	-4.5	-4.2	-20.2	-20.5	-20.0	-15.9	-16.1	-15
BOTTOM CHORD														
L 3-C 4	244.0	+9.8	+13.6	+13.0	+11.7	+6.8	+7.5	+5.9	+22.9	+24.3	+21.0	+16.1	+16.3	+15
C 4-U 5	252.0	+9.8	+13.1	+14.5	+11.5	+6.6	+7.2	+5.8	+22.9	+24.3	+21.3	+16.4	+17.0	+15
L 5-C 6	242.0	+11.8	+11.8	+13.5	+11.4	+6.9	+6.8	+5.7	+23.6	+25.3	+23.2	+17.7	+18.6	+17
C 6-U 7	252.0	+12.3	+11.3	+13.0	+11.2	+5.7	+6.5	+5.6	+23.6	+25.3	+23.5	+18.0	+18.8	+17
L 7-C 8	228.0	+13.7	+9.9	+12.6	+11.4	+4.9	+6.3	+5.7	+23.6	+26.3	+25.1	+18.8	+20.0	+19
C 8-U 9	244.0	+14.0	+9.7	+12.2	+11.0	+4.9	+6.1	+5.5	+23.7	+26.9	+25.0	+18.9	+20.1	+19
L 9-C 10	240.0	+14.4	+9.1	+10.0	+9.4	+4.6	+5.0	+4.7	+23.5	+24.4	+23.8	+19.0	+19.4	+19
C 10-U 11	258.0	+14.6	+9.0	+9.9	+9.4	+4.5	+5.0	+4.7	+23.6	+24.5	+24.0	+19.1	+19.6	+19
MAIN DIAGONALS														
U 3-L 5	192.0	-6.7	-13.5	-13.8	-10.0	-6.7	-6.9	-5.9	-20.2	-20.5	-18.7	-13.4	-13.6	-11
U 5-L 7	192.0	-9.0	-11.6	-12.3	-9.6	-5.8	-6.1	-4.8	-20.6	-21.2	-18.6	-14.8	-15.1	-13
L 7-L 9	181.5	-10.6	-9.8	-11.0	-9.3	-4.9	-5.5	-4.7	-20.4	-21.6	-19.9	-15.3	-16.1	-15
U 9-L 11	181.5	-11.9	-7.8	-9.3	-8.4	-3.9	-4.6	-4.2	-19.7	-21.2	-20.3	-15.8	-16.5	-16
L 11-L 13	163.4	-12.6	-6.6	-6.8	-6.7	-3.3	-3.4	-3.3	-19.2	-19.4	-19.3	-15.9	-16.0	-15
U 13-L 15	130.5	-8.7	-6.7	-9.3	-5.4	-4.3	-4.6	-3.7	-17.6	-17.9	-17.0	-13.0	-13.2	-11
U 15-L 17	147.5	+5.0	+18.6	+19.2	+13.2	+9.3	+9.6	+9.6	+24.2	+24.8	+19.2	+14.0	+13.2	+12
U 17-L 19	1306.3	-12.8	-6.8	-6.3	-6.7	-3.4	-3.4	-3.4	-19.2	-19.3	-19.3	-15.0	-15.0	-15

ISLAND CANTILEVER ARM, EAST

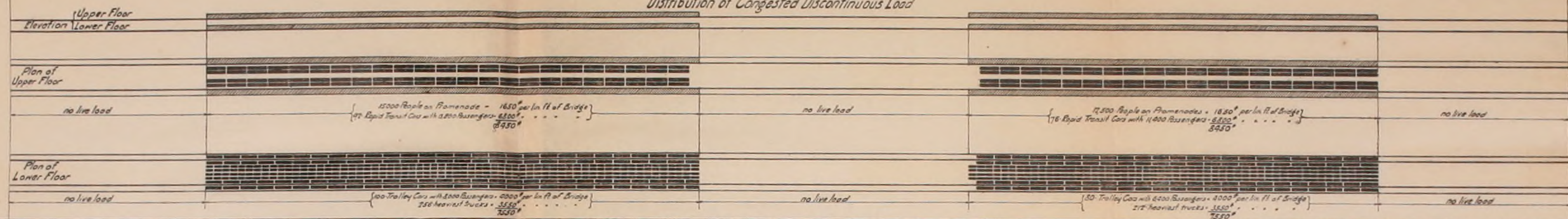
TOP CHORD							
U 7-U 17	400.0	+21.8	+17.3	+17.1	+16.0	+8.7	+8.7
U 17-U 19	321.0	+21.4	+15.0	+15.3	+14.6	+9.0	+9.2
U 19-U 21	240.0	+20.5	+15.4	+15.3	+14.8	+9.2	+9.8
U 21-U 23	162.8	+20.0	+10.5	+11.0	+10.9	+9.8	+11.0
U 23-U 25	98.0	+18.3	+31.2	+25.4	+19.3	+12.7	+9.7
U 25-L 27	88.5	+8.9	+15.3	+18.1	+12.2	+7.6	+9.1
U 27-U 29	36.0	+5.2	+18.2	+22.0	+12.6	+9.1	+11.0
BOTTOM CHORD							
L 17-L 19	630.8	-11.8	-9.1	-9.3	-8.5	-4.6	-4.6
L 19-L 21	247.0	-11.3	-9.6	-9.0	-8.0	-4.2	-5.0

Diagram Showing Congested Loading

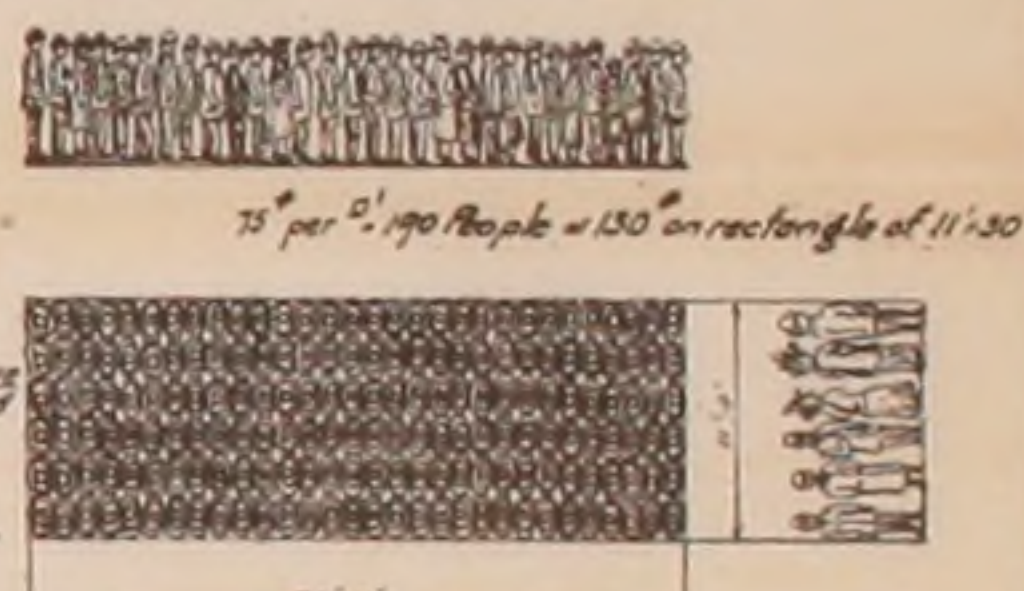
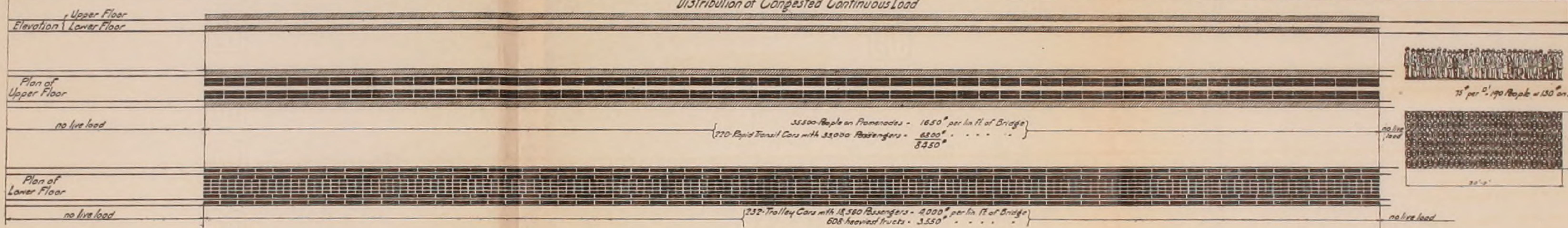




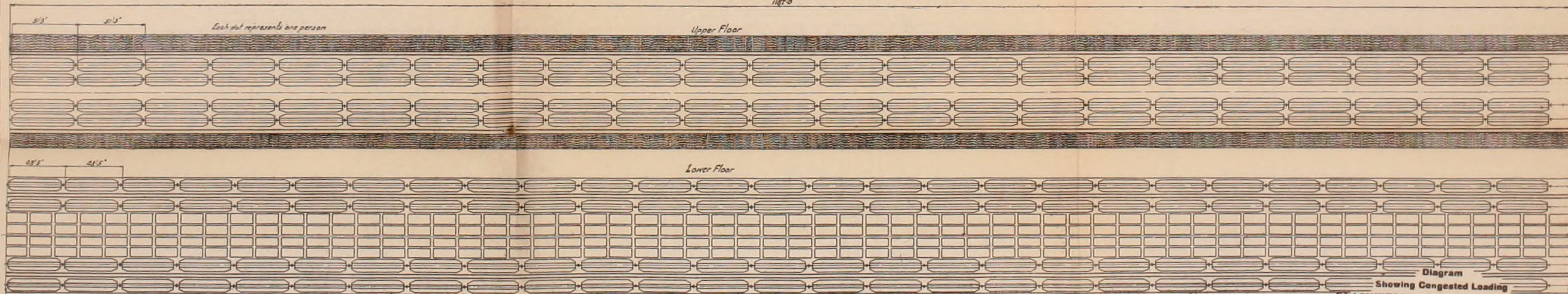
Distribution of Congested Discontinuous Load



Distribution of Congested Continuous Load



Distribution of Congested Load on Manhattan River Span (shown in enlarged scale)



Height of one trolley car - 35,000'
80 Passengers at 130' - 10,400'
total - 45,400' - 1000' per lin ft of track

Height of people on promenade - 75' per sq ft
Height of trucks on roadway - 100' per sq ft

Height of Elevated R.R.
5 Trolley Cars at 71,780' - 358,900'
3 Trailer Cars at 51,300' - 153,900'
8 - 150 Passengers at 150' - 150,000'
total for one train of 8 cars - 662,800' - 1700' per lin ft of track

Diagram
Showing Congested Loading
BLACKWELL'S ISLAND BRIDGE NO 4
over East River - New York City.
BUILT BY
THE PENNSYLVANIA STEEL CO.,
Bridge & Construction Dep't
Steelton, Pa.
Jan 1907

Scale - 1 in. = 100 ft. 1/4 in. = 30 ft.