COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY 40601

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COMMISSIONER OF HIGHWAYS

March 27, 1973

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H.3.33

MEMORANDUM TO: J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT: Research Report No. 361; "Fatigue Analysis of Central Bridge Deduced from Strain Gage Data and Probability Analysis"; KYP-72-33; HPR-1(8), Part III.

The Central Bridge at Newport was constructed in 1891 and modified in 1914, 1930, 1964 (steel grid floor installed), and 1968. A structural analysis made in 1947 qualified it for H-20 loading provided necessary repairs were made. An H-15 loading was assigned in 1968. Early in 1970, severe corrosion of eye-bar members was discovered where "paired" members came together at a common pin (no spacer between them). The loss of section was reported in July of 1970 *(1)*. The greatest loss of section was 23%. Not considering wind loads, cursory analysis indicated that the stresses remained within the allowable over-stress limit. However, the possibility remained that the attendant increase in dead-load stress together with certain live-load stresses could cause fatiguing (exceed the endurance limit) -- beginning at some point in history.

[1] "Replication of Eye-Bars and Measurements of Losses in Cross Section due to Corrosion;" Indexed as Report No. 296.

Prior to the disclosure of corrosion, we had undertaken a more general study of the fatigue life of older bridges in the State -- more particularly, six Ohio River bridges. It was necessary to statistically reconstitute (or synthesize) the traffic history for each bridge. Records were searched to obtain the nearest applicable traffic volumes and loadometer data back to the earliest date. One-day, on-site loadometer measurements were made in 1968 by the Department of Motor Vehicle Transportation to aide us in this program and to guide the Department otherwise in posting load limits. In order to use traffic volumes (AADT'S) and weight data in a fatigue analysis, it was necessary to know the probability of various combinations of vehicles occurring on a span at a given time. To do this, we made spot-speed surveys (with a radar meter) and time-interval and sequence-of-type measurements and surveys. From these surveys, we were able to totalize probability percentages -- which, when multiplied by ADT's year by year, gave us total repetitions of loadings (then converted to number of stresses in the member considered). A model fatigue-damage diagram (equation) was constructed on the log S-vs-log N basis (2×10^6 repetitions at the endurance limit -- i.e. stress below which no damage occurs, chosen for the particular type of steel). The computations of fatigue life in this way are feasible only by use of a high-speed computer. We have, on a previous occasion referred to the whole program as the "soft

synthesis" of "used up" and "remaining" fatigue life.

The enabling work was started by Robert L. Lynch in 1967 *(2, 3)*. Lynch ventured to Australia to pursue a doctoral degree but left an unfinished manuscript and the beginning of a computer program. The report was finished by R. C. Deen and me. It was issued belatedly in January 1972 *(4)*. To that

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[2]"Analysis of Traffic Loads on Bridges," February 1968; Indexed as Report No. 251.

[3]"Analysis of Traffic Loads on Bridges; Report II: Characteristics of Traffic on Ohio River Bridges -- 1968;" March 1969; Indexed as Report No. 275.

[4]"Bridges: Synthesis of Load Histories and Analyses of Fatigue;" Report No. 318.

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time, only hypothetical problems had been solved. Meanwhile, R. J. Bruner, III, a mid-year graduate (1970-71), was assigned to the study. He has advanced through the learning process and is the author of the report being submitted herewith. The report specifically addresses the analysis of Central Bridge because the loss of section in eye-bar members brings the fatigue-life estimates into a current time domain. Certain assumptions have been made to encompass extreme possibilities -- that is, to bracket possible inaccuracies in input parameters, etc. Even so, factors such as wind loads and temperature stresses have not been included. The life estimates are, therefore, conservative estimates in the sense that possibly a great number of stress cycles remain unaccounted.

It is interesting perhaps, but saddening to us, to cite an error in Equation 4, page 3, of Report No. 318, which should have read $P_{niG} = P_i^n P_{G(t)}^{(n-1)}$. This error delayed the final writing of the computer program. Another error which delayed the analysis was an iterative statement which required multiplication of damage factors by zero when the probabilities of the load occurrence was zero. Until this was corrected, the computer time estimate was 2.5 hours.

Basically, the analysis considers 0.0 and 23.0% loss of section; the 23% loss of section relates to a specific eye bar. Where no loss of section is involved, the fatigue damage (or percent of fatigue life "used up") remains nil. Intuitively, we might expect similar analyses of other bridges to yield similar results. The Bridge Division has completed plans for the replacement of the affected eye bars on Central Bridge; and the analysis presented here tend to superimpose a degree of urgency upon the scheduling of the work. We do understand that work is being proposed for a letting in April. We are unable at this time to determine whether or not loads (trucks) should be restricted or prohibited from using the bridge in the interim. Our estimates of some affecting factors simply are not that accurate. Mainly, we do not know how the corrosion progressed with time. Neither the linear rate nor parabolic curve represents the real case -- or else our other assumptions are grossly inaccurate.

Upon disclosure of the losses in section of the eye bars, there was immediate concern also about the possibility of imbalance of load on the paired members. Mindful that an imbalance could, perhaps critically, overstress one or the other of the members, Prewitt Scratch Gages were installed on three sets of eye-bar pairs (U15L'5L'4-3&4, U14L6L'5-3&4, and D14L3L2-3&4); strain records were obtained from August 31 to November 10, 1970. These records were analyzed and reported *(5)* as to the number

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[5]"Stress Histories of Bridge Members from Scratch Gage Records;" by R. D. Hughes; Report No. 323, February 1972.

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of strain events (and thus live-load stresses); however, it was not possible to compare single events on paired members. Judging from the first set of scratch records obtained, it appeared that a full year of strains could be recorded; the gages were re-started and were scheduled to be retrieved in September 1971. Unfortunately, during the month of August, the gages were removed by parties unknown and have not been recovered. A new set of gages was installed April 17, 1972; and records were obtained through 4 1/2 months. These were translated into stress events and used to estimate fatigue life. Eye-bar U14L6L'5-3 would, thus, have a fatigue life of 112 years - that is, assuming loading to be constant through all years and the rate of corrosion to be constant (total = rate x number of years).

Finally, in order to determine more accurately the distribution of load between paired members, SR-4-type strain gages were affixed to bars D14L3L2-3&4. These gages were monitored (on a strip-chart recorder) for 3 hours on September 15 and for 3.5 hours on September 26, 1972. Results are discussed in the attached report.

In summary, it appears that the loss of section (due to corrosion) in identified eye bars has reduced fatigue life from infinite time to a finite time base. The confidence limit (or variability) associated with the assumed value for the endurance limit of the steel remains undefined -- therefore, the probability of failure at a given calendar time remains somewhat undefined also.

This has been an interesting progression of study for us. At times, it has been difficult to maintain continuity through lapses of inactivity. Much remains to be done in the area of bridge life and obsolescence. We timorously mention design life and planned retirement of bridges. Few bridge engineers in this country dare to speculate on how long a bridge will endure; whereas, in Europe, there is an international committee on bridge life. Some recent studies made in this country on modern bridges (some designed for stiffness) have indicated fatigue lives of 400, 1600, and 4000 years, etc. if the present level of loading persists and if other decaying processes do not beset the structure. We may find on other bridges we started to study, as in the case of Central Bridge, that fatigue life is unlimited unless corrosion becomes the affecting factor or loads severely exceed limits set by engineering analysis.

NOTE: As a matter of interest and convenient reference, I have appended behind the report a copy of the engineer's report on the design and construction of Central Bridge (from **Transactions**, ASCE, August 1892).

Respectfully submitted,

Jas. H. Havens

Jas. H. Havens
Director of Research

JHH:dw
Attachment
cc's: Research Committee

TECHNICAL REPORT STANDARD TITLE PAGE

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16. Abstract

This report presents evaluations of the load history of the Central Bridge from both strain gage data and probability analysis. Estimation of remaining service life is made through fatigue criteria. Also included is a comparison (from strain gage data) of live load stresses carried by parallel eye bars. An appendix provides a user's manual for the computer program for probability-based analyses of load events and fatigue-life computations.

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Research Report
361

FATIGUE ANALYSIS OF CENTRAL BRIDGE DEDUCED FROM STRAIN GAGE DATA AND PROBABILITY ANALYSIS

KYP-72-33, HPR-1(8), Part III

by

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Division of Research
DEPARTMENT OF HIGHWAYS
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March 1973

SUMMARY AND CONCLUSIONS

Fatigue analysis of the Central Bridge was approached in this study in two ways: 1) by use of "Prewitt Scratch Gages" and 2) from probability analysis. The short-term scratch gage measurements of strains, expanded through time, indicated a remaining service life of 30 to 40 years; whereas, the probability analysis indicated that failure could have occurred between 1936 and 1961. The large discrepancies between two analyses arose from extremely large loads (up to ten combination trucks on a 254-foot span) synthesized by the probability analysis. Such loads were not observed on the scratch gage records. The effect of very large live loads superimposed on high dead-load (DL) stresses magnifies the damage factors (the EBL, Equivalent Bridge Loadings). Also, a small variance in the assumed or estimated endurance limit of the steel will produce great differences in predicted life.

There are indications from the analysis that fatigue failure may be imminent unless loads are reduced or certain eye bars strengthened or replaced. Plans have been prepared by the Division of Bridges to strengthen those eye bars which have been weakened by corrosion.

INTRODUCTION

The Central Bridge, over the Ohio River between Newport and Cincinnati, was completed in 1891 and is now in danger of fatigue damage. A progression of studies have been undertaken by the Division of Research to determine the likelihood of fatigue failure and to estimate remaining service from the standpoint of fatigue analyses. In one investigation *(1,2)*, strains were recorded and the loss of section due to corrosion of several sections was measured. Another study *(3)* developed a methodology for determination of fatigue damage from probability analysis of traffic data.

During the current study, both the probability analysis and the use of strain gages were utilized to evaluate the fatigue damage which has been incurred by the Central Bridge. Extensive strain gage data were gathered using Prewitt Scratch Gages and SR-4 resistivity gages. A computer program was developed to compute accumulated fatigue damage from traffic data. Certain assumptions were necessary; these were evaluated by comparing results obtained from adjudged, probable extremes for paired input parameters.

STRAIN GAGE ANALYSIS

PROCEDURE

Scratch Gages -- On April 18, 1972, Prewitt Scratch Gages were placed on four members of the Central Bridge. Two additional gages were attached on April 26. The gages were placed on the following paired eye bars:

April 18 - D14L3L2-3
D14L3L2-4
U14L6L'5-3
U14L6L'5-4
April 26 - U15L'5L'4-3
U15L'5L'4-4

Bars selected for instrumentation were those which had the maximum loss of section according to a previous study *(1, 2)*. Gages were 48-inch, temperature compensating Prewitt Scratch Gages. The operation and use of those gages were also reported previously *(1, 2)*.

Gages were attached to the eye bars with C-clamps (Figure 1). The threads of the clamps were soldered to provide a more permanent attachment. Restraining straps made of aluminum foil were placed at one-foot intervals along the gage to prevent possible buckling which might induce errors in the records. The gages were then covered with plastic to provide protection (Figure 2). Two gage targets showed no record -- one indicated two complete rotations and could not be read, thus accounting for the differences in total number of days of record noted in the results. All targets were sent to Baganoff Associated, Inc. in St Louis for computer reduction and analysis.

SR-4 Resistivity Gages -- On August 23, 1972, SR-4 resistivity strain gages were placed on Bars D14L3L2-3 and D14L3L2-4. The gages were placed parallel to each other on a normal section of the eye bar so that any differences in recorded strain could be attributed to differences in stresses on those members.

The strain gages used were BLH Electronics, Inc., Type FAE-50-1256-ET with a gage factor of 2.06 ± 1 percent. A three-wire system was used to eliminate any error caused by the length of hthe lead wire. Three 120-ohm ± 1 percent resistors were used to form the wheatstone bridge along with the active strain gage. Brush, dual-channel strain gage amplifier and recorder were used to balance the bridge and record strains (Figure 3). Settings used on the amplifier and recorder were as follows:

Volts per chart line	0.05
Gain	1.55 x 10
Multiplier	5

Prior to attaching gages to the bars, all paint was removed from the bars using a grinding stone and file. Surfaces were then prepared according to gage manufacturer's specifications for chemical cleansing; gages were applied using Eastman 910 Adhesive (Figure 4).

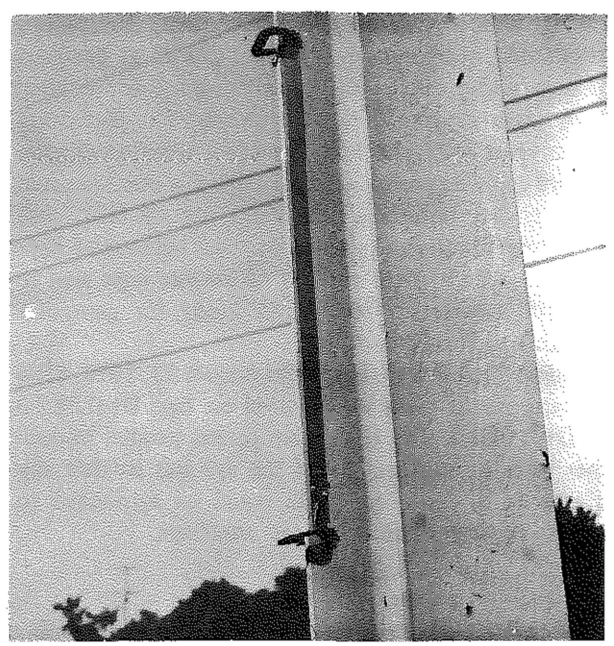


Figure 1. Method of Attaching Scratch Gage.

RESULTS

Scratch Gages -- Scratch gages were monitored for approximately 4 1/2 months. Data collected from the discs are listed in Table I. These data were analyzed by the equivalent-bridge-load criterion and a Goodman diagram to determine fatigue damage. In the EBL calculations, it was assumed that loading was constant (at the current rate) and that corrosion occurred linearly throughout the life of the bridge. Differences in stresses on parallel bars were also determined.

To calculate stresses listed in Table II, the following equation was used:

$$L = 100\ (DL + LL)/C \qquad 1$$

where L = total stress,
DL = dead-load stress from Table I,
LL = live-load stress from Table I, and
C = percent of section remaining from Table II.

The equivalent bridge load factor *(3)* was calculated from

$$EBL = N_E\ (10^{\log N_E[(L - \sigma_E)/(\sigma_u - \sigma_E)]}, \qquad 2$$

where N_E = number of events to failure at the endurance limit,
σ_E = endurance limit, and
σ_u = ultimate strength.

A table with predetermined EBL's can be found in APPENDIX B of Reference 3. The number of cycles of each load was found from

$$N' = N \times EBL \qquad 3$$

where N' = number of equivalent loads corresponding to total stress level L and
N = number of events from Table I for live-load stress level LL corresponding to total stress level L.

The yearly damage caused by the recorded loads was found from

$$D = 365\ \Sigma N'/N_E T \qquad 4$$

where D = percent damage per year caused by recorded loads and
T = elapsed time of record in days.

Since the loss of section was not known for Bar U14L6L'5-4, the above-referenced calculations could not be made. The values used in making the EBL calculations were as follows:

Ultimate strength of steel (σ_u) = 60,000 psi,
Endurance limit of steel (σ_E) = 16,500 psi, and
Events to failure at endurance limit (N_E) = 2,000,000.

From data shown in Table II, it was apparent that damage caused by the recorded loads was significant when the EBL criterion was used. The most critical member noted in the analysis was U14L6L'5-3, which showed a yearly loss of service life of 0.89 percent. This would yield a service life of 112 years if damage remained constant over the life of the bridge. Assuming

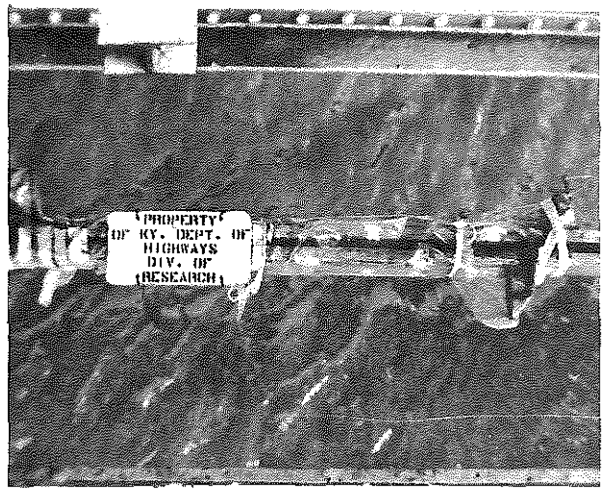


Figure 2. Scratch Gage in Place on Central Bridge; Showing Covering and Identification.

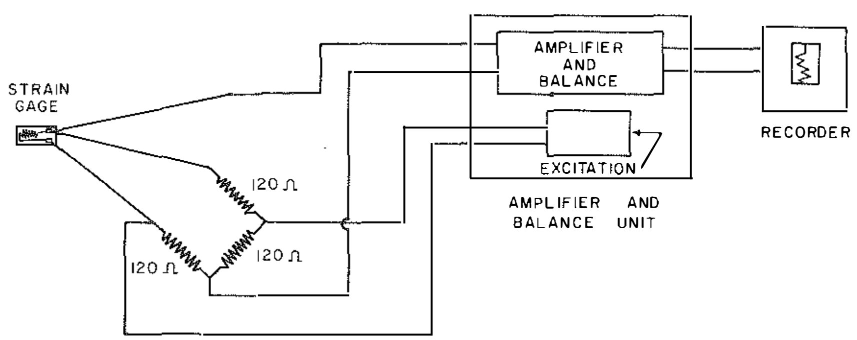


Figure 3. Schematic of Circuitry Used for SR-4 Strain Gage Recording.

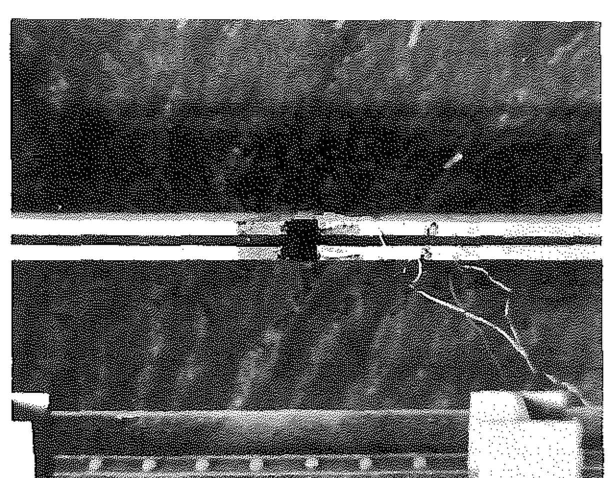


Figure 4. SR-4 Resistivity Strain Gages in Place on Central Bridge and Covered with Barrier E (BLH Electronics) Protective Coating.

TABLE 1

NUMBER OF EVENTS PER STRESS LEVEL

LIVE-LOAD STRESS (PSI)	BAR NUMBER D14L3L2-3	D14L3L2-4	U14L6L'5-3	U14L6L'5-4	U15L'5L'4-3	U15L'5L'4-4
< 200	324	338	278	513	323	44
200	597	502	462	830	543	91
400	1009	530	367	779	338	129
600	742	313	267	448	157	132
800	99	60	103	97	69	32
1000	33	25	45	29	35	31
1200	13	9	44	21	26	11
1400		4	9	3	9	4
1600	3	1	6	2	5	1
1800			3		2	2
2000	1		4		1	
2200		1			1	
2400			1		2	
2600						
2800					1	
Total Events	2821	1783	1589	2722	1507	467
Total Time Days	129	91	69	129	121	83
Average Stress (psi)	491	433	489	425	423	570
Dead-Load Stress (psi)	14180	14180	14260	14260	14260	14260
Percent of Original Section Remaining	78	85	78	NA	77	85

TABLE II

EBL LOADINGS (DL + LL) WITH LOSS OF SECTION CONSIDERED

TOTAL STRESS (PSI)	BAR NUMBER				
	D14L3L2-3	D14L3L2-4	U14L6L'5-3	U15L'5L'4-3	U15L'5L'4-4
16750		372			48
17000		602			109
17250		689			168
17500		438			185
17750		92			49
18000		42			35
18250	586	16	503		20
18500	1164	8	901	630	8
18750	2159	2	785	1162	2
19000	1729		622	787	4
19250	249	2	259	395	
19500	90		122	187	
19750	26		87	70	
20000			29	85	
20250	10		21	32	
20500			11	19	
20750	4		17	8	
21000				5	
21250			5	5	
21500				10	
21750					
22000				6	
Total	6017	2263	3362	3401	628
Damage (%/year)	0.85	0.45	0.89	0.48	0.14

that corrosion occurs uniformly over the life of the bridge, the loss of fatigue life which has occurred can be computed. When damage was computed in this way, it was found that 30 percent of the service life had been used. Another computation was made which extended present conditions into the future; this showed that the bridge had 40 years of remaining service life if corrosion continued to increase at the same uniform rate previously considered.

In these calculations, wind and temperature loadings were not considered. These loads could have considerable effect on the service life of the bridge.

In Figure 5, the maximum damage stress (DL = 18,500 psi and LL = 3,650 psi of U15L'5L'4-3) was plotted on a Goodman diagram to show its relationship to the endurance limit thus determined. It can be seen that the stress is well within the safe limits according to that criterion. Because of wind and temperature loadings, and age and condition of the steel, the more conservative EBL criterion is probably more appropriate for this situation.

Comparisons were also made of scratch gage data to determine what percent of the load was being carried by each of the paired parallel bars. Results of this analysis are shown in Figures 6, 7, and 8. Differences in stresses are apparent for all pairs. These differences are more prominent at low stresses but also occur at higher stress levels. These differences do not appear on the figures at the higher stress levels because of the low percentage of events at those stresses.

The cause of the differences in stresses in the members cannot readily be identified but several possibilities are apparent:

1) there may be loose pin connections in the eye bars,
2) the strain gages might not have been placed on sections of equal areas, and (or)
3) the strain gages might not have been exactly parallel.

SR-4 Resistivity Strain Gages -- Further analysis of stress differences in the eye bars was made using SR-4 resistivity strain gages on Bars D14L3L2-3 and D14L3L2-4. Resistivity gages were placed on the Central Bridge to determine whether or not equal strains were occurring in parallel bars. A simultaneous record (Figure 9) was made of strain in each bar. These data were then used in a least squares analysis to obtain equations relating stress in one bar to that of the companion bar. Channels of the recorder were then reversed and the least squares analysis was rerun. An average equation was then computed so that any differences in recorder channels would be eliminated. The equations and their plots are shown in Figure 10.

Differences in stresses in the instrumented paired members were relatively small. These differences could be attributed to any of the reasons mentioned earlier regarding differences found from scratch gage data.

General -- According to the equivalent-bridge-load criterion and data obtained from the scratch gages, there is noticeable fatigue damage occurring in corroded eye bars of the Central Bridge. Although the service-life calculations are vague as to life remaining in the bars, they do show that possible danger exists.

It was also found that strains in parallel members were nearly equal. Some differences were recorded, but this was more than likely due to gage locations and recording differences rather than actual differences in strains in the bars themselves. The only large differences in recorded strains were for Bars U15L'5L'4-3 and U15L'5L'4-4. In that case, there were also large differences in numbers of events per day and in percent of events per load increment, so it is possible that errors in the records for these bars may be present.

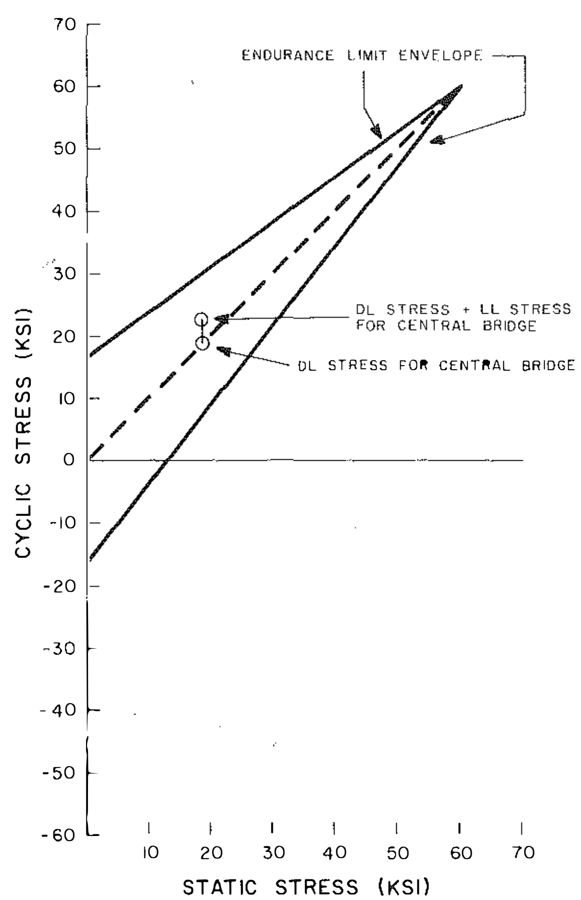


Figure 5. Goodman Diagram Showing Maximum Recorded Stress with Section Loss Considered.

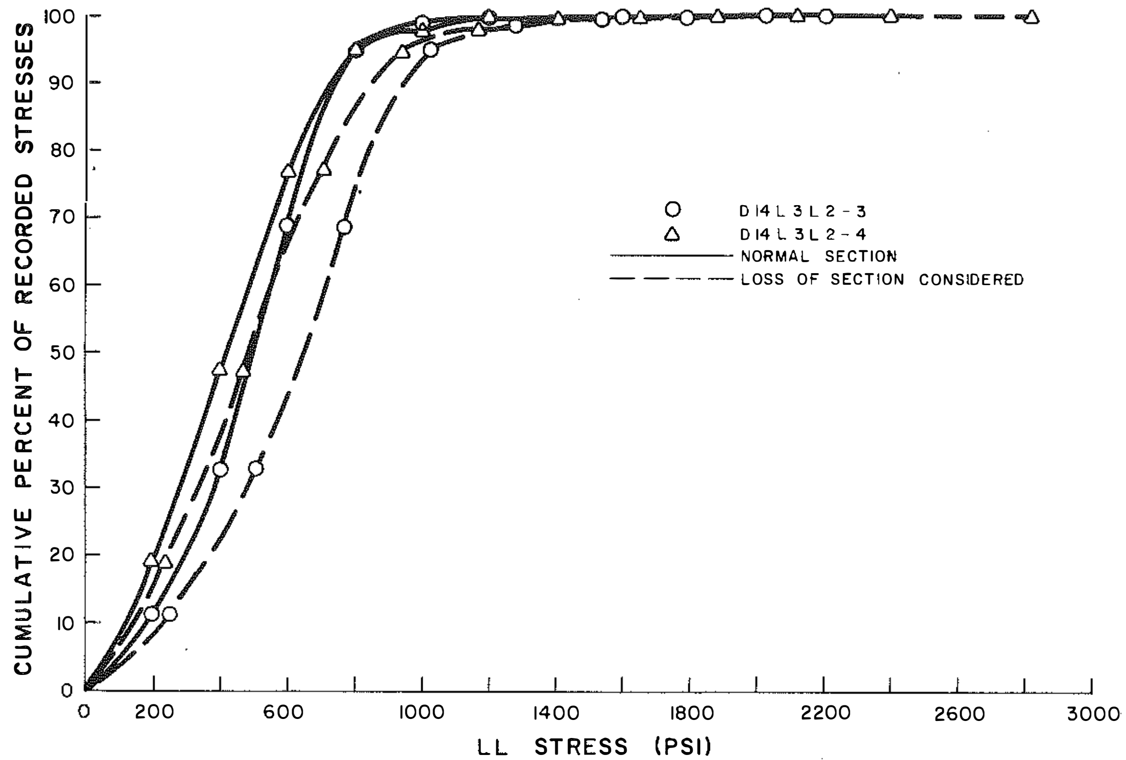


Figure 6. Cumulative Percent Vehicles vs Live-Load Stress.

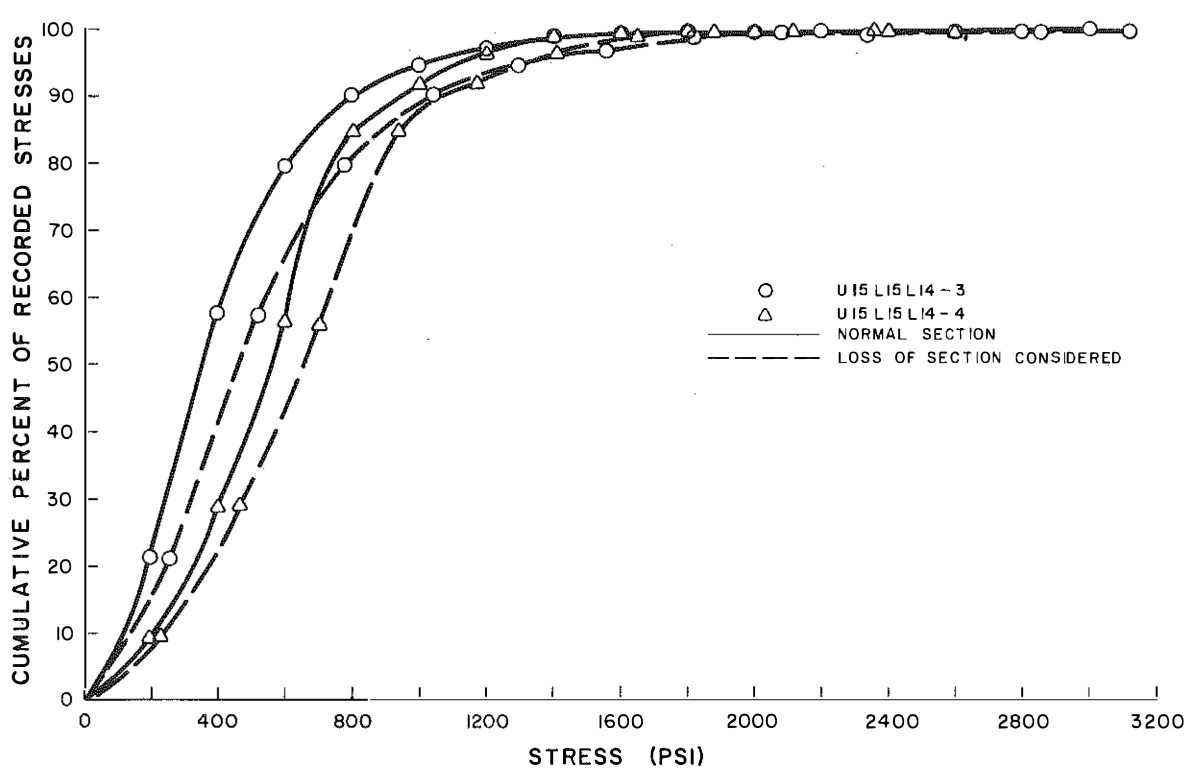


Figure 7. Cumulative Percent Vehicles vs Live-Load Stress.

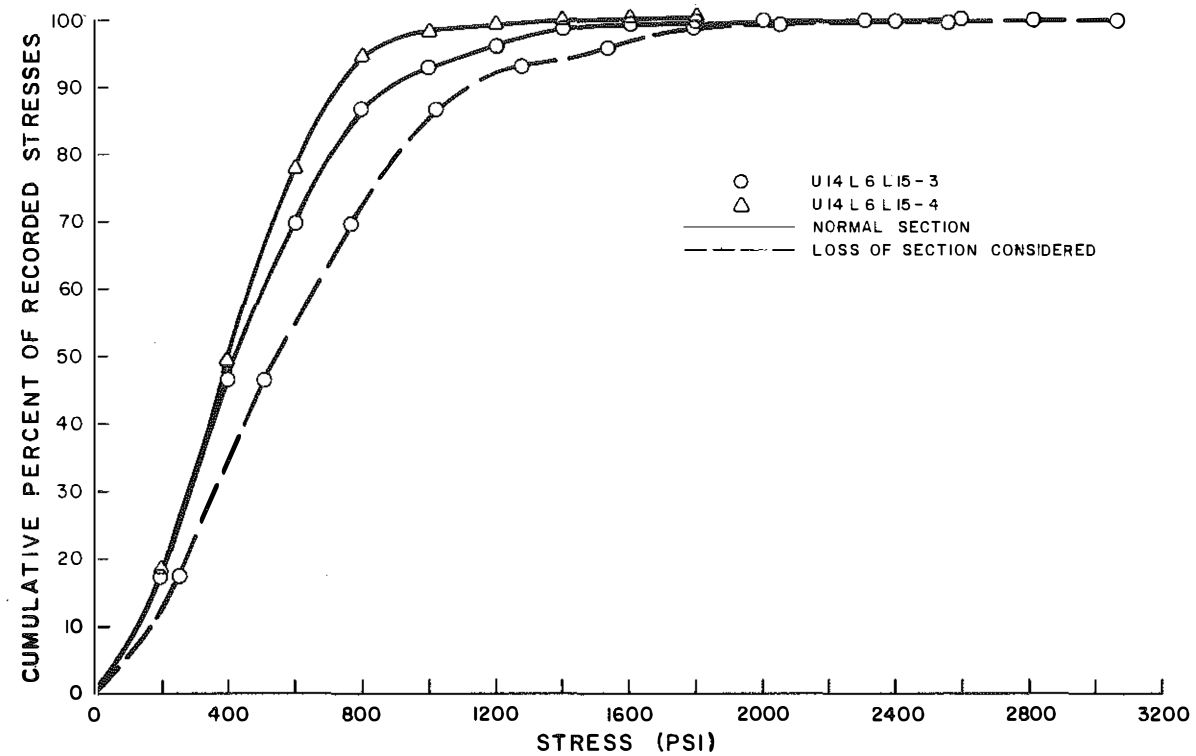


Figure 8. Cumulative Percent Vehicles vs Live-Load Stress.

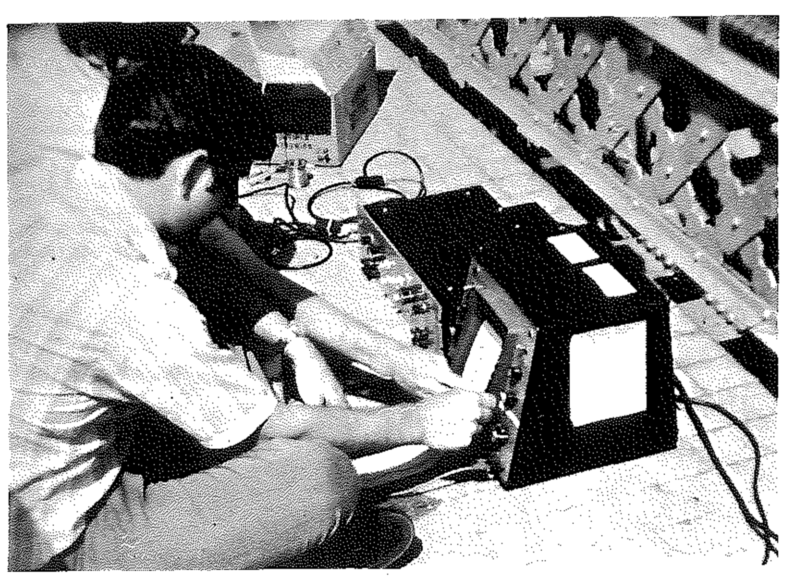


Figure 9. Equipment Utilized in Reading SR-4 Resistivity Strain Gages.

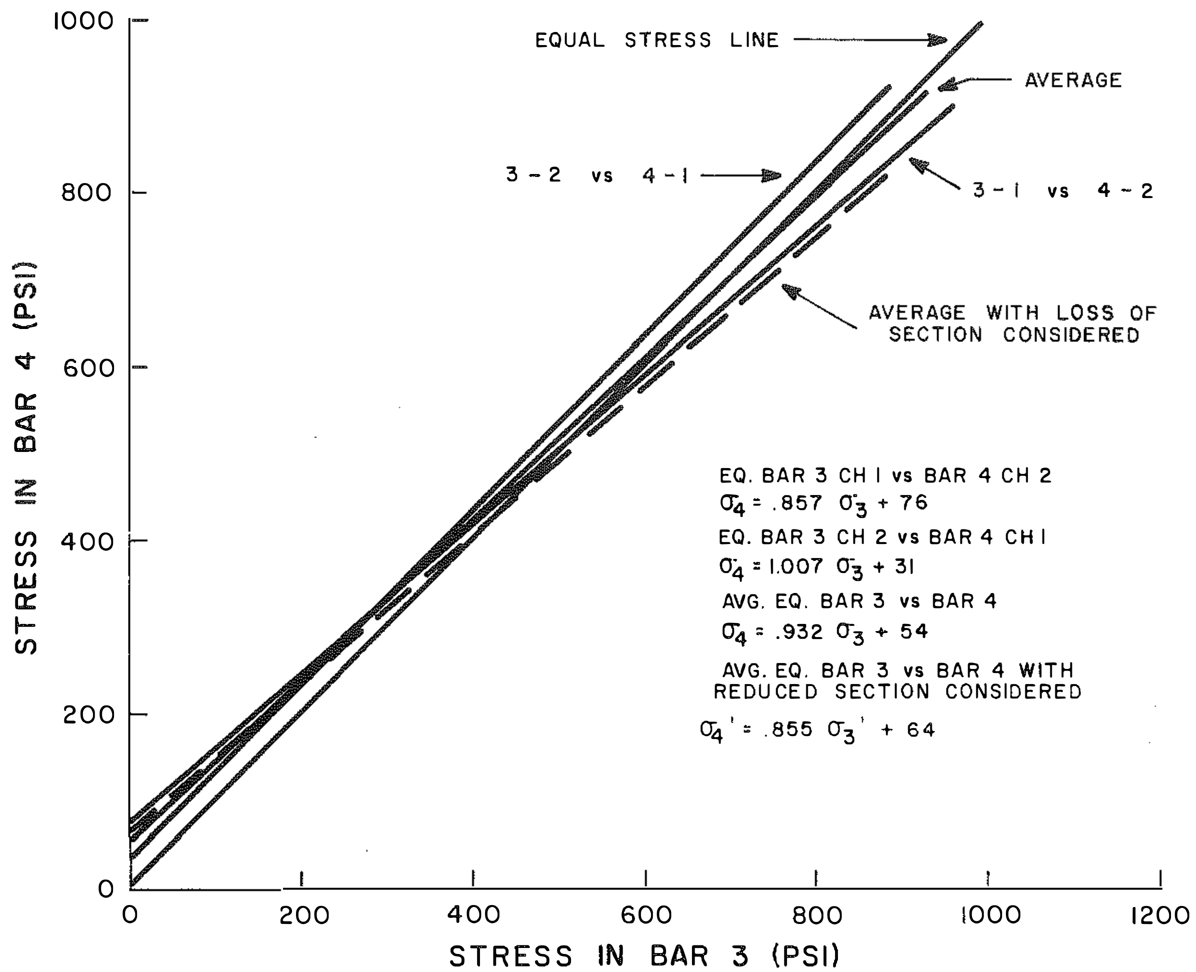


Figure 10. Stress in D14L3L2-3 vs Stress in D14L3L2-4.

PROBABILITY ANALYSIS

PROCEDURE

A computer program was developed to calculate loss of fatigue life from the probability analysis presented in Reference 3. All traffic data used in this analysis came from References 4 and 5. Input data were as follows:

Vehicle Data	
Percent of Total Traffic	
Cars	91.4 percent
Trucks	7.3 percent
Combination Trucks	1.3 percent
Average Length	
Cars	20 feet
Trucks	25 feet
Combination Trucks	47 feet
Average Spot Speed	28.2 mph
Average Weights	See Figure 11
ADT	See Figure 12
Gap Probabilities	See Figure 13

Material Data	
Yield Strength	33,000 psi
Ultimate Strength	60,000 psi
Endurance Limit	As Indicated
Events to Failure at Endurance Limit	2,000,000
Bridge Data	
Length of Span	254 feet
Width of Span	23 feet
Design Load	75 psf
Critical Member Data	
Dead-Load Stress	14,260 psi
Design Live-Load Stress	5,950 psi

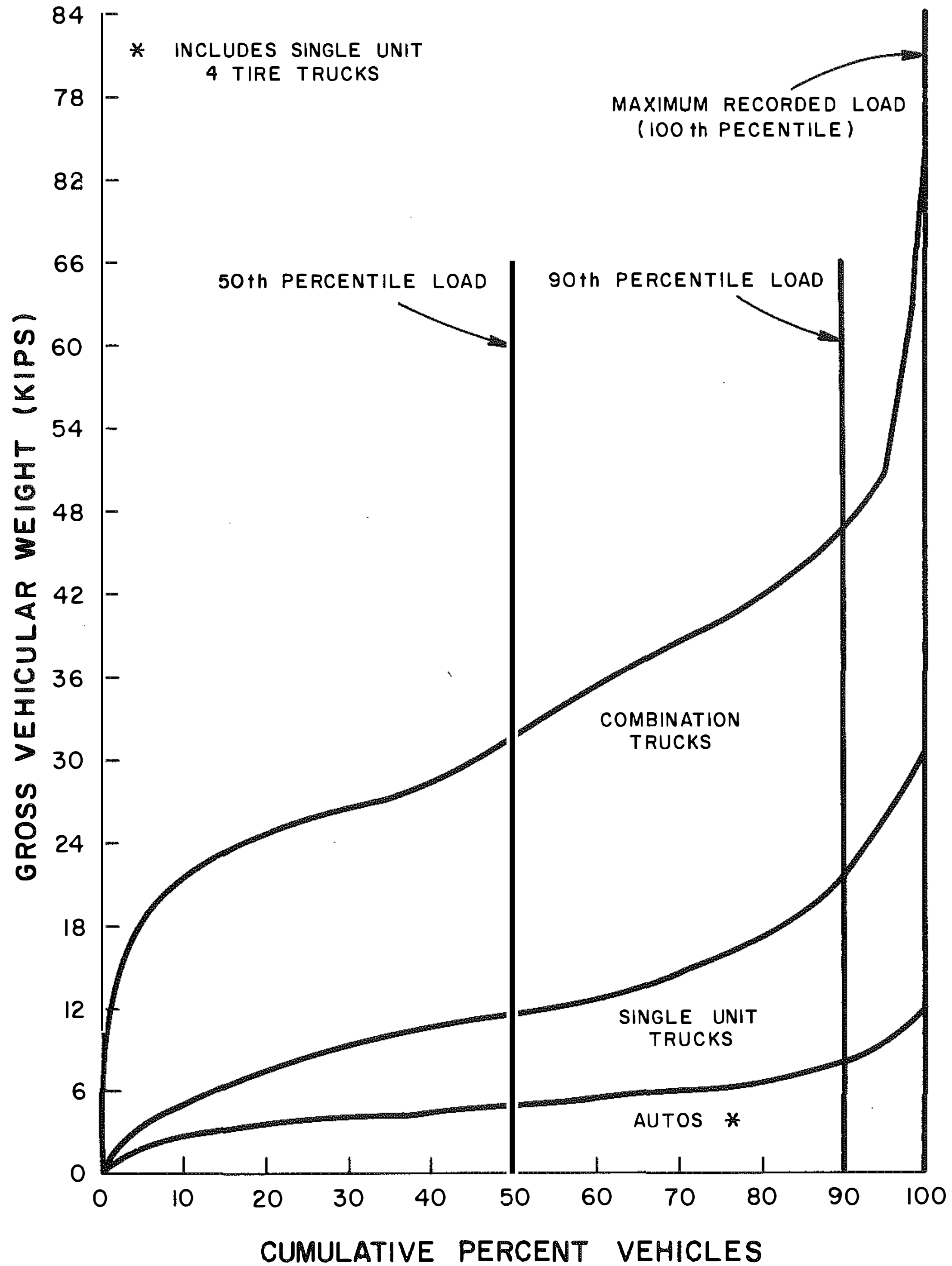


Figure 11. Cumulative Percent Vehicles vs Gross Weight for All Vehicle Types on Central Bridge.

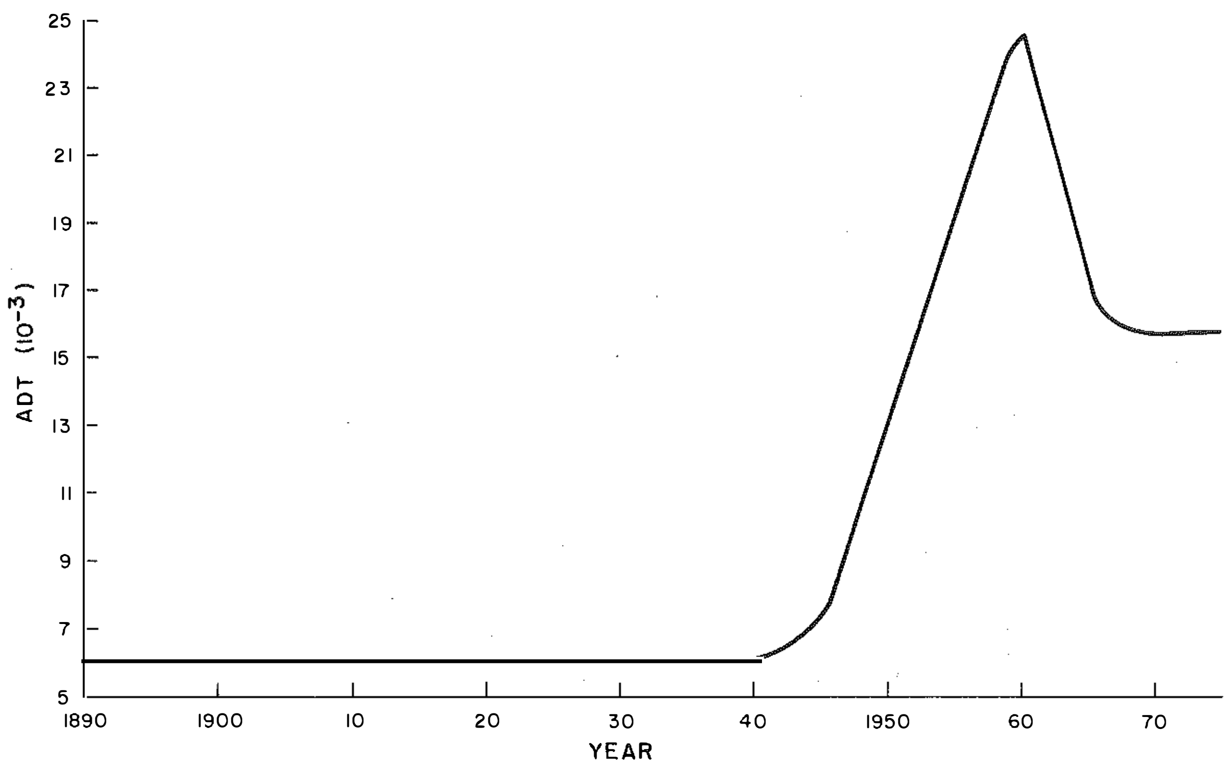


Figure 12. ADT vs Year for All Vehicle Types on Central Bridge.

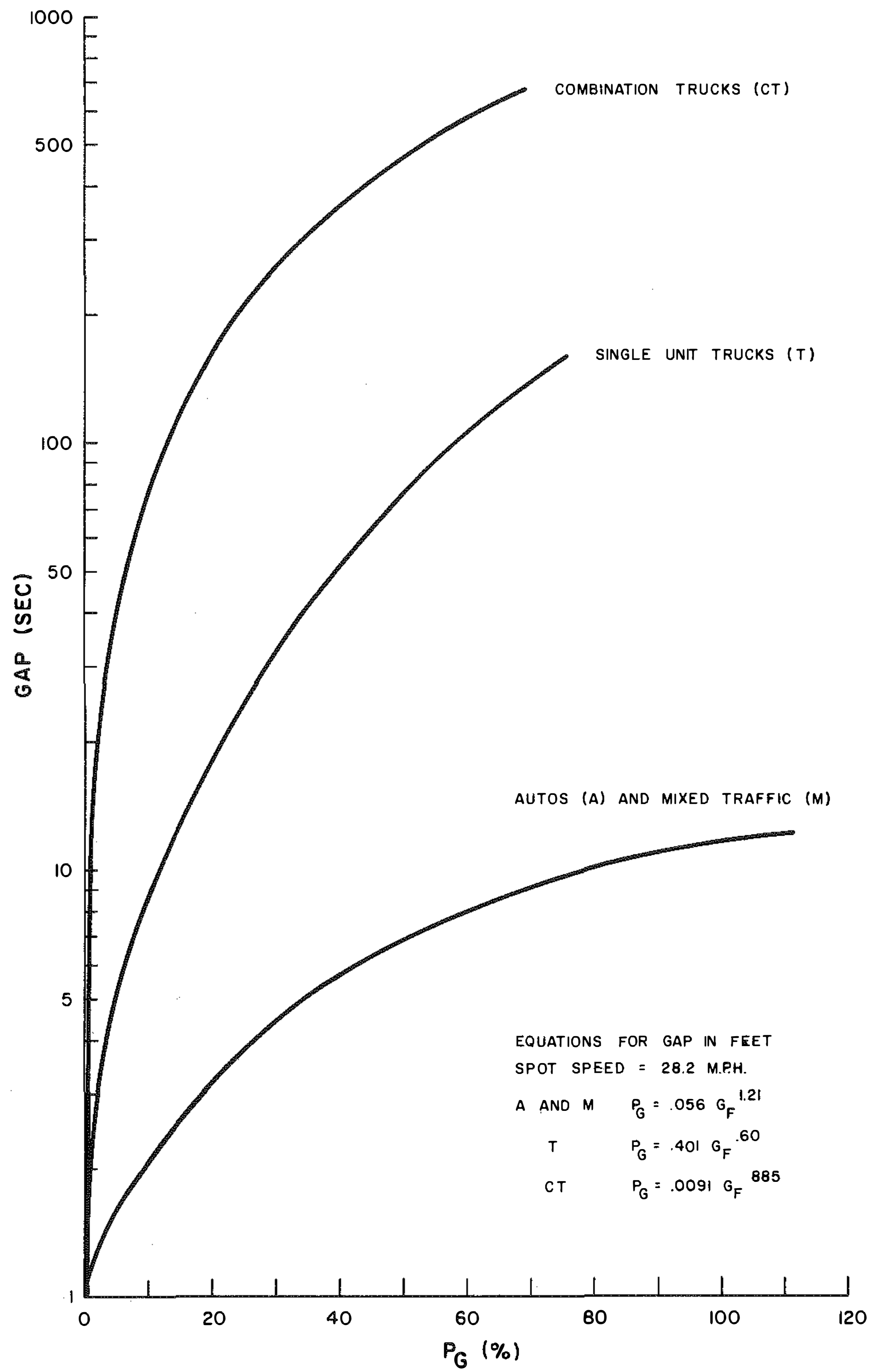


Figure 13. Gap vs Probability of Occurrence (P_G) for All Vehicle Types on Central Bridge.

All computations covered a period of 81 years (from 1891 to 1972). When corrosion was taken into account, the section was considered normal in 1891 but advanced to a 23 percent loss of section by 1972. Both uniform and parabolic aging (due to corrosion) were considered (Figure 14). For the complete computer program and an explanation of its use, see the APPENDIX. Wind and temperature stresses were not considered in this program because of the difficulty in measuring such stresses accurately.

RESULTS AND DISCUSSION

Eight computer runs were made using different loads, considerations of corrosion, and endurance limits. Results of these runs are listed in Table III. In Runs Nos. 1, 2, and 3, loss of section due to corrosion was not considered; it was found that very little damage resulted even when all vehicle classes were considered at their maximum recorded weight (Figure 11) and all recorded ADT's were doubled. All other runs took corrosion into account. These runs considered loading at the 50th-percentile level and ADT's as recorded (Figure 12); variables were endurance limit and type of corrosion aging. From these results, it became obvious that the most important factor is the range between the dead-load stress and the endurance limit of the steel. Also, the loss-of-section-vs-time relationship assumed, as seen from Runs 6 and 7, greatly affects the "duration" of the range. Small changes in the assumed endurance limit caused great changes in the calculated service life of the bridge member. Inasmuch as failure was predicted in all runs where corrosion was considered, it appears that some assumptions regarding the severely corroded members in the Central Bridge are too extreme.

Failure was predicted when the dead-load stress in the member reached a value near that of the endurance limit; thereafter, all vehicles crossing the bridge became damaging loads. However, fatigue damage is a function of dynamic (live-load) stress and static stress; and the Goodman diagram, Figure 5, tends to moderate the damage attributable to the live loads in similar situations. Inasmuch as wind and temperature stresses have not been considered in these analyses, the original condition of the steel is not known, and insamuch as the effects of aging on the steel are not known (at this time), the calculations are somewhat overly conservative in assessing fatigue damage.

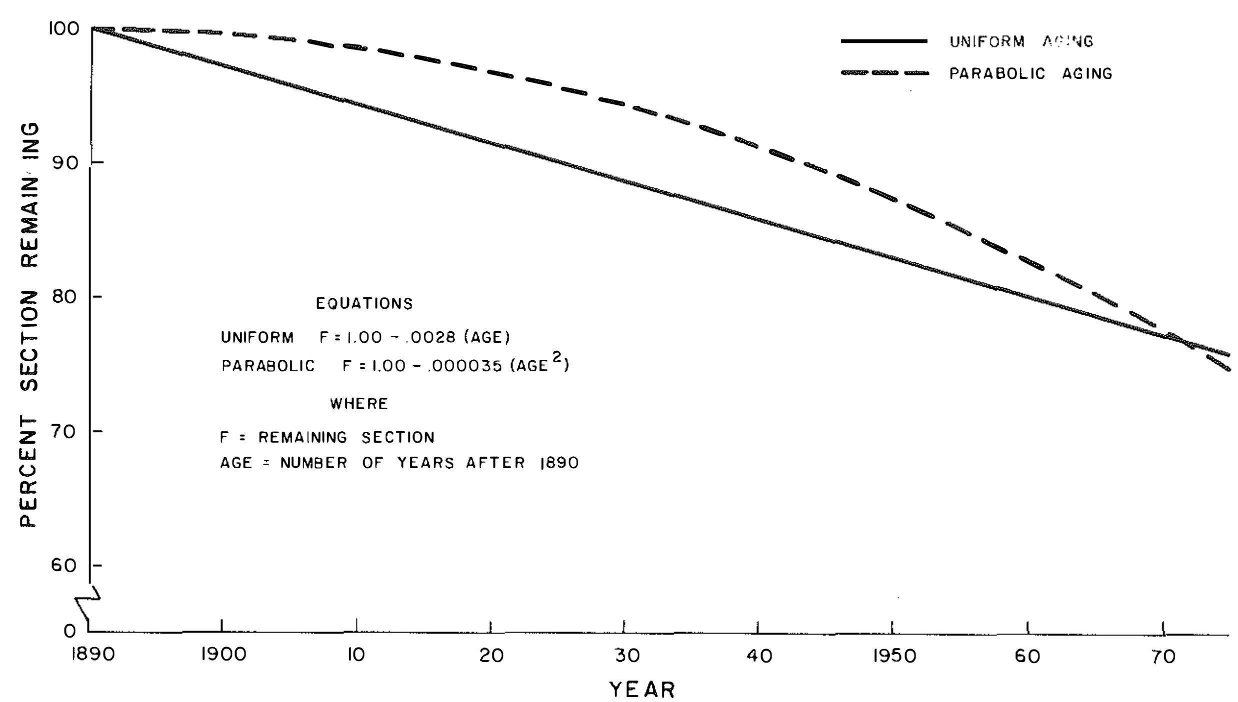


Figure 14. Percent Section Remaining vs Year for the Critical Member on Central Bridge.

TABLE III

LIFE ESTIMATES FROM PROBABILITY ANALYSIS

RUN NO.	PERCENT LIFE USED	AGE IN YEARS	CALENDER YEAR	ADT*	GROSS VEHICLE WEIGHT (PERCENTILE)**	PERCENT LOSS OF SECTION DUE TO CORROSION	ENDURANCE LIMIT (PSI)	DL STRESS AT CALENDER YEAR SHOWN (PSI)
1	0	81	1972	Figure 12	50th	0	16500	
2	0	81	1972	Figure 12	90th	0	16500	
3	5	81	1972	Figure 12 (Doubled)	100th***	0	16500	
4	100+	25	1916	Figure 12	50th	23, Linear	15000	14780
5	100+	45	1936	Figure 12	50th	23, Linear	16000	16060
6	100+	55	1946	Figure 12	50th	23, Linear	17000	16820
7	100+	66	1957	Figure 12	50th	23, Parabolic	17000	16800
8	100+	68	1959	Figure 12	50th	23, Linear	18000	17580

*See Figure 12; From Reference 4
**See Figure 11
***Maximum Recorded Loading (from 1968 weighings, Reference 4)

REFERENCES

1. Hughes, R. D., *Stress Histories of Bridge Members from Scratch Gage Records,* Kentucky Department of Highways, February 1972.

2. *Kentucky Obtains Bridge Load Histories with Newly Developed Scratch Gage Device,* Highway Research News, Highway Research Board, No. 36, Summer 1969.

3. Havens, J. H., and Deen, R. C., *Bridges: Synthesis of Load Histories and Analysis of Fatigue,* Kentucky Department of Highways, January 1972.

4. Lynch, R. L., *Analysis of Traffic Loads on Bridges,* Kentucky Department of Highways, February 1968.

5. Lynch, R. L., *Analysis of Traffic Loads on Bridges; Report II: Characteristics of Traffic on Ohio River Bridges - 1968,* Kentucky Department of Highways, March 1969.

APPENDIX

PROBABILITY ANALYSIS COMPUTER PROGRAM

PROGRAM INPUT
(FORMATS IN PARENTHESIS)

Card 1

Columns	1-4	Length of bridge in feet (F4.0)
	5-10	"A" in gap probability equation for mixed traffic (F6.4)
	11-16	"A" in gap probability equation for cars (F6.4)
	17-22	"A" in gap probability equation for trucks (F6.4)
	23-28	"A" in gap probability equation for combination trucks (F6.4)
	29-34	"B" in gap probability equation for mixed traffic (F6.4)
	35-40	"B" in gap probability equation for cars (F6.4)
	41-46	"B" in gap probability equation for trucks (F6.4)
	47-52	"B" in gap probability equation for combination trucks (F6.4)

The gap probability equation is of the form:

$$P_G = A(G^B)$$

where G = gap length in feet,
P_G = probability of gap less than or equal to G, and
A, B = constants (see Figure 13).

Card 2

Columns	1-5	Percent cars expressed as a decimal fraction (F5.4)
	6-10	Percent trucks expressed as a decimal fraction (F5.4)
	11-15	Percent combination trucks expressed as a decimal fraction (F5.4)
	16-18	Length of average car in feet (F3.0)
	19-21	Length of average truck in feet (F3.0)
	22-24	Length of average combination truck in feet (F3.0)

Card 3

Columns	1-6	Weight of car in pounds (F6.0)
	7-12	Weight of truck in pounds (F6.0)
	13-18	Weight of combination truck in pounds (F6.0)

Card 4

Columns	1-3	Width of bridge in feet (F3.0)
	4-9	Live-load stress in psi which occurs at design loading (F6.0)
	10-13	Design load in psf (F4.0)
	14-19	Dead-load stress in psi (F6.0)

Card 5

Columns	1-9	Ultimate strength of material in psi (F9.0)
	10-18	Endurance limit of material in psi (F9.0)
	19-27	Number of events which will cause failure at endurance limit (F9.0)
	28-30	Loss of section due to corrosion expressed as a decimal fraction (F3.2)
	31-32	Factor which determines type of corrosion aging used. Use -1 for parabolic aging and 0 for linear aging (I2)

Card 6

Columns	1-3	Number (N) of different ADT's used (I3)
	4-7	Average spot speed in mph (F4.1)

Cards 7, 8, ...N

Columns	1-7	ADT (F7.0)
	8-11	Number of years at constant ADT (F4.0)

PROGRAM DESCRIPTION

Caution: Run time for this program can become very long; runs should be made only after thorough study of program and input data.

Initialization Section -- In this section, all arrays except AGL(I) (average gross load) are initialized to zero. AGL(I) is initialized from AGL(2) = 14,500 psi by increments of 1000 psi up to AGL(25) = 37,500 psi. AGL(1) is set equal to 10,000,000 psi so that asterisks will be printed in the printout; these asterisks are subsequently defined as being stresses less than 14,000 psi.

Format Section I -- In this section, most of the input data are read and then printed in the printout section. The type of corrosion being considered is printed and headings for a subsequent table are prepared. See "Input Data Symbols" at beginning of program for list of variables.

Computation Section I -- In this section, probabilities of occurrence of different vehicle configurations where only one vehicle type is considered are computed. MN1 is the maximum number of autos (vehicle type 1) of length VL1 which can be placed on a span of length L.

The probability array is a four dimensional array as follows:

P(Number of Autos + 1, Number of Trucks + 1, Number of Combination Trucks + 1, Lane Number)

For example, P(3, 4, 1, 2) = b would be interpreted as follows:

b = probability of indicated load occurring.

Indicated load in lane 2 = 2 autos, 3 trucks and no combination trucks.

The reason for adding 1 to the number of vehicles was that 0's cannot be used as array subscripts. Since I is the number of vehicles being considered ITOT = I + 1 was used for array subscripting and IR was defined as real for use in computations. G1, G2 and G3 are gap lengths for each vehicle type -- autos, trucks, and combination trucks, respectively. These are used in decision statements and stop computations when the gap is less than or equal to zero.

Computations occur as follows:

1) Gap probability is found from PG1 = $AC(G1^{BC})/100$ where AC and BC are constants derived from traffic data.
2) If the value of PG1 is greater than one, it is set equal to one.
3) Probability of load occurrence is calculated.
4) If the probability of occurrence is less than or equal to 1.0×10^{-15}, it is set equal to zero.

This procedure is then repeated for Type 2 and Type 3 vehicles.

Computation Section II -- In this section, the balance of the probability calculations is made (i.e., probabilities involving two or more vehicle types). For this purpose, MN2 and MN3 are calculated and used along with MN1 to determine the maximum number of times each loop is run. The next computation is MN1 = MN1 + 1, MN2 = MN2 + 1, and MN3 = MN3 + 1. This is done since in this section the loop counter is used as the subscript for the probability array thus requiring it to be carried to 1 plus the maximum number of vehicles possible. I, J, K, IR, JR and KR are used for computational purposes, I, J, and K being the number of vehicles under consideration for Types 1, 2 and 3 vehicles, respectively, and IR, JR, and KR being defined as real values of I, J, and K.

The first decision statement causes the program to skip all further computations in this section if the calculations have been made previously in Computation Section I. The second decision determines which gap equation should be used according to how many vehicle types are being considered (two vehicle types being considered, Statement 102 is used; three vehicle types being considered, Statement 99 is used).

Next, the gap probability (PGM) is calculated if the gap is greater than zero. If this gap probability is greater than 1.0, it is reduced to 1.0. The probability of occurrence is then calculated in Statement 107, and as before, this value is set equal to zero when it drops below 1.0×10^{-15}. FAC(I,J,K) is a function written into this program and is not a previously stored function.

Final Data Input -- In this section, average spot speed (SP), average daily traffic (ADT(I)), number of years at constant ADT (YEARSN(I)), and number of different ADT's (NYRS) are read into the program. For example, traffic data would be stored in the program as follows:

YEAR	AVERAGE ADT
1955	15,000
1956	15,000
1957	15,000
1958	16,000
1959	17,000
1960	18,000

Average spot speed = 29.5 mph

for data cards as follows:

NYRS = 4	SP = 29.5
ADT(1) = 15,000	YEARSN(1) = 3
ADT(2) = 16,000	YEARSN(2) = 1
ADT(3) = 17,000	YEARSN(3) = 1
ADT(4) = 19,000	YEARSN(4) = 1

(Note: When corrosion is considered, accuracy can be gained by using the smallest possible values for YEARSN(I) -- i.e., yearly incrementation of ADT(I); however, this will not be possible under all circumstances due to the practical limits on run time.)

Computation Section III -- This section is composed of seven nested DO loops; therefore, any modifications requiring loops will have to be made in the form of subprograms. In this section, cross correlations are made to compute the probabilities of any possible loading in one lane occurring in conjunction with any other loading in other lanes. Stresses and the number of events at that stress are also calculated and these are placed in stress increments in subroutine INCREM. Finally, the percent damage (PERLIF) is calculated as percent life used.

Statements 126 and 130 are decision statements which prevent the program from making unnecessary, time-consuming calculations when Z (Statement 148) would equal zero. Statement 137 also causes the elimination of unnecessary calculations by skipping computations when no vehicles would be present on the span.

The time-dependency calculations are used to find the probability that loads in adjacent lanes would occur within a certain time period sufficiently small to be considered as one load. Statement 138 causes this time-dependency probability (PD) to be set equal to 1.0 when vehicles are considered to be on one lane only.

The computation of load and stress is accomplished as follows:

1) Total weight of vehicles (W) on bridge is computed.
2) W is then changed to LOAD in psf.
3) Corrosion factor (F) is then determined according to either linear or parabolic aging as specified in data input (Statements 151 through 157).
4) Dead-load stresses (DLSTR) and design live-load stress (DSTR) are corrected to take corrosion into account.
5) Live-load stress (LLSTR) is found as a proportion of the factored design live-load stress (FDSTR).
6) Total stress (TSTR) is then computed as the sum of the factored dead-load stress (FDSTR) and the live-load stress.

Computation of the EBL factor (FACTOR) and the life used (PERLIF) occurs as follows:

1) EBL factor is computed if average gross load AGL(IS) is greater than endurance limit. For AGL(1) and when AGL(IS) is less than endurance limit (ENDLIM), BLOAD and LODFAC(IS) (the number of events corresponding to AGL(IS)) are defined to be zero, thus not changing LODTOT (the total number of equivalent events).
2) When AGL(IS) is greater than ENDLIM, BLOAD is defined to be LODFAC(IS).
3) LODFAC(IS) is redefined to its new value, which includes the loading just considered.
4) LODTOT is redefined to its new value, which includes the loading just considered.
5) PERLIF is computed.
6) If PERLIF is greater than 100 percent, computation stops; if not, a new loading is considered.

Data Printout -- This section prints all the data which have been determined during the computation stages. All computations made in this section are to effect easier interpretation of data printed.

```
              C
              C
              C          FATIGUE ANALYSIS FROM PROBABILITY STUDY
              C
              C     THEORY BASED ON RESEARCH REPORT #318, BRIDGES  SYNTHESIS OF LOAD
              C     HISTORIES AND ANALYSIS OF FATIGUE
              C
              C
              C            INPUT DATA SYMBOLS
              C
              C     L=LENGTH OF BRIDGE
              C     P1,P2,P3=VEHICLE TYPE PROBABILITIES
              C     VL1,VL2,VL3=AVG. VEHICLE LENGTH BY TYPE
              C     VW1,VW2,VW3=AVG. VEHICLE WEIGHT BY TYPE
              C     WIDTH=BRIDGE WIDTH
              C     DSTR=DESIGN STRESS
              C     DLOAD=DESIGN LOADING IN PSF
              C     DLSTR=DEAD LOAD STRESS
              C     ULTST=ULTIMATE STRENGTH OF MATERIAL IN MEMBER
              C     ENDLIM=ENDURANCE LIMIT OF MATERIAL
              C     ENDNUM=NUMBER OF CYCLES TO FAILURE AT ENDURANCE LIMIT
              C     NYRS=NUMBER OF DIFFERENT ADT'S TO BE EVALUATED
              C     ADT(I)=AVG. DAILY TRAFFIC FOR YEAR GROUPING I
              C     YEARSN(I)=NUMBER OF YEARS COVERED IN YEAR GROUPING I
              C     COR=CORROSION DAMAGE (0.0 TO 1.0 )
              C     AM,BM AC,BC AT,BT ATC,BTC=CONSTANTS USED IN GAP PROBABILITY
              C          EQUATION OF FORM PG=AM*(GM**BM) WHERE PG=PROBABILITY OF
              C          GAP GM OCCURING
              C
              C     NFAC=-1 FOR PARABOLIC CORROSION AGEING
              C     NFAC= 0 FOR LINEAR CORROSION AGEING
              C
 0001               IMPLICIT REAL(A-H,L,P,R-Z), INTEGER(O,Q)
 0002               REAL P(35,28,15,2),ZT(25),AGL(25),IR,JR,KR,MR,NR,OR,QR
                   1,ADT(50),YEARSN(50),LODFAC(25)
              C
              C     INITIALIZATION SECTION  STATEMENT 3 THRU 26
              C
 0003               DO 1 I=1,35
 0004               DO 2 J=1,28
 0005               DO 3 K=1,15
 0006               DO 4 M=1,2
 0007               P(I,J,K,M)=0.
 0008             4 CONTINUE
 0009             3 CONTINUE
 0010             2 CONTINUE
 0011             1 CONTINUE
 0012               AGETOT=0.
 0013               LODTOT=0
 0014               DO 5 I=1,25
 0015               ZT(I)=0.
 0016               LODFAC(I)=0.
 0017             5 CONTINUE
 0018               DO 300 I=1,50
 0019               ADT(I)=0.
 0020               YEARSN(I)=0.
 0021           300 CONTINUE
 0022               AGL(1)=10000000.
```

```
              DO 120 I=1,24
              IR=I
              AGL(I+1)=14500.+(1000.*(IR-1))
          120 CONTINUE
        C
        C     FORMAT SECTION I   STATEMENT 27 THRU 53
        C
              READ(5,100)L,AM,AC,AT,ATC,BM,BC,BT,BTC
          100 FORMAT(F4.0,8(F6.4))
              READ(5,101)P1,P2,P3,VL1,VL2,VL3
          101 FORMAT(3(F5.4),3(F3.0))
              READ (5,201) VW1,VW2,VW3
          201 FORMAT(3F6.0)
              READ(5,202) WIDTH,DSTR,DLOAD,DLSTR
          202 FORMAT(F3.0,F6.0,F4.0,F6.0)
              READ(5,2001) ULTST,ENDLIM,ENDNUM,COR,NFAC
         2001 FORMAT(3(F9.0),F3.2,I2)
              WRITE(6,3011) VW1,VW2,VW3
         3011 FORMAT(2X,'VW1=',F6.0,6X,'VW2=',F6.0,6X,'VW3=',F6.0,/)
              WRITE(6,3012) DLOAD,DSTR,DLSTR
         3012 FORMAT(2X,'DLOAD=',F4.0,6X,'DSTR=',F6.0,5X,'DLSTR=',F6.0,/)
              WRITE(6,3013) L,COR
         3013 FORMAT(2X,'L=',F4.0,10X,'COR=',F3.2,/)
              IF(COR.LE.0.001) GO TO 6012
              IF(NFAC.EQ.-1) GO TO 6010
              WRITE(6,6011)
         6011 FORMAT(2X,'LINEAR CORROSION AGEING',///)
              GO TO 6012
         6010 CONTINUE
              WRITE(6,6013)
         6013 FORMAT(2X,'PARABOLIC CORROSION AGEING',///)
         6012 CONTINUE
              WRITE(6,8000)
         8000 FORMAT(2X,'LIFE USED',3X,'TOTAL STRESS',3X,'AGE',/)
        C
        C     COMPUTATION SECTION I   STATEMENT 54 THRU 82
        C
              MN1=L/VL1
        C
        C     INITIALIZE PROBABLITIES SO THAT WITH NO VEHICLES IN LANE
        C     PROBABLITY WILL EQUAL 100%
        C
              P(1,1,1,1)=1.0
              P(1,1,1,2)=1.0
        C
        C     COMPUTATION LOOP THRU STATEMENT 82
        C
              DO 1001 I=1,MN1
              ITUF=I+1
              IR=I
              G1=(L-(IR*VL1))/(IR-.5)
              G2=(L-(IR*VL2))/(IR-.5)
              G3=(L-(IR*VL3))/(IR-.5)
        C
        C     VEHICLE TYPE 1
        C
              IF(G1.LE.0) GO TO 8
              PG1=(AC*(G1**BC))/100
```

```
0065                IF(PG1.GE.1.) PG1=1.0
0066                P(ITOT,1,1,1)=((P1**I)*(PG1**(I-1)))
0067                IF(P(ITOT,1,1,1).LT.1.0E-15) P(ITOT.1.1,1)=0.
0068                P(ITOT,1,1.2)=P(ITOT,1,1,1)
          C
          C         VEHICLE TYPE 2
          C
0069              8 IF(G2.LE.0.) GO TO 11
0070                PG2=(AT*(G2**BT))/100
0071                IF(PG2.GE.1.) PG2=1.0
0072                P(1,ITOT,1,1)=((P2**I)*(PG2**(I-1)))
0073                IF(P(1.ITOT,1,1).LT.1.0E-15) P(1.ITOT,1,1)=0.
0074                P(1,ITOT,1.2)=P(1,ITOT,1,1)
          C
          C         VEHICLE TYPE 3
          C
0075             11 IF(G3.LE.0.) GO TO 10
0076                PG3=(ATC*(G3**BTC))/100
0077                IF(PG3.GE.1.) PG3=1.0
0078                P(1,1,ITOT,1)=((P3**I)*(PG3**(I-1)))
0079                IF(P(1,1,ITOT,1).LT.1.0E-15) P(1,1.ITOT.1)=D.
0080                P(1.1.ITOT.2)=P(1,1,ITOT,1)
0081             10 CONTINUE
0082           1001 CONTINUE
          C
          C         COMPUTATION SECTION II  STATEMENT 83 THRU 113
          C
0083                MN2=L/VL2
0084                MN3=L/VL3
0085                MN1=MN1+1
0086                MN2=MN2+1
0087                MN3=MN3+1
          C
          C         LOOP THRU STATEMENT 113
          C
0088                DO 1002 ITOT=1,MN1
0089                I=ITOT-1
          C
          C         LOOP THRU STATEMENT 112
          C
0090                DO 1003 JTOT=1,MN2
0091                J=JTOT-1
          C
          C         LOOP THRU STATEMENT 111
          C
0092                DO 1004 KTOT=1,MN3
0093                K=KTOT-1
0094                IR=I
0095                JR=J
0096                KR=K
          C
          C         DECISION STATEMENTS
          C
0097                IF((I.EQ.0.AND.J.EQ.0).OR.(J.EQ.0.AND.K.EQ.0).OR.(I.EQ.0.AND.
               1K.EQ.0)) GO TO 1005
0098                IF(I.EQ.0.OR.J.EQ.0.OR.K.EQ.0) GO TO 12
0099                GM=(L-(IR*VL1+JR*VL2+KR*VL3))/((IR+JR+KR)-1.5)
0100                GO TO 13
```

```
0101            12 CONTINUE
0102               GM=(L-(IR*VL1+JR*VL2+KR*VL3))/((IR+JR+KR)-1.0)
0103            13 CONTINUE
0104               IF(GM.LE.0.) GO TO 1005
0105               PGM=(AM*(GM**BM))/100
0106               IF(PGM.GE.1.) PGM=1.0
                C
                C     CALCULATION OF PROBABILITY OF OCCURENCE
                C
0107               P(ITOT,JTOT,KTOT,1)=(FAC(I,J,K)*(((P1*PGM)**I)*((P2*PGM)**J)*
                  1((P3*PGM)**K))*(IR+JR+KR))/((IR*PGM)+(JR*PGM)+(KR*PGM))
0108               IF(P(ITOT,JTOT,KTOT,1).LT.1.0E-15) P(ITOT,JTOT,KTOT,1)=0.
0109               P(ITOT,JTOT,KTOT,2)=P(ITOT,JTOT,KTOT,1)
0110          1005 CONTINUE
0111          1004 CONTINUE
0112          1003 CONTINUE
0113          1002 CONTINUE
                C
                C     FINAL DATA INPUT  STATEMENT 114 THRU 120
                C
0114               READ(5,102) NYRS,SP
0115           102 FORMAT(I3,F4.1)
0116               DO 301 I=1,NYRS
0117               READ(5,203) ADT(I),YEARSN(I)
0118           203 FORMAT(F7.0,F4.0)
0119               AGETOT=AGETOT+YEARSN(I)
0120           301 CONTINUE
                C
                C     COMPUTATION SECTION III  STATEMENT 121 THRU 202
                C
0121               AGE=0.
                C
                C     LOOP THRU STATEMENT 196
                C
0122               DO 1012 L=1,NYRS
                C
                C     LOOP THRU STATEMENT 191
                C
0123               DO 1006 I=1,MN1
                C
                C     LOOP THRU STATEMENT 190
                C
0124               DO 1007 J=1,MN2
                C
                C     LOOP THRU STATEMENT 189
                C
0125               DO 1008 K=1,MN3
0126               IF(P(I,J,K,1).LE.0.0) GO TO 4005
                C
                C     LOOP THRU STATEMENT 187
                C
0127               DO 1009 M=1,MN1
                C
                C     LOOP THRU STATEMENT 186
                C
0128               DO 1010 N=1,MN2
                C
                C     LOOP THRU STATEMENT 185
```

```
                C
0129                  DO 1011 O=1,MN3
0130                  IF(P(M,N,O,1).LE.0.0) GO TO 4006
0131                  IR=I
0132                  JR=J
0133                  KR=K
0134                  MR=M
0135                  NR=N
0136                  OR=O
0137                  IF(I.EQ.1.AND.J.EQ.1.AND.K.EQ.1.AND.M.EQ.1.AND.N.EQ.1.AND.O.EQ.1.
                     1) GO TO 852
                C
                C     TIME DEPENDENCY CALCULATIONS  STATEMENT 138 THRU 146
                C
0138                  IF((I.EQ.1.AND.J.EQ.1.AND.K.EQ.1).OR.(M.EQ.1.AND.N.EQ.1.AND.O.EQ.1
                     1)) GO TO 850
0139                  GO TO 851
0140              850 CONTINUE
0141                  PD=1.0
0142                  GO TO 853
0143              851 CONTINUE
0144                  PD=(ADT(O)*50.)/(255640.*SP)
0145                  IF(PD.GE.1.) PD=1.0
0146              853 CONTINUE
                C
                C     COMPUTATION OF TOTAL PROBABILITY OF OCCURENCE
                C
0147                  PS=PD*(P(I,J,K,1)*P(M,N,O,2))
                C
                C     COMPUTATION OF NUMBER OF EVENTS
                C
0148                  Z=365.*ADT(O)*PS*YEARSN(O)
                C
                C     COMPUTATION OF LOAD AND STRESS  STATEMENT 149 THRU 161
                C
0149                  W=((IR+MR-2)*VW1)+((JR+NR-2)*VW2)+((KR+OR-2)*VW3)
0150                  LOAD=W/(WIDTH*L)
0151                  IF(NFAC.EQ.-1) GO TO 6001
0152                  F=1.00-(((AGE+(YEARSN(O)/2.))/AGETOT)*COR)
0153                  GO TO 6002
0154             6001 CONTINUE
0155                  CORFAC=COR/(AGETOT**2)
0156                  F=1.00-(CORFAC*((AGE+(.6667*YEARSN(O)))**2))
0157             6002 CONTINUE
0158                  FDSTR=DSTR/F
0159                  FDLSTR=DLSTR/F
0160                  LLSTR=(LOAD*FDSTR)/DLOAD
0161                  TSTR=LLSTR+FDLSTR
                C
                C     INCREMENTING OF TOTAT STRESSES INTO STRESS INTERVALS
                C     AND SUMMATION OF EVENTS  STATEMENT 162 THRU 168
                C
0162                  IF(TSTR.LT.14000.) GO TO 125
0163                  CALL INCREM(TSTR,Z,ZT,IS)
0164                  GO TO 126
0165              125 CONTINUE
0166                  ZT(1)=ZT(1)+Z
0167                  IS=1
```

```
  126 CONTINUE
C
C     COMPUTATION OF EBL FACTOR AND LIFE USED  STATEMENT 169 THRU 182
C
      IF(AGL(IS).GT.1000000.) GO TO 3001
      IF(AGL(IS).LT.ENDLIM) GO TO 3001
      ROP1=(ALOG10(ENDNUM))*((AGL(IS)-ULTST)/(ULTST-ENDLIM))
      FACTOR=ENDNUM*(10.**ROP1)
C
C     COMPUTATION OF TOTAL NUMBER OF EQUIVILENT CYCLES
C
      BLOAD=LODFAC(IS)
      LODFAC(IS)=ZT(IS)*FACTOR
      GO TO 3002
 3001 CONTINUE
      BLOAD=0.
      LODFAC(IS)=0.
 3002 CONTINUE
      LODTOT=LODTOT+LODFAC(IS)-BLOAD
      PERLIF=(LODTOT*100.)/ENDNUM
      IF(PERLIF.GE.100.0) GO TO 4000
  852 CONTINUE
 4006 CONTINUE
 1011 CONTINUE
 1010 CONTINUE
 1009 CONTINUE
 4005 CONTINUE
 1008 CONTINUE
 1007 CONTINUE
 1006 CONTINUE
C
C     DATA PRINT OUT  STATEMENT 192 THRU 194
C
      AGEPR=AGE+YEARSN(Q)
      WRITE(6,4003) PERLIF,TSTR,AGEPR
 4003 FORMAT(5X,F5.1,8X,F6.0,6X,F3.0)
      AGE=AGE+YEARSN(Q)
 1012 CONTINUE
C
C     DATA PRINT OUT, STATEMENT 197 THRU 227
C
      AGEPR=AGE
      GO TO 6000
 4000 CONTINUE
      AGEPR=AGE+YEARSN(Q)
 6000 CONTINUE
      WRITE(6,5010)
 5010 FORMAT(//)
      IPR=I-1
      JPR=J-1
      KPR=K-1
      NPR=N-1
      MPR=M-1
      OPR=O-1
      WRITE(6,8001)
 8001 FORMAT(10X,'CARS',3X,'TRUCKS',3X,'COMBINATION')
      WRITE(6,8002)
 8002 FORMAT(28X,'TRUCKS',/)
```

```
           WRITE(6,8003) IPR,JPR,KPR
      8003 FORMAT(2X,'LANE 1',2X,I3,5X,I3,8X,I3)
           WRITE(6,8004) MPR,NPR,OPR
      8004 FORMAT(2X,'LANE 2',2X,I3,5X,I3,8X,I3,/)
           WRITE(6,4009) PERLIF,TSTR,AGEPR
      4009 FORMAT(2X,'LIFE USED=',F5.1,'%',4X,'TOTAL STRESS=',F6.0,5X,'AGE=
          1',F3.0,////)
           WRITE (6,132)
       132 FORMAT(4X,'EVENTS',9X,'STRESS',10X,'EBL',///)
           DO 130 I=1,25
           WRITE (6,131) ZT(I),AGL(I),LODFAC(I)
       131 FORMAT(2X,E9.3,8X,F6.0,8X,E9.3)
       130 CONTINUE
           WRITE(6,133)
       133 FORMAT('0',15X,'****** INDICATES NUMBER OF LOADINGS LESS THAN 1400
          10 PSI',///)
           CALL EXIT
           END
```

```
FORTRAN IV G1  RELEASE 1.1          MAIN          DATE = 73008       12/00/08          PAGE 0001

      C
      C
      C
      C       FUNCTIONAL SUBPROGRAM USED IN CALCULATION OF THE PROBABLITY
      C       OF OCCURENCE - STATEMENT 107 OF MAIN PROGRAM
      C
      C
              FUNCTION FAC(N1,N2,N3)
              IFN1=1
              IF(N1.EQ.0)  GO TO 11
              DO 10 I=1,N1
              J=N1-I+1
              IFN1=IFN1*J
           10 CONTINUE
           11 CONTINUE
              IFN2=1
              IF(N2.EQ.0)  GO TO 21
              DO 20 I=1,N2
              J=N2-I+1
              IFN2=IFN2*J
           20 CONTINUE
           21 CONTINUE
              IFN3=1
              IF(N3.EQ.0)  GO TO 31
              DO 30 I=1,N3
              J=N3-I+1
              IFN3=IFN3*J
           30 CONTINUE
           31 CONTINUE
              IFNCOM=1
              N4=N1+N2+N3
              DO 40 I=1,N4
              J=N4-I+1
              IFNCOM=IFNCOM*J
           40 CONTINUE
              IFTOT=IFN1*IFN2*IFN3
              FACTOP=IFNCOM
              FACBOT=IFTOT
              FAC=FACTOP/FACBOT
              RETURN
              END
```

```
FORTRAN IV G1  RELEASE 1.1            MAIN            DATE = 73008          12/00/08              PAGE 0001

                C
                C
                C
                C     SUBPROGRAM USED TO PLACE COMPUTED STRESSES INTO STRESS INTERVALS
                C     AND TO SUM THE NUMBER OF EVENTS CORRESPONDING TO THAT INTERVAL
                C     STATEMENT 163-MAIN PROGRAM
                C
                C
                C
                      SUBROUTINE INCREM(A,B,C,J)
                      REAL C(25),IR
                      DO 4 I=1,24
                      IR=I
                      IF(A.GE.(14000.+((IR-1)*1000.)).AND.A.LT.(15000.+((IR-1)*1000.)))
                     1GO TO 5
                      GO TO 6
                    5 CONTINUE
                      C(I+1)=C(I+1)+B
                      J=I+1
                      GO TO 7
                    6 CONTINUE
                    4 CONTINUE
                    7 CONTINUE
                      RETURN
                      END
```

EXAMPLE OF INPUT

```
254. .056  .056  .401  .00911.21  1.21   .60   .885
.914 .073 .013 20.25.47.
 4800.12000.31800.
23. 5950. 75.14260.
   60000.   17000. 2000000..23 0
 3028.2
  6000. 25.
  6000. 10.
  6000. 10.
  6000.  5.
  6400.  2.
  6400.  2.
  6400.  1.
  8000.  1.
  9500.  1.
 10700.  1.
 11900.  1.
 13200.  1.
 14600.  1.
 15800.  1.
 17100.  1.
 18400.  1.
 19700.  1.
 20700.  1.
 22000.  1.
 23200.  1.
 24000.  1.
 24500.  1.
 23000.  1.
 21700.  1.
 20000.  1.
 18600.  1.
 17200.  1.
 16200.  1.
 15900.  1.
 15800.  4.
```

EXAMPLE OF OUTPUT

```
VW1= 4800.          VW2=12000.          VW3=31800.

DLOAD= 75.          DSTR= 5950.         DLSTR=14260.

L=254.              COR=.23

LINEAR CORROSION AGEING


LIFE USED     TOTAL STRESS     AGE

      0.0         15866.        25.
      0.0         16728.        35.
      0.0         17264.        45.
     12.8         17689.        50.
     29.8         17894.        52.
     84.7         18014.        54.


            CARS     TRUCKS     COMBINATION
                                  TRUCKS

LANE 1       0         0           0
LANE 2       1         1           0

LIFE USED=104.4%     TOTAL STRESS=17141.       AGE=  55.


   EVENTS             STRESS             EBL


0.0                   ******          0.0
0.132E+09             14500.          0.0
0.646E+08             15500.          0.0
0.111E+09             16500.          0.0
0.177E+07             17500.          0.209E+07
0.774E+01             18500.          0.128E+02
0.148E-05             19500.          0.345E-05
0.492E-14             20500.          0.160E-13
0.633E-24             21500.          0.289E-23
0.0                   22500.          0.0
0.0                   23500.          0.0
0.0                   24500.          0.0
0.0                   25500.          0.0
0.0                   26500.          0.0
0.0                   27500.          0.0
0.0                   28500.          0.0
0.0                   29500.          0.0
0.0                   30500.          0.0
0.0                   31500.          0.0
0.0                   32500.          0.0
0.0                   33500.          0.0
0.0                   34500.          0.0
0.0                   35500.          0.0
0.0                   36500.          0.0
0.0                   37500.          0.0

                ****** INDICATES NUMBER OF LOADINGS LESS THAN 14000 PSI
```

APPENDIX

THE CANTILEVER HIGHWAY BRIDGE AT CINCINNATI

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

545.
(Vol. XXVII.—August, 1892.)

THE CANTILEVER HIGHWAY BRIDGE AT CINCINNATI.

By GUSTAVE KAUFMAN, M. Am. Soc. C. E., and
F. C. OSBORN, M. Am. Soc. C. E.

GENERAL DESCRIPTION OF THE WORK.

By GUSTAVE KAUFMAN, M. Am. Soc. C. E.

During the years 1890 and 1891 the Cantilever Highway Bridge described in this article was built across the Ohio River, between the cities of Cincinnati, O., and Newport, Ky. The terminus in Cincinnati is at the corner of 2d Street and Broadway, and in Newport it is at the corner of York and 3d Streets. The roadway of the bridge is 24 feet wide in the clear, with two sidewalks each 7 feet wide. The total length of the structure is 2 966 feet. The main engineering feature is the cantilever span, 520 feet from center to center of piers.

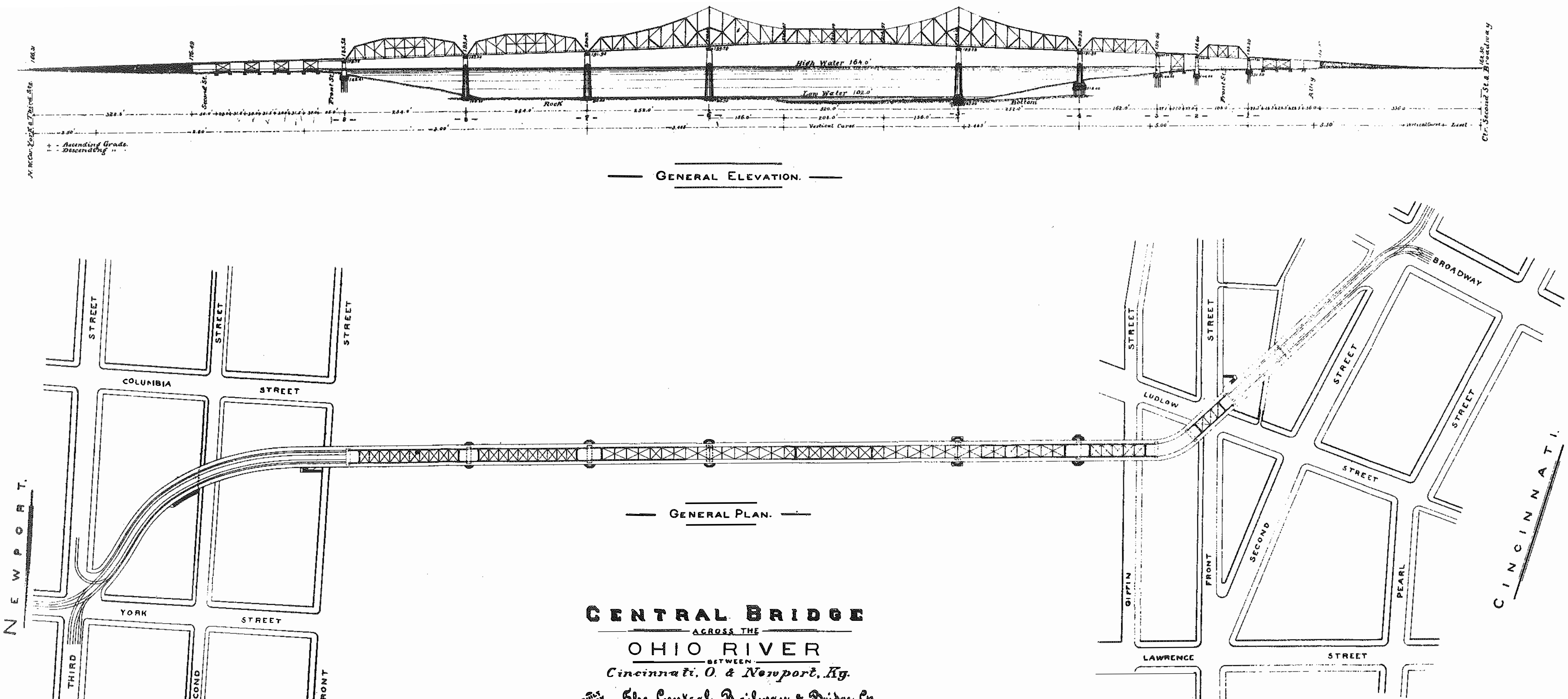
The bridge is located between the Louisville and Nashville Railroad Bridge and the Old Cincinnati Suspension Bridge, and but a short distance above the mouth of the Licking River. Prior to the construction of this bridge, the highway traffic between the two cities was principally accommodated by a ferry company with two large ferry boats. The Louisville and Nashville Bridge, which is supplied with very narrow roadways and sidewalks, accommodated the street-car traffic and a portion of the other highway traffic, but was not popular on account of its location and the interruption of highway traffic during the passage of trains.

The site of the new bridge is very favorable for economical construction, from the fact that a peculiar limestone formation extends across the river at this point. The top of this formation is an irregular triangle in shape, the base of which is on the Kentucky side and the apex on the Cincinnati side. On the Kentucky side at extreme low water this formation is exposed; the base of the triangle is about 1 400 feet long, and extends from the mouth of the Licking River to a point midway between the bridge under discussion and the Louisville and Nashville Bridge. The top of the formation maintains the level of extreme low water about two-thirds the distance across the river, where it drops suddenly, and for the balance of the distance across the river the top of the rock is from 5 to 7 feet below low water. The sites of other bridges built across the Ohio River at Cincinnati were by no means so favorable, and it was necessary to go to considerable depths to obtain suitable foundations for their piers; notably the Chesapeake and Ohio Railway Bridge, where rock was found about 52 feet below low water.

For many years the fear of insufficient revenue prevented the construction of a highway bridge at this place; but the development of the electric street railway and the necessity for rapid transit between the cities of Cincinnati and Newport furnished the requisite impetus and led to the formation of the Central Railway and Bridge Company for the purpose of constructing this bridge. The preliminary surveys and the general lay-out were prepared by G. Bouscaren, M. Am. Soc. C. E., in the early part of 1887. The general plans and the location of the channel span were approved by the Government in April, 1888, and authority at the same time was given for the construction of the bridge.

The general elevation and plan of the bridge; the gradients, lengths of spans and height above water are shown on Plate XXVII. A gen-

PLATE XXVII
TRANS. AM. SOC. CIV. ENGRS
VOL. XXVII. No 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



CENTRAL BRIDGE
ACROSS THE
OHIO RIVER
BETWEEN
Cincinnati, O. & Newport, Ky.
BUILT BY The Central Railway & Bridge Co.
1890-91.

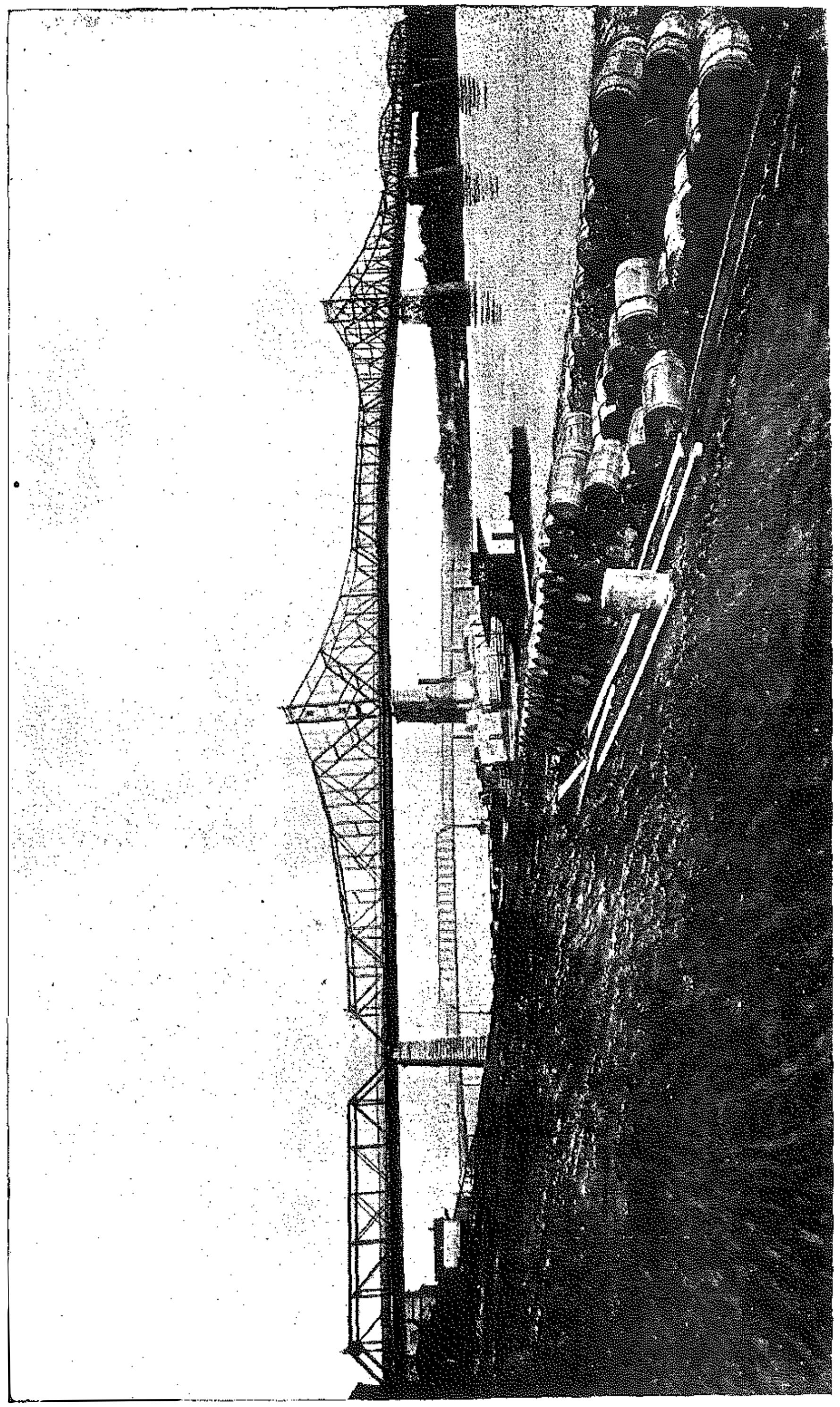
SCALE - 1 Inch = 200 Feet.

CHIEF ENGINEERS,
Ferris, Kaufman & Co.,
OF PITTSBURGH, PA.

RESIDENT ENGINEER,
A. A. Stuart.

Contractors for the entire structure.
The King Iron Bridge & Man'f'g Co.
OF CLEVELAND, O.

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 545.
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



eral view is shown in Plate XXVIII. The approaches were located entirely on private property occupied by a large number of buildings of all kinds and descriptions, which were owned and leased by almost an equal number of individuals. All the street and alley crossings are overhead. The maximum grade, which occurs on the Cincinnati end, is $5\frac{1}{2}$ per cent. This grade is necessary to enable the structure to give the clear height over the channel, required by the Act of Congress governing the construction of bridges across the Ohio River. Between the mouth of the Big Sandy River and the Suspension Bridge at Cincinnati, all bridges must have a channel span 500 feet wide in the clear at low water. The lowest part of the channel span must be 100 feet above low water-mark, and 40 feet above local highest water. The highest water known at Cincinnati prior to 1883, the time of the passage of the act, was in 1832, when the river reached the height of 64 feet on the Government gauge. In 1884 the river rose to the unprecedented height of 71 feet and three-quarters of an inch. Since that time the law has been construed by the Government authorities to refer to the flood of 1832 and in fixing the height of channel spans it is only necessary to have 40 feet clear above that flood. Therefore, in the case of the Central Bridge, it was required to have a clear height of 102 feet above low water-mark, which is 2 feet on the gauge.

No active work of construction was done on the bridge until March, 1890, when, through the perseverance of Mr. V. Morris, the Southwestern Agent of the King Bridge Company, arrangements for its construction were perfected with the Company which he represented. This company was to construct the bridge complete in all details by January 1st, 1891, under the specifications to be prepared by Ferris, Kaufman & Co., who were at this time appointed Chief Engineers of the Central Railway and Bridge Company. On account of the limited time, but twenty-four hours were taken in preparing the specifications, copies of which will be found in the appendices.* The general plans adopted by the Government were of course adhered to, and many items in the original specifications of the company were adopted. On the 31st day of March, 1890, the formal contract was executed. By its terms the King Bridge Company was bound, under heavy forfeiture, to complete the structure satisfactorily to the engineers by the time mentioned.

* See pages 194 to 220 inclusive.

The preliminary estimate of quantities was as follows:

SUBSTRUCTURE:

First-class masonry, Piers 4, 5, 6, 7 and 8	10 420	cubic yards.
" " " " 1, 2, 3 and 9	1 353	"
" " " Pedestals	140	"
Second " " Abutments and Ramps	3 200	"
Concrete	2 100	"
Piles in foundations	20 400	linear feet.
Timber "	152 000	ft. B. M.
Iron	15 000	pounds.
Excavation in coffer-dams	1 750	cubic yards.
" foundations on shore	2 750	"
Filling between ramp walls	3 000	"
Granite paving	2 500	sq. yds.

SUPERSTRUCTURE:

Structural iron and steel	2 500	tons.
Lumber for floor	550 000	ft. B. M.
Hand rail	6 000	linear feet.
Toll houses, gas pipe, etc.		

The right of way on the Newport side was practically all secured, but nothing had been done in the way of clearing it at the time of the closing of the contract, and no property had been obtained on the Cincinnati side. The undertaking of the King Bridge Company to complete this work in nine months was large; but, at the same time, it seemed that the contract could be successfully carried out if proper energy were used by all, and if no extraordinarily unfavorable circumstances should arise. The company, in April, sub-let the contract for the substructure and paving of approaches to Mr. J. Le Duke, of Berea, Ohio. In accordance with this contract the various parts were to be completed, as follows:

Piers Nos. 1, 2, 3, 4, 6, 7, 8, 9, pedestals and approach masonry not later than October 1st, 1890; Pier No. 5 not later than October 31st, 1890, and all the earth filling, granite paving, flagging and the entire contract not later than December 1st, 1890. The work was to be commenced not later than May 1st, 1890, and the contractor was placed under heavy bonds for the completion of the work as stated. The ordinary clause giving the contractor more time for the completion of his work, on account of delays through causes beyond his control. was omitted in this contract, and the contractor was given to understand clearly that he was to take all the chances and finish the work in time to enable the erection of the bridge to be completed by January 1st, 1891.

The contract for the iron and steel was given to Messrs. Carnegie, Phipps & Co.; the material was to be delivered to the King Bridge Company at Cleveland within sixty days from the time the order was placed, and this would enable the latter Company to deliver the finished material at the bridge site as agreed upon.

The contract for the erection of the superstructure and the laying of the floor, etc., was awarded to Messrs. Baird Bros., of Pittsburgh, Pa., who were to be given possession of the piers and material at the time shown in Le Duke's contract, and were to complete the work by January 1st, 1891.

For a short time the work in all dopartmonts progressed very satisfactorily and according to programme; but soon delays from various causes arose, until finally, as it became evident that the bridge could not be completed as contracted for, the masonry contractors became demoralized; and it required great patience and energy to maintain the prosecution of the work.

The Ohio River has for some miles above and below the bridge site a narrow and tortuous channel; and as the mouth of the Licking River is almost directly opposite the site, it is subject to very rapid and wide fluctuations. The variation between extreme high water and extreme low water is about 69 feet. These conditions conspired to render the foundation work hazardous and expensive. The year 1890 is now noted for the number of floods which occurred in the Ohio River. A hydrograph showing the stage of the water, the condition of the weather and the temperature for each day during the construction of the work, will be found on Plate XXIX. This will indicate one of the great difficulties under which this work was carried on. Usually during the summer and fall months the water stage at Cincinnati is below 15 feet, with an occasional rise above that point, but during 1890 the reverse was the case. There were about five wooks of low water. During this period Piers 6 and 7 were fairly started, but Pier 5 was caught by the high water. The delay in completing this pier prevented the opening of the bridge until August, 1891, some eight months behind time.

The year 1890, as is well known, excelled all other years in the product of steel and iron, and great difficulty was encountered in getting the structural material to the bridge shops. The mills failed in a great measure to complete their contract in this case, and the time for the delivery had elapsed by several months before the King Bridge Company obtained all the material they had ordered. The delay from this cause was not particularly noticeable on account of the fact that the masonry was not finished.

Surveys.—Surveys and locations for the construction work, and, in fact, all the substructure work, were made under the direction of Mr. A. A. Stuart,* M. Am. Soc. C. E., who was Resident Engineer, and Mr. L. V. Rice, Assistant Engineer, to both of whom much credit is due for the accuracy and excellency of the work. The general situation was favorable for accurate triangulation work, and to this is largely attributable the excellent results obtained.

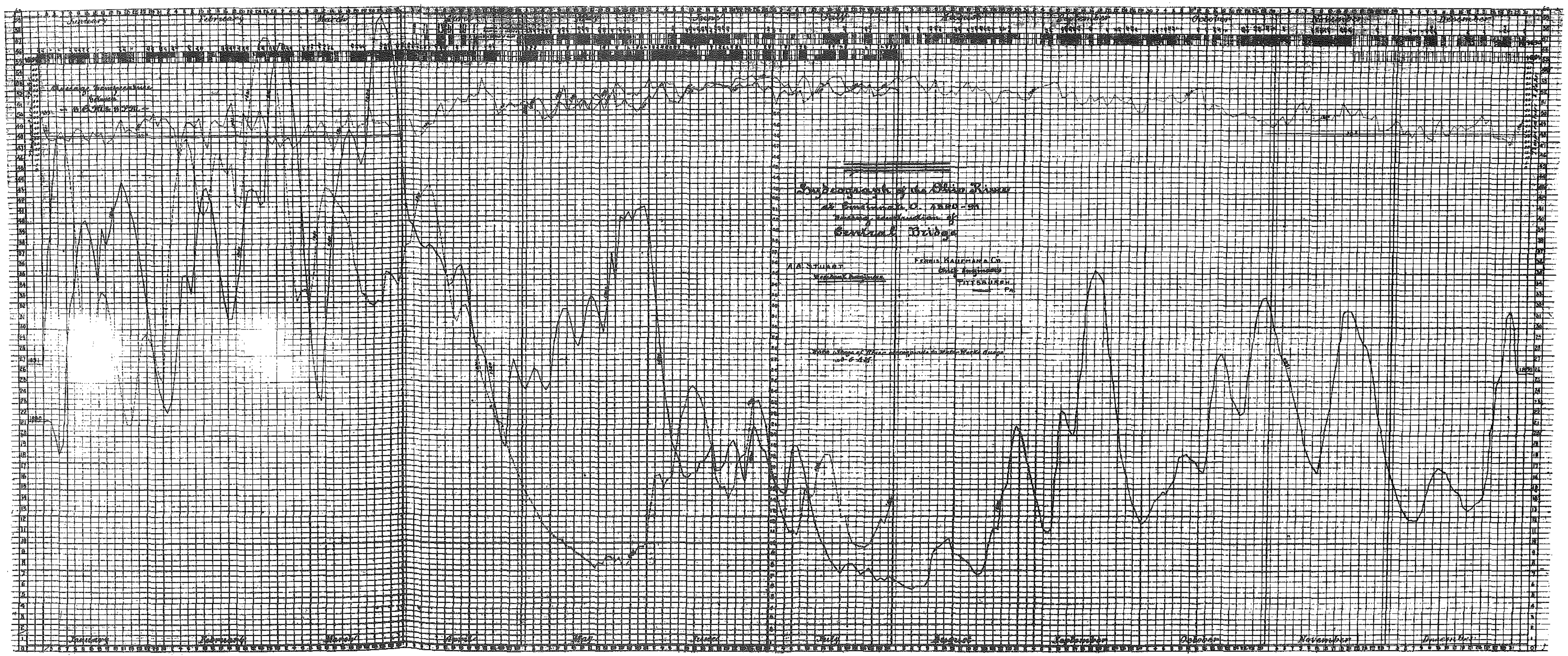
A test of the accuracy of the field operations was made by computing the distance between the base line on the Cincinnati side and a point on the bridge axis common to the two base lines on the Newport side, using the three triangles formed on the three base lines, and the results were respectively as follows: 1 696.076 feet; 1 696.074 feet; 1 696.053 feet. With the assurance of accuracy of field work which these results gave, the remaining elements of the triangles were computed ready for use in locating the river piers.

Masonry.—The length of the structure between the termini is 2 966 feet. Beginning in Cincinnati, this distance is made up as follows:

Granite paving and masonry ramp..........	285	feet.
Steel viaduct..............................	151	"
One truss span across Ludlow street	108	"
Viaduct..................................	81	"
One truss span............................	162	"
Cincinnati cantilever arm..................	252	"
Two river arms and suspended span	520	"
Newport cantilever arm....................	252	"
Two truss spans, each 254 feet..............	508	"
Steel viaduct..............................	319	"
Granite paving and masonry ramp...........	328	"
Total	2 966	"

* The reports of Mr. Stuart were frequently drawn upon in preparing this paper.

PLATE XXIX
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXVII. Nº 545
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



The superstructure is supported by two abutments, twenty-eight pedestals and nine piers. The abutments and ramp walls are built of second-class masonry and entirely of Ohio River freestone, except the coping, which is of Berea sandstone. All the pedestal piers are built of first-class masonry. All the cement used on the work, with the exception of a few barrels of Portland for main coping stone and pointing, was Louisville cement, and was tested at the mills by Messrs. Mead & Shaw, Cement Inspectors, of Louisville, and all accepted barrels which were shipped were branded by them. The ramp walls, abutments and pedestals being far removed from the water, are founded on the natural earth upon a course of concrete 2 feet thick, the material affording ample resistance to bear the superimposed loads without the aid of piling.

Piers Nos. 1, 2, 3 and 9 are similar in all respects, except as to size and height, and are all founded on piles driven to a firm resistance from short blows of a hammer weighing 4 000 pounds. The foundation beds were from 7 to 10 feet deep, and after sawing the piles off 18 inches above the bottom of the pits, concrete was put in, varying in thickness from 3 to 4½ feet, thoroughly imbedding the piles in a plastic mass upon which the foundations and footing courses were started. These piers were built entirely of Ohio River freestone, except the coping which was Bedford oolitic limestone. They are rectangular in plan throughout their height, battering one-half inch to the foot, and as they stand above the average high water, no difficulties were encountered in constructing their foundations.

Piers Nos. 4 and 8 are similar in construction, but different in kind of foundations and dimensions. Pier No. 4 rests upon one hundred and fifty piles, driven to solid rock, having heavy cast-iron shoes, the points of which were seated in the rock by repeated light blows from the hammer. They were cut off 18 inches above the bottom of the foundation bed, and their heads were imbedded in concrete 3 feet 6 inches thick. Upon this the foundation footing courses, four in number, were laid.

Pier No. 8 is located at about extreme low water-line on the Kentucky shore, and rests upon solid rock. This foundation was begun July 14th, 1890, and the laying of masonry was begun July 27th, the stage of the river being about 7 feet. A clay dam was built around the site of the pier, and with this protection from the water the excavation was made. A pit 18 inches deep was excavated in the rock, and in this the footing courses were started.

These two piers have semicircular nosings up to the belt course, where they are contracted in length and become rectangular in plan. From the foundation to the belting course the face work is built of Berea sandstone, with concrete backing. The face stones were laid in Flemish bond, headers and stretchers alternating with each other in every course. The concrete backing was put in as fast as the face stones of each course were laid, and was allowed twelve hours to set before any masonry was laid upon it. Above the belting course these piers are built entirely of Ohio River freestone, except the coping, which is of Bedford limestone. The table on next page gives the complete record of the building of Pier No. 4, and it is believed by the writer that this is the first instance that such a record has been kept.

Each face stone and each backing stone was measured and its contents calculated. The sum of the two was deducted from the contents of the full course and the balance was taken as the amount of mortar in that course. In this way the column giving the cubic yards of mortar in all joints was obtained. The number of barrels of cement used for the face work and backing was obtained by actual count. The column giving the number of barrels of cement per cubic yard of backing, gives also the amount of cement in a yard of concrete, as the backing is concrete. The cost of laying up to the starling course refers only to the face stones.

Piers Nos. 5, 6 and 7 are similar in construction and are located in the river. Piers 6 and 7 are founded on the solid rock, and their foundations were put in without difficulty by the use of single wall coffer-dams. The solid rock bed in the river at this point has no deposit upon it, and in landing the coffer-dams it was necessary first to sink a crib composed of timbers and stone above the pier site in order to hold the coffer-dam in place. The coffer-dams for Piers 5, 6 and 7 were all alike in construction, being rectangular in plan and 30 x 70 feet in size out to out. The walls were built of horizontal courses of 12 x 12-inch timber bolted together for a height of 6 feet, and above this the walls were composed of 6 x 4-inch stuff. At intervals of 8 feet along the longitudinal walls 12 x 12-inch vertical timbers were bolted, and into each pair of verticals 12 x 12-inch horizontal struts were dovetailed.

CENTRAL BRIDGE, CINCINNATI–NEWPORT.—BUILT 1890–91.—DATA FROM PIER NO. 4.

Number of course.	Mean size of course out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in all joints.	Face work. Percentage of whole.	Barrels cement per yard of face work.	Barrels cement per yard of backing.	Cost per yard for laying, inc. cem. and sand.	REMARKS.
	Feet.					Percent				
1	23.18 x 59.18	1' — 7"	30.90	46.05	1.45	40	0.26	1.70	$1.90	Footings, Polygonal ends—concrete backing.
2	21.35 x 57.35	1' — 7"	29.35	29.67	1.05	43	0.37	1.64	1.90	
3	19.44 x 55.18	2' — 8"	53.34	47.37	1.30	53	0.32	1.52	1.90	
4	16.44 x 52.18	2' — 4"	32.35	38.95	0.93	45	0.37	1.79	1.90	
5	13.69 x 49.44	2' — 2"	28.40	22.70	0.88	58	0.35	1.68	1.65	
6	13.50 x 49.25	"	26.70	23.89	0.79	53	0.30	1.65	"	
7	13.31 x 49.06	"	28.86	20.79	0.87	58	0.21	1.80	"	
8	13.12 x 48.87	"	28.31	20.48	0.87	58	0.25	2.00	"	Concrete backing.
9	12.93 x 48.68	"	28.74	19.25	0.87	60	0.24	1.71	"	
10	12.74 x 48.49	"	27.85	19.34	0.86	59	0.25	1.94	"	
11	12.55 x 48.30	"	26.48	19.90	0.83	57	0.30	1.71	"	
12	12.36 x 48.11	2' — 1"	26.42	17.66	0.84	60	0.26	1.81	"	
13	12.18 x 47.93	2' — 0"	23.99	17.43	0.80	58	0.27	1.71	"	
14	12.00 x 47.75	"	25.11	15.69	0.82	62	0.24	1.60	"	Semicircular ends.
15	11.81 x 47.56	"	24.78	15.29	0.81	62	0.24	1.77	"	
16	11.62 x 47.37	2' — 1"	25.74	15.17	0.83	63	0.31	1.71	"	
17	11.43 x 47.18	"	24.84	15.25	0.82	62	0.36	1.67	"	
18	11.24 x 46.98	"	25.50	14.05	0.75	64	0.33	1.74	"	
19	11.05 x 46.80	"	24.71	14.02	0.80	64	0.42	1.64	"	
20	10.86 x 46.61	"	24.61	13.47	0.80	65	0.45	1.74	"	
21	10.66 x 46.42	2' — 0"	24.07	11.01	0.80	69	0.29	1.63	1.65	
22	11.76 x 47.50	1' — 10"	23.11	12.71	6.20	65	0.42	Included with face work.	1.96	Starling.
23	10.70 x 46.44	1' — 5"	15.09	10.49	4.99	59	0.47		"	
24	10.58 x 35.66	"	15.17	8.46	4.56	64	0.53		"	Hood courses.
25	10.46 x 35.52	1' — 6"	10.02	12.13	8.40	45	0.53		"	
26	10.34 x 35.37	1' — 6"	12.25	7.60	3.50	62	0.53		"	Rectangular in plan.
27	10.22 x 35.23	1' — 5"	11.81	7.54	3.64	61	0.55		"	
28	10.10 x 35.08	1' — 4"	11.39	6.60	3.23	63	0.56		"	
29	10.00 x 34.96	1' — 4"	11.68	6.08	2.83	66	0.68		"	
30	9.87 x 34.84	1' — 5"	12.53	5.93	2.03	67	0.64		"	Courses 22 to 38 inclusive have stone backing.
31	9.75 x 34.72	1' — 3"	10.16	5.92	2.52	63	0.55		"	
32	9.62 x 34.61	1' — 4"	10.23	6.62	3.77	61	0.56		"	
33	9.48 x 34.49	1' — 5"	11.07	6.62	2.98	63	0.56		"	
34	9.35 x 34.37	1' — 6"	11.36	7.00	4.30	62	0.54		"	
35	9.25 x 34.25	1' — 3"	9.21	5.92	2.77	61	0.72		"	
36	9.15 x 34.15	1' — 3"	9.18	5.75	3.00	62	0.60		"	
37	9.03 x 34.03	1' — 3"	8.88	5.84	2.95	60	0.47		1.96	
38	10.00 x 35.00	1' — 6"	13.80	6.16	2.99	69	0.50		2.06	Sub-coping.
39	11.00 x 36.00	1' — 6"	21.82	......	0.72	100	0.50		2.06	Coping.

Amount of cement per yard of face work, courses 1 to 21 inclusive = 0.30 barrels.
" " " " " " concrete " " " " " = 1.71 "
" " " " " " masonry " 22 " 39 " = 0.56 "
Percentage of face work in " 1 " 21 " = 55 per cent.
" " " " " " 22 " 39 " = 62 "

NOTE.—From the foundation to the starling course, the masonry is built of Berea sandstone with concrete backing, but above this it is built entirely of Ohio River freestone. All face stones were required to have a width of 1½ times their thickness, and below the starling they were laid with Flemish bond. All mortar was mixed 1 cement to 2 of sand, and the concrete was mixed 1, 2 and 4, the latter being broken stone. Louisville cement was used throughout.

The bottom edges of the walls were padded with cotton waste 6 inches thick, held in place by cotton ducking. The coffer-dams were towed into place without bottoms and sunk by loading them with stone intended for use in the piers. On July 10th, the crib for Pier No. 7 was located when the water was about 14 feet high and the current very swift. On the 11th of July the coffer-dam was towed into place and sunk, the stage of water being about 10 feet. After sinking it was discovered to be uneven, the southeast corner being about 1 foot higher than the others, a 12 x 12-inch stick of timber having lodged in this corner. This was removed by a diver. Considerable difficulty was experienced in pumping out this coffer-dam, the cotton ducking having been torn out in a number of places in launching, causing leaks which were finally stopped by throwing in bags of sand. This work consumed considerable time, but by July 22d a bed 2.7 feet deep was excavated in the rock and the first course of masonry begun.

On July 20th the coffer-dam of Pier No. 6 was located after considerable trouble on account of a very swift current. The stage of the water was at this time 8 feet, and a week was consumed in stopping the leaks around the bottom of this coffer-dam. After excavating a bed 3.8 feet deep in the rock, masonry was started August 1st.

On August 10th the coffer-dam for Pier No. 5 was located and sunk in position, but as the rock bottom at this point was overlaid with about 2 feet of river silt, much time was taken in an effort to make the bottom edge of the coffer-dam tight. Before this was accomplished the river began to rise and all operations were suspended on August 26th. From this time the water stage fluctuated between 11 and 20 feet, until on September 18th it reached a 35-foot stage, which was never known to occur before in the month of September. The condition of the work at this time, starting from the Cincinnati end, was as follows: The right of way from Broadway to Giffin Street had not been entirely obtained, many suits of condemnation having made the work of securing it extremely tedious, but it had been so far obtained as to allow the foundation of Pier No. 1 to be put in; also Piers Nos. 2 and 3. On Pier No. 4 nothing had yet been accomplished, owing to litigation as to the right of the company to condemn property at this point. Pier No. 5, as stated above, had not yet been started, but a coffer-dam had been sunk. Pier No. 6 had 33 feet of masonry yet to be laid to complete it. Pier No. 7 had yet 30 feet of masonry to be laid on it. To complete Pier No. 8, 36 feet of masonry were required. Pier No. 9 was completed. The pedestals on the Newport side, with the exception of a few cap stones; the abutment on the Newport side, and most of the ramp walls were completed. No filling between the same, however, had yet been done. Out of a total of 13 000 cubic yards of masonry, 7 000 yards had been laid.

During the whole season to this date, there had been but five weeks in which the stage of the water was below 10 feet, and only for one day was the water below a 6-foot stage, and during this time there were a number of very rainy days. The masonry contractors were becoming very much demoralized, and they realized that it was impossible for them to complete their work by the specified time. Considerable iron had been delivered on the ground and the contractors for the erection were on hand ready to proceed with their work.

As winter was rapidly approaching it became obvious that if the work was to be finished approximately on time, it was absolutely necessary that the river work be completed first. The condition of Pier No. 5 and the condition of the river made it clear that some radical move had to be made. It was therefore determined on September 15th to use the pneumatic process in founding Pier No. 5, notwithstanding the fact that bed-rock was only about 7 feet below low water. Plans and specifications for a caisson were made as rapidly as possible.

The plan of the caisson is shown on Plate XXX and the specifications in Appendix 3. It was 12 feet high from the shoe to top of the deck, with a coffer-dam about 24 feet high, so that the work could be prosecuted in a 24 to 26-foot stage of water after the caisson was landed on the rock bottom of the river. An examination of the records gave sufficient reasons to expect that the work could be thus carried on without interruption during the early winter months and that the pier could be finished before very cold weather set in.

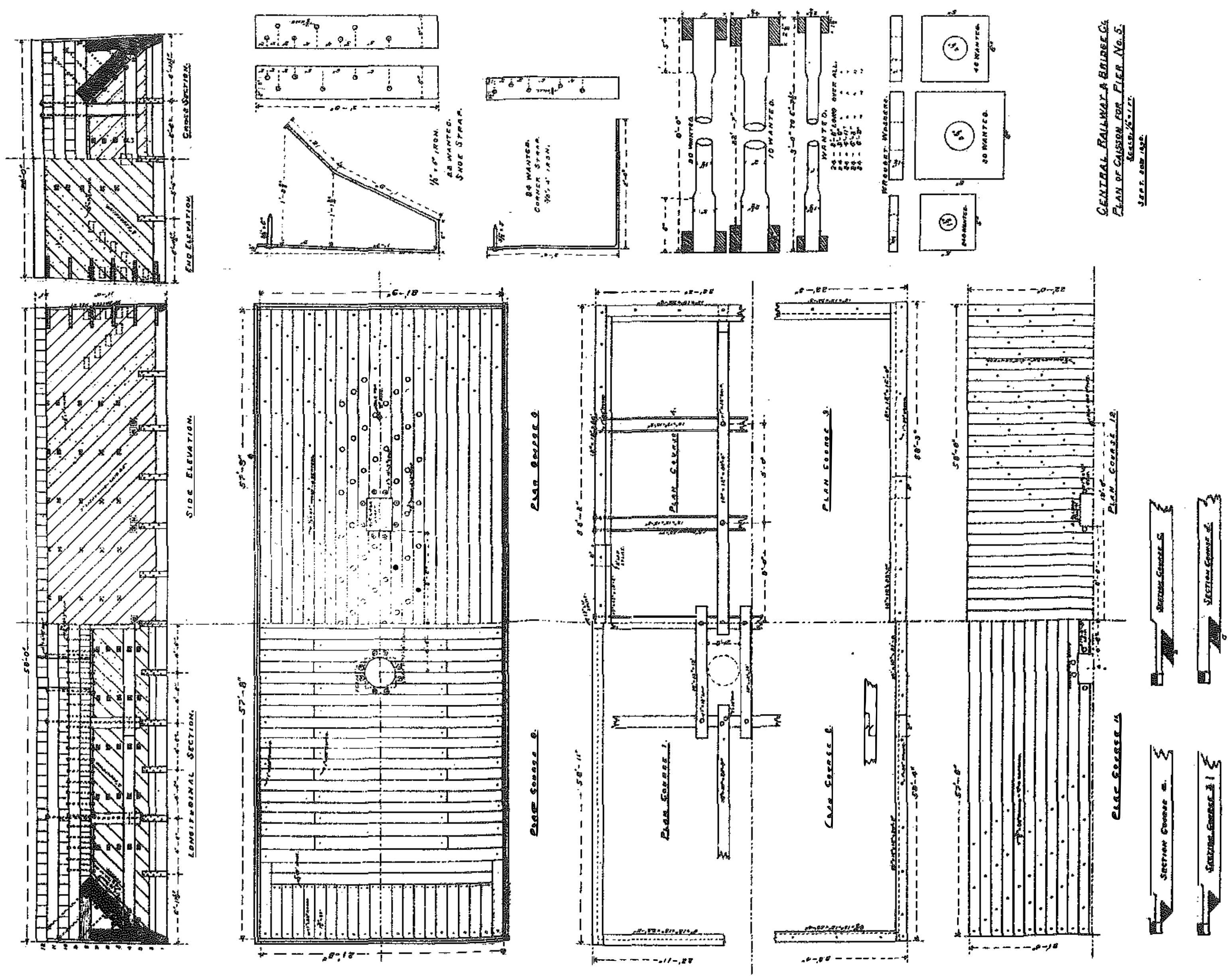
Work was accordingly begun on the caisson September 23d, and on October 17th it was launched. A pressure plant belonging to Messrs. Sooysmith & Co. was obtained from Louisville, and this firm was contracted with to do the work. At this time another freshet came in the river, and as this pier stands in the main channel where the current is swiftest, it was deemed unwise to attempt to locate it until the water would fall somewhat. On November 7th, with about 22 feet of water in the river, the caisson was towed into position, and on November 9th the work of laying masonry began. Owing to the rapid fluctuations in the water level, the cutting edge was not landed on the bottom until November 28th, and on November 29th the air compressor was started and the work of excavating and removing material in the working chamber was begun at once.

During the month following this date, the river stage permitted the work of excavating and sinking of caisson to go on without interruption. On account of the material being rock, the laying of masonry was subject to careful regulation so that the cutting edge would not be liable to injury from excessive pressure. On December 27th, 1890, the caisson had penetrated into the rock about 5 feet and had 38 inches yet to go. Seven full courses of masonry had been laid, and about ten or twelve working days were yet necessary to complete the sinking, seal the air chamber, and to bring the masonry up high enough to be out of the way of a 32-foot stage of water. On that day, however, the river rose above the shafts and suspended all operations.

At this time the condition of the work was such that the contractors could work in a 28-foot stage of water by the aid of the coffer-dam, but from December 27th until April 18th, 1891, as shown on the hydrograph, at no time did the stage of the river allow the contractors to do anything toward the completion of Pier No. 5. On April 28th the sinking was completed, and by May 8th the working chamber and shafts were filled with concrete. Laying masonry was not resumed until April 28th, from which time it was continued without further interruption from high water.

The caisson in this foundation was of the Morrison type, except that the iron shoe was omitted. The sinking was accomplished without accident or injury to any of the men engaged on it, and required seven hundred and twenty hours actual working time to penetrate 8 feet into the solid rock, or an average of 3.2 inches for each twenty-four hours. The rock penetrated consisted of ledges of fairly hard shaly formation alternating with thin ledges of hard fossiliferous limestone. Where first struck it was not well adapted to make a good foundation, and in order to get the deck of the caisson 3 feet below extreme low water, it was necessary to penetrate the rock 8 feet. The last piece of coping was set on Pier No. 5 at 9 A.M., June 18th, 1891, entirely completing the substructure within twelve months from the time of beginning the work.

PLATE XXX.
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXVII. No. 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



Piers Nos. 5, 6 and 7 have semicircular nosings up to the belting course. The under side of each is set at water stage 66 feet, and they, from the belting course to the under side of the coping, are built entirely of Ohio River freestone. The coping is 2 feet thick on Piers Nos. 5 and 6, and 18 inches thick on the other piers, all being of Bedford oolitic limestone, from Bedford, Ind.

The following table gives data of the construction of Pier No. 5. It gives the size and thickness of each course, number of cubic yards in the face stones and number of cubic yards of backing; also number of cubic yards of mortar in bed joints, the amount of cement required, and it also shows the cost of laying masonry, including sand and cement.

CENTRAL BRIDGE, CINCINNATI-NEWPORT.—BUILT 1890-91.—DATA FROM PIER No. 5.

Number of course.	Mean size of courses out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in bed joints.	Face work, percentage of whole.	Barrels cement per yard of masonry.	Cost per yard for laying, including sand and cement.	REMARKS.
	Feet.	Feet.				Pr. ct.			
1	19.50 x 55.50	2.25	88.29	52.61	1.620	80.8			Footings. Polygonal ends.
2	19.00 x 56.00	2.16	82.24	48.09	1.550	40.0			" "
8	19.00 x 55.00	"	28.54	51.01	1.525	86.0			Semicircular "
4	17.25 x 53.57	"	30.08	38.79	1.825	43.6			
5	17.07 x 63.39	2.08	28.94	86.46	1.310	44.0			
6	16.90 x 53 22	"	29.26	35.30	1.280	45.0			
7	16.73 x 58.05	"	27.38	36.89	1.260	43.0			
8	16.56 x 52.88	"	29.47	33.41	1.250	47.0			
9	16.39 x 52.71	2.00	28.50	31.23	1.245	47.7			
10	16.22 x 52.54	"	26.00	32.94	1.220	44.1			
11	16.04 x 52.36	2.16	29.40	33.38	1.207	47.1			
12	15.86 x 62.18	"	29.77	33.01	1.194	48.0			
18	15.68 x 52.00	2.08	27.30	81.34	1.180	46.5			
14	15.50 x 51.82	"	25.23	82.67	1.165	43.6			
15	15.32 x 51.64	"	27.48	30.43	1.150	47.4	0.3289		
16	15.15 x 51.47	2.00	26.72	27.39	1.130	49.4		$1.127	
17	14.98 x 51.80	"	28.16	25.20	1.110	52.7			
18	14.81 x 51.13	"	25.56	27.05	1.095	48.4			Semicircular ends.
19	14.64 x 50.96	"	25.27	26.59	1.081	48.7			
20	14 47 x 50.79	"	25.00	26.12	1.066	48.8			
21	14.30 x 50.62	"	25.42	24.95	1.050	50.4			
22	14.13 x 50.45	"	25.22	24.40	1.035	50.8			
23	18.96 x 50.28	1.88	21.86	22.88	1.020	49.7			
24	18.79 x 50.11	"	28.38	20.88	1.005	52.8			
25	13.62 x 49.94	"	21.90	21.60	0.990	50.4			
26	13.45 x 49.77	"	20.78	21.96	0.978	48.0			
27	13.28 x 49.60	1.75	21.90	18.84	0.960	54.4			
28	13.14 x 49.46	"	22.52	17.19	0.940	56.6			
29	13.00 x 49.32	1.83	22.35	18.65	0.933	52.0			
30	12.85 x 49.17	"	20.63	20.07	0.920	50.6			
31	12.69 x 49.01	1.66	20.25	16.00	0.907	62.4			
32	12.54 x 48.86	1.58	18.47	15.53	0.897	54.0			
33	12.40 x 48.72	1.83	22.25	16.38	0.882	58.7			

CENTRAL BRIDGE. CINCINNATI-NEWPORT.—BUILT 1890-91.—DATA FROM PIER No. 5—(Continued).

Number of course.	Mean size of courses out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in bed joints.	Face work, percentage of whole.	Barrels cement per yard of masonry.	Cost per yard for laying, including sand and cement.	REMARKS.
	Feet.	Feet.				Pr. ct			
34	13.32 x 49.64	1 83	23.33	17.54	0.860	57.0			Starling course,
35	12.26 x 48.20	1.42	17.77	11.61	0.863	61.0		$1.121	
36	12.14 x 46.20	"	16.77	10.79	0.798	60.0			Hood courses.
37	12.02 x 39.89	"	15.05	8.10	0.677	65.0			
38	11.90 x 35.80	"	13.75	8.72	0.660	61.0			
39	11.78 x 35.78	1.33	14.33	6.48	0.651	69.0			
40	11.66 x 35.66	"	12.27	8.21	0.641	60.0			
41	11.54 x 35.54	1.42	14.81	6.79	0.632	68.5			
42	11.42 x 35.42	1.33	13.85	6.12	0.625	69.0			
43	11.30 x 35.30	"	12.55	7.04	0.616	54.0			
44	11.19 x 35.19	"	12.32	7.13	0.608	63.0	0.3010		
45	11.07 x 35.07	"	12.73	6.44	0.598	66.0		$1.470	
46	10.96 x 34.96	"	12.21	6.71	[illegible]	65.0			Rectangular in plan.
47	10.84 x 34.84	"	13.19	5.46	[illegible]	71.0			
48	10.73 x 34.73	"	13.70	4.71	[illegible]	74.0			
49	10.61 x 34.61	"	11.96	6.17	[illegible]	66.0			
50	10.50 x 34.50	"	12.05	5.95	0.560	67.0			
51	10.38 x 34.38	"	12.36	5.29	0.550	70.0			
52	10.27 x 34.27	"	12.33	5.01	0.540	71.0			
53	10.16 x 34.16	"	12.85	4.28	0.537	75.0			
54	10.05 x 34.05	1.16	10.37	4.33	0.530	70.0			
55	11.00 x 35.00	2.00	20.63	7.89	0.525	72.0	0.348	$1.500	Sub-coping.
56	12.00 x 36.00	"	32.00	None.	0.594	100.0	0.349	1.536	Coping.

Percentage of face work in courses.........................1 to 33 inclusive = 46.42 per cent.
" " " " ....34 to 54 " = 65.81 "

NOTES.—From top of caisson to the starling course the masonry is built of Berea sandstone with Ohio River freestone backing; but above this it is built entirely of Ohio River freestone.

All face stones were required to have a width of one and one-quarter (1¼) times their thickness.

All mortar was mixed 1 cement to 2 of sand, Louisville cement being used throughout.

Cost of cement and sand rated at $1.20 per barrel of cement used.

LABOR PER DAY OF TEN HOURS RATED AT.

Foreman............	$4 00
Mason............	3 25
Cutter............	3 50
Engineer............	2 00
Common labor............	1 50

It is very interesting to note in connection with this table the ratio of increase of the cost of laying as the pier increases in height. On comparing the table for Pier No. 4 with that for Pier No. 5, it will be noted that the cost of laying is higher in the former than in the latter. The reason for this is that Pier No. 4 was constructed during the winter months and the foreman in charge was not so expert as the one on Pier No. 5. The writer believes that the information in the table is valuable, and if the cost of quarrying and cutting for the various building stones were known, exact estimates of the cost of bridge masonry could readily be made.

Before closing the description of the substructure it may be well to explain why Piers Nos. 4 and 8 were constructed with Berea sandstone face, and concrete heartings, while Piers Nos. 5, 6 and 7 were constructed with the same face stone and Ohio River freestone backing. The original specifications required that the face of the piers should be constructed of limestone obtainable in a quarry near Cincinnati and the backing should be of freestone.

Considerable difference of opinion as to the reliability of this limestone was found among engineers in Cincinnati, and as we had no experience with it, nor time to investigate the matter thoroughly, we decided to take the safe course and not permit its use. A quarry at North Vernon, Indiana, which furnished stone of undoubted quality, could not deliver the quantity as rapidly as necessary. There were then available only the Indiana oolitic limestone and the Berea sandstone quarries, each of undoubted character and of sufficient magnitude to furnish rapidly the quantity of stone required. It was found, however, that the oolitic quarries could give no guarantee of prompt delivery on account of other contracts, and it was decided to obtain the face stone for the important piers from the Berea sandstone quarries. In order to reduce the cost of masonry to that originally specified, the King Bridge Company requested permission to use concrete heartings in the piers in question. The writer had no experience in constructing masonry in this way, but knowing it had been done satisfactorily in several instances of moderate-sized piers, readily granted permission to the contractors to construct Piers Nos. 4 and 8 as requested.

In the absence of information regarding the comparative elasticity of concrete and Berea sandstone, consent to the use of concrete in the interiors of the large Piers Nos. 5, 6 and 7, in which the pressure on the lower course is very great, was withheld. The Berea stone is comparatively soft, and if the concrete should compress more than it did, the periphery of the pier would be subjected to crushing. We did not care to take any chance in the matter. We are not aware of any piers of this size thus constructed except some granite piers on the

Mississippi River, which no doubt could withstand a pressure of this kind.

The contract price for the substructure was as follows:

First.—Filling in abutments on Newport and Cincinnati sides, earth or gravel, per cubic yard, $0.22.

Second.—Excavation of Piers 1, 2, 3, 9, pedestals and abutment foundations, per cubic yard, $0.44.

Third.—Concrete foundations, per cubic yard, $4.40.

Fourth.—Pile foundations, per linear foot, 30.8 cents.

Fifth.—First-class masonry, including the cost of foundation excavation, etc., for Piers Nos. 4 and 8, $11.43 per cubic yard.

Sixth.—First-class masonry, including the cost of foundation complete for Piers Nos. 5, 6 and 7, $12.50 per cubic yard.

Seventh.—First-class masonry, Piers Nos. 1, 2, 3, 9, and pedestals, per cubic yard, $9.90.

Eighth.—Second-class masonry in abutments and ramps, per cubic yard, $7.98.

The quantities of masonry in each part of the work is shown in the following table:

CENTRAL BRIDGE, CINCINNATI-NEWPORT.—BUILT 1890–91.

Pier Number.	Size under Sub-Coping.	Height Over All.	Size at Base of Shaft.	Thickness of Coping.	Cubic Yards of Masonry.	Remarks.
1........	4'.98 × 29'.98	26'.22	6'.42 × 31'.42	18"	146.18	Square Shaft.
2........	4'.98 × 29'.98	39'.41	7'.63 × 32'.63	18"	271.68	" "
3........	6'.00 × 30'.00	47'.00	9'.07 × 33'.07	18"	393.92	" "
4........	9'.00 × 34'.00	73'.96	13'.77 × 49'.52	18"	1 432.85	Circular Shaft.
5........	10'.00 × 34'.00	112',79	17'.34 × 53'.66	24"	2 357.61	Circular Shaft, inc. Caisson in H't.
6........	10'.00 × 34'.00	104'.09	17'.80 × 54'.16	24"	2 475.59	Circular Shaft.
7........	9'.00 × 34'.00	93'.43	16'.00 × 51'.75	18"	1 974.12	" "
8........	7'.00 × 32' 00	87'.14	13'.47 × 46'.83	18"	1 393.30	" "
9........	7'.00 × 32' 00	37'.28	9'.58 × 34'.58	18"	330.06	Square Shaft.
					10 775.31	
Cin'ti Ramp.	....	....	....	12"	663.03	....
Newport Ramp.	....	....	....	12"	1 510.23	....

Superstructure.—The superstructure of this bridge consisted of the following spans as shown on the general lay-out:

Viaduct spans: one span, 36 feet; four spans, 28 feet 6 inches; one through span, 108 feet center to center of piers; three spans, 27 feet each; one through span, 162 feet center to center of piers; one cantilever arm, 252 feet; one cantilever arm, 156 feet; one suspended span, 208 feet; one cantilever arm, 156 feet; one cantilever arm, 252 feet; two through spans, 254 feet each. Viaduct spans: one, 55 feet; three, 30 feet; three, 61 feet; one, 29 feet; and one, 50 feet. The clearance is 16 feet above the top of roadway, and the clear width between trusses is 24 feet.

The specifications governing the construction of this portion of the work are very full, and contain a number of features which will be discussed by F. C. Osborn, M. Am. Soc. C. E., in his paper on the cantilever span of the bridge. A copy of the specifications will be found in Appendix 2. Full information in regard to loads and unit stresses allowed, the character of the material used and the required tests will be found therein.

The inspection of the material at the mills, the work at the bridge shops and the erection was conducted by the firm of G. W. G. Ferris & Co., of Pittsburgh. About six hundred and fifty tests, taken from rolled sections, were made. The material proved to be of first-class character, all that was accepted falling within the limitations of the specifications. The full-sized eye-bars tested all broke in the body of the bar with the exception of one which failed in the head on account of a flaw. The defective head was cut off and another put on. Upon a re-test this bar broke in the body of the bar.

Central Bridge.—Detailed Weights, Etc.

Superstructure:

	Pounds.
36-foot span and bent 1	26 640
28-foot 5-inch span	11 545
Two 28-foot 5-inch spans and bents 2, 3 and 4	54 545
28-foot 5-inch span	11 215
108-foot truss span	124 725
27-foot span	14 020
27-foot span and bents 5 and 6	35 385
27-foot 2-inch span	12 340
Carried forward	290 415

		Pounds.
Brought forward		290 415
162-foot truss span		202 010
Shore arms of cantilever		1 376 978
River arms of cantilever		691 360
Suspended span		335 185
Two 254-foot spans		809 160
55-foot girder spans		45 550
30-foot spans and bents 7 and 8		37 396
31-foot 5-inch span		15 025
30-foot span and bents 9 and 10		33 566
31-foot 5-inch span		15 235
30-foot span and bents 11 and 12		33 396
31-foot 5-inch span		15 070
29-foot span and bents 13 and 14		28 432
50-foot girder		36 715
		3 965 483
Rivets, bolts, etc.	31 790	
Hook bolts, nails and spikes	36 945	
Finials and cresting	17 075	
Name plates and castings	5 345	
Hand railing, Rail	176 695	
Braces	2 405	
Posts	12 955	
Lamps, posts and braces	3 590	
Newel posts	5 635	
Stairways	44 865	
		337 300
		4 302 783

Lumber (white oak) for floor...............551 434 feet B. M.
Paint, 650 gallons (two coats).............1 gallon for $3\frac{1}{3}$ tons.

Erection of Central Bridge.—About the middle of September, 1890, although the masonry was in somewhat of a sad plight, and it was clearly evident that the contractors could not complete their work as agreed upon, there was sufficient work done to enable hopes to be formed that the work could be completed somewhere in the neighborhood of February or March, 1891.

On the Cincinnati end nothing could be done owing to the right of way not yet being clear and the superstructure not yet completed. Work was therefore begun on the Newport end. In the early part of October the viaduct on the Newport side was started and was completed by the 25th of the month. On October 30th the girders across Front Street were placed in position. On November 12th the raising of the iron span between Piers 8 and 9 was begun, the false work having been placed between these piers during the latter part of October, and by the 15th the spans were connected. On the 24th it was fully riveted up and the stringers and floor beams were in place. On November 25th the false work for the span between Piers 7 and 8 was started and was finished December 3d, and by December 9th this span was fully coupled up.

The question as to whether the Newport shore arm of the cantilever between Piers 6 and 7 should be erected at this time now arose; the work on the caisson for Pier No. 5 was progressing favorably, and indications were that it would be prosecuted to a finish without interruption. The river looked very favorable (water stage 15 feet), and it was decided to erect this arm. On December 14th the work on the false work was started and was finished by the 20th, and on the 21st the raising of iron began. On the 22d reports from the head waters of the river indicated that considerable of a rise could be expected, so that additional energy was used, and by the 23d this span was coupled up without the floor beams or stringers.

It was calculated that the trusses could stand their own weight alone, if found necessary to remove the false work before the river arms of the cantilever were erected. As the indications were that the river would reach a stage of over 30 feet, in which false work could not be held, it was determined to take it out, and this was done by the 28th of the month. The work on the Newport side was then stopped until some reasonable assurance of the completion of Pier No. 5 could be had, and the work on the erection of the river arm was not started until the following May. When this work was begun the material for the cantilever arm was taken out in barges and hoisted into place. This work was very difficult and expensive, but it was not safe to take any material out on the shore arm without having false work to support it.

After extending the river arm three panels and hanging to it the traveler weighing about 60 tons, permission was given the contractor for erection, to place the floor system of the shore arm in position and to carry out the material along the bridge. The erection towards the center of the cantilever was carried on slowly, as there was no occasion to push this until Pier No. 5 was completed. It, however, was finished June 18th, at the same time as Pier No. 5.

Prior to January 1st the right of way on the Cincinnati side had been fully obtained, cleared, and the substructure finished. On January 15th the Cincinnati viaduct was erected and fully riveted.* By March 11th the 162-foot span between Piers 3 and 4 was coupled up. On May 25th the false work on the Cincinnati shore arm was started and finished June 12th. July 3d this arm was completely coupled up, and by the 14th the inside traveler necessary to erect the cantilever was completed, and the work of erecting the Cincinnati cantilever arm started. On July 22d the two arms of the cantilever met and were coupled up without any difficulty. The wedges and screws used gave great satisfaction and enabled the span to be readily adjusted. On August 29th, 1891, the whole work was completed and opened for the use of the public.

THE CANTILEVER SPAN.

By F. C. Osborn, M. Am. Soc. C. E.

One of the objective points in the designing of this structure was the elimination, as far as possible, of undulatory and vibratory motion from passing loads. To this end the stringers were riveted rigidly to the floor beams, the floor beams in turn rigidly attached to posts and suspenders, and the latter made in compression form in order to better resist any tendency to vertical vibration. The lower lateral bracing is made of angles, arranged in a double triangular system, the angles attached to stringers at all intersections and to each other at center of panel; the attachment to chords is by means of wing plates directly to main truss pins.

* Plate XXXI.

PLATE XXXI.
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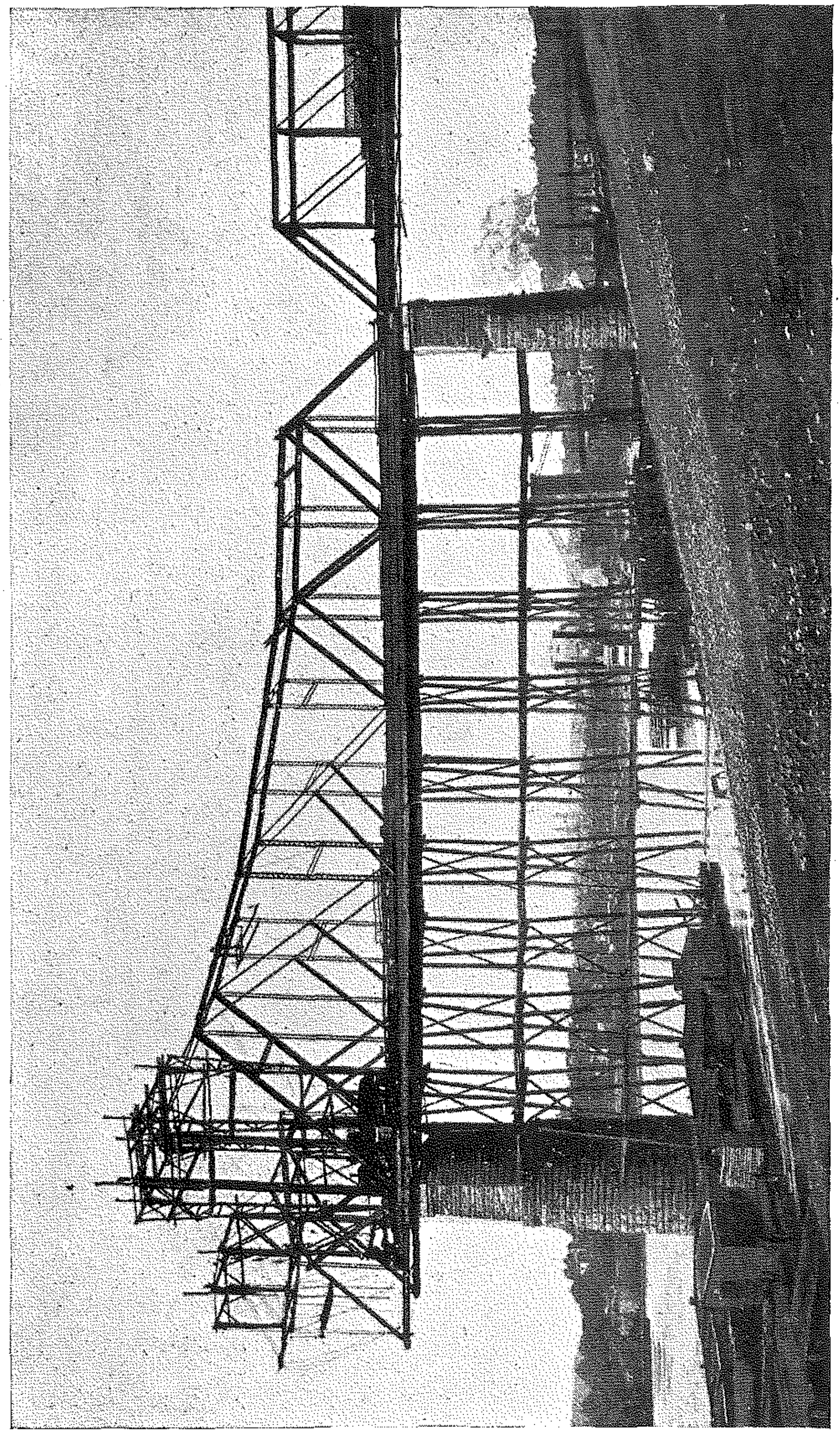
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



ANCHORAGE ARM OF CANTILEVER ON CINCINNATI SIDE, JULY 14TH, 1891.

The portal bracing at the anchorage end of the shore arm is in box form, taking hold of both top and bottom flanges of the end post by means of large gusset plates and also attaching securely to the top and bottom flanges of the top chord. The portal rods are made double and attach to long pins passing through the gussets.

The top chord bars of the river cantilever arms are made in two-panel lengths, and are supported by the light vertical posts and top lateral struts in such a way as to clamp them securely in position and at the same time effectively transmit the wind pressure at the panel point to the top lateral bracing.

Owing to the sharp grade of the cantilever spans the proper position for the tall posts over the piers became an interesting question. If they were made vertical, the other posts and suspenders being normal to the bottom chord and grade, it would make a short panel on one side of the post and a long one on the other, and give the post the appearance of leaning up-grade, as well as an awkward look on account of not being parallel with the posts on either side of it. These objections could have been met, of course, by making all posts and suspenders vertical instead of normal to the grade. If this was done, however, it would have to be done on the adjacent spans, two of 254 feet each and one of 162 feet, and the extra expense would have been greater than was thought justifiable. After considerable study and the making of scale drawings of the several combinations, it was decided to make all posts, including the large ones over the pier, perpendicular to the grade.

The camber calculations for the river arm of the cantilever were made on the basis of the full dead load and one-half only of the maximum live load strain. The compression members were lengthened and the tension members shortened by an amount corresponding to their change in length from the strains caused by the above loading. The lengths of members in the shore arm were calculated as though the bottom chord was perfectly straight and the posts perpendicular to it, no allowance being made for upward or downward deflection. After swinging, and before the erection of the river arm, this span would deflect downward; under the dead load alone of the completed structure it would deflect upward; with the live load covering this arm alone it would deflect downward at the anchorage end and upward at the other, the bottom chord taking the form of a reverse curve.

The calculated maximum anchorage strain is 136 000 pounds at the end of each truss, and is taken up by a 6 x 1½-inch steel eye-bar passing 20 feet into the masonry and attached to a box girder 7 feet square and 2 feet deep

Provision for alternate strains of tension and compression in the top chord of the anchorage arm is made by using eye-bars for the full tension strain and a built member for the full compression strain. The compression chord is prevented from taking up any tensile strain, by means of oblong pin holes which permit the compression members to separate at the joint.

APPENDIX I.

SPECIFICATIONS FOR SUBSTRUCTURE OF THE "CENTRAL BRIDGE" OVER THE OHIO RIVER, BETWEEN CINCINNATI AND NEWPORT, FOR THE CENTRAL RAILWAY AND BRIDGE COMPANY.

Abutments.—The abutments or ramps on the Cincinnati and Newport sides shall be respectively about 277 and 330 feet long; they shall be formed of two side walls of masonry capped, as shown on plans, with coping courses 12 inches thick and 2 feet wide, supporting the side railings, and form the remainder of the width of the sidewalks with Berea sandstone flags not less than 9 inches thick, laid in cement with parallel joints, and a front wall supporting the end of the iron superstructure, capped with limestone not less than 18 inches thick. The ends of the flagstone on the roadway side shall be supported on a good and suitable foundation, as the engineer may direct.

These walls shall be built of freestone ashlars not less than 10 inches thick; they shall be founded on a bed of concrete or on pile foundations, as the nature of the ground may require. The spaces between the walls shall be filled with earth, gravel or broken stone, or brick deposited in 12-inch layers, upon which the pavement and tracks for the wagon-ways and tramways shall be laid.

Pedestals.—There shall be twelve pedestals of masonry for the viaduct approach on the Cincinnati side, and sixteen for the viaduct approach on the Newport side. The pedestals shall be built of free-

PLATE XXXII.
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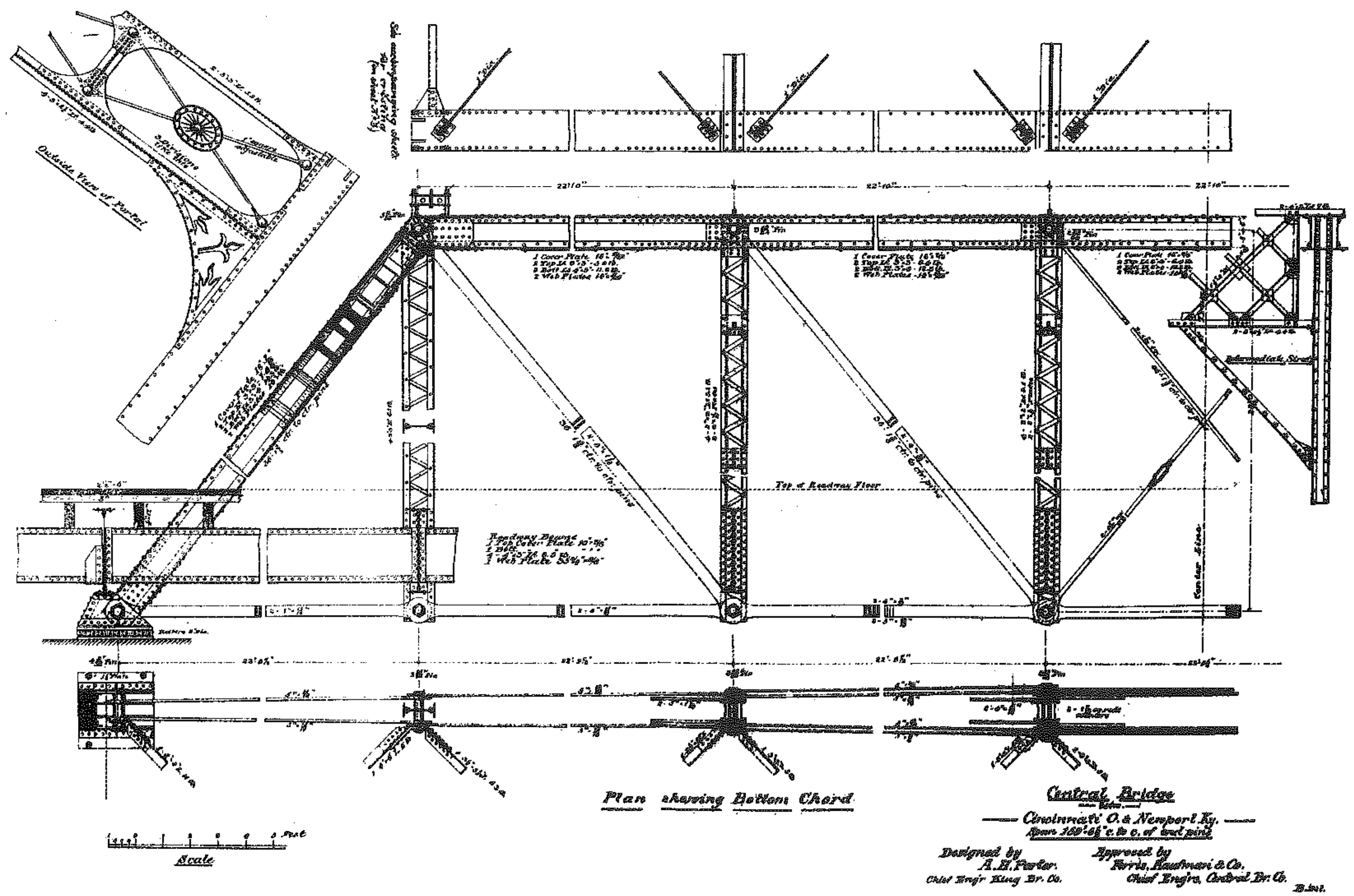
CANTILEVER HIGHWAY BRIDGE.



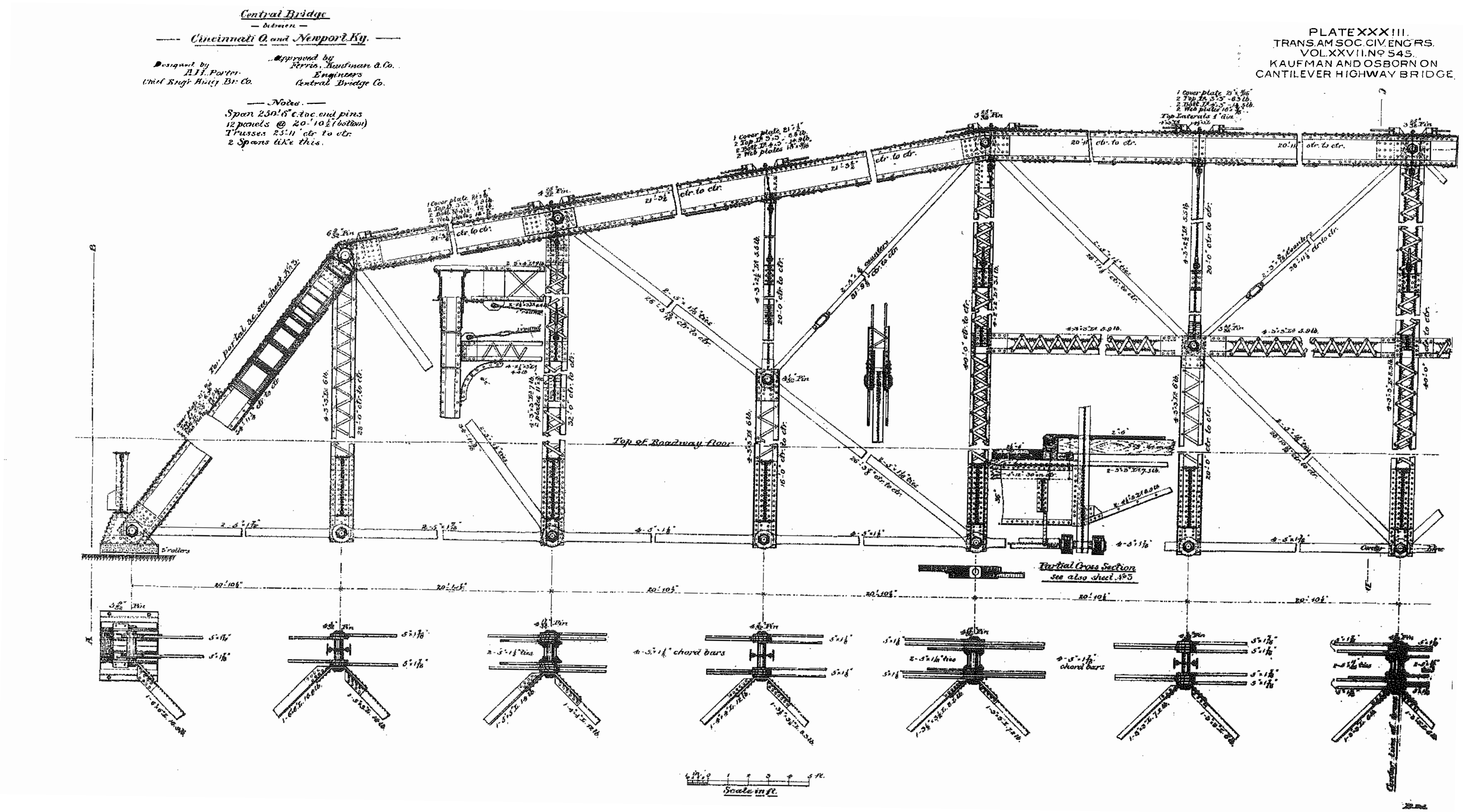


PLATE XXXIII.
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KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

PLATE XXXIV
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KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

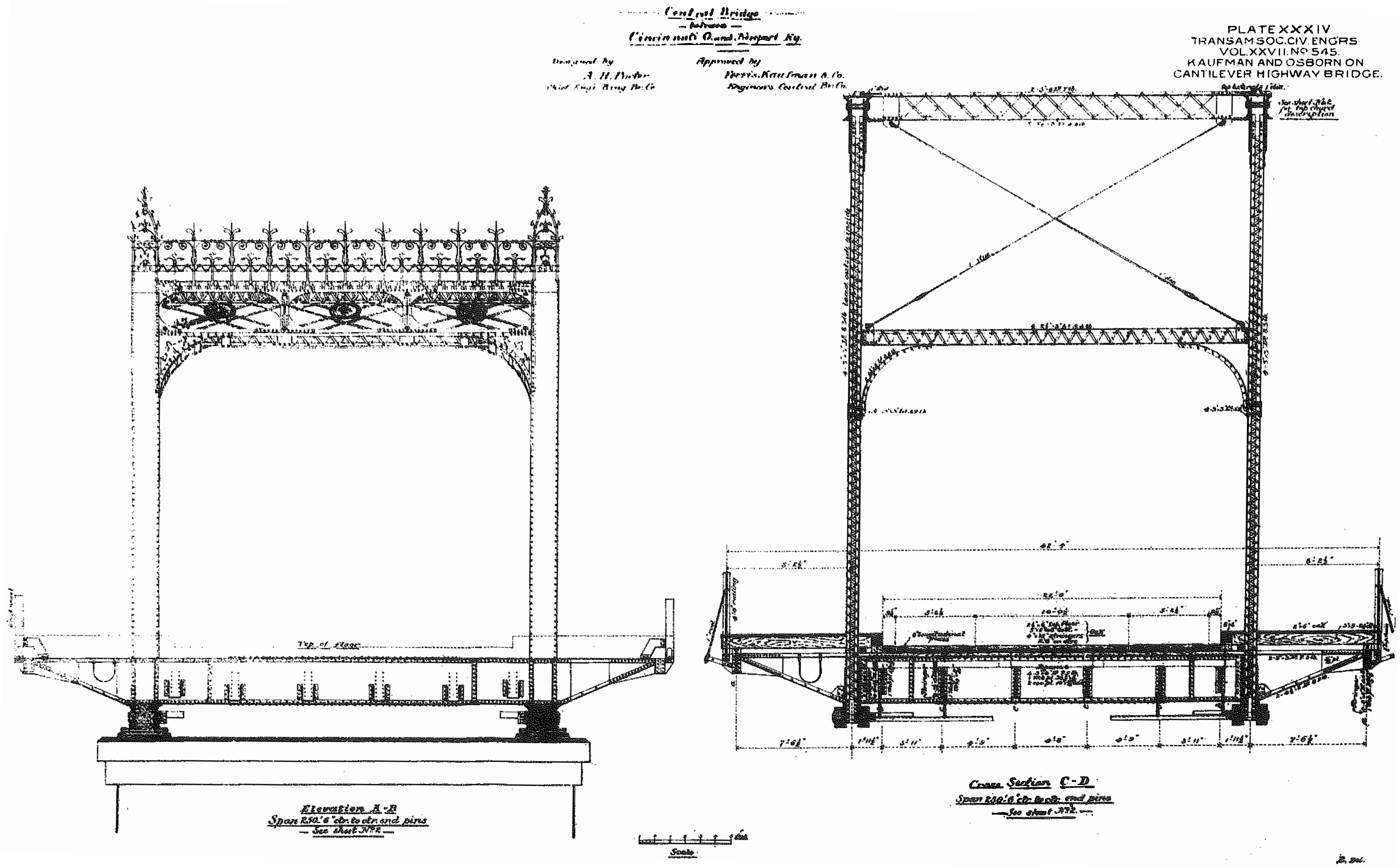

PLATE XXXV.
TRANS. AM. SOC. CIV. ENG'RS.
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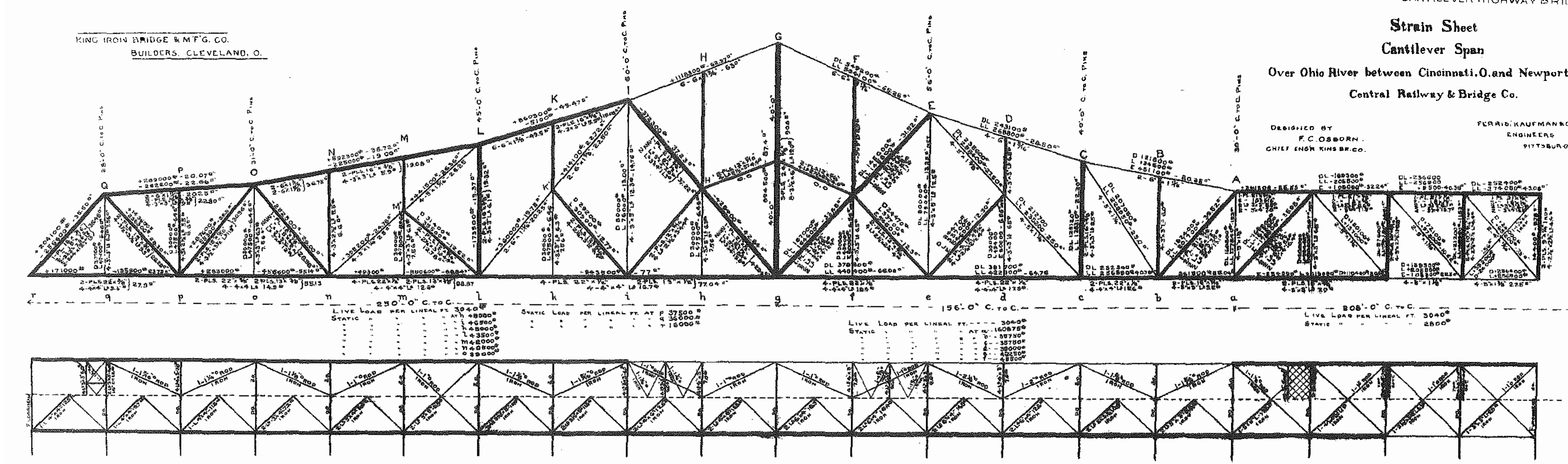
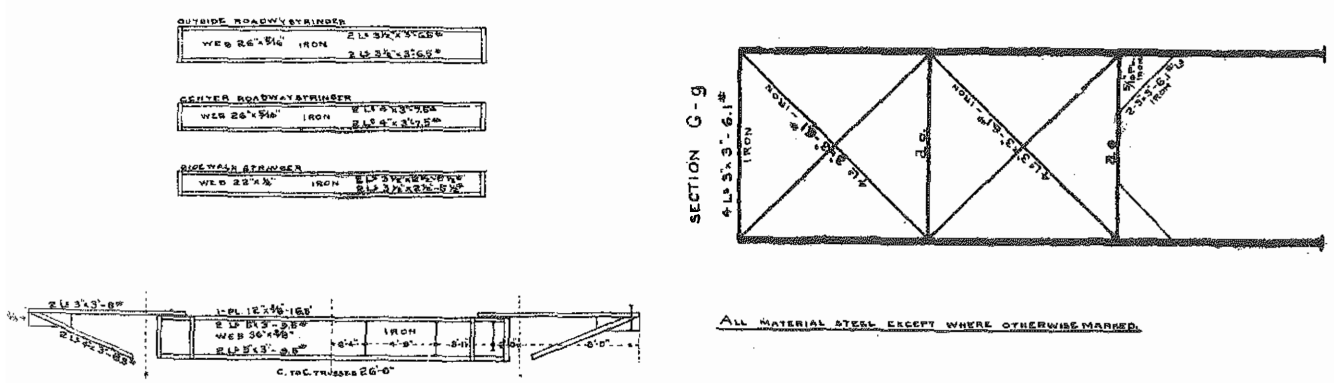
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

Strain Sheet
Cantilever Span
Over Ohio River between Cincinnati, O. and Newport, Ky.
Central Railway & Bridge Co.

DESIGNED BY
F. C. OSBORN.
CHIEF ENG'R KING BR. CO.

FERRIS, KAUFMAN & CO.
ENGINEERS
PITTSBURGH, PA.





LIVE LOAD – For all truss members receiving more than one panel load, eighty (80) pounds per square foot for suspended span and cantilever arms. For stringers, floorbeams, and long suspenders one hundred (100) pounds per square foot of clear roadway and sidewalks, or an Aveling and Porter fifteen ton steam road roller.

In the calculation of stresses the following conditions of live load will be assumed.

For main truss members, the roadway and both sidewalks will be considered loaded

For long suspenders, roadway and one sidewalk only will be considered loaded, and for floorbeams, the roadway will be considered loaded with sidewalks unloaded, also roadway unloaded with sidewalks loaded.

The above 80 pounds per square foot is taken on clear roadway and sidewalks.

WIND LOAD – Stresses calculated for a pressure of thirty (30) pounds per square foot on the exposed surfaces of both trusses and railings and a moving load surface of six (6) □ feet per lineal foot of bridge.

LATERAL STRUTS – Lateral struts will be apportioned to resist the resultant due to an initial stress of ten thousand (10,000) pounds per square inch upon all rods attached to them, when this is in excess of wind stress. The fiber stress due to weight of strut must be considered and be deducted from the unit stress specified.

Approved:

Ferris Kaufman & Co.
by [illegible]

May 31, 1890.

ALLOWED STRAINS PER SQ. INCH
TRUSSES

Tension Members	Iron	Steel
Eyebars and counters	10000(1+[illegible])	16000(1+[illegible])
Shapes and and angles (net area)	10000 ;	12000
Lateral rods	15000	
Compression Members		
Square ends	9000–30 $\frac{l}{r}$	; 15000–50 $\frac{l}{r}$
One square and one pin end	9000–35 $\frac{l}{r}$	; 15000–70 $\frac{l}{r}$
Pin ends	8000–40 $\frac{l}{r}$	; 15000–80 $\frac{l}{r}$
Lateral struts	11000–50 $\frac{l}{r}$	

In which L = distance between supports in inches
r = least radius of gyration in inches

Alternate Tension and Compression
For compression only use compression formula
For greater stress 10000(1–[illegible]) 16000(1–[illegible])
Use the one giving the greater area of section.

Floorbeams and Stringers
Tension (net section) 10000(1+[illegible]) ; 12000(1+[illegible])
Compression – use same gross area as tension flange.

PLATE XXXVI.
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KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

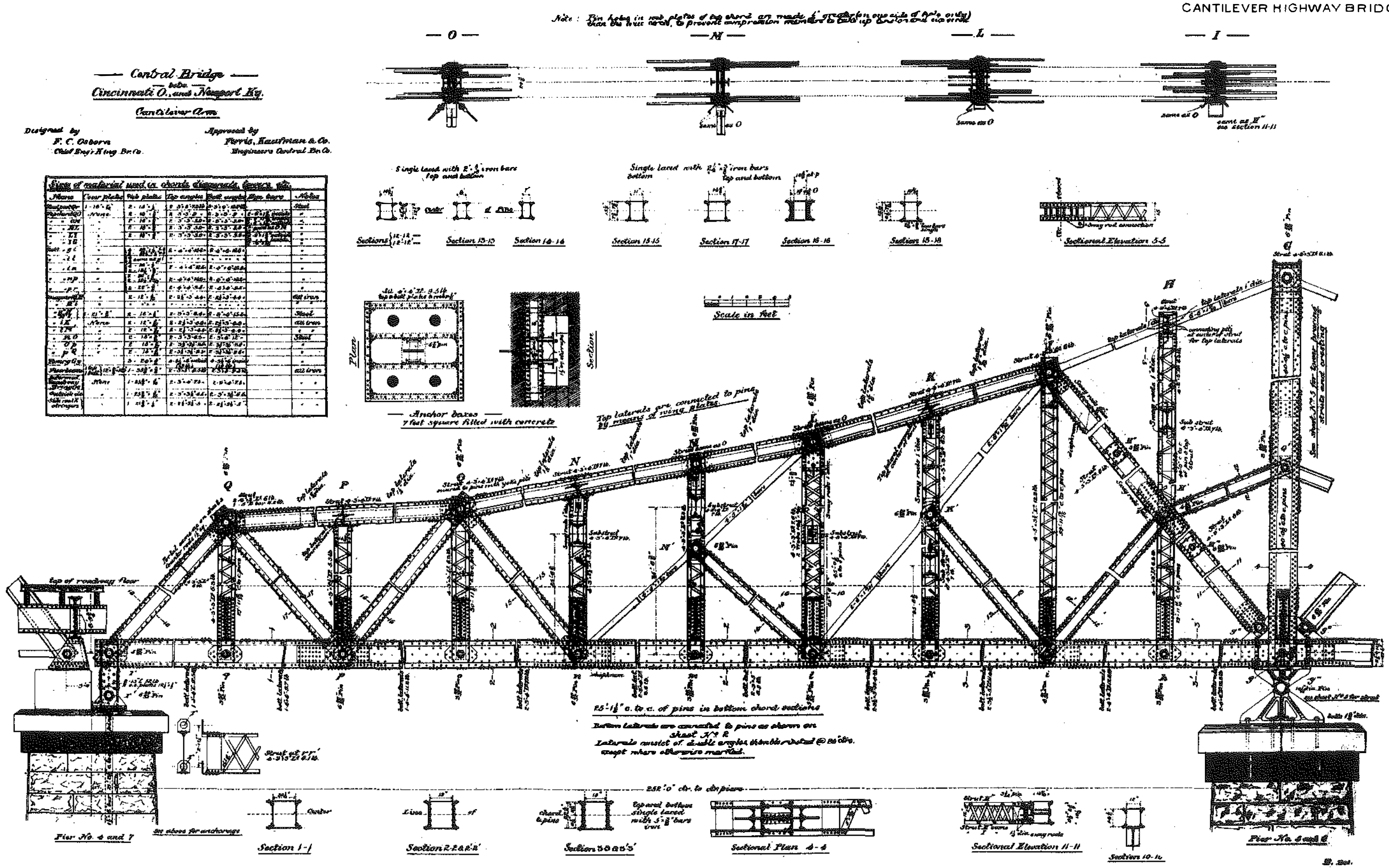

PLATE XXXVII.
TRANS. AM. SOC. CIV. ENGRS
VOL. XXVII. N° 545
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE

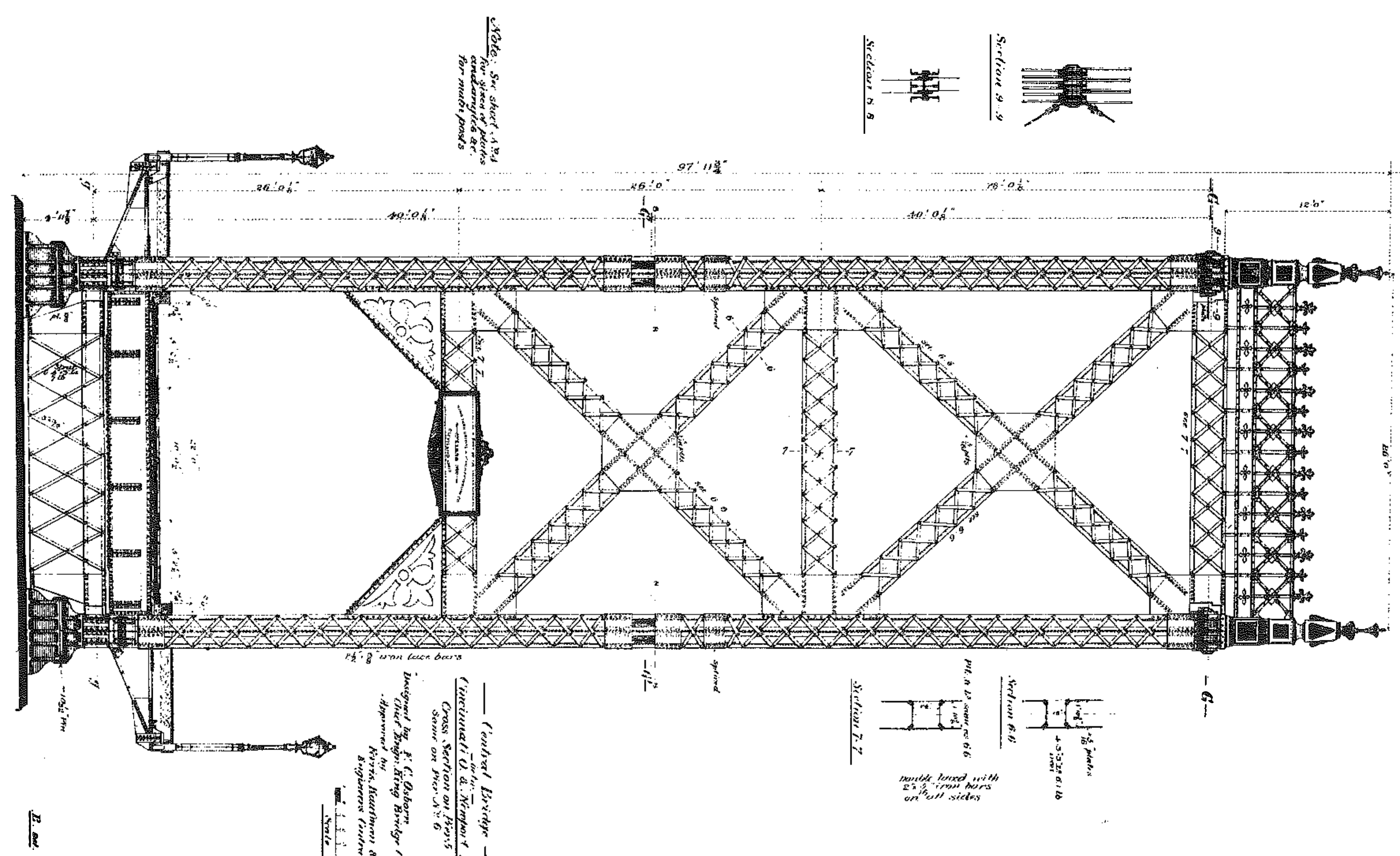

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENG'RS.
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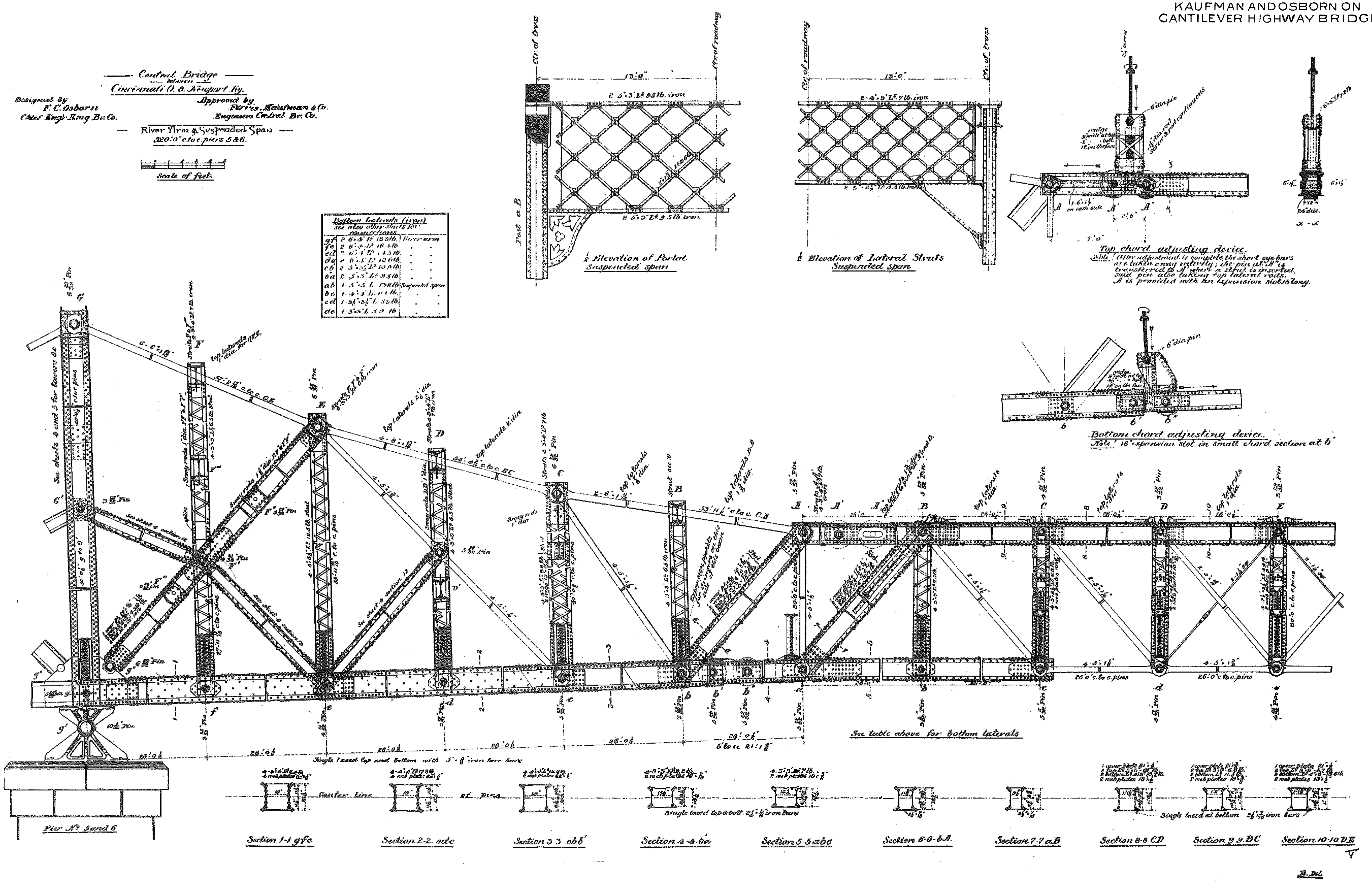
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



Bottom laterals (iron) see also other sheets for connections		
gf	2 6"×4" L^s 18.5 lb.	River arm
fe	2 6"×4" L^s 16.4 lb.	" "
ed	2 6"×4" L^s 14.5 lb.	" "
dc	2 6"×4" L^s 12.6 lb.	" "
cb'	2 5"×3½" L^s 10.9 lb.	" "
b'a	2 5"×3" L^s 9.5 lb.	" "
ab	1-5"×5" L 17.8 lb.	Suspended span
bc	1-4"×4" L 11 lb.	" "
cd	1-3½"×3½" L 8.5 lb.	" "
de	1-5"×5" L 5.9 lb.	" "

PLATE XXXIX.
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KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

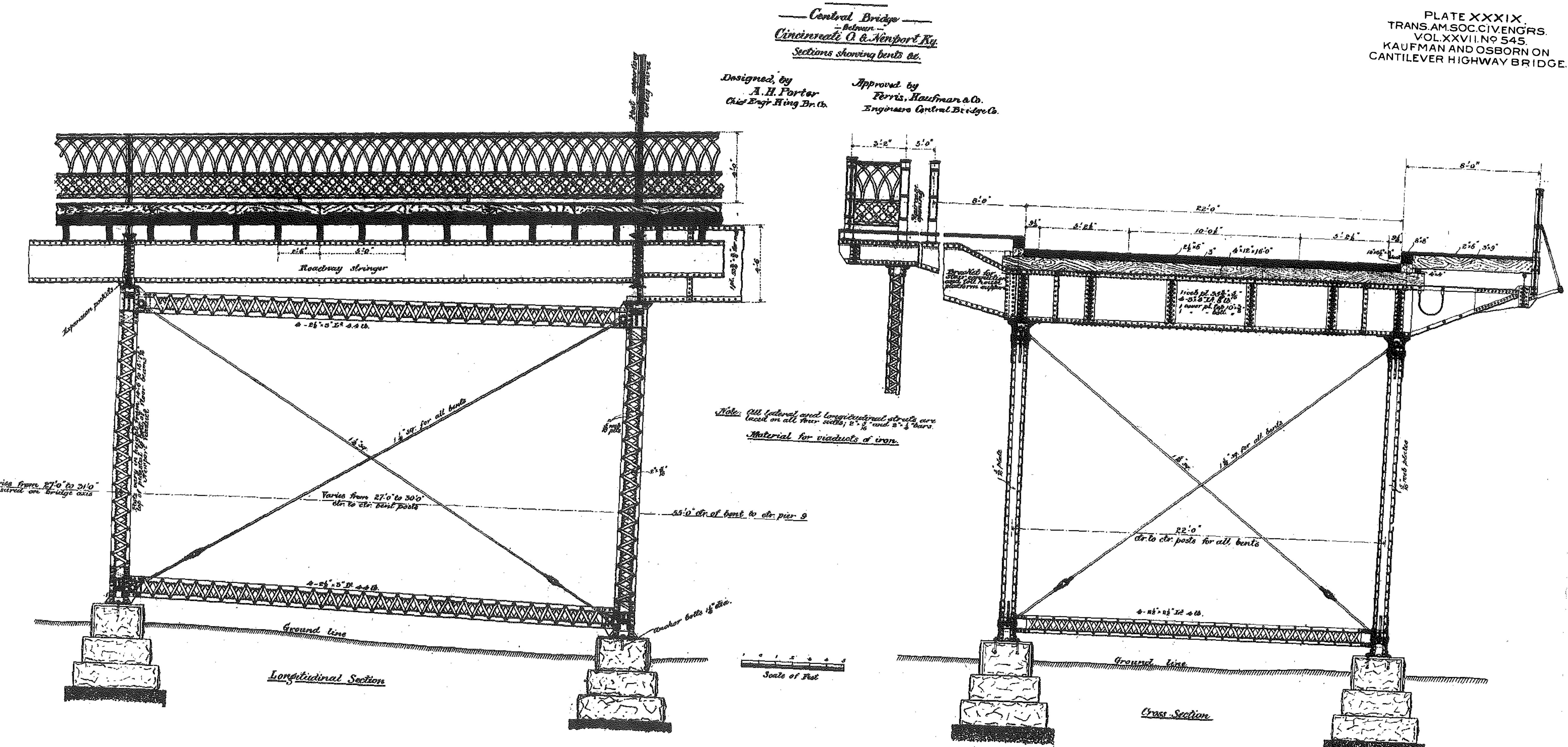

PLATE XL.
TRANS. AM. SOC. CIV. ENGS.
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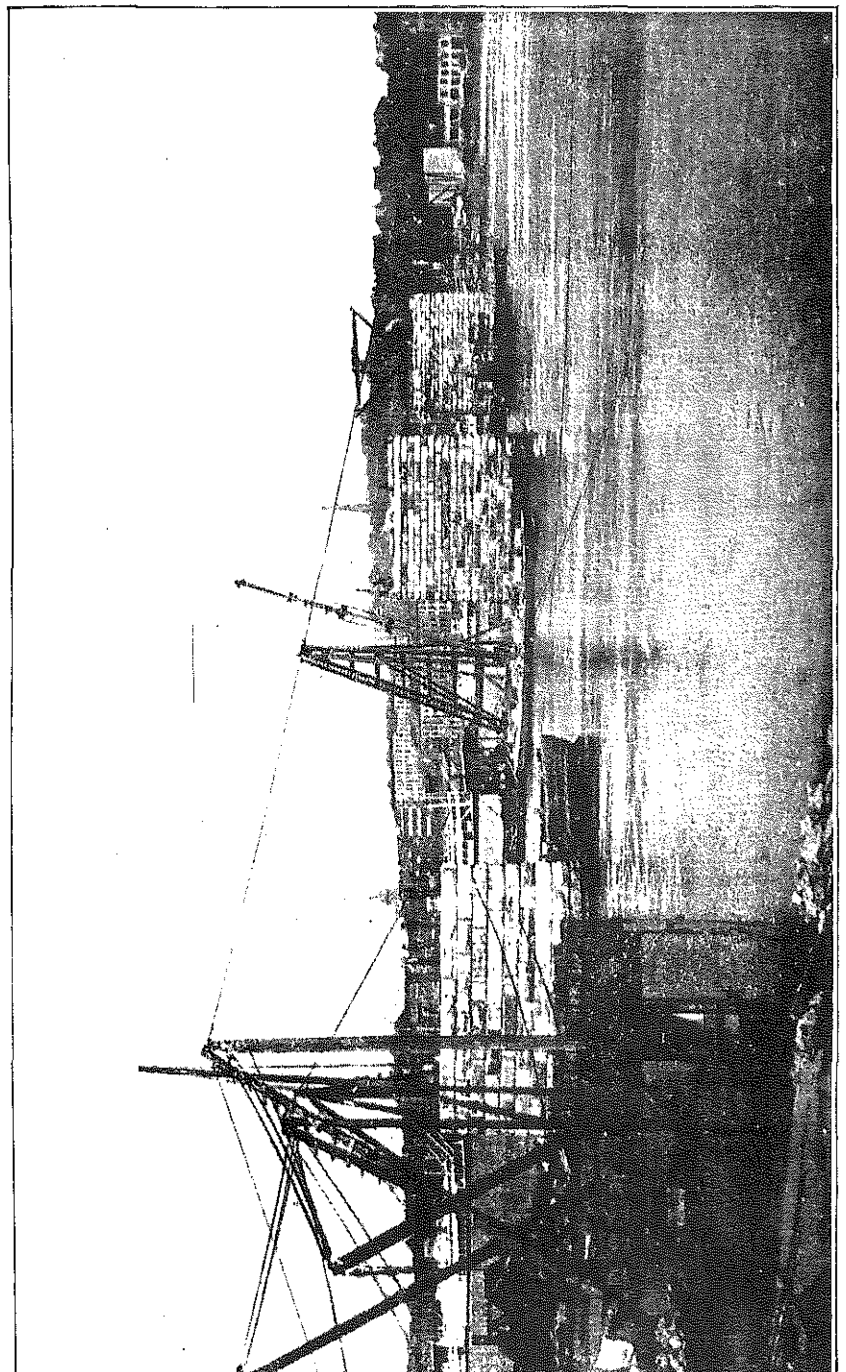
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



GENERAL VIEW OF RIVER PIERS, SEPTEMBER 20TH, 1890.

PLATE XLI.
TRANS. AM. SOC. CIV. ENGS.
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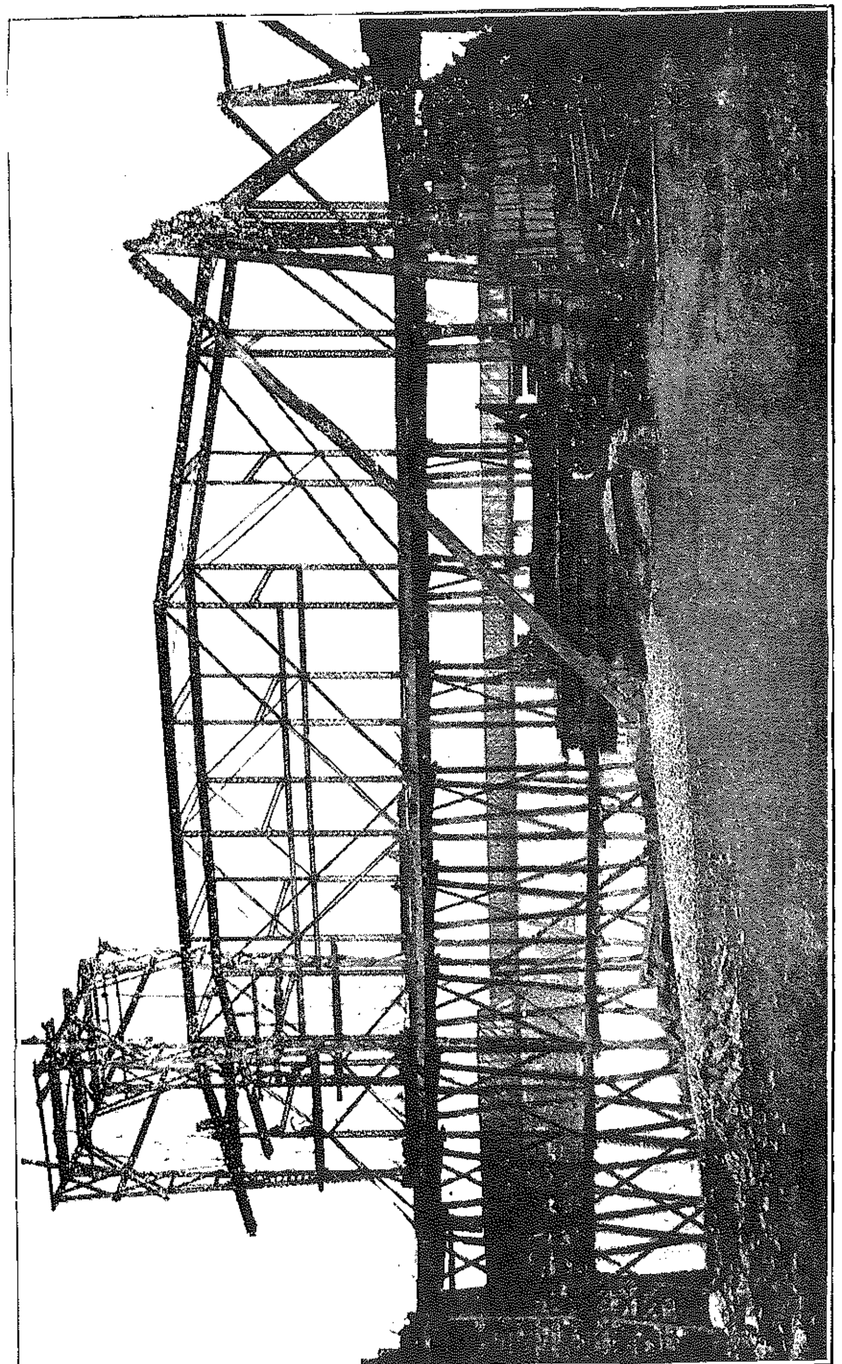
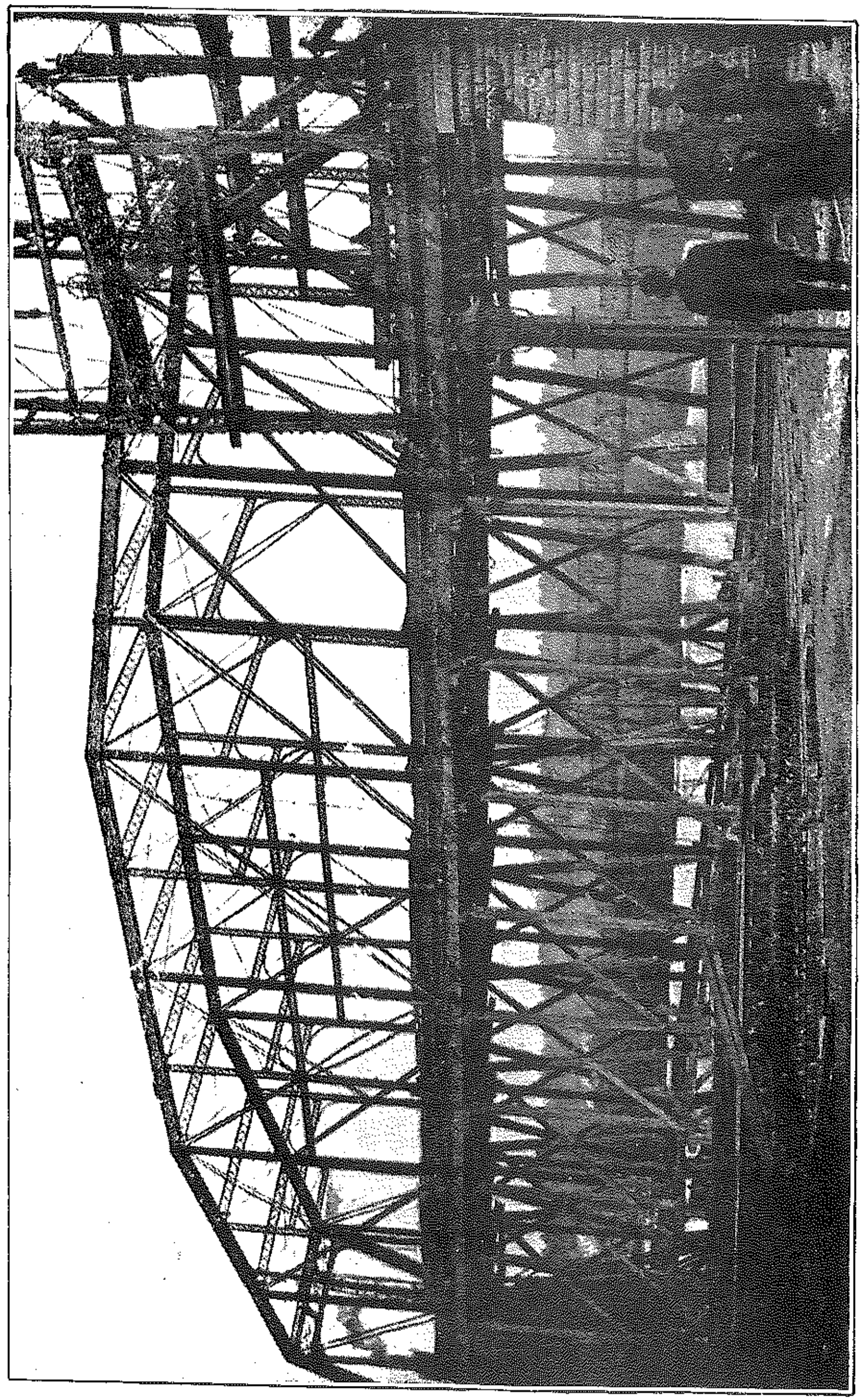
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



SPAN 8 AND 9, NEWPORT SIDE, NOVEMBER 14TH, 1890.



SPAN 8 AND 9, NEWPORT SIDE, NOVEMBER 17TH, 1890.

PLATE XLIII.
TRANS. AM. SOC. CIV. ENGS.
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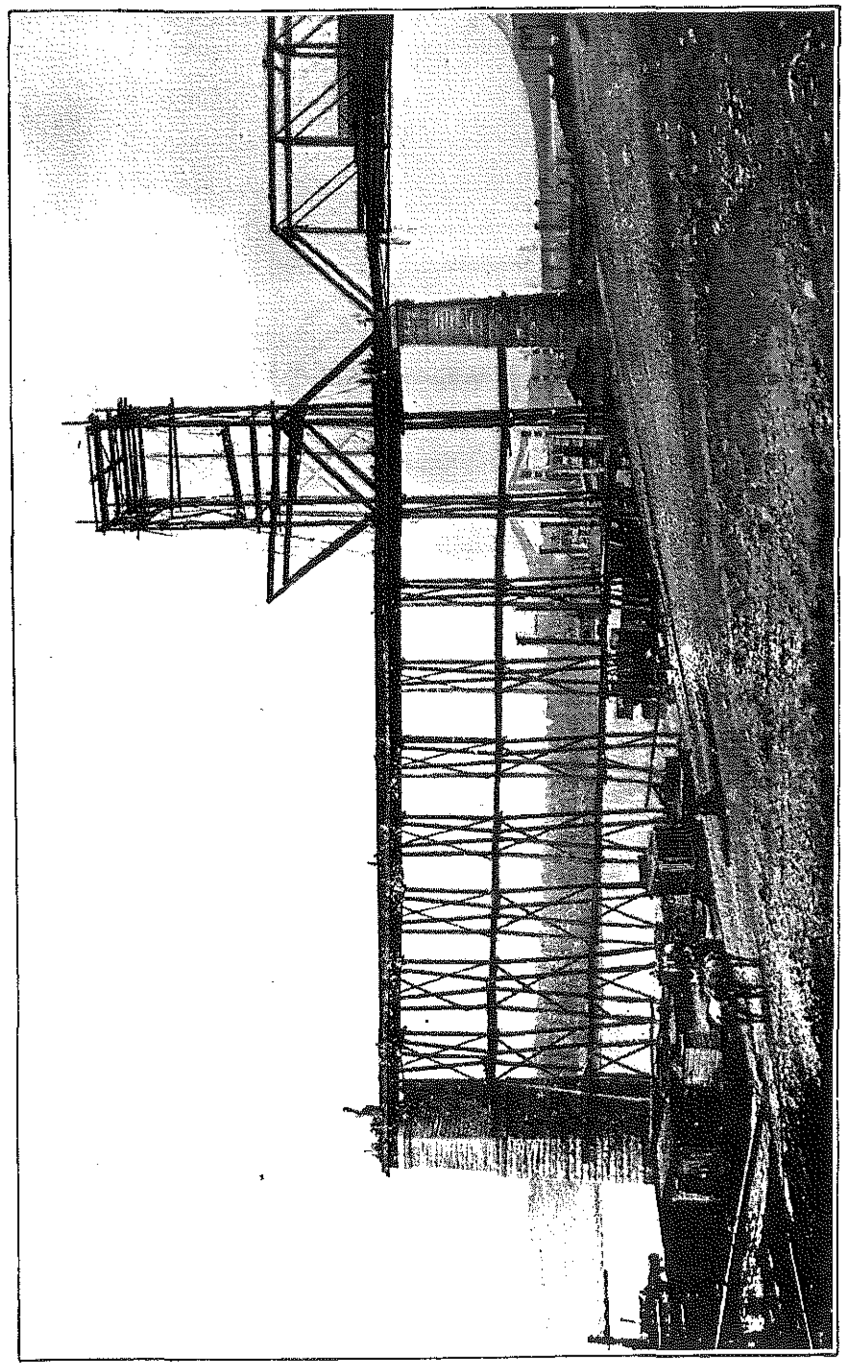
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



ANCHORAGE ARM OF CANTILEVER, CINCINNATI SIDE, JUNE 30TH, 1891.

stone ashlars not less than 18 inches thick, capped with a single block of limestone not less than 18 inches thick. They shall be founded on a bed of concrete, or on a pile foundation, as the nature of the ground may require.

Piers.—Numbering from the Cincinnati side, Piers Nos. 1, 2, 3, 4 and 9 shall be founded on pile or concrete foundations, the bottom of the masonry being from 6 to 10 feet below the surface of the ground.

Piers Nos. 5, 6, 7 and 8 shall be founded on the bed rock of the river. The rock shall be excavated from 4 to 6 feet in depth, and properly dressed to receive the first course of masonry; the spaces between the side walls of the pit and the masonry of the piers shall be filled with concrete to an even elevation with the bottom of the river.

Piers Nos. 1, 2, 3 and 9 shall be rectangular in shape, in a horizontal section, with a batter of one-half inch to the foot on all faces.

Piers Nos. 4, 5, 6, 7 and 8 shall also be rectangular in shape from the top of coping down to the elevation of high water. From high water down they shall have a semi-circular nosing at each end, as shown on plans.

The general dimensions for each pier shall be approximately as follows:

	Coping.	Heights.		Total.
Pier 1,	8 x 32	..	..	23 feet.
" 2,	8 x 32	..	..	34 "
" 3,	8 x 32	..	..	42 "
" 4,	11 x 36	29	41	70 "
" 5,	12 x 36	34	79	113 "
" 6,	12 x 36	34	71	105 "
" 7,	11 x 36	29	70	99 "
" 8,	9 x 32	21	70	91 "
" 9,	9 x 32	..	..	36 "

The masonry of Piers Nos. 1, 2, 3 and 9 shall be of freestone ashlars not less than 16 inches thick; the coping shall be no less than 18 inches thick, and of approved stone.

The masonry of Piers Nos. 4 and 8 shall be of Berea sandstone ashlars for the face, with concrete hearting below high water.

Piers Nos. 5, 6 and 7 shall be of Berea sandstone facing and Ohio River freestone hearting below high water.

The masonry of Piers Nos. 4, 5, 6, 7 and 8 shall be composed entirely of freestone ashlars above high water to the under side of coping. The depth of the courses shall not be less than 16 inches. The coping courses shall be of limestone of approved quality not less than 24 inches thick for Piers Nos. 5 and 6, and 18 inches thick for piers Nos. 4, 7 and 8.

Each pier shall have one or more footing courses. The anchorage for the ends of the Cantilever spans on Piers Nos. 4 and 7, as well as the anchor bolts in pedestals of the viaducts, shall be put in by the contractor for the substructure; the iron work for the same shall be furnished by the Bridge Company.

General.

All coffer-dams and scaffoldings used for the construction of the piers, as well as all surplus material excavated for the foundations of the same, shall be removed by the contractor before payment of the final estimate. The material excavated from the foundations of the river piers shall be deposited in such a place as not to cause any obstruction in any portion of the river. The place of deposit shall be satisfactory to the proper authorities.

Piers and pedestals shall be built of first-class masonry. The abutments shall be of second-class masonry. All joints of the masonry shall be neatly pointed off with rich cement mortar. No masonry shall be laid in freezing weather without permission of the Engineer.

All materials shall be inspected, and shall be used only when approved and accepted by the engineer. All work shall be done under the direction and to the acceptance of the engineer. All defective work shall be promptly taken down by the contractor on orders from the engineer, and rebuilt properly at the contractor's expense.

In the absence of the contractor from any part of the work, the engineer shall give his orders respecting that work to whomsoever it may be in charge of, or executing the said work, and his orders shall be respected and obeyed. The contractor assumes all risks arising from the weather, accidents or causualties of any kind.

Masonry details shall be prepared by the engineer for each structure, and a copy of the same shall be furnished the contractor before beginning the work.

Masonry shall be divided into two classes: first and second-class masonry.

First-class Masonry.—For Piers Nos. 4 and 8: The thickness of the courses shall vary from 16 inches thick to 30 inches thick, and the courses shall decrease uniformly in thickness from the bottom to the top of piers. The piers shall consist of alternate courses of headers and stretchers. They shall not be less than 3½ feet nor more than 6 feet long, and shall be no less than 16 inches nor more than 30 inches thick, nor less in width than one and one-quarter times the depth of the course to which they belong.

The casings shall be of Berea sandstone and the piers shall be filled with concrete made of Louisville selected and inspected cement. The casings shall be laid in alternate courses of headers and stretchers; the face stones shall be square; the joints shall be three-eighths of an inch in thickness. The vertical joints three-eighths of an inch in thickness shall extend backward from the face of the wall no less than 12 inches, and as much more as the stone will admit. The concrete filling shall be placed in the pier upon the completion of each course of the casing. It shall be mixed and deposited in place as specified under the head of "concrete."

All face stone must hold their size back in the heart of the wall that they show on the face.

All stone must lie on their natural quarry bed, and must be cleaned carefully and dampened before setting. They must have their beds and joints well dressed, and true to the proper plane. The beds shall be made as large as the stones will admit of. All face stones shall break joints not less than 12 inches.

No hammering on the stone will be allowed after it is set, but small inequalities may be pointed off carefully.

The masonry shall be rock faced, with no projections of more than 3 inches from the proper plane.

The belting and coping courses, as well as quoins, shall have drafts 1½ inches wide. The coping stones shall have parallel joints dressed throughout. They shall be of such dimensions as may be required by the engineer. They shall be tied together with iron clamps made of seven-eighths inch square iron; they shall extend 9 inches within the edge of each stone and their points shall extend 4 inches into each stone; the clamps shall be set in lead.

First-class Masonry for Piers Nos. 5, 6 *and* 7.—The piers shall consist of headers and stretchers, and there shall be at least one header to every three stretchers, or more frequently if necessary in the opinion of the engineer.

Headers and stretchers shall not be less than 3½ feet nor more than 7 feet long, according to thickness, nor less in width than one and a quarter times the depth of the course to which they belong.

The thickness of the courses shall not be less than 16 inches nor more than 30 inches, and they shall decrease uniformly from the bottom to the top of walls.

Face stones must hold the size back in the heart of the wall that they show on the face.

All stones must lie on their natural quarry bed and must be cleaned carefully and dampened before setting. They must have their beds and joints well dressed and true to their proper plane.

The beds shall be made as large as the stones will admit; the vertical joints of the face must be in contact at least 6 inches measured in from the face. The face stones shall break joints not less than 12 inches. The backing shall be of good-sized, well-shaped stones, laid so as to break joints, and thoroughly bond the work in all directions. The joints shall not be less than three-eighths of an inch nor more than five-eighths of an inch thick. There shall be no spaces larger than 6 inches between the backing stones; they shall be filled with small stones laid flush in cement mortar. The whole of the masonry shall be laid flush in cement mortar, so as to fill thoroughly all joints, beds and spaces between stones. To remove all doubts as to this point, each course shall also be grouted, if required by the engineer.

No hammering on the stone will be allowed after it is set, but small inequalities may be pointed off carefully.

The masonry shall be rock faced, with no projections of more than 3 inches from the proper line.

The belting and coping courses, as well as all quoins, shall have drafts 1½ inches wide. The coping stone shall have parallel joints dressed throughout. They shall be of such dimensions as may be required by the engineer. They shall be tied together with iron clamps made of seven-eighths of an inch square. They shall extend 9 inches within the edge of each stone, and their points shall extend 4 inches into each stone; the clamps shall be set in lead.

First-class Masonry for Piers Nos. 1, 2, 3 *and* 9.—Shall be built entirely of Ohio River freestone. The piers shall consist of headers and stretchers, and there shall be at least one header to every three stretchers, and more frequently if necessary in the opinion of the engineer.

Headers and stretchers shall not be less than 2½ feet nor more than 6 feet long, according to thickness, and not less than 1½ feet wide, nor less in width than the depth of the courses to which they belong. In all other respects the specifications governing the construction of Piers Nos. 5, 6 and 7 shall be used in the construction of Piers Nos. 1, 2, 3 and 9.

Second-class Masonry.—Face stones shall not be less in thickness than specified for on each piece of work.

Joints.—Vertical joints on the face must be in contact at least 4 inches measured in from the face, and as much more as the stone will admit of.

Backing.—Shall be of large, well-shaped stones, having good natural or scabbled beds, the thickness corresponding to the face stones of the same course. Bond of face and backing stones shall not be less than 12 inches.

In all other respects second-class masonry shall be constructed as specified under the heads of "First-class Masonry" for Piers Nos. 5, 6 and 7.

Concrete.—Concrete shall be composed by actual measurement of four measures of broken stone of uniform size, not more than 2½ inches in any direction, free from clay and soapstone and well screened, two measures of sand and one measure of cement. The broken stone is to be well watered, and stone and mortar thoroughly turned and mixed on a tight plank floor immediately before using until every stone is coated with the mortar.

All concrete is to be laid in sections or layers, not exceeding 9 inches in thickness, and is to be thoroughly rammed. It must be mixed so dry that the water will not flush to the surface until the ramming is nearly completed. The ramming must be completed within fifteen minutes after the water has been mixed with the cement. Concrete shall be allowed at least twelve hours to set before masonry is laid on it.

Pile Foundations.—The piles shall be of white oak, not less than 8 inches in diameter at the small end and 12 inches at the butt end, with the bark peeled off. They shall be straight and carefully pointed or shod with approved cast or wrought iron shoes, if required by the engineer.

They shall be driven to such a depth as the engineer may direct. They shall be driven with suitable rings and with a heavy hammer, with short falls, if necessary, to avoid splitting. All piles badly split or otherwise injured in driving, or driven out of place by the fault of the contractor, shall be replaced with others at the contractor's expense.

The piles shall be cut off level 2 feet below the bottom of the masonry, and 1 foot above the bottom of excavation. The pit shall then be filled with concrete to the level of the top of the piles, and the piles capped by 12 x 12-inch timbers of one length drift-bolted to the piles with iron drifts 1 inch in diameter. The spaces between the caps shall be filled with concrete, and the platform made of 12 x 12-inch timbers laid closely together.

After the first course of masonry is laid, the spaces between it and the sides of the excavation shall be filled with concrete to a height of 12 inches above the top of the platform. After the masonry has been carried above ground, the remainder of the pit shall be filled with the material excavated, well rammed in, and the pavement, if any, that was removed for the excavation, shall be carefully relaid and left in as good condition as it was before.

The spaces between the walls of ramps shall be paved, guttered and curbed with granite in accordance with the specifications in force for similar work in the City of Cincinnati.

Materials.

Stone.—The freestone shall be equal to the best quality of what is known as the Ohio River freestone, procurable on the river between Maysville and Portsmouth. The limestone shall be equal to the best quality of Indiana compact limestone, procurable near North Vernon and Greensburg, Ind.

All stone must be sound, of sufficient strength to stand the required pressure without danger of crushing, not liable to be affected by the weather, and shall be thoroughly seasoned before using.

Cement.—The cement shall be equal to the best quality of Louisville hydraulic cement. It shall stand, without breaking, a tensile strength of not less than 100 pounds per square inch in briquettes seven days old. It shall not swell or crack in the process of hardening.

Sand.—The sand shall be clean, sharp river sand, properly screened from all dirt, clay, soapstone, or other impurities.

Mortar.—The cement mortar shall be generally composed of one measure of cement to two measures sand well mixed with clear water in clear water beds and used immediately after mixing. Different proportions of sand and cement shall be used when required by the engineer.

Timber.—All the permanent timbers used in the foundations shall be of sound white oak cut from living trees, free from worm holes, dry rot, decayed and loose knots, wind shakes, and all other defects impairing its strength and durability. It shall be sawed true and of full size. Sap angles measuring over 1½ inches on the face shall not be allowed.

Iron.—All wrought iron in bolts, spikes, clamps, pile-shoes and other parts used in the substructure, shall be of the best quality of tough, ductile metal, that will stand 50 000 pounds tension per square inch before breaking, with 15 per cent. elongation in specimens three-quarters of an inch square, 12 inches long, and bend cold 180 degrees on a circle 1½ inches in diameter.

All cast iron shall be of tough, gray metal that will stand 18 000 pounds tension per square inch before breaking.

APPENDIX II.

SPECIFICATIONS FOR THE CENTRAL RAILWAY AND BRIDGE COMPANY'S HIGHWAY BRIDGE OVER THE OHIO RIVER FROM CINCINNATI, OHIO, TO NEWPORT, KY.

Form of Truss.—The form and dimensions of trusses shown on the general drawings will be satisfactory.

Strain Sheets.—The strain sheets submitted must show for each member of the truss and for cross bracing the maximum live and dead load stresses sustained, together with the wind stresses in the top and bottom chord and end brace, the dimensions and area of cross-sections, the kind of metal used, also the dead load assumed in the calculations, which must not be less than the actual weight of the structure.

Detail Drawings.—Tracings of complete detail drawings must be submitted for approval and be approved by the engineer before work is commenced. A copy of every approved strain sheet and drawing shall be furnished the engineer within ten days after its approval, and all working drawings required by the engineer will be furnished free of cost.

Material.—All parts of the structure shall be of wrought iron or steel, except washers, separating spools (over 1 inch in thickness) and ornamental work, which may be of cast iron, and the flooring hereinafter specified.

Clearance.—All through spans shall have a clear height of not less than 16 feet above the top of roadway floor, a clear width between trusses of 24 feet and two sidewalks 7 feet in the clear. The clear width between guard rails shall not be less than 22 feet.

Temperature.—Provision must be made in all parts of the structure for the expansion and contraction corresponding to a variation of 150 degrees Fahr. in temperature.

FLOOR.

Depth.—The depth of floor from top of roadway to lowest point of iron work shall not exceed 5 feet.

Plan.—The floor beam may be riveted to Zee iron or steel posts by means of gusset plate above pin; or, if the posts are composed of channels or built up with plates and angles, the webs of floor beams can pass through slots in the posts and be riveted to angles on the interior of same. Tension on rivet heads must be avoided.

Bottom Laterals.—The bottom laterals must be attached directly to the bottom chord pins by wing plates or by other effective means. If a stiff system is used the members must be riveted at their intersections; if rods are used they must be securely clamped together.

Roadway.—The roadway shall consist of six lines of iron stringers riveted to floor beams. These stringers shall be covered with cross timbers of suitable dimensions, spaced at not more than 30 inches centers. These cross timbers to be covered with two layers of planking, the lower course being 3 inches in thickness and not exceeding 8 inches in width, laid longitudinally with one-fourth-inch open joints. The top course will be 2½ inches thick, laid transversely and with close joints, excepting that adjacent to track rail there will be one longitudinal strip 8 inches in width; the remaining top course of planking must not exceed 6 inches in width. Each cross-tie must be notched over the stringers at least one-half inch, and be securely fastened to the outside flanges of outside stringers by five-eighths-inch hook bolts and in addition at two intermediate stringers, and so arranged to alternate in each consecutive tie. These hook bolts must be provided with a wrought-iron washer under nut. The cross-ties must extend without break over all stringers. The bottom course of planking shall be securely fastened to cross-ties by wrought-iron spikes at least $7 \times \frac{3}{8}$ inches, two at each end and at alternate edges over each cross-tie. The top course of planking will be securely fastened to the bottom course by fifty-penny nails of approved quality.

Guard Rail.—The wheel guards will be 8×8 inches, supported by blocks $3\frac{1}{2}$ inches high, 10 inches wide and 10 inches long, spaced center to center a distance of 5 feet, or the distance center to center of every other cross-tie. These blocks are to be beveled at one end to a width of 8 inches at their top; the guard rail will be beveled on lower inner edge not less than 1 inch between faces of blocks; these blocks will be supported directly on the lower course of planking. The guard rail, block, lower course of planking and cross-tie shall be securely fastened by a three-quarter-inch bolt, provided with wrought-iron washers above and below.

Sidewalk.—The sidewalk floor shall consist of one course 6×2-inch planking laid with one-fourth-inch open joints on cross-ties of suitable size, spaced not more than 2 feet on centers, and laid transversely on iron stringers; separating strips must be used at least 6 inches long, one-fourth inch thick and 2 inches in width over each cross-tie between each line of planking. These separating strips are to be securely nailed to edge of planking. The cross-ties shall be secured to top flanges of outside stringers by five-eighths-inch hook bolts provided with wrought-iron washers. The flooring of the sidewalk shall be fastened with forty-penny nails—two at each end of planks, and at alternate edges of each consecutive cross-tie. Suitable provision must be made for completely boxing in the lower chord of spans.

Sidewalk Railing.—A substantial iron railing, not less than 4 feet in height and of approved design, shall be erected on the outer line of sidewalks the entire length of the bridge and approaches. The railings on superstructure shall have posts resting directly on the floor beam, and be securely braced thereto by outside braces of proper inclination, and extending nearly to the top of posts. The railings over the approaches shall be securely fastened by bolts let into the masonry and properly leaded. Intermediate stays shall be provided at center of panels.

Lighting.—Provision for lighting the entire structure must be made, using lamps and posts of design approved by the engineer.

Gas Main.—Suitable provision must be made for carrying under the sidewalk one line of 15-inch gas main.

Loads.

Dead Loads.—The structure shall be proportioned to carry the following load, viz.:

First.—The weight of iron and steel in the structure, the weight of iron being assumed at $3\frac{1}{3}$ pounds for 12 cubic inches and the weight of steel 2 per cent. heavier.

Second.—The weight of wooden floor, considering each foot B. M. to weigh $4\frac{1}{2}$ pounds for white oak. No extra allowance need be made for spikes, and railings may be assumed to weigh 30 pounds per lineal foot each. Track rails may be assumed to weigh 80 pounds per lineal foot of bridge.

The total dead panel loads will be distributed at top and bottom points as follows:

1st. On loaded chords:

(*a.*) One-half load resulting from weight of trusses.

(*b.*) The panel loads resulting from weight of lateral system in the plane of the chord.

(*c.*) One-half the weight of sway system at panel points where occurring.

(*d.*) The panel loads resulting from weight of wooden floor, floor beams, stringers, sidewalk brackets, sidewalk railings and track rails.

2d. On unloaded chords:

(*a.*) One-half the panel load resulting from weight of trusses.

(*b.*) The panel loads resulting from weight of lateral system in the plane of the chord.

(*c.*) One-half the weight of sway system at panel points where occurring.

Live Loads.—For all truss members receiving more than one panel load 75 pounds per square foot of clear roadway and sidewalks for 254-foot spans; 80 pounds per square foot for suspended span and cantilever arms; 85 pounds per square foot for 162-foot span; 100 pounds per square foot for 108-foot span. For stringers, floor beams, long suspenders and iron trestle 100 pounds per square foot of clear roadway and sidewalks, or an Aveling & Porter 15-ton steam road roller.

In the calculation of stresses the following conditions of live load will be assumed:

For main truss members, the roadway and both sidewalks will be considered loaded.

For trestle legs and long suspenders, roadway and one sidewalk only will be considered loaded, and for floor beams, the roadway will be considered loaded with sidewalks unloaded, also roadway unloaded with sidewalks loaded.

ALLOWED STRESSES.

Allowed stresses per square inch in pounds for different members will be as follows:

Tension Members:

	Wrought Iron.	Steel.
Eye-bars and counters,	$10\,000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$	$12\,000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$
Shapes and angles (net section),	10 000	12 000
Lateral rods,	18 000	

Compression Members:

	Wrought Iron.	Steel.
Square ends,	$9\,000-30\frac{l}{r}$	$15\,000-60\frac{l}{r}$
One square and one pin end,	$9\,000-35\frac{l}{r}$	$15\,000-70\frac{l}{r}$
Pin ends,	$9\,000-40\frac{l}{r}$	$15\,000-80\frac{l}{r}$

Lateral Struts:

In which l equals distance between supports in inches, r equals least radius of gyration in inches,

$$11\,000-50\frac{l}{r}$$

Flanges of floor beam, stringers and plate girders	Tension—Same formula as for eye-bars and counters.

Compression:

In which l equals unsupported length in inches, b equals width of flange in inches,

$$\frac{10\,000}{1+\dfrac{l^2}{5\,000\,b^2}}$$

Alternate Tension and Compression:

For compression only. Use compression formula.

For greater stress,

$$\text{Wrought Iron. } 10\,000\left(1-\frac{\text{max. lesser stress}}{2\text{ max. greater stress}}\right) \quad \text{Steel. } 12\,000\left(1-\frac{\text{max. lesser stress}}{2\text{ max. greater stress}}\right)$$

Use the one giving the greater area of section.

	Wrought Iron.	Steel.
Shearing: On webs of floor beams, stringers and plate girders	4 000	
Pins and rivets	6 000	8 000
Bearing: On diameter of pin holes	12 000	18 000
On diameter of rivet holes	12 000	15 000
Bending: Stress in extreme fiber of pins	15 000	21 000
Field Rivets.—All field rivets must be iron, and provision will be made for 50 per cent. in excess of above requirements.		
Timber: On extreme fibers in bending; tension and compression	1 200	
On bearing surfaces	400	

Wind.—Wind strains shall be calculated:

1. For a pressure of 30 pounds per square foot on the exposed surfaces of both trusses and railings, and a moving load surface of 6 square feet per lineal foot of bridge.

2. For a wind pressure of 50 pounds per square foot on the exposed surfaces of both trusses and railings, the direction of wind giving larger surface being assumed in the calculations. The greatest results shall be taken in the proportioning of parts.

Lateral Struts.—Lateral struts will be proportioned to resist the resultant due to an initial stress of 10 000 pounds per square inch upon all rods attached to them, when this is in excess of wind stress. The fiber stress due to weight of strut must be considered and be deducted from the unit stress specified.

Camber.

All spans shall have an estimated camber of $d = \frac{l^2 s}{ch}$, in which d is camber in inches, l equals length of span in feet, s equals mean stress per square inch on chords in tons of 2 000 pounds, h equals depth of truss in feet, and c equals $900 + 8.4$ span for spans under 250 feet, and c equals 3 000 pounds for spans over 250 feet.

Paint.

All iron or steel, before leaving the shop, shall be cleaned from all loose scales and rust, and be given one good coat of pure linseed oil. All surfaces in contact with each other shall receive one coat of oxide of iron paint before assembling, and all planed or turned surfaces shall be coated with white lead or tallow. After erection all iron and steel work shall be thoroughly and evenly painted with two coats of paint of such quality and color as the engineer may select.

Plate Girders.

Compressed Flanges.—The compressed flanges shall be stayed transversely when their length is more than twenty-five times their width.

Web Splices.—All joints in webs shall be spliced by a plate on each side of web.

Flange Area.—No part of the web will be considered as available in flange area. The web is assumed to sustain shear only.

Stiffeners.—All web plates shall have stiffeners at the inner edges of end-bearing plates and at all points of local concentrated loadings. Intermediate stiffeners will be used if the shearing stress per square inch in web exceeds $\frac{10\,000}{1 + \frac{1}{3\,000}\left(\frac{d}{t}\right)^2}$, in which d equals clear depth between flange angles or clear distance between stiffening angles, and t equals thickness of web.

Details of Construction.

Unsymmetrical Sections.—Sections composed of two rolled or riveted channels and one plate shall have the center of pin, in all cases, in the center of gravity of the section, and any abutting members shall be so proportioned that the centers of pins shall be on the same line. All eccentricity of stress shall be avoided. Provision must be made in all chord sections for bending from its own weight.

Bottom Laterals.—The bottom laterals will be attached directly to the bottom chord pins and not to the floor beams.

Top Lateral Struts.—The top lateral struts will have the full depth of the chord, and be securely riveted thereto.

Portals.—The portals will consist of top and bottom struts connected by cross bracing. The struts will be of neat design, be provided with ornamental brackets, and be securely attached to the trusses. The portal at each end of bridge will be provided with a name plate of approved design having such appropriate inscription as may be directed.

Top Lateral Rods.—Top lateral rods will be attached directly to the chord pins by wing plates or other effective means.

Rollers.—All spans shall have at one end nests of turned friction rollers of wrought iron or steel bearing upon planed surfaces. The rollers shall not be less than 2½ inches in diameter, and the pressure per lineal inch of roller shall not exceed 700 $\sqrt{d}$ for wrought iron or 900 $\sqrt{d}$ for steel, d representing the diameter of roller in inches.

Bed Plates.—Bed plates shall be of sufficient thickness to transmit the pressure on them uniformly to the rollers or masonry, as the case may be. It will be sufficient in determining this thickness to consider the plate having the load upon it uniformly distributed over its entire bearing surface, a continuous beam of uniform section over the walls of shoe as points of support, span lengths being taken as distances center to center of walls and from outer edge of plate to center of outer wall.

Bending moments must be taken at the center of each span and at a section cut by a plane normal to plate and parallel to walls of shoes through center of gravity of angles supporting walls to plate. The maximum bending moments must be taken. An extreme fiber strain of 18 000 pounds is allowed for steel and 15 000 pounds for iron. In no case shall the bed plates be less than 1 inch thick. The bed plates shall be so proportioned that the pressure upon masonry will not exceed 300 pounds per square inch.

Roller Plates.—Roller plates must not be less than seven-eighths of an inch thick.

Bolsters.—There will be wrought iron or steel bolsters at each end of bridge, securely anchored to the masonry, proper provision being made for expansion.

End Braces.—End braces will have pin bearings at both ends.

Long Tension Braces.—Long intersecting tension braces shall be clamped together at intersections to avoid rattling.

Long Compression Members.—In built posts and struts of trusses the angles shall be of one length without break, but the web plates may be spliced at intermediate supports if desired.

Sub-struts and Diagonals.—Sub-struts and overhead diagonal bracing will be provided at each vertical post in through spans, when depth of truss center to center of pins is 30 feet or over.

Vertical Suspenders.—All vertical suspenders will be designed to resist compression. If eye-bars they will be stiffened by zigzag bracing or otherwise to avoid vibration.

Trestle Towers.—The base of trestle bents shall be sufficient to avoid tension under the highest wind specified and sufficient anchorage shall be provided to resist not less than one-half the overturning moment. The trestle bents shall be united in pairs to form towers and each tower thus formed shall be thoroughly braced in both directions. Cross-section and longitudinal struts shall be provided at bottom and at intermediate joints; also at top in the absence of floor beam or girders acting as such.

Raising Appliances.—At the foot of all towers and under bolsters of spans provision shall be made either by lengthening the pins, or by suitable lugs, for raising the structure to make any necessary repairs. These lugs shall be designed to resist the total weight of the structure and 1 200 pounds per lineal foot of bridge.

Effective Section of Members Built of Angles.—Whenever a member is composed of angles, both flanges of angles must be connected, else only one flange will be considered as effective.

General Conditions.

Punching.—In punching rivet holes in iron, the diameter of the punch shall not exceed the diameter of the rivet by more than $\frac{1}{16}$th of an inch, and the diameter of the die shall in no case exceed the diameter of the punch by more than $\frac{1}{16}$th of an inch.

Punching and Reaming.—All rivet holes in steel work in the cantilever span shall be reamed. In the other spans all holes that are not fair when parts are assembled shall be reamed at the option of the inspector. In punching steel the play between die and punch shall not be more than $\frac{1}{16}$th of an inch. In all reamed work the size of punch shall be one-eighth of an inch less than diameter of rivet to be used, and hole shall be reamed to $\frac{1}{16}$th of an inch larger than diameter of rivet. One-sixteenth of an inch must be taken out of all parts of the hole in reaming. Sharp edges of reamed holes shall be so trimmed as to make a slight fillet under the heads.

Effective Diameter of Rivets.—The effective diameter of the driven rivets in reamed holes will be assumed $\frac{1}{16}$th of an inch larger than its diameter before driving, and in making deductions for rivet holes in tension braces the same allowance will be made. In iron, this allowance must be one-eighth of an inch more than the diameter of the rivet.

Pitch of Rivets.—The pitch of rivets shall not exceed 8 inches nor be less than three diameters of the rivet. At the ends of compression members the pitch shall not exceed four diameters of the rivet for a length equal to twice the depth of the member. In stringers, where the cross-ties rest directly on the flange, the pitch of rivets must be the same throughout as at the ends.

Distance from Center of Rivet to Edge of Plate.—The distance from center of rivet to edge of plate shall not exceed eight times the thickness of plate nor be less than one and one-half times the diameter of the rivet.

Distance between Rivets in Compression Members.—The distance between centers of rivets or plates strained in compression shall not exceed twenty times the thickness of plate in line of stress nor forty times the thickness of plate at right angles to line of stress.

Length of Compression Members.—No compression member shall have a length exceeding forty-five times its least width.

Least Thickness of Plates.—No plate or shape shall be less than one-fourth of an inch thick when both faces are accessible for painting, nor less than $\frac{5}{16}$ths of an inch thick if only one face is accessible.

Bearing Plates.—All pin holes shall be reinforced, when necessary, so as not to exceed the allowed pressure on the pins, and the reinforcing plates must be provided with a sufficient number of rivets to transfer the pressure which comes upon them.

Tie Plates and Splice Plates.—The open sides of all compression members composed of two channels only, and trough-shaped sections composed of two channels and one plate, shall be stayed by tie plates at ends and diagonal lacing bars at intermediate points. The tie plates shall be square. Intermediate joints in the top chord shall be provided with tie plates at bottom, and side plates of sufficient length to hold the parts truly in position.

Lacing.—The sizes of diagonal lacing bars shall be as follows:

On the cantilever span 4 x $\frac{3}{8}$ inches.

For other spans:

$1\frac{1}{2}$ x $\frac{1}{4}$-inch	for members	having a depth of	6 inches and under.	
$1\frac{3}{4}$ x $\frac{1}{4}$	"	"	" 7 to 8 inches.	
2 x $\frac{5}{16}$	"	"	" 9 to 12	"
$2\frac{1}{4}$ x $\frac{3}{8}$	"	"	" 13 to 16	"
$2\frac{1}{2}$ x $\frac{7}{16}$	"	"	" 17 to 20	"
$2\frac{1}{2}$ x $\frac{1}{2}$	"	"	" 21 inches and upward.	

The distances between connections of the lacing bars shall not exceed eight times the least width of the segments connected, and in no case shall exceed an angle of 60 degrees.

Area of Rods.—No lateral or diagonal rod shall have a less area than three-quarters of a square inch.

Upset Rods.—The area at root of thread in the upset ends of rods shall be greater than the area of the rod by at least 17 per cent.

Weight of Member.—For all horizontal or inclined compression members the weight of members shall be considered, and in fixing sections the fiber stress due to weight of member shall be deducted from unit stress allowed by formula. Tensile stress shall be avoided in a transverse direction to the fiber, and shearing stress in a parallel direction to the fiber of the iron.

Effect of Wind.—If the strain from the wind in the chords or a possible temperature strain should neutralize or reverse the strain in the chord from the dead load, provision must be made for same; and if the combined strains from the dead, live and wind loads in the chords exceed 25 000 pounds per square inch, additional section must be added until the above allowed unit strain is not exceeded. Again, provision must be made in all built members for bending for wind. If the strain per square inch in such members, due to bending from wind, combined with the direct strain per square inch from dead, live and wind loads, exceeds 25 000 pounds per square inch, additional section must be added until this allowed unit strain is not exceeded.

Washers and Nuts.—Washers and nuts shall have a uniform bearing. All nuts shall be easily accessible with a wrench for the purpose of adjustment, and shall be effectively checked after the final adjustment.

Lateral Adjustment Rods.—All lateral and adjustment rods shall be provided with open turn-buckles so that the length of thread may be verified.

Wing Plates.—The amount of metal immediately in front of pin hole in wing plate is to be determined in the following manner: The shearing area is to be considered as a section of twice the thickness of wing plate multiplied by the distance parallel to line of stress, from edge of plate to a point which is the intersection of a chord, equal to one-half the diameter of pin hole and taken normal to the line of stress, with the circumference of pin hole.

Details.—Details shall be of such nature that their strength can be accurately calculated, which strength shall be at least equal to that of the member or members which they are designed to connect.

Workmanship.

Pins and Pilots.—Pins shall be turned true to size and straight. They shall be turned down to a smaller diameter at the ends and be driven in place with a pilot nut when necessary to save the thread. There shall be a washer one-half an inch thick under each nut.

Inspection.—The inspection of work shall be made as it progresses, and at as early a period as the nature of the work permits.

All workmanship must be first-class. All abutting surfaces of compression members, except flanges of plate girders where the joints are fully spliced, must be planed or turned to even bearings so that they shall be in such contact throughout as may be obtained by such means. All finished surfaces must be protected by white lead and tallow. The rivet holes for splice plates of abutting members shall be so accurately spaced, that when the members are brought into position the hole shall be truly opposite before the rivets are driven. The chord pieces must be fitted together in the shops in lengths of at least four pieces and rivet holes in splice plates reamed while in position. Wherever it is impossible to ream together the parts which will come together in the field, the holes in both shall be reamed to an iron template.

When members are connected by bolts which transmit shearing strains, the holes must be reamed parallel, and the bolts turned to a driving fit.

Rollers must be finished perfectly round, and roller beds planed.

Rivets.—Rivets must completely fill the holes, have full heads concentric with the rivet, of a height not less than .6 diameter of the rivet, and in full contact with the surface, or be counter sunk when so required, and machine driven when practicable. Rivets must not be used in direct tension.

Built members must, when finished, be true and free from twists, kinks, buckles or open joints between the component pieces.

Eye-Bars and Pin Holes.—All pin holes must be accurately bored at right angles to the axis of the members, and in pieces not adjustable for length. No variation of more than $\frac{1}{32}$d of an inch will be allowed in the length between centers of pin holes; the diameter of the pin holes shall not exceed that of the pins by more than $\frac{1}{32}$d of an inch, nor by more than $\frac{1}{50}$th of an inch for pins under $3\frac{1}{2}$ inches diameter. Eye-bars must be straight before boring; the holes must be in the center of the heads and on center line of the bars. All links belonging to the same panel, when placed in a pile, must allow the pins at each end to pass through at the same time without forcing. No welds will be allowed in the body of the eye-bars, laterals or counters, except to form loops of laterals, counters and sway rods; eyes of laterals, sway rods and counters must be bored.

The heads of eye-bars shall be so proportioned and made that the bars will preferably break in the body of the bar rather than in any part of the head or neck. The form of the head and the mode of manufacture shall be subject to the approval of the engineer. A variation from the specified dimensions of the heads of eye-bars will be allowed in thickness of $\frac{1}{32}$d of an inch and in diameter of a quarter of an inch in either direction.

Thimbles or washers must be used wherever required to fill vacant spaces on pins or bolts.

Punching and Reaming.—Rivet holes must be accurately spaced; the use of drift pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to distort the metal about the holes; if the hole must be enlarged to admit the rivet, it must be reamed.

Steel Plates.—All steel plates must be straightened in the straightening machine and not by hammering.

Annealing.—In all cases where a steel piece in which the full strength is required has been partially heated, the whole piece must be subsequently annealed. All bends in steel must be made cold, or if the degree of curvature is so great as to require heating, the whole piece must be subsequently annealed.

Interpretation of Drawings and Specifications.—The decision of the engineer shall control as to the interpretation of the drawings and specifications during the execution of the work thereunder.

Distribution of Material.

All spans of the superstructure with the exception of the 108-foot span, the viaduct trestle legs and floor system in all spans shall be of steel (Class A). All rivets in steel work shall be of steel (Class B).

Quality of Material.

Wrought Iron.—All wrought iron must be tough, ductile, fibrous and of uniform quality for each class, straight, smooth, free from cinder pockets or injurious flaws, buckles, blisters or cracks.

The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece not less than one-fourth of an inch thick, cut from the full-sized bar, and planed or turned parallel. The area of cross-section shall not be less than one-half square inch. The elongation shall be measured after breaking on an original length of 8 inches.

All iron shall have a limit of elasticity of not less than 26 000 pounds per square inch.

All iron used in tension shall have an ultimate strength of not less than 50 000 pounds per square inch, and elongate not less than 18 per cent.

Angles and other shapes, and plates 24 inches wide and under, shall have an ultimate strength of not less than 50 000 pounds per square inch and elongate not less than 15 per cent.

Plates over 24 inches wide shall have an ultimate strength of not less than 46 000 pounds per square inch and elongate not less than 12 per cent.

When full-size tension members are tested to prove the strength of their connections, a reduction in their ultimate strength of (500 x width of bar) pounds per square inch will be allowed.

All iron shall bend cold 180 degrees around a curve whose diameter is twice the thickness of pieces for bar-iron and three times the thickness of pieces for plates and shapes without fracture.

Iron which is to be worked hot in the manufacture must be capable of bending sharply to a right angle at a working heat without sign of fracture.

All rivet iron must be tough and soft, and capable of bending cold until the sides are in close contact without sign of fracture on the convex side of the curve.

Steel.—Steel may be made either by the Bessemer or by the open-hearth process. All blooms, billets or slabs will be examined for surface defects, flaws and blow holes before rolling into finished sections, and such chippings and alterations must be made as will secure perfect solidity in the rolled sections.

The slabs for plates must, in all cases, be as nearly rectangular as possible and straight their whole length.

The steel must be uniform in quality for each class, and after heating to light cherry red (as seen in the dark) and quenching in cold water, shall comply with the bending requirements provided in this specification for such class of steel.

In order to grade the steel used in this work at the steel mills, the following form of selecting the test pieces shall be rigidly enforced. From every cast of metal there shall be made one test. If this test is satisfactory the whole cast may be accepted, subject to tests made on rolled sections. This test must be made from a three-quarter-inch round rolled from a 4-inch square billet, which has been reduced from original ingot; this billet to be taken during the blooming down of ingot, and reduced in such way that the reduction of section into three-quarters of an inch round will be as nearly as possible equivalent to reduction of section on finished material when rolled from original ingot. The manner and time of selecting this billet will be left to the convenience of the manufacturer.

In addition to this tension test a bending test will be required. The pieces used in this test may be either three-quarters of an inch round or three-quarters of an inch square, preferably the latter. This piece must bend cold 180 degrees about its own diameter for steel of Class A, and

180 degrees and close down flat upon itself for steel of Class B. The tests on three-quarter-inch round specimens, above mentioned, must satisfy the following requirements:

Class A.—Ultimate strength, 62 000 to 70 000 pounds per square inch. Elastic limit not less than 36 000 pounds per square inch. Elongation not less than 22 per cent. in 8 inches. Reduction at point of fracture not less than 40 per cent.

Class B.—Ultimate strength, 56 000 to 60 000. Elastic limit not less than 30 000. Elongation, 25 per cent. in 8 inches. Reduction at point of fracture, 50 per cent. Phosphorus in all steel Class A and Class B to be not over .085 of 1 per cent.

Tests at Rolling Mills.—The finished bars must be free from injurious flaws or cracks, and must have a smooth, clean finish.

At least one test will be required from every heat or furnace full of billets, slabs or blooms to prove condition of metal after rolling it into finished sections. This test piece must be cut from some rolled section of said heat and shall be generally one-half of a square inch in area and must conform to requirements as specified for three-quarter round at steel mills in every respect, except that ultimate strenght may vary 2 per cent. below minimum, and 5 per cent. above maximum.

The original number of heat or cast at steel mills must be stamped on all billets, blooms or slabs, and when rolled into finished sections this same number must be stamped on every piece rolled.

Number of Tests on Full-sized Eye-bars.—The method of making full-sized tests on eye-bars will be as follows: One full-sized test will be required in each size of bar, and if the number of bars of any size exceeds twenty, then one additional test will be required for each multiple of twenty or part thereof exceeding ten. The extra bars required for test must be ordered at the steel mills with the original order. When the Bridge Company have finished an item of bars as represented on the shop bill, the inspector shall then select from this lot of bars the bars for test, and if these tests are satisfactory the whole item may then be accepted. Should this first test fail to stand the requirements of this specification, and if in this test no blame attaches to the Bridge Company on account of poor work, the latter can demand to have two other tests made on this same item of bars, and should those two tests be satisfactory, the whole item may be accepted, and so on with other items until the whole order of bars is completed.

Heads of Eye-bars.—The manufacturers must provide sufficient excess of material in the heads of eye-bars to insure their breaking in the body rather than in the head, but any bar which breaks in the head at a stress higher than called for will be accepted; provided, only, that the elastic limit is up to specified requirements. The minimum ultimate strength of full-sized bars when tested to destruction will be 60 000 pounds per square inch. The minimum elastic limit shall be 34 000 pounds per square inch.

Castings.—All castings shall be of tough gray iron, free from injurious cold shuts or blow holes, true to pattern and of workmanlike finish. Sample pieces 1 inch square, cast from the same heat of metal in sand moulds, shall be capable of sustaining on a clear span of 4 feet 6 inches a central load of 500 pounds when tested in the rough bar.

General Requirements.

Any full-sized tension member of iron or steel tested to destruction shall be paid for at cost, less its scrap value to the contractor, if it proves satisfactory. If it does not stand the specified test, it will be considered rejected material and be solely at the cost of the contractor. The contractor shall furnish testing machine of the proper capacity, and shall prepare and test, without charge, such specimens of iron as may be required by the engineer or inspector to prove that it comes up to the requirements mentioned; he shall also furnish, prepare and test, without charge, such specimens of the several grades of steel, at steel mills and rolling mills, as may be required by the inspectors. Every facility for inspection of material and workmanship shall be furnished by the contractor, and the engineer or inspector shall be allowed full access to all parts of the establishment in which any portion of the materials are made or workmanship executed. Timely notice will be given to the engineer by the contractors when they are ready for the inspectors, and the inspectors will test and inspect the material at the mills as rapidly as it is made. All material must be inspected, weighed and stamped by the inspector before shipment. The acceptance of any material or manufactured member by the inspector shall not prevent its subsequent rejection if found defective after delivery, and such materials or members shall be replaced by the contractor without extra charge.

Engineer.—Wherever the term engineer is used throughout these specifications, it is distinctly understood that such term shall mean the chief engineers or their authorized assistants.

Timber.—All timber must be of the best quality of white oak, sawed true to size and out of wind. It must be free from sap, except in sticks having a depth of 16 inches or upward, when 1 inch of sap will be allowed on two corners. It must be free from wind shakes, loose or rotten knots or other defects that will impair its strength or durability.

Erection.—An available portion of the river shall at all times be left open to navigation, and proper lights shall be displayed at night in accordance with the regulations and requirements of the United States Government. The contractor will furnish all staging, piling, cribbing, and material of every description required in the erection of the superstructure, and remove the same after erection is completed, leaving the river as free from obstructions as when he commenced. The contractor shall assume all risks from floods and storms, damage to persons and properties and casualties of every description until the final acceptance of the completed structure.

The erection is to be carried on subject to the approval and inspection of the engineer, and is to be completed, ready for use, to his satisfaction.

APPENDIX III.

CENTRAL RAILWAY AND BRIDGE COMPANY'S SPECIFICATIONS FOR A CAISSON FOR PIER No. 5.

The caisson to be used in building the foundation for Pier No. 5 shall be 22 x 58 feet at top, 12 feet high over all, and have a batter of one-half inch per foot from a point 1 foot below the top to the cutting edge, thus making the bottom 22 feet 11 inches by 58 feet 11 inches.

All timber used in its construction must be of a good quality of sound white or poplar, free from rotten knots, shakes or other defects, and shall be of such dimensions as shown on the plans attached hereto and forming a part of these specifications. Courses Nos. 1, 2, 10, 11 and 12 must be of oak, and as much more of the entire timber as can be secured of oak, the intention being to use no other timber except as an enforced expedient to secure the speedy execution of the work. All framing must be accurately done so as to secure close joints throughout the work.

All the iron used in spikes, bolts, drift bolts and straps must be of a ductile, fibrous wrought iron, having a tensile strength of 50 000 pounds per square inch of section. Drift bolts must be driven into holes bored their full length, and of diameter $\frac{1}{16}$th of an inch less than the iron.

All seams between the timbers in the walls and ceiling of the air chamber, and in the outside walls, must be thoroughly caulked with oakum, and driven in with a heavy hammer. After the 3-inch sheathing is laid and thorougly spiked, the seams between the boards must also be thoroughly caulked with oakum, and wherever spike or bolt heads are to be exposed to the air pressure of the working chamber, they must be wrapped with oakum before being drawn up tight or driven home.

In order to decrease the leakage, and to secure more uniform bearing for the timbers, all the vertical seams in courses Nos. 9, 10, 11 and 12 must be poured full of a thin cement grout, mixed neat. After this is done, all the seams in course No. 12 must, in addition, be caulked with oakum.

Air lock and excavating shafts will be located as shown on the plans, and all seams in them must be thoroughly caulked. Provision must be made for carrying these shafts up through the masonry as fast as laid, either by cylinders of boiler iron or by matched oak boards capable of being made water-tight. Some approved form of iron air lock must be provided, and also excavating apparatus equal in efficiency to the "O'Conner bucket" for removing solid materials which may be encountered during the sinking.

On the sides and ends of the caisson heavy iron rings must be secured for use in suspending it during the preliminary stages of sinking, and for attaching lines to hold it in position against a swift current.

After reaching a depth satisfactory to the engineers, the air chamber and shafts must be filled with concrete made of one part by volume of cement, two parts of sand, four parts of limestone, rock broken so as to pass through a ring 2½ inches in diameter, and sufficient water to bring the mortar to proper consistency, the whole to be thoroughly rammed before the setting of the cement.

The triangular space between the air chamber and outside walls must also be filled with concrete, mixed in the proportions above stated, and as the superimposed course of timber is being laid, care must be taken that each stick is well bedded in a freshly mixed mortar spread on top of the concrete.