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# A summary and analysis of bridge failures

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# A SUMMARY AND ANALYSIS OF BRIDGE FAILURES

by

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TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

# LIST OF TABLES

# LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

# INTRODUCTION

## General

It is the general purpose of this study to acquaint the reader with a history of bridge failures. Some of these failures are very major with much loss in lives and dollars, while some are rather minor. However, it is frequently impossible to draw a line between major failures and minor failures when considering the lessons which may and should be learned. Lessons can be learned by a summary of all bridge failures and an analysis of the factors associated with those failures.

There are many failures which occur and are never heard about by the general public. They may be listed and described in detail, but kept forever in some secret file for various reasons. Some may be settled out of court with respect to responsibility, or the person responsible chooses to keep the failure secret to protect his reputation. Many failures are learned about through technical papers and magazines, particularly failures of a minor nature. Thus, only technical people become aware of them.

Experience can be a very expensive teacher but it is usually the best teacher. In reviewing past bridge failures it is natural that the ones with the greatest loss of life will probably be the ones to remain in the minds of the public, although the technical person must realize that all types of past failures must be given equal considerations as a recurrence may be under different circumstances.

The bridge failures summarized in this report are grouped under headings listing the type of failure. It is unfortunate that there is one cause of failure which exists probably more often than any other -- carelessness during construction -- which is an error which may always be present. Ignorance, however, may be a factor and there are times when the economics of the construction sacrifices many lives. The most critical period in the life of a structure is often during the construction period. There is a critical stage during construction, and after this stage the engineer can partially relax and be satisfied that his design is stable. Of course there are other tests which the structure must also face during its early performance. But after construction is satisfactorily completed, a very large part of the battle is won. Failures resulting not from insufficiencies of the structural design of the completed work but from unexpected movements and loadings during construction are, in the public mind, not distinguished from structural design failures. Such incidents occur quite often near the completion of a job when progress is at the maximum scheduled rate and manpower is not sufficient to provide all the necessary precautions against failure.

Failures are very often due to lack of inspection or the economics involved in a particular project which leads to carelessness or neglect. Many major bridge failures in this report are primarily caused by lack of inspection. The Kings Bridge in Melbourne, Australia is a well known and recent example where the designer failed to accept or demand responsibility of the field inspection (15, 45, 53). A very essential safeguard against failure is strict and competent supervision and inspection by skilled foremen, architects and engineers throughout the construction operation.

Much is known about structures today and failures of completed structures are not as frequent as they once were, at least those which are caused by improper design. Most of the failures in completed structures are a result of dishonest performance and noncompliance due to ignorance or a matter of economics rather than improper design.

Structural failures have occurred since the beginning of time and in all types of structures. Most of these failures are not publicly known nor are they known even by engineers who will design a structure using criteria which possibly has already been proven wrong or unsatisfactory in a failure in the past. Reasons for a collapse may not be determined before a lengthy investigation has taken place and by that time there is probably little interest remaining in the incident. The cause of minor accidents is seldom announced. There are more lessons which can be learned by the knowledge of past failures than by the successes. Therefore there should be reports of all types of failures -- failures not necessarily meaning collapse of a structure but using the engineering definition "...whenever a structure ceases to perform in a manner for which it was designed."

Following is a list of causes of failures which are very common and will become apparent to the reader in this summary:

1. Ignorance

    a. Incompetent men in charge of design, construction or inspection

    b. Supervision and maintenance by men without necessary knowledge or experience

    c. Lack of sufficient preliminary information

d. Revision of design by persons lacking knowledge of the original requirements

2. Economy

   a. Restrictions in initial cost

   b. Lack of maintenance

3. Carelessness during construction

   a. An engineer, usually competent and careful, shows negligence in some part of a design

   b. A contractor takes a chance while completely aware of the risks involved

   c. Lack of coordination in production of plans, construction procedures and inspection

4. Unusual occurrences

## Legal Responsibilities

Every time a failure occurs a legal finger must point to someone which may cause embarrassment and may even mean professional ruin for many. It is not the intention of this summary to point a finger at anyone, but only to familiarize the reader with the failures in some detail along with the cause, if available, so that there may be lessons learned.

The matter of where to place the legal responsibility is a factor which is very difficult to obtain, especially in the case of foundation failures. The responsibility is frequently placed predominantly with the engineer, even though the failure is the result of ignorance or negligence of the construction contractors. This is caused by the fact that any errors made during construction should be spotted by the engineer or his inspector on the job. Errors of judgment are no longer used as a defense. There is a well known phrase -- "A medical doctor buries his mistakes, an architect covers his mistakes with ivy, and the engineer must write a long report on his mistake."

There may soon be a need for a meeting of all the phases of the construction industry to clarify the limits of responsibility for project concept, design, detail, material production and assembly, construction direction, and supervision. If each performs his service properly by hiring the necessary experienced personnel, and if he does not attempt to do the job required of another, then there may be much greater success and freedom from failure.

## Methods of Control

A safety factor which has been developed over the years is the establishment of quality standards for materials and workmanship, and for design practice, by the American Society for Testing Materials, the American Welding Society, the American Concrete Institute, the American Institute of Steel Construction, and other similar organizations. Also becoming important is specialization brought about through registration. Quite often a person qualified to perform a specific service will attempt to perform services for which he is not qualified. This may not be apparent to the person hiring him for his services and is unethical as a member of his profession.

A technical control bureau has been established in Europe which was started as a result of a great number of construction failures occurring in the 1920's. This is a private organization set up to qualify projects for construction liability and damage insurance. This organization, which investigates ways to control construction practices, has control over field inspections and supervision and also approves designs. A technical control bureau would seem to be a practical way of maintaining standards of public safety while avoiding greater government control. Some advantages would be:

1. To unify the responsibility for control of a project

2. More efficient use of trained construction personnel could be made. A trained inspector could be stationed on a job, or visit a number of jobs on a coordinated schedule, rather than imposing sporadic interruptions in design office schedule when the engineer must perform job inspection.

3. Testing laboratory standards could be raised by abolishing price competition.

4. A direct financial motive for insuring good designs and conforming construction would be provided. Since the control bureau is tied in with an insurance plan which must pay off in cash in event of failure, motivation will be toward strictness of control, for example, the high standards of inspection now maintained by fire insurance companies.

5. It provides a technical staff trained to see projects with fresh and unbiased viewpoints.

This would seem to be a very desirable system and would eliminate many failures which recur time after time, indicative of the insufficiency of voluntary participation in promoting existing knowledge.

It is apparent that much improvement has been made when comparing the number of bridges built and the number of failures in recent years with those at the beginning of the twentieth century. There are many more specifications and building codes today to guide the designer and builder. These building codes have been based partly on experience learned from failures in the past and partly on experimental and theoretical investigations through the years.

## Objectives

Many lessons have been learned and will continue to be learned. By summarizing the past failures of bridges and the causes of the failures it is hoped that the same type of failure will not recur. It is, however, apparent that they do recur and are possibly caused by the same error. If designers, inspectors, owners, and builders would all become aware of the types of failures and their causes, a great advancement would be made toward safer structures. It is hoped that the engineering profession and the construction industry can learn by the mistakes of others and can thereby avoid unnecessary repetition of failures in the past. This report is written in the hope that by contributing to the better understanding of reasons for failure and a knowledge of bridge failures and types of bridge failures in the past, that future failures of the types discussed may be substantially reduced.

# FAILURES DUE TO DESIGN ERRORS

## General

There are six bridge failures summarized in this report which were caused mainly by error in design. This is one of the most common causes of collapse or failure.

Errors in design are many times a result of carelessness on the part of the designer and many times just a result of human error. Most of the design errors today are eliminated by the use of electronic computation methods and by a check of the design by a competent person before the detailed plans are distributed to the contractors for bids.

During the first 20 or 30 years of this century the designs were made by the steel fabricator or the bridge contractor. This would obviously result in more chance for error and would certainly place a greater burden of responsibility on the inspector. The designer would compute the allowable stresses for each member and these would be shown on a drawing of the structure. During the construction operation the inspector at the erection site would be required to calculate the actual stresses in each member as the erection progressed.

The following report is intended to make the reader more aware of the seriousness of design errors and the circumstances by which they occur.

### Second Narrows Bridge

There were 18 lives lost as a result of the collapse of two spans of the Second Narrows Bridge at Vancouver, B.C., during its construction on June 17, 1956 (61, 74, 92). The construction cost was estimated at $\$3\frac{1}{2}$ million at the time. This bridge was a $16 million structure which was to connect Vancouver and suburban Vancouver. The completed structure was to be 4250 ft. long and was being built for the provincial government as part of a $23 million express highway project included in the Trans-Canada highway network.

The collapse involved a cantilever portion which was being extended to reach a pier. This cantilever was being supported by temporary piers, one of which was in place at the time of the collapse. There was 345 ft. of cantilever beyond a main pier when, according to eye witnesses, the temporary supporting legs buckled. This impact moved the top of the main pier and caused the collapse of the preceding span. There were 79 workmen on these spans at the time of the collapse. It was estimated that further construction would be delayed about six months and that about half the steel could be salvaged and re-used.

The cause of the collapse was due to buckling of the webs of steel beams at the base of the temporary pier being used to support the cantilevered portion as it was being extended to a main pier. Instability of the temporary pier was due to omission of stiffeners and effective diaphragming in the steel grillage atop the concrete pile system. Two of the 18 who died in the accident were engineers responsible for the error in calculation. The grillage consisted of four rolled steel I-beam sections at the top of each pile group. It was testified that two mistakes had been made in the calculations by the Dominion Bridge Company, the steel erectors, which led to weaknesses in the grillage. One mistake was that the entire area of the beam had been used in computing the shear strength whereas only the web area should have been used. As a result the computed shear stress was only about one-half of the correct value. The second mistake was in checking the need for web stiffeners. This was based on a 1 in. flange instead of the 0.653 in. web thickness.

It had been testified that one of the mistakes had been found prior to the collapse by someone in the design office but, unfortunately, no action was taken.

Lincoln Highway Bascule

The east leaf of a two-leaf drawspan over Hackensack River between Jersey City and Newark, New Jersey, failed while being lowered on December 15, 1928 (2, 8, 43, 51, 52). Each of the leaves were 98 ft. long and 48 ft. wide. It had been in operation less than two years and carried a large amount of vehicular traffic.

The bridge consisted of a counterweight system as shown in Figure 1. The counterweight consisted of a block of concrete 20'-0" high, 11'-6" wide and 48'-10" long and weighed 750 tons. The vertical distance of the bottom of the counterweight above the roadway varied from 7 to 29 feet. The failure took place when the counterweight was approximately at the half-way point, at which time the span together with the counterweight fell into the river. The failure took place in the north leg of the counterweight tower. There was a complete fracture at the base of the leg. The leg being made up of channels with lacing showed a partly rusted tension break in the rear channel and a bending and tensile break in the other resulting in a forward thrust of the counterweight into

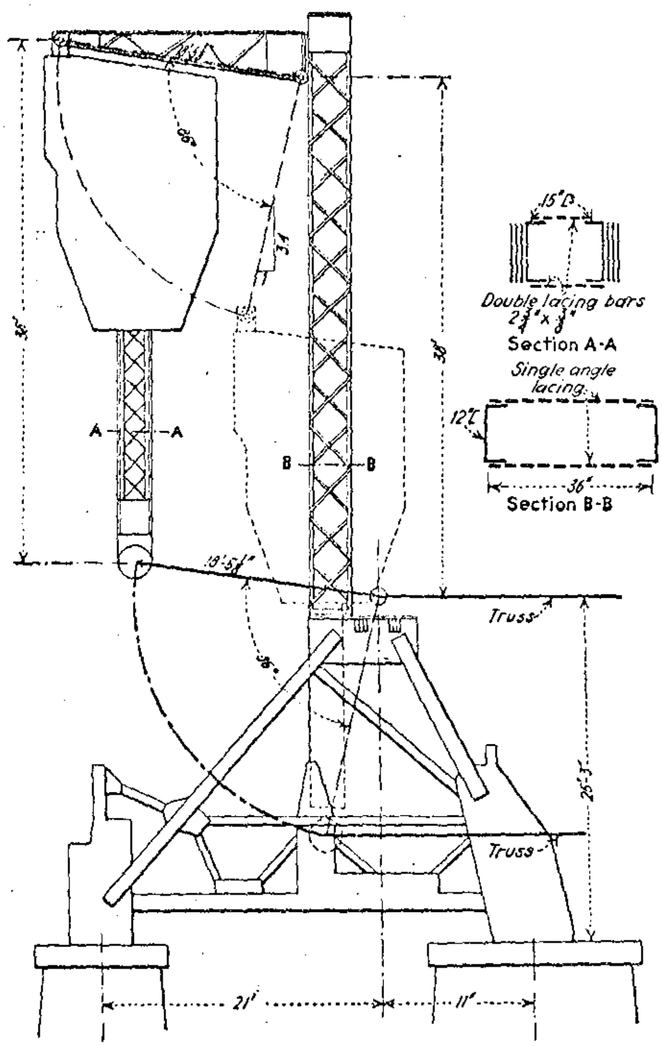



Figure 1. Diagram of counterweight system of Hackensack River Bridge (52)

the river.

A board of engineers was appointed to make an investigation into the failure and they made stress measurements on the undamaged west leaf during its operation and found it to be in a state of potential failure. The board reported on observations on the failed structure, observations on the remaining structure, and computed stresses. It was observed that the failure was caused entirely by fracture of the north leg of the east counterweight tower. There were indications of old cracks which had occurred at different times previous to the accident. The metal within the tower, however, appeared to be of good quality.

Observations made on the west structure while being operated at a slow rate showed signs of great stress in the counterweight tower with a result of bowing of the tower lacing. Strain gauges were placed on the channels near the base and a stress of 25,500 psi was observed. A deflection of the top of the tower was found to be over a range of 0.48 feet.

The factors considered when computing stresses were: wind on the counterweight of 30 psf, the torque produced by trunnion friction, stresses from starting and stopping plus stresses produced by periodic motion. The result was a very great overstress in the counterweight tower legs.

The Strauss Engineering Corporation of Chicago, the bridge designers, made their own analysis as to the cause of the failure. They reported that the failure was due to abnormally high friction developed at the counterweight trunnion bearing (tail bearing) of the north truss and the condition aggravated by the poor quality of the bronze bushings. Investigation of the trunnion bearings showed discolored surfaces which indicated possible faulty lubrication. Also, some bearings were plugged in such a way that lubrication was impossible and poor workmanship was found in the polished surfaces of the bearings. A chemical analysis was made of the bushings which indicated a lack of uniformity in the metal and showed the presence of antimony which was not indicated in the specifications. A review of the files indicated that some of the bearings had been rejected but had been shipped without inspection and were installed.

After an investigation by Strauss Engineering Corporation it was concluded that the prime cause of the failure was the abnormally high friction developed at the counterweight trunnion bearing of the north truss which was a result of poor lubrication and poor quality bronze bushings.

The final report of the investigating board of engineers reasserted what was brought out in their preliminary report and computed the separate items of stress action affecting the counterweight tower. These were combined as follows:

1. Combined effect of emergency brakes, specified friction, 15 lb. wind load and measured elastic vibration
   61,700 psi

2. Combined effect of motor brakes, emergency brakes and specified friction
   60,600 psi

3. Combined effect of motor brakes, emergency brakes, friction and 15 lb. wind load
   74,600 psi

4. Combined effect of motor brakes, emergency brakes, specified friction and measured elastic vibration

   74,700 psi

5. Combined effect of motor brakes, emergency brakes, specified friction measured elastic vibration and 15 lb. wind load

   88,700 psi

Any one of the above combinations could have resulted in failure as they were greater than the yield point of the steel. There also was a large amount of stress reversal and repetition which brought into play an endurance limit below the ultimate strength.

The final report of the board of engineers appointed to investigate the cause of the failure indicated that inadequate design was the primary cause of the collapse of the counterweight tower.

The above results of calculations were challenged by C. E. Paine, Vice President of Strauss Engineering Corporation (2). The following is the result of calculations up to this point (2).

Table 1. Calculations of stresses in counterweight tower

| | C. E. Paine | Board of engineers |
|---|---|---|
| Due to deceleration | + 22,200 | + 69,600 |
| Due to 15 lb. wind | + 14,000 | + 14,000 |
| Due to friction | - 8,500 | - 9,000 |
| Total with wind | + 27,700 | + 74,600 |
| Total without wind | + 13,700 | + 60,600 |

The board checked the calculations made by C. E. Paine and found several errors which are shown in Table 2 (2).

Table 2. Revised calculations of stresses in counterweight tower

| | C. E. Paine | Paine analysis (as corrected by board) |
|---|---|---|
| Deceleration | + 22,200 | + 29,300 |
| Oscillation | Not given | + 14,650 |
| 15 lb. wind | + 14,000 | + 14,000 |
| Friction | - 8,500 | - 3,800 |
| Total primary stress with wind | + 27,700 | + 54,150 |
| Total primary stress without wind | + 13,700 | + 40,150 |
| Secondary stress | Not included | + 12,450 |
| Maximum unit stress | + 27,700 | + 66,600 |
| Yield point | Not given | + 36,000 |

From the tabulated stresses consider a combination of deceleration stress, oscillation stress, one-half wind, and trunnion friction. The result of this combination, as corrected by the board, is 47,150 psi which is large enough to account for failure. The greatest error above was due to the fact that oscillations would be damped by friction and that the effect of oscillation may be ignored. The fact that the fracture in the tower channel was progressive and occurred at three different times, as clearly shown by the appearance of the fracture, demonstrates that the failure was caused by recurring conditions of overstress.

Tacoma Narrows Bridge

Probably one of the most dramatic bridge failures was the collapse of the Tacoma Narrows suspension bridge which spanned Pudget Sound at its narrowest point at Tacoma Narrows (36, 56, 62, 75, 98, 104). (See Figure 2). The channel was 4600 ft. wide at the bridge site.

The bridge consisted of a central span of 2800 ft., two side spans of 1100 ft. each, a west approach 450 ft. long of continuous steel girder construction and an east approach 210 ft. long of reinforced concrete frame construction. There were two cable anchorages, a 26 ft. roadway, two 5 ft. sidewalks and two 8 ft. deep stiffening girders.

The reinforced concrete deck was supported by 5 longitudinal steel stringers which were tied together by floor beams 25 ft. apart which were connected to the two main girders. The girders were 39 ft. apart and were in a vertical plane with the two suspension cables. The two main towers which supported the three suspended spans were 450 ft. above low water level of the channel. The anchorages to which the suspension cables were connected contained 20,000 cu. yds. of concrete, 270,000 lbs. of reinforcing steel, and 600,000 lbs. of structural steel. The total cost of the bridge was $6,469,770. It was opened to traffic on July 1, 1940. For details of the bridge see Figure 3.

The collapse of the center span took place at 10:00 a.m. on

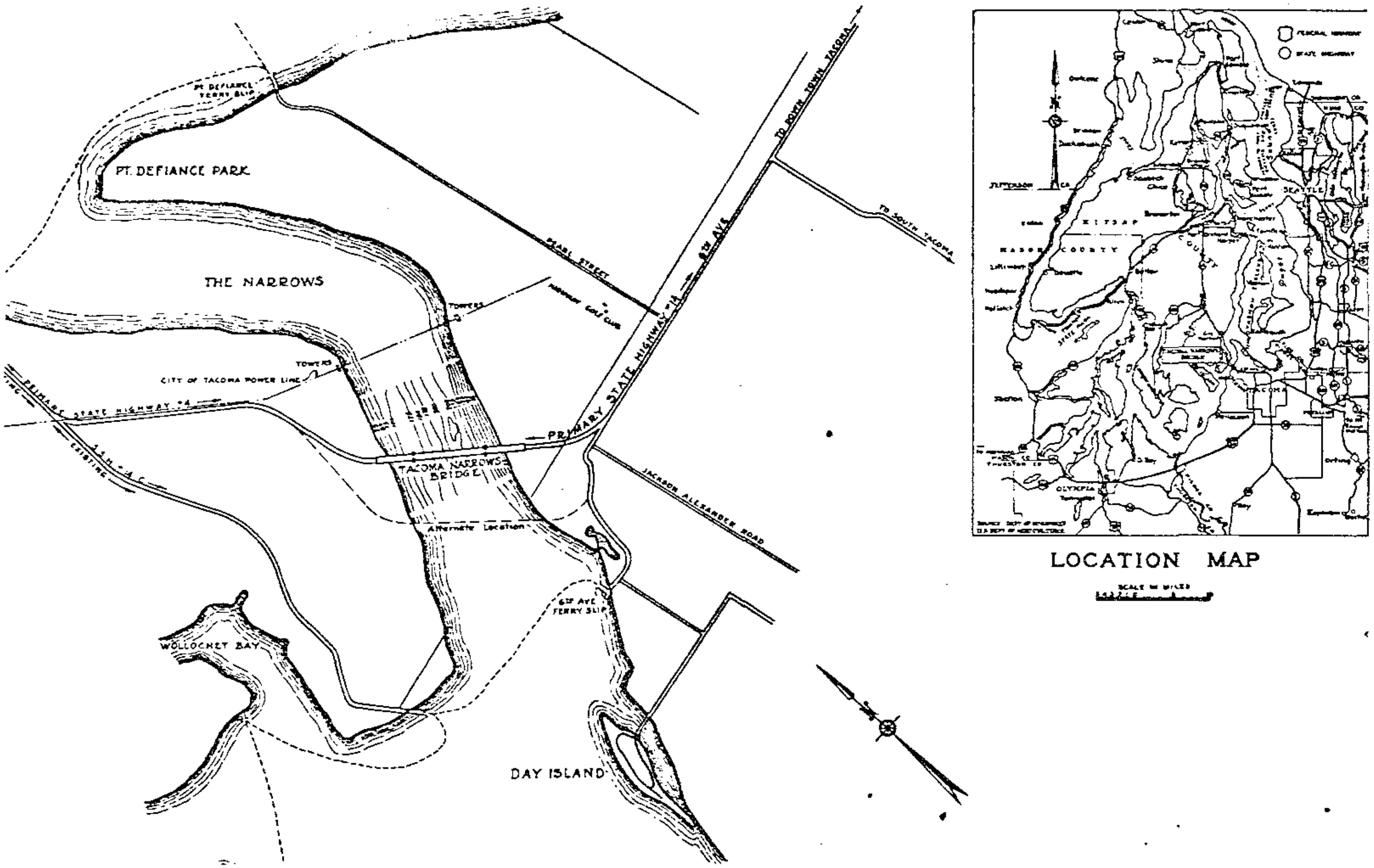



Figure 2. Location of Tacoma Narrows Bridge (56)

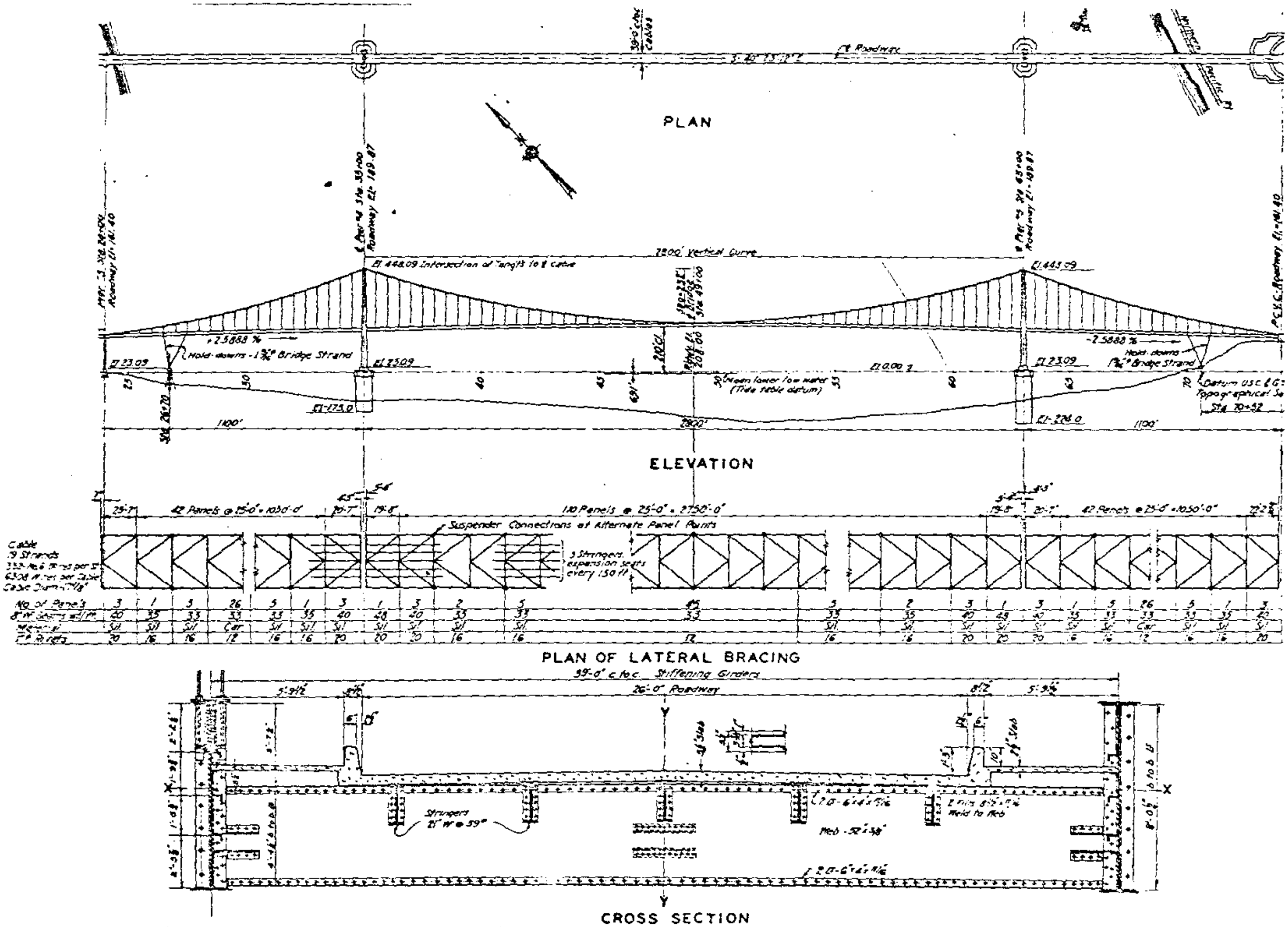



Figure 3. Details of Tacoma Narrows Bridge (56)

November 7, 1940 during a 42 m.p.h. southerly wind blowing at about $45^{\circ}$ to the roadway. Large vertical oscillations of large amplitudes took place. At first the roadway remained horizontal in the transverse direction but eventually the opposite sides of the roadway began to oscillate out of phase with each other and the main span began to break up and drop into the Sound.

There had been a considerable amount of comment from motorists prior to the collapse about the motion of the bridge deck. Even the workmen had noticed some motion during the final stages of construction. Many motorists detoured long distances to avoid re-crossing the bridge. There were various types of wave motion observed with amplitudes as great as 5 ft. in the vertical direction, but with very little side sway. Very large wave motion was sometimes noticed with a very low velocity wind and sometimes very little motion observed with high winds, so the amount of wave motion seemed to be independent of the wind velocity. The wave motion most often observed was that of two nodes and a frequency of 12 to 14 cycles per minute. For the various modes observed, see Figure 4.

About one hour and thirty minutes prior to the collapse of the center span the bridge received special attention because the frequency increased considerably. The main span began vibrating in about eight segments with double amplitudes of about three feet and a frequency of about 36 cycles per minute.

About an hour before collapse the vibration was in two segments with a node at the center of the main span, the two sides of the roadway began to vibrate out of phase with each other resulting in one side

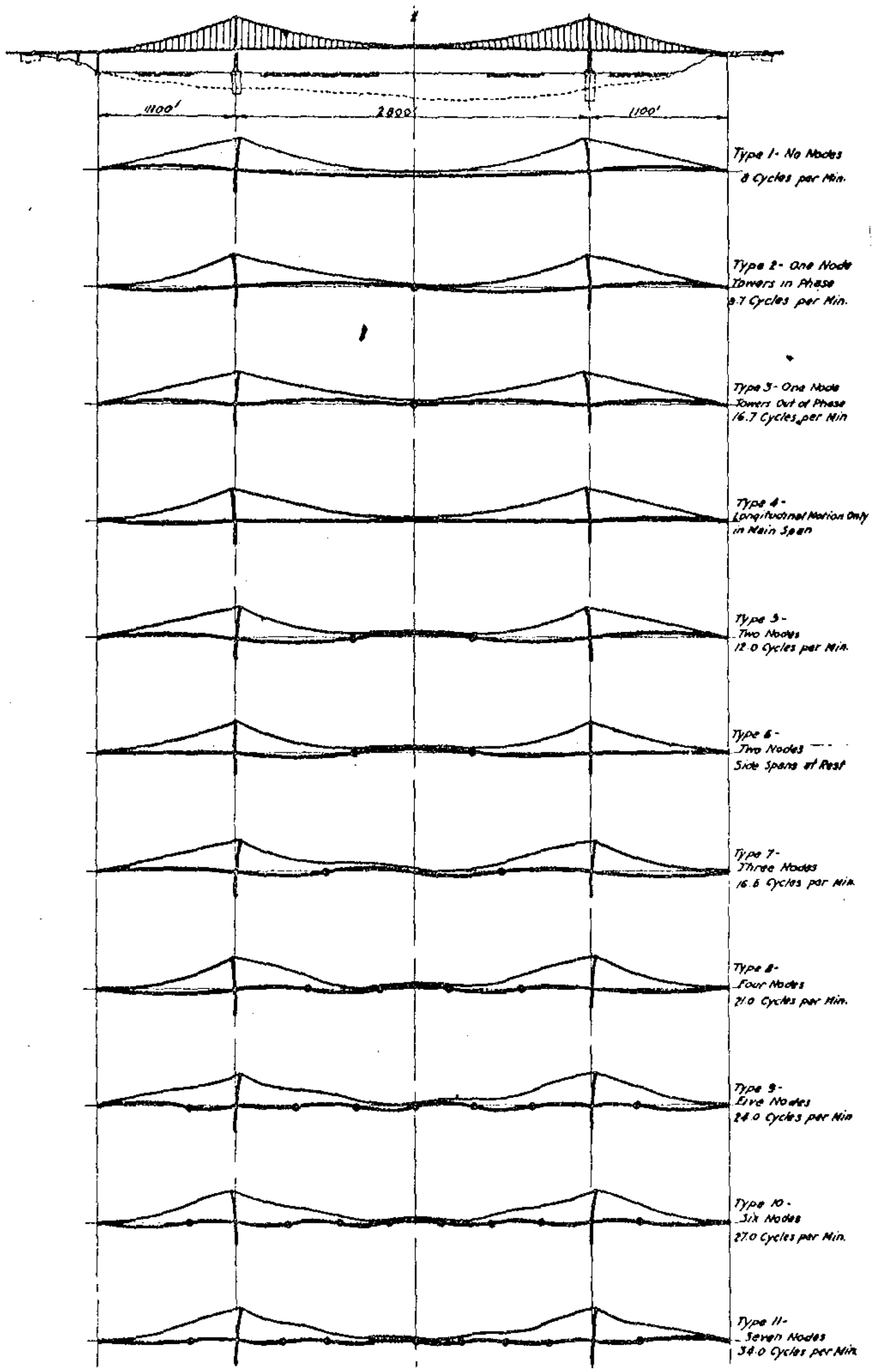



Figure 4. Modes of oscillations of Tacoma Narrows Bridge (56)

going up while the other side was going down. Many photographs have shown this particular action. The deck would tilt to about $30^{\circ}$ with the horizontal and the amplitudes increased to about 14 feet. The difference in elevation of the two sides of the roadway would be about 28 ft. at maximum amplitude.

There were two cars on the bridge at the time of maximum motion of the deck. The occupants of the cars escaped without injury but were quite shaken. As the cars were tossed from one curb to the other the occupants jumped out and were able only to crawl to safety on their hands and knees.

After the collapse there was only about 400 ft. remaining of the center span deck, the end spans had about 30 ft. of deflection and the towers were deflected about 12 ft. towards the shore. The towers deflected as much as 17 ft. just prior to their obtaining an equilibrium condition.

The investigation board's report on the cause of the failure showed that there was no evidence of faulty workmanship or poor materials. The bridge was very adequate in its ability to carry the live loads for which it was designed. The primary cause lies in the general proportions of the bridge. When comparing this bridge with others in the past which have had no real problems such as this one, it was found that forces which were almost negligible in those of large ratio of width to span length became dominant forces in this bridge. It was not stiff enough to withstand the aerodynamic forces and the design was lacking in capacity to dissipate energy and to dampen the distortions caused by wind forces.

It was apparent that a system of truss type stiffener girders would have been affected less by the wind forces than the solid type girders. An investigation was made in a laboratory wind tunnel following the collapse and it was observed that the actual motion of the bridge would be even more uncertain due to gusts and wind variations.

Hydraulic buffers had been designed to dampen the oscillations. These were located at the main towers but seemed to be ineffective in the purpose for which they were designed. All means of damping known were used in the structure and more were being studied at the time of the collapse. The methods tried had been used on other suspension bridges and seemed quite satisfactory with very little wave motion of the bridge. The prime concern was the discomfort to people traveling over the bridge and the wear on moving parts. There never was a fear of failure of the Tacoma Narrows Bridge.

Diagonal guys had been installed at the center of the main span to help resist longitudinal motion of the deck as vibration took place. As the floor began to oscillate there was a change in the sag of the cable. This, of course, could only be possible by horizontal motion of the cable at the center of the span. It had been concluded that these diagonal guys had a great effect on the failure of the bridge. It was found during the investigation that one of the guys had been slipping along the main cable. When one cable became ineffective the cable on the other side of the roadway became stressed much greater and an out-of-phase oscillation took place. The sudden slipping of the cable would also have an effect on the dynamic stresses. The sensitivity of the bridge to torsion was greater with only one guy acting than with no guys at all. The increased distortions dislodged the lateral members holding the stringers and the deck began falling into the Sound. This decreased the torsional resistance of the bridge and it did not have the capacity to dissipate the energy which was building up. As a result the amplitude continued to increase until most of the main span had collapsed.

The initial cause of the failure was the slipping of the diagonal guys. The results following were unavoidable as the forces became too great. The structure was very much affected by the wind forces due to its slender proportions combined with solid stiffening girders.

The problem of oscillation caused by wind forces was not new. Several bridges in the past have had problems due to lack of stiffening or rigidity (62). The problems seemed to be solved with the use of heavy stiffening trusses.

The first suspension bridge was designed and built by James Finley in 1801. This bridge had a 70 ft. span with truss type stiffening girders and wrought iron suspension.

The stiffening truss idea was later disregarded in the design and construction of a chain cable suspension bridge in Great Britain in 1817 with a 448 ft. span, but it was wrecked six months later because of the lack of stiffening against vertical motion. Another suspension bridge designed by the same man suffered failure in 1823 and again in 1836 due to the identical problem. Thus, the importance of stiffening was brought out by failures and should have been recognized.

In 1838 there was a partial failure of the Montrose Bridge built in 1828-29. It had a suspended roadway 26 ft. wide and a span of 412 feet. It was suspended by four chains, a pair vertically above each other on each side of the roadway. It had a lightweight railing along each side only to serve as a railing and not for the purpose of stiffening. A portion of the center span was lost during a strong wind while witnesses watched the undulating motion of the deck.

The Menai Straits Bridge built in Wales in 1826 by Thomas Telford had a span of 550 feet. The deck was suspended by four wrought iron cables and had very little stiffening. A month after its completion a heavy gale caused considerable motion in the chains and roadway which resulted in breaking of several suspender rods and floor beams. The motion described by witnesses was recognized as being similar to that of the Tacoma Narrows Bridge and being of two waves with very little vertical motion, if any, at the center of the span. Repairs and some means of strengthening was made but in 1836 during a severe storm the deck was reported to have amplitudes of 8 ft. at the quarter points with very little damage. The bridge, however, received additional damage in 1839.

The most remarkable parallel to the Tacoma Narrows failure was that of the Wheeling Bridge in West Virginia in 1854. This bridge had adopted the use of wire cables for suspension. It was built by Colonel Charles Ellet, Jr., in 1847-48 over the Ohio River. It had a span of 1010 ft. and a roadway of 17 feet. It had ten cables of 550 No. 10 iron wires. During a strong wind in 1854 the bridge collapsed in much the same way as the Tacoma Narrows Bridge according to past records. The deck was said to have lifted almost as high as the towers at times with half of the floor being nearly reversed. The final result of that action was a fall into the Ohio River below. It was following this failure that the stiffening truss again came to be used.

At about the same time of the Wheeling Bridge failure an 825 ft. railroad and highway bridge was being built across the Niagara gorge. It was of double deck construction with a railroad on the upper deck and the lower deck for horse drawn vehicles. It had a truss or support between the two decks and it was discovered that it gave considerable stiffening to the bridge. The bridge served well until it was replaced in 1899.

The Niagara Clifton suspension bridge at the mouth of the gorge just below the Falls was built in 1867-69 by a Canadian engineer, Samuel Keefer. It had a span of 1260 ft. and failed in a wind storm of 74 m.p.h. winds. It failed during the night and its action was described by a lone traveler as "rocking like a boat in a heavy sea with its deck almost on its very edge." The bridge was replaced in 1898 by the famous Honeymoon Bridge which was of arch construction and 840 ft. long. The Honeymoon Bridge was wrecked by an ice jam in 1938 (88).

The decline of the popularity of the stiffening truss began with the George Washington Bridge in 1931. This was to be of the double deck construction but only the upper deck was constructed first. The truss required for support between the two decks was also left out. However, the bridge behaved very well due to its large dead load and the rigidity was considered quite satisfactory.

Following this, bridge engineers began to design narrow, light, suspension bridges with shallow girders instead of trusses for stiffening. Lateral movement was expected but not vertical undulations. The Golden Gate Bridge with a span of 4200 ft. has been observed to have great undulations due to very large winds but it has a stiffening truss and the most that is expected of the motion is some annoyance to travelers. The Whitestone Bridge which is well known for its slenderness and grace misbehaves to the point where it is carefully watched.

The problem which took place with the Tacoma Narrows Bridge was not a new problem but one which had been forgotten. Increased stiffness against wind action is very important and can be accomplished by several methods of design such as stiffening trusses, greater dead load, or guy cables.

Tay Bridge

The Tay Bridge failure on December 29, 1879 resulted in the loss of 75 lives (99, 110). This bridge was made up of 85 spans of various types of construction and design. The length of the bridge was 10,320 ft. long and crossed the Firth of Tay between Newport and Dundee in Scotland. The spans were as follows: eleven spans of 245 ft., two of 227 ft., one bowstring girder of 166 ft., one span of 162 ft., thirteen spans of 145 ft., ten of 129 ft. 3 in., eleven of 129 ft., two of 87 ft., twenty-four of 67 ft. 6 in., three of 67 ft., one of 66 ft., and six of 28 ft. 11 inch. The eleven spans of 245 ft. and two of 127 ft. were the spans that collapsed and were located about midway across the channel. The bridge was a railway bridge with one set of tracks. The height of the track above the water was approximately 90 feet.

The bridge consisted of many types of girders and piers. The spans which fell into the water were trussed girders 27 ft. in height and the roadway was supported on the bottom chord. The piers were made up of six vertical steel pipes 15 in. in diameter placed in a hexagonal shape and supported on a concrete filled steel cylinder and lined with brick. The base was 31 ft. in diameter and extended about 5 ft. above high water. The vertical pipes were filled with concrete and were tied together with diagonal bracing. The height of these piers was 82 ft. from the base to the bottom of the truss.

The remainder of the bridge was intact after the collapse with only the thirteen spans mentioned previously having fallen into the water. Also the bases of the piers were not damaged which seemed to indicate that the vertical members were broken due to a lateral wind force.

The accident occurred about 7:30 p.m. as a passenger train was crossing the bridge during a very strong wind measured at about 70 m.p.h. with gusts up to 90 m.p.h. A signal was sent to the opposite end of the bridge informing them that there was a train crossing. This was a standard procedure as there was only one set of tracks. At the time that the train proceeded across the bridge it was intact but shortly thereafter, as one of the foremen watched, the train suddenly seemed to disappear. No word was ever received from the other end of the bridge confirming the arrival of the train which indicated that the communication system had been broken and collapse of the bridge was feared. This was later confirmed and efforts were made to pick up any survivors but all were lost along with the train which was found lying at the bottom of the channel.

Possibly an important factor in the collapse was the interruption of the construction by the death of the first contractor. The original design of the piers was revised by the next contractor and required a much smaller base. Also it was impossible to find a firm rock footing for one of the piers so the spans were increased, thus requiring one less pier.

Another factor which played a part in the failure was the ratio of the width of the pier to the height as compared to other structures of similar construction but of less exposed conditions. Also the Tay Bridge had comparably longer spans. It was thought that had the original design been carried out and not revised by the new contractor the bridge would have been structurally sound.

The wind pressure was determined to be about 40 lbs. per sq. ft. on the exposed surface areas of the bridge. This pressure corresponds to a wind velocity of 80 to 90 m.p.h. It was felt that the bridge was not properly designed for such a force. Calculations made by a London engineering firm showed that the bridge as revised could not resist a wind pressure of greater than 30 lbs. per sq. foot. Had the design been made according to the received American practice it would have resisted the force of any storm.

The Board of Inquiry, who investigated the collapse, placed the blame mainly on Mr. Bouch, the original designer, and stated that the bridge was badly designed, badly constructed and badly maintained and that its downfall was due to defects in the structure which would sooner or later bring it down. This meant disgrace and professional ruin for Mr. Bouch.

In those days there were no British Standard Specifications regarding the wind pressures to be used in bridge design. The wind pressure of 12 lb. per sq. ft. used in the design of the Tay Bridge was considered very inadequate.

The cause of the collapse was in part due to inadequate lateral wind bracing and failure of cast-iron lugs which connected the bracing to the cast-iron posts of the pier. The design and construction of the piers were severely criticized having been modified from the original design. The revision assumed solid rock and this rock did not even exist.

The fabricating of the cast-iron columns for the piers was under very poor supervision and blowholes in the columns were filled with a substance known as Beaumont's Egg which could be made to match the cast-iron perfectly. This, of course, weakened the columns. The substance was made up of beeswax, fiddler's rosin, very fine iron borings and lamp black. Many columns, which were honeycombed and filled with Egg, were shipped to the bridge site. The inspection of the foundry was left wholly up to the contractors which was, of course, a great mistake on the part of Mr. Bouch.

Mr. Bouch lost his desire to go on living after the accident and died only a few months later at an age of 58. His hair turned white overnight as a total of 95 lives were lost, 20 during the construction and 75 as a result of the collapse. A black cloud seemed to hang over the construction of this bridge because the first contractor died before completion of the contract, the second went insane and died, the third was ruined by the collapse and the resident engineer under Bouch developed paralysis from which he never recovered and which he had said was the result of his anxiety about the bridge.

It was felt at the time that the reason for the poor design was mistaken economy, with the great number of pier foundations in 30 to 35 ft. of water and with very rapid tidal currents the cost would have been considerably greater if the original design had been followed. It seems incredible that anyone would put economy ahead of the value of many human lives.

The lessons learned in the Tay Bridge disaster were remembered when the Forth Bridge was designed a few years later and the Tay Bridge was also rebuilt. The use of cast-iron was not considered in the rebuilding of the Tay Bridge nor in the Forth Bridge. The piers of the new Tay Bridge were built as shown in Figure 5.

## Moenchenstein Bridge

On Sunday, June 14, 1891 about 500 people from Basel, Switzerland were traveling by train to Moenchenstein, which was only about three miles away. The train was very heavily loaded, consisted of twelve different vehicles and required two engines. When the train was one-fourth mile from its destination it crossed the Birs River by way of a

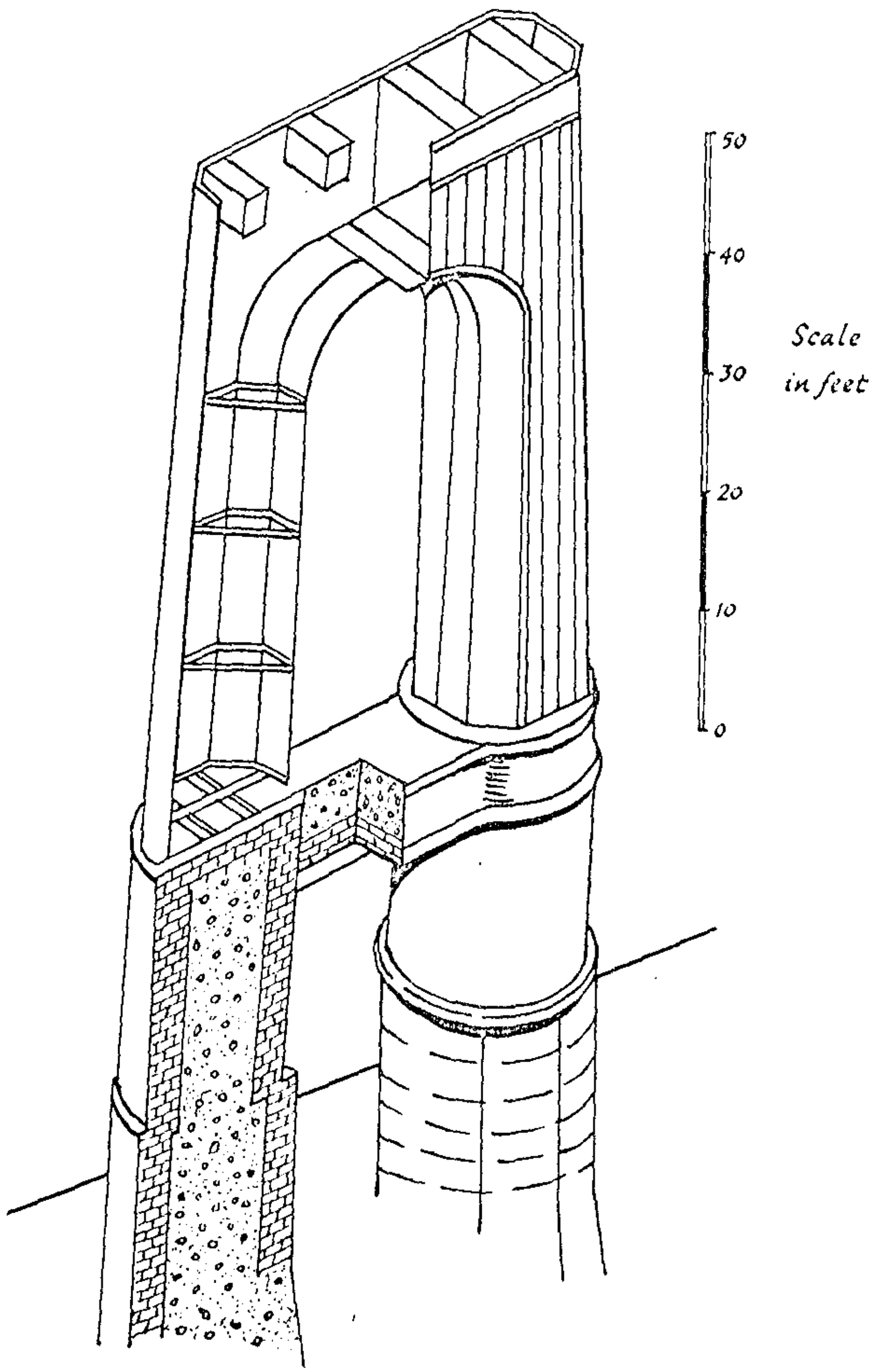



Figure 5. Pier of rebuilt Tay Bridge (99)

single track truss bridge which collapsed under the weight of the loaded train (18, 55, 76). There were 74 lives lost and some 200 injuries. The vehicles which were completely destroyed contained 266 seats, all of which were occupied.

The bridge was of the truss type with a simple span of 138 ft., and divided into six panels. The width between trusses was 15 ft. and had a height of 20 feet.

The bridge was designed by a well known bridge designer in 1874 and it was hard to realize that such an error could have been made to cause such a disaster. Following the accident, the designer did not want to accept any of the responsibility for the design although his signature was on the plans. He also was the contractor who built the bridge. The design was a complete alteration of one originally designed by a prominent Swiss engineer. The original design was an arched span of 143 feet.

It was quite a common practice in those days for the contractors to actually design a structure from sketches and load diagrams supplied them by engineers. The contracting party would be required to sign the plans and be responsible for any problems which would arise as a direct result of the design.

During the years following the completion of the bridge, it became evident that trains were getting larger and an investigation took place to determine its load capacity. It was found that the trusses were capable of carrying the customary loads as the design stresses were very low at 8500 psi. The floor system, however, was found to be deficient in strength and changes were made.

One factor which accounts for the great loss of life was the poor construction of the passenger cars above the lower running framework. As some of the passenger cars fell through the bridge, others were pulled in and with the forward momentum seemed to fall directly on top of the forward cars. The coupling gears also were considered weak which accounted for the cars literally flying through the air individually and dropping on top of those first to hit the river. There had been seven cars, four of which were passenger cars with 250 passengers, piled together in one large heap of wreckage. The statement was made that had the train been an American made train such a heavy loss of life would have been practically impossible due to the superior construction of its vehicles. The couplings of European rolling stock were compared to clay-pipe stems.

The cause of the accident was not attributed to a derailment as no signs of any wheelmarks were made on the ties. There was no sign of anything wrong until the bridge suddenly gave way, according to those who survived the accident.

It was determined with much certainty that the initial fracture occurred at the end panel at which point the locomotives had reached after passing over the bridge. The fracture could not have occurred at the center of the span because that had already been subjected to heavier loads. The chords at the center were stressed much higher than those at the ends and the lower chord had the same cross-section area throughout its length. In fact, strips of metal had been added to the lower chord near the abutments but were discontinued within one panel length. This was done as a part of the "beefing up" for greater load capacity but it would be expected that any additional material would have been added at the center of the lower chord.

According to two Swiss technical experts who were selected to report on the disaster, some of the design errors arrived at were: the thickness of the web of the lower chord was insufficient; there were too many rivets around the joints at the connection of the lower chord to the ties and struts which weakened the webs; and the eccentric fastening of the ties and struts to the chords. The material of the structure was of a variable nature and not sufficient for the design loads. The angles which tied the plates of the chords together were of poor quality with low ductility. The webs of the floor beams and stringers were also very inferior.

Vibration was also considered a major factor in the collapse of the bridge. The diagonal strut at the center of the bridge was designed only for tension which is correct with a static load at midspan but during the passing of a moving load the vibration set up in the span caused this member to go into compression. This vibration could also be increased due to the skew of the bridge which would cause like panels to deflect at different times and cause bending in the structure. The struts near the center of the span were overstressed when a compressive load, equal to the tensile load for which they were designed, was added.

The cause of the failure was finally explained as follows: the motion of the train as it neared the center of the bridge caused a buckling of the sixth strut bowing it outward. This would result in a settling of the truss and overloading of other struts. The struts near the abutment gave way and the one truss collapse was followed immediately by the other.

The main cause was the weak struts near the middle of the span which caused overloading of the others under vibrations. Other causes were the poor quality of the material and the eccentric connection of the ties and struts.

Tardes Viaduct

The failure of the Tardes Viaduct in France took place during erection on January 26, 1884 (108). This viaduct was to span the Tardes Valley near Evoux, France as an important link in the railroad from Montlucon to Eygurande. The depth of the gorge below the rail was 300 ft. and the length of the structure required to span the gorge was 821 feet. The plan was for a three span bridge with the center span being 328 ft. and the two end spans of 227.8 feet. The two piers were to be of granite construction of quite an enormous size. One pier was to be 195.6 ft. in height and the other 157.4 ft. both with an area at the top of 14'-9" x 26'-3". Evidently no steel reinforcing of any type was considered and its use was probably unknown at that time.

The term "granite" of which the piers were to be built, appears to indicate a form of concrete according to actual photographs taken at the site of the collapse.

The deck was supported by two trusses below which were 18 ft. apart and 27 ft. deep. The trusses were made up of riveted lattice with bars at $45^{o}$ and crossing each other five times in each panel.

The great height of the structure above the bottom of the gorge required a great amount of falsework for the erection. It was decided by the engineers that a method of launching from one of the approaches would be more economical. The launching of such long spans, as the 328 ft. across the center span, was the greatest ever attempted up to that time.

The approaches at both ends of the bridge were curved with rock projecting upward on both sides of the approach. In order to make room for the erection of the truss, a considerable amount of rock excavation was necessary.

Because of the great center span a method of reducing the open distance was accomplished by building a scaffolding at the far pier overlooking the gorge. This projected out 33 ft. over the gorge. Also a false truss was added unto the end of the center span proper of 98.4 feet. The open span was now only 197 feet. It is not quite clear as to the advantage of the 98.4 ft. addition to the end of the center portion prior to the launch beyond the first pier unless it was of some lighter construction. According to the diagram it appears to be of the same construction as the rest of the truss.

The launching was accomplished by a mechanism on top of the pier consisting of a set of rollers 21 in. in diameter. The mechanism was constructed in a manner such that the vertical position of the rollers would be automatically adjusted by any irregularity of the bottom of the chord and deflection of the truss.

At the time of the accident the truss had been launched to a point 174 ft. beyond the pier leaving only 121 ft. remaining to reach the scaffolding on the opposite pier. A violent wind occurred during the night and caused the destruction of all the truss beyond the abutment, a length of 433 ft. and 450 tons. Only that which was erected on the approaches remained intact. No one was at the site to witness the collapse.

The most probable theory of the cause of the failure was that the horizontal force of the wind produced oscillation of the projecting truss and produced lateral movement at the launching mechanism. There were no restraints against lateral movements between the launching mechanism and the top of the pier which was regarded as a faulty part of the design.

Calculations were made in an attempt to determine the wind velocity needed to move the bridge laterally at the pier. By using a coefficient of friction of 0.20 and the weight of the truss a wind force of 35 lb. per sq. ft. would have been sufficient to overcome the frictional force. The exposed area of the truss was a minimum when considering the wind to act perpendicular to the bridge and would be greater if the wind acted obliquely.

It is a well known fact that a wind force causes an uplift on a bridge thereby decreasing the horizontal force required to overcome the friction.

The final decision was that the engineers erred in not taking precautions to guard against and to control this lateral motion. It was assumed that lateral displacement began when the wind force reached about 30 lbs. per sq. foot. This is not an unusual wind and it is difficult to understand the reasoning of the design engineers in not requiring some means of restraint against lateral movement during launching.

## Summary

A discussion of bridge failures in this report shows that errors in design are the cause of some very major failures and, in most cases, the thought arises as to why these failures cannot be prevented. The disastrous failures, of course, are the most disturbing due to the great loss of life and the great cost. The following causes of failure in this chapter are: correct design given too little importance when designing a very critical part of a structure, dynamic effects disregarded, lack of sufficient knowledge regarding the effects of wind forces on a structure, revision of original design with too great an emphasis on economics, and ignoring the effects of increasing live loads beyond the design loading.

Second Narrows Bridge:

There were 18 lives lost as a result of this failure which took place during construction in 1956 (61, 74, 92). The cause of the collapse was due to failure of a network of steel grillage supporting a temporary pier required for a 345 ft. cantilever.

The design of the steel grillage was faulty and had been discovered in the design office of the Dominion Bridge Company prior to the collapse but it was evidently considered unimportant.

Lincoln Highway Bascule:

This was a relatively new bridge and collapse of one leaf took place in 1928 when the concrete counterweight fell from its supports above the roadway (2, 8, 43, 51, 52). The board appointed to investigate the collapse reported that the counterweight towers were overstressed during operation of the leaves and that an old break was observed in one tower leg of the collapsed portion. There were very high stresses caused by dynamic action during starting and stopping operation. The designer was held responsible for the collapse.

Tacoma Narrows Bridge:

The failure of the Tacoma Narrows suspension bridge in 1940 was very dramatic and probably the most remembered bridge failure by people living today (36, 56, 62, 75, 98, 104). It was a very costly failure but fortunately there were no lives lost.

The bridge had a suspended main span of 2800 ft. and was adequately designed for the live loads and dead load. The error was in the design for aerodynamic forces to which the bridge was subjected during wind velocities. The bridge was considered too flexible for its great span lengths in comparison to other large suspension bridges. It did not have sufficient dead load as a result of its narrow roadway. The effect of the wind on the stiffening girders was too great, thus causing the snaking action which eventually led to collapse.

Tay Bridge:

The failure of a portion of this very well known bridge took place in 1879 with the collapse of thirteen spans and the loss of 75 lives (99, 110). The total length of the bridge was 10,320 ft. and made up a total of 85 spans. The bridge was used to cross the Firth of Tay in Scotland. The collapse took place during a severe wind storm and while a passenger train was traveling across the bridge. The height of the track above the water was about 90 feet.

The cause of the collapse was due to a change in the design of the piers by a new contractor following the death of the original contractor. Also, there was faulty material used in the vertical steel supports of the piers. Flaws in the steel were covered up with a material which could not be distinguished from the actual steel.

Moenchenstein Bridge:

This was a disaster which took the lives of 74 passengers on a train loaded with about 500 passengers (18, 55, 76). The tragedy took place in 1891 near Basel, Switzerland when a truss bridge collapsed under the strain of the loaded train.

The cause of the failure was due to inadequacy of the bridge to carry the live loads which had increased over the years. The great loss of life was attributed in part to the poor construction of the passenger coaches, which literally resulted in a heap of burning rubble at the bottom of the gorge.

Tardes Viaduct:

This failure took place in France in 1884 during erection with no loss in lives (108).

An erection procedure which consisted of launching the bridge from one pier to the next was attempted because of an enormous amount of falsework required. A system of rollers was constructed at the top of the first pier but the designer neglected to provide sufficient lateral support.

The collapse took place when 174 ft. of the bridge had been launched beyond the first pier. A relatively strong wind forced the truss and the launching mechanism off the top of the pier.

# FAILURES DUE TO INADEQUATE INSPECTION

## General

There are four bridge failures summarized in this report which were a direct result of inadequate inspection by responsible and competent persons.

Careful inspection during construction is very essential. If a bridge is constructed in a faulty manner much of the care placed in the design is lost. An inspector is probably the most important person involved in the construction of a structure although this importance is often overlooked. Confidence is often placed in the honesty of the contractor which does not always result in the best workmanship.

An inspector must have close contact with the building contractor at all times. He must insist on good workmanship and discuss methods of solving problems as they arise. He must also try to maintain communication among personnel.

An inspector should be aware of design procedures. Conversely it would be ideal for the designer to be familiar with actual construction procedures.

It is hoped that the reader will gain some knowledge of the importance of adequate inspection by the summary which follows.

### First Quebec Bridge

The Quebec Bridge over the St. Lawrence River failed during erection on August 29, 1907 with the loss of 74 lives (58, 69, 80, 85, 95, 100). This bridge was the largest of its type and was of the truss cantilever type with a suspended center portion (see Figure 6).

The river was about a half mile wide at the site and 200 ft. deep at mid channel. The span of 1800 ft. was to be the longest up to that time. The total length of the bridge was 3240 ft. being made up of the center suspended span with a length of 675 ft., the two cantilever arms 562 ft. 6in. each, the two anchor arms 500 ft. each and two approach spans of 220 ft. each. The clearance above high water was to be 150 feet. The two main towers rose to a height of 400 ft. above high water. This was a gigantic structure for its time and was unequaled by any other bridge. As an example of the great size of members, a cross-section of a compression member was 4'-6" x 5'-6". The structure was, in general, pin connected. The bridge provided for two railway tracks, two street car tracks, two roadways and two footways. It was an important link in the Canadian railway system.

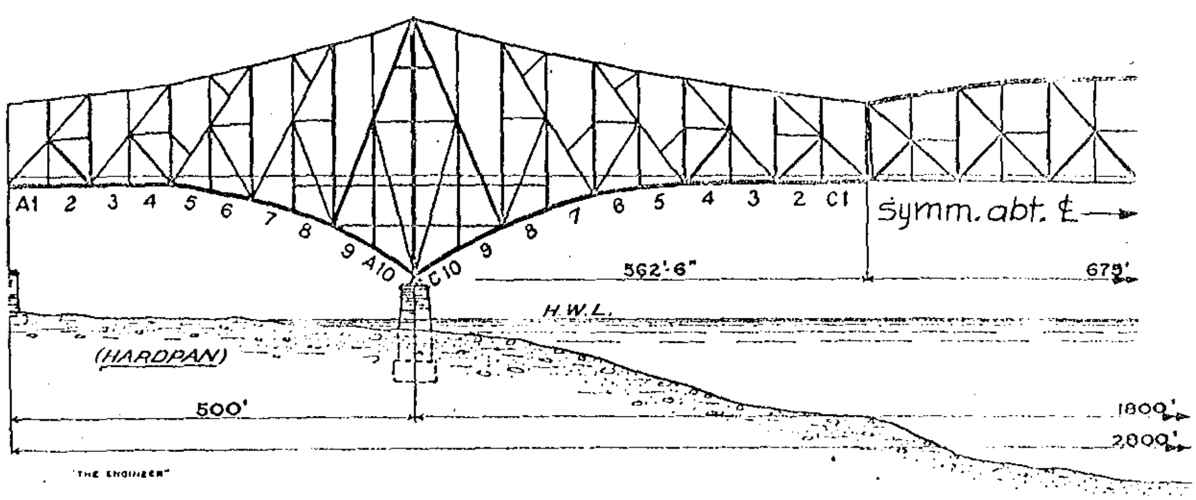



Figure 6. Final design of First Quebec Bridge (69)

The initial planning for the bridge began in 1852 when the citizens of Quebec advocated construction of the bridge across the St. Lawrence River for communication purposes between the Maritime Provinces and the United States. Otherwise communication was restricted by freezing of the river. Plans were prepared for a suspension bridge but the construction did not come to be a reality.

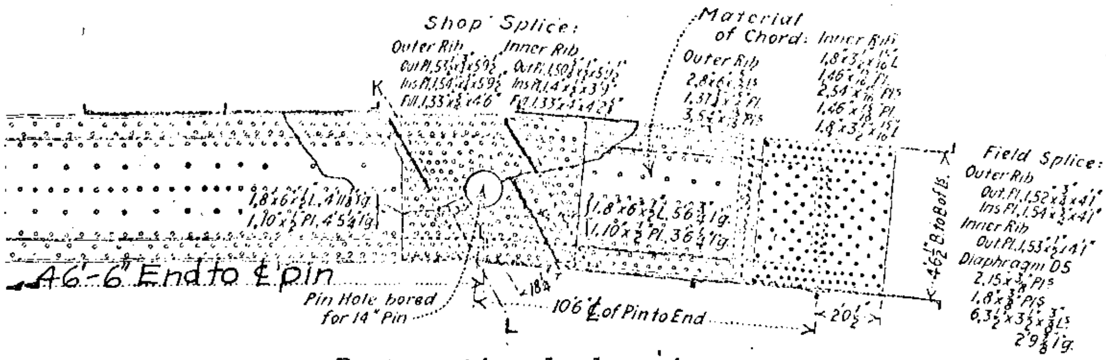
In 1887 the Quebec Bridge Company was given authority to build a bridge but due to a lack of funds it was postponed until 1903. In that year the Quebec Bridge Company was assured of financial assistance by the Canadian government by an act of the Canadian Parliament which declared the bridge to be for the general advantage of Canada. The contract for construction was awarded to the Phoenix Bridge Company of Phoenixville, Pa., on June 19, 1903.

In February of 1899 the Phoenixville Bridge Company submitted a plan which was signed by Mr. Hoore, the Chief Engineer of the Quebec Bridge Company, following the approval and recommendation of Mr. Cooper, a prominent bridge engineer who had been chosen by the Quebec Bridge Company to act as consultant. The design was revised by Mr. Cooper from a 1600 ft. center span to 1800 ft. as a result of poor foundation material. Mr. Cooper was very much interested in the project because he was at the age of retirement and it seemed a rewarding way to complete his career. His duties were to examine, correct and approve the plans submitted and prepared by the contractor, and to advise Mr. Hoore when requested. There were personality conflicts between these two men as Mr. Hoore felt he should be chief engineer in charge of erection of the bridge. Instead he was made resident engineer of erection. Everyone seemed to have more respect for the advice of Mr. Cooper and he was given the complete responsibility for the project. His word was final and not liable for alteration by anyone. Mr. Cooper made decisions with absolute honesty and was uninfluenced by other parties involved. The conflicting feelings between Mr. Hoore and Mr. Cooper probably had some influence on the great tragedy.

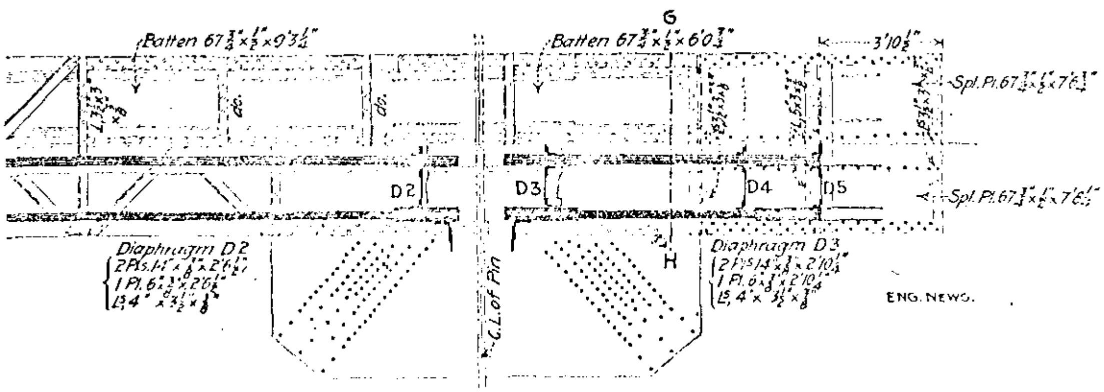
The south cantilever was nearly completed at the time of the collapse and work had begun on the north cantilever. A huge traveler derrick was used on the erection, the weight of the traveler being 1100 tons. It was of sufficient height for work to be done on the highest portion of the 315 ft. tower. At the time of the accident the large traveler was being dismantled to be used on the north cantilever. A smaller traveler of 250 tons was to continue with the work on the south cantilever. At the time of the collapse of the cantilevered portion the smaller traveler was erecting the suspended portion and was 751 ft. from the main pier. The larger traveler, with about 800 tons of it left to be removed, was at the ninth panel or about 500 ft. out from the pier. It was questionable as to the added weight being the cause of the failure and that the work should have stopped until the larger traveler had been completely removed. The Phoenix Bridge Company made calculations as to the stresses at the time of the failure and found that the stresses in the chords were less than 20,000 psi, which was less than that for which it was designed. The cantilever portion was designed on the basis of the larger traveler being used to erect the suspended span.

The bridge seemed to fall vertically with no side motion. The cantilever sloped towards the water until the tower's base slipped off the pier tops. It then fell directly and immediately into the river. The thought seemed reasonable that the collapse began with the failure of a compression member in the lower chord just south of the main tower in the anchor portion because of the position of the wreckage and the completeness of the collapse. There was no breakage of any member of the top eyebars and it was finally discovered under the wreckage that the bottom chord member A9L of the second panel south of the tower was buckled very critically (see Figures 7 and 8). It was later reported that this compression chord had deflected about 2 in. prior to the collapse but nothing was done to correct it. None of the other chords had been found to be buckled which threw a great deal of suspicion on member A9L. It was bent in the shape of an S and the question arose as to why would a member fail when stressed much less than that for which it was designed. The dead load produced a stress of only about 15,000 psi and the live load of the two travelers was less than that of the uncompleted flooring had it been in place. The total stress in the compression member A9L was computed as about 18,000 psi and a temporary stress of even 24,000 psi would be considered safe.

The deflection of the member was discovered about three days prior to the collapse. A careful watch was made of it for the next two days and no more deflection was noticed but the latticing began to show signs of severe stress. A telegram was sent by the inspector in charge of erection, Mr. McClure, to Mr. Cooper. Mr. Cooper immediately wired the




Part sectional elevation




Part top plan and section on center line

Figure 7. Member A9L of First Quebec Bridge (57)

Phoenix Bridge Company and told them that no more weight should be added to the bridge until this problem had been solved, but action was delayed and the bridge fell into the river along with 85 men, 11 of which were rescued.

The compression member A9L was constructed of four massive plate webs each made up of four rolled plates, stitch-riveted together to form

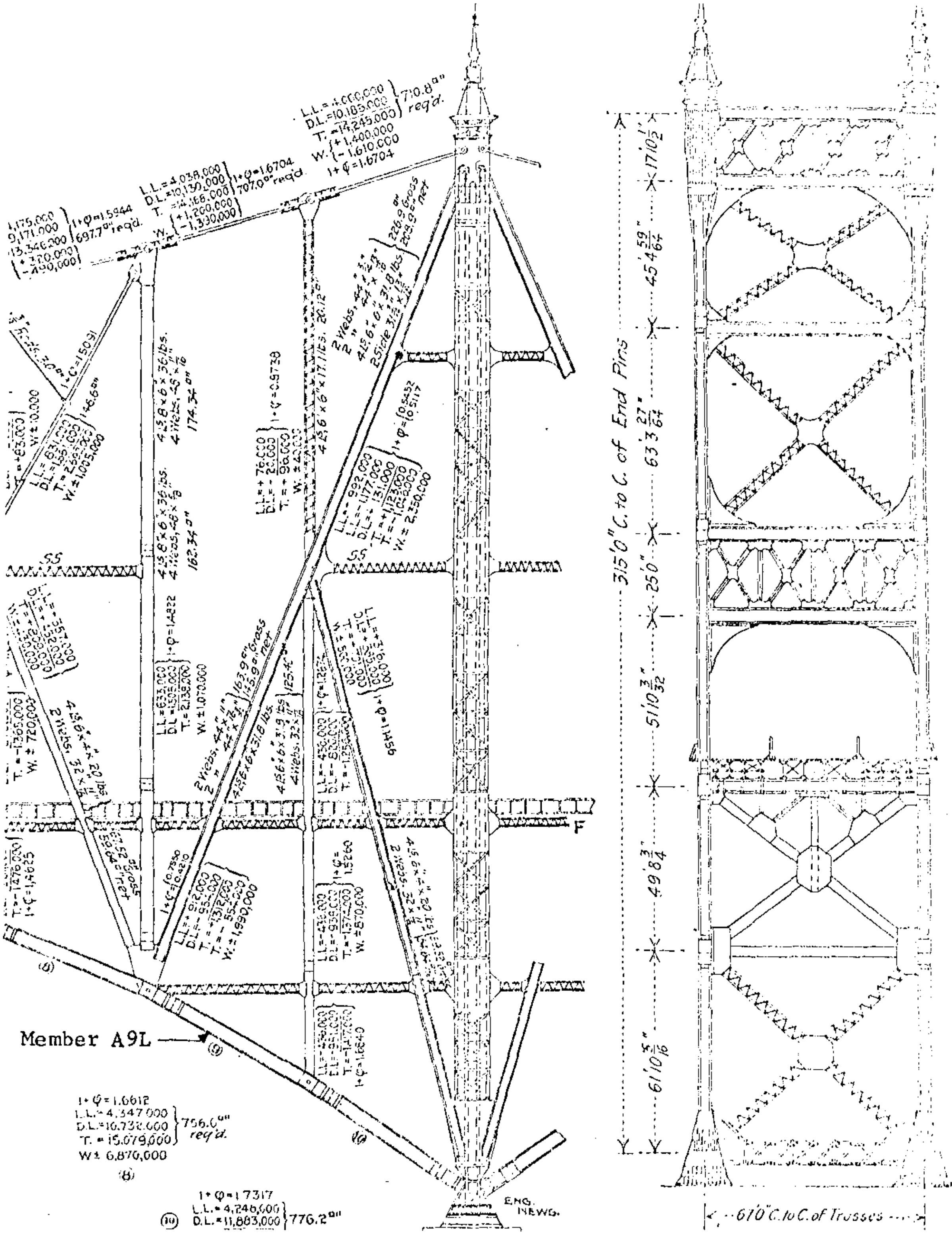



Figure 8. Part stress diagram of First Quebec Bridge (57)

one built up plate nearly 4 in. thick. These four webs were bound together by latticing so as to act as one member. The outside dimensions of the member were 5'-7 1/2" x 4'-6 5/8". The member was reported to have had a kink in one of the inner webs before it left the fabricating shop. The member was also involved in a railway accident during shipment to the site. It was reported that the member had been repaired with excellent workmanship. It is very improbable that the workers would have placed the member in its position in the bridge had it not been straight. The length of the member was 57 ft. and if the deflection of 2 in. existed before erection it would have been noticed and rejected. The resident engineer said the deflection had always been in the member and the deflection was not caused by overstress. This was not the feeling of the other responsible men on the job. Every member was inspected by several people and it is not probable that it would go unnoticed in such an important and massive structure.

It was felt that the method of latticing the web plates together in the member was poor. Near the ends of the member were cover plates used to tie the web plates together and it was demonstrated in the wreckage that this was an excellent means of holding the plates together. The amount of material in the stiffening lattice that binds the parts of a member together ought to be in proportion to the parts which they join together. Theoretically very little material would be needed to hold the compression members together under direct loading but under practical conditions this is often untrue. The plates must be free from any initial strains and this is impossible due to shearing and punching of rivet holes along with riveting.

Another fact placing more suspicion on the member A9L was that the member A9R, directly across in the right truss member, was also badly bent where none of the other bottom chord members were damaged as such. This appears to have resulted in the transfer of the load to the right truss as soon as the member A9L began to yield. This would lead to the feeling that other members of the bridge were stressed somewhat higher than expected.

Mr. Cooper was reported to have complained about the capability of the chief engineer hired by the Quebec Bridge Company, the designers of the bridge. He also felt the engineer representing the Phoenix Bridge Company, the contractors, was not qualified for his responsible position connected with the erection of such an important structure.

The practice at that time regarding bridge design was for the contractor in charge of the fabrication of the bridge to make up the working plans. As a rule, no engineer could afford to maintain a staff of such character and no corporation would listen to a fee that would cover any such expense. The engineer supplies the fabricator with the loading, both dead load and live load, the stresses in the members and the areas required to resist these stresses. The fabricator then builds the members from the stress sheet (see Figure 8).

Mr. Cooper believed that if both the Phoenix Bridge Company and the Quebec Bridge Company would have had responsible representatives who were qualified and experienced for such an important job the work would have been stopped in time and the bridge probably saved or at least the workers would have been removed from the bridge. Mr. Cooper's inspector reported to him in person about the bent compression member and action was at once taken to notify the contracting firm to stop immediately, but word was received too late. He believed that the member A9L could have been prevented from further deflection with about three hours work and $100.

The testimony of the chief engineer of the Phoenix Bridge Company revealed that the matter of the deflection of member A9L was seriously considered by the engineer, shop officials, and inspectors but decided that the deflection could not be due to an overstress as the stress was only about three-fourths of the design stress. The resident engineer on the bridge, Mr. Hoore, informed them that the member had been bent for a long time and the erection continued without waiting for advice which they all agreed was the correct thing to do. At about that instant the bridge fell.

Also a contributing factor was the design of the bridge using assumed weights of members and due to haste in completing the final plans the design calculations were not revised using actual weights which were 20 per cent higher than the assumed. Mr. Cooper neglected to make the check in the weights assuming it had already been done by the designer. The problem was discovered when the members were actually weighed in the fabricating shop and erection had already begun. This error accounted for the apparently low stresses of the individual members as compared with those in the specifications. The actual stresses were, therefore, much greater.

The matters of weights of members was discussed with Mr. Cooper by the designer and it was decided that the bridge would be safe as designed and that erection should continue. The actual stress in the critical bottom chord member A9L was finally calculated at a compressive stress of 29,700 psi.

The Royal Commissioners who were investigating the collapse felt that the structure should have been condemmed when such an error was discovered but undoubtedly Mr. Cooper felt that the extreme conditions used as criteria in the design would not actually exist.

There was no doubt as to the primary responsibility of the Phoenix Bridge Company which made the designs. Also secondary responsibility should lie with Mr. Cooper as the consultant who approved the designs and plans. Each party here involved had admitted reliability to some extent on the other party. Some aspects of the design were probably considered too lightly by one party due to its confidence in the other party but this does not relieve either from their responsibilities.

The correct way of constructing any structure is for each party involved to have a qualified inspector on the job and to work together and check each other's errors. Also each may have a different view of some particular problem. The advice of people involved with the actual construction and who are well experienced should be well accepted by the designer or the inspectors. This is not always the case.

Regarding the design of this bridge there had been no precedent set as there had never before been such an enormous structure built. The members were made up with latticing which had been compared to that of smaller structures. When the members were fabricated the workmen were impressed with the lack of stiffness and rigidity and the president of the company suggested this to the designing engineer but the idea was rejected as unimportant.

The primary cause of failure was determined to be the member A9L in the lower chord which showed signs of distress during erection but its condition was not thought important by some of the officials at the site because the member was apparently operating at a stress below that for which it was designed. This points to the fact that had not the allowable working stresses been so high, 24,000 psi, the condition of the member would have been given more consideration. The fact that the structure actually failed under a unit-stress, which was considered a safe working load, indicated that deficiencies in the design of the failing members were to be placed ahead of the high unit-stresses as the chief cause of the disaster.

In 1908 the Canadian Government decided to assume the responsibility for providing a bridge at this site as an important link in the Transcontinental Railway which was then under construction. A board of three engineers was appointed to act under the Minister of Railways. The board was to decide whether or not any part of the collapsed bridge could be re-used and construction continued or if a totally new design should be made. The Board decided that no use could be made of the existing plans, specifications or material. Even though the north cantilever was well along in construction it was found that the bottom of the pier was 46 ft. above bedrock and they recommended a totally new pier 57 ft.

farther out from shore.

Alternative plans were submitted by various steel companies and the one decided on was the one submitted by the St. Lawrence Bridge Company as seen in Figure 9. There was much disagreement within the Board and finally after several resignations and new appointments there had been eleven different men involved in and responsible in the decision of the Second Quebec Bridge contrary to only one, Mr. Cooper, in the First Quebec Bridge. The same procedure was followed regarding the design by the contractor and not by an engineering firm. This procedure was quite common and followed often in design of large bridges.

The general plan was for a similar type of bridge with bottom chord members being in a straight line rather than in a curve pattern as in the first bridge. The bottom chord section contained four webs, but consisted of twice the cross-sectional area.

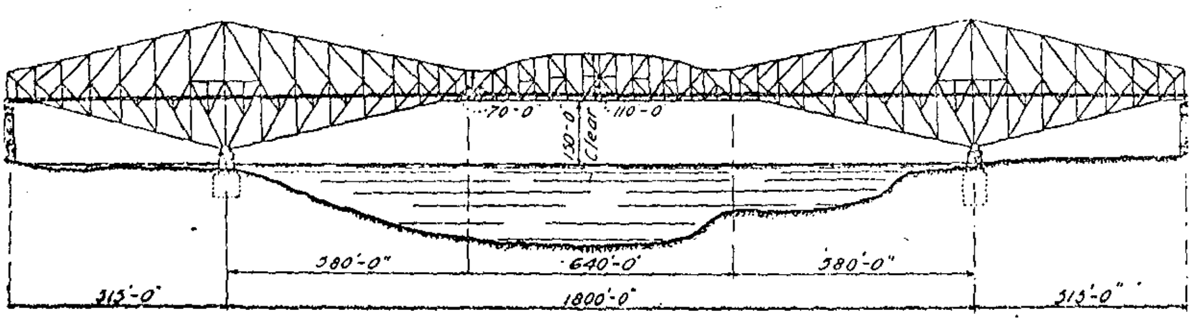



Figure 9. The Second Quebec Bridge, as built (69)

Palm Beach Arch Bridge

A bridge consisting of thirteen concrete arched spans, a steel swing span and a causeway at each end was the site of a collapse on December 29, 1922 at Lake Worth between Palm Beach and West Palm Beach, Florida (34, 117). The total length of the bridge was to be 1,885 ft. with a 30 ft. roadway and cantilevered 5 ft. sidewalks on each side of the bridge making the total width 40 ft. out to out.

The arches were barrel arches which were being filled with sand pumped up from the bottom of the lake. The filling was nearly completed at the time of the collapse. In fact the collapse took place on December 29 and the bridge was to be opened to traffic on January 1. This collapse is quite typical in that many failures take place during construction and usually near the completion of the bridge. Placing the final material adds greatly to the dead load of the structure at a rapid rate and can cause unequal loading conditions. This, however, was not considered the cause of this failure.

Three spans of 63 ft. fell into the lake along with two piers and the falsework for the arches. The three spans which collapsed were those nearest the opening for the swing span. The failure of arches or especially of spandrel-filled barrel arches was practically unknown at that time and a detailed investigation was called for by the design engineers who were also carrying out the supervision of construction. The report was made public on January 9 and contained the following results of the investigation.

The arch spans were 69 ft. with a clear span of 63 feet. The pier at the swing span opening was 19 ft. thick and the other piers varied from 9 ft. to 9 ft. 6 in. with the tops of the piers being 2 ft. below the surface of the water.

The arches were of reinforced concrete barrel construction with 12 in. walls at roadway level and 24 in. walls at the bottom where they rested on the pier top. There was no vertical reinforcement in the piers and there was no bond between the piers and the arch walls.

The piers were constructed in sheet-pile cofferdams which were cleaned out to rock. The concrete was placed under water by use of a tremie and the concreting was not continuous. The rock on which the piers were placed was a soft coral rock and sheet piling could be driven right through it.

Following the collapse the three arches along with the two piers were completely gone from view below the water surface. Part of one pier was intact below the water surface but the other pier had completely disappeared.

The concrete was inspected and it was revealed that too much sand had been used and also some very poor and large aggregate. The concrete could, at places, be crushed between the finger tips.

The question arose as to the amount of care taken in investigating the foundation material. The thickness of this rather soft rock was quite variable and in places it was not more than one or two feet thick. Sometimes the rock would disappear completely. The rock had been penetrated by pine piles which indicated the poor quality of the rock.

There was also doubt as to the quality of the pier concrete because of the manner in which it was placed. The absence of vertical reinforcing also indicated structurally unsound piers.

The cause of the collapse was due to the failure of the one pier that completely disappeared and the actual investigation of that pier could not be accomplished. The investigation showed that as a result of noncontinuous placing of concrete under water there was a considerable amount of soft and defective material between the adjacent pourings. There was no aggregate and the soft material was unable to carry any loads. Some places consisted of very large voids which resulted from the washing away of this soft material.

The cause of the collapse was established as a combination of the following:

1. The unusually light type of design and reinforcement
2. The poor quality of the concrete at critical points
3. The doubtful nature of the foundation strata, which should have been rigidly developed at each pier over its entire length
4. The tremie concreting, and discontinuous concreting, in combination with the omission of vertical reinforcement in the piers.

The specifications which were to be followed in the construction called for removal of any scum or laitance from the surface of the preceding pour before beginning the next pour. This requirement was completely ignored. Some of the concrete near the top of the pier was actually poured in water. It was reported that these operations were carried on with the knowledge of the inspector.

It also was discovered that some of the arches had been constructed out of alignment. It was thought possible that the fallen arches also may have been in a position other than that called for in the design. The plans and specifications were considered accurate, thus the responsibility for the collapse lies with the contractor and the inspector representing the design engineer.

Attica Bridge

A bridge consisting of six pratt truss spans over the Wabash River was the site of a collapse on Sunday, April 5, 1914 (7, 107). Two spans collapsed allowing a passenger engine and three passenger coaches to fall 30 feet. Three people were killed and 35 injured, which was surprisingly low.

The length of the bridge was 1,795 ft. with truss spans varying in length from 110 ft. to 156 feet. The collapse occurred at an end span and an adjacent span. The accident took place over dry ground otherwise many passengers could have drowned.

The failure was caused directly by an accident to one of the end posts on a 110 ft. span, which was one of the end trusses of the bridge. This end post had been struck by a derailed freight car prior to the collapse. The impact of the derailed car caused the neutral axis of the end post to be displaced outward about 7 1/2 inches. Also there were severe splits in the cover plates for a considerable distance parallel to the rivet lines. Some of the cracks occurred within the rivet lines.

The passenger train which fell through the bridge was from Williamsport, Indiana. Its crew had received word of the damaged bridge and that they should proceed very slowly over the bridge at speeds of 3 or 4 m.p.h.

When the passenger train reached the approach of the bridge a freight locomotive was being used to remove the derailed car from the bridge. Apparently the crew took it upon themselves to cross the bridge as the bridge did not deflect under the load of the engine being used to clear the deck. When the pilot wheels of the locomotive reached the far abutment the span dropped, along with the adjacent end of the second truss.

The damage to the end post should have been inspected more fully and allowable loads determined. The load of the passenger train resulted in considerably more stress in the member than that caused by the individual freight locomotive. The weakened condition should have been apparent and its seriousness realized with immediate suspension of traffic. The testing of the bridge with the use of the locomotive clearly was endangering the lives of the employees. The load on the end post as a result of the freight locomotive was 147,800 pounds and that of the passenger train 186,800 pounds. A detailed inspection had been ordered following the damage to the end post but apparently the bridge seemed to be strong enough and the passenger train proceeded across. It had not been established if the crew had received word to cross and the responsibility for permitting the bridge to be used in its weakened condition could not definitely be placed due to lack of evidence.

A partial analysis of the stresses in the end post following the collapse showed a probable compressive stress of 7,700 psi which is not unreasonable. The allowable stress calculated according to the column formula, $P = 16{,}000 - 70\ l/r$ was 11,980 psi with $l$ being 408 in. and $r$ being 7.1 inches. If, however, the end of the post remained fixed in such a way as to cause an eccentricity of 7 1/2 in. an additional fiber stress of 11,080 psi could have resulted. This added to that caused by direct load could result in a total stress of 18,780 psi which is about 57 per cent overstress.

The material otherwise was considered of good quality along with the riveting and workmanship. The failure was a direct result of the damaged end post but it is not clear where the actual responsibility was placed. The attempt to have the damaged bridge inspected by an experienced bridge inspector was evident, but the death of the engineman, fireman, and an express messenger in the collapse makes it impossible to determine if actual word was received for the passenger train to cross the damaged bridge.

Chester River Bridge

A failure of an unusual type occurred on a bridge over the Chester River in Chester, Pa., on the evening of September 10, 1921 (13, 41). This could be considered a very minor failure in regard to the damages to the bridge, but the cause of the failure requires attention along with the fact that 24 lives were lost.

On this evening on September 10 a crowd of people were gathered on a sidewalk cantilevered out from the bridge girder to watch the rescue of a boy from the river. The sidewalk suddenly began to slope towards the river and 75 people fell into the river, of which 24 drowned.

The structure was a welded plate girder bridge with a span of 70 feet. The sidewalk was cantilevered out from the girder a distance of 12 ft., and supported by brackets made up of angles and connected to a stiffener angle of the girder by gusset plates of 3/8 in. thickness. The brackets were 12 ft. center to center. The bracket which failed was bolted to the girder stiffener by the gusset plate which was riveted to the top angle of the bracket. All the other brackets were connected to the girder by gusset plates which were riveted only. This led investigators to believe that the failed bracket had been under repair at some time previously.

The gusset plate had failed along a vertical line just beyond the edge of the girder stiffener. About 8 in. at the top appeared to have been an old failure because the fracture was rusty.

The design of the brackets was considered faulty because of the fact that wrought iron had been used for the gusset plate and that the rolling grain was vertical.

The bridge was designed in 1886 and in 1910 a canal boat became wedged beneath the bridge and had bent the bracket. The fact as to how the bracket was repaired was not considered important. Some said it was straightened without removing it from the bridge which is evidenced by the fact that the gusset was riveted at the top and bolted to the girder stiffener. The courts decided that the county was to be held responsible due to poor inspection of the bridge. It was felt that a defect of the kind that had obviously existed in the gusset plate should not have gone unnoticed by the inspectors. The collapse therefore showed negligence on the part of the inspectors.

## Summary

It is the purpose of this discussion to remind the reader of the great importance of proper inspection both by the owner and by the designer. Inspection is often taken too lightly. It is the final stage for a structure to be constructed properly since the early planning stages and the drawing board. If this stage is faulty an eraser cannot make it correct.

The following brief summary includes causes of failure due to lack of ability and knowledge on the part of the inspector, the inspector being aware of faulty workmanship, and lack of maintenance inspection.

<u>First Quebec Bridge</u>:

The failure of this bridge, which was being built to span the St. Lawrence River, took place in 1907 during erection with the loss of 74 lives (58, 69, 80, 85, 95, 100). It was a truss cantilever type with a suspended center span.

Failure took place in a compression member of the lower chord of an anchor span. Even though unusual deflections were being observed as failure was impending, erection was not discontinued due to unqualified inspectors at the erection site. The result was the collapse of the anchorage span and cantilever.

<u>Palm Beach Arch Bridge</u>:

This bridge consisted of thirteen arch spans with a steel swing span included (34, 117). Failure of three spans took place in 1922 just three days prior to the opening to traffic.

Cause of the collapse was due to use of faulty concrete in the piers, discontinuous concreting by use of tremie, insufficient steel reinforcement in piers, and inadequate foundation material. These faulty construction procedures were all carried out with the knowledge of the inspector.

Attica Bridge:

This bridge consisted of six Pratt truss spans of which two end spans collapsed under the weight of a passenger train in 1914 (7, 107). Three persons were killed in the collapse which was a result of a damaged end post. A derailment of a railroad car just prior to the collapse caused severe damage to the end post.

The cause of the accident was determined to be the damaged end post but it was not established where the responsibility lay. A qualified inspector had been notified but before he could reach the site the train proceeded across the bridge. Whether or not the train engineer had received word allowing the train to cross was not established due to the death of the crew.

Chester River Bridge:

This failure consisted of the failure of a cantilevered sidewalk on the bridge (13, 41). There were 24 lives lost due to drowning when 75 persons fell into the river.

One of the brackets which supported the sidewalk failed due to a previous accident. It had at one time been repaired but not replaced. The fracture at the time of collapse consisted of a rusted break.

Lack of inspection was established as the chief cause of the disaster.

# FAILURES DUE TO INADEQUATE CONSTRUCTION PROCEDURES

## General

Four bridge failures are discussed in this report and were a direct result of inadequate construction procedures which are closely associated with inadequate inspection. Adequate inspection would not permit some construction procedures to take place.

In the past, when truss bridges were quite common, it was often found that the erector would add members to a truss or increase the size of members to fit the field conditions. As a result the stresses in the members would change from those used in the design.

Often it is found necessary in construction to use methods which are different from methods used in the past and may be unfamiliar to the contractor. According to specifications an alternative method must be approved by the engineer. This, of course, requires the engineer to be well experienced with the procedures available. The engineer must, however, be truthful in his opinion and knowledge concerning the methods.

In the following summary various methods of inadequate construction are discussed.

### Ashtabula Bridge

A great tragedy took place on December 29, 1876 when a Pacific Express passenger train loaded with 160 passengers fell through a bridge and resulted in the death of 92 of its passengers (4, 5, 6). The bridge was of the Howe truss type with two tracks being supported on the top chord. It had a single span of 156 ft., was 20 ft. deep from lower to upper chord and consisted of twelve panels. It was the first Howe truss built of wrought iron instead of wood and was somewhat of an experiment. One factor which caused great difficulty and may have had something to do with the collapse was the extra dead weight resulting from the use of wrought iron instead of the usual wood.

The train was made up of two engines, four baggage and express cars, two ordinary coaches, one smoking car, one palace car and three sleepers. The train was traveling from Erie to Ashtabula, Ohio on the evening of December 29 and was slowed considerably by a great snowfall. When the train arrived at the bridge it was only a few hundred feet from the depot and moving about 10 to 12 miles per hour. When the front engine came within two panel lengths of the far abutment, the center of the bridge apparently began to deflect as reported by the engineer of the front engine. He reported hearing a loud crack and as the engine was apparently traveling upgrade he opened the throttle. This broke the connection and allowed the engine to reach the abutment before the rest of the train began to fall. It fell to the bottom of a 75 ft. deep gorge.

It was certainly reasonable to assume that the bridge had sometime been subjected to heavier loads than this over its 12 years in operation. The design load, which was two trains on the bridge simultaneously, could have been reached at some time. The matter of low temperatures making the iron more brittle was given some consideration but was overruled in that a dynamic blow would have to had taken place, low temperature not being a factor under static load. The only way the bridge could have been exposed to a dynamic blow would be if there had been some cars derailed and the wheels were striking the cross ties but this was not considered to have happened. The matter of brittle fracture resulting from low temperatures and chemical analysis was considered almost a century ago so this aspect of structural design certainly is not a new one. It seems that over this length of time a great deal should be known about brittle fracture through experience and experimentation.

The designer of the bridge was Mr. Tomlinson, an engineer. The design was modified by Mr. Stone who was president of the road at the time. Mr. Tomlinson was not in agreement with Mr. Stone and refused to sign the plans as he did not approve of a wrought iron Howe truss because of the additional dead load. He thought the bracing should be made of larger material and because of the disagreement was fired. There were no members in the bridge with a depth greater than 6 in. and this was considered too small for the compression members. Even when supported at the center some of these members were 23 ft. long.

A civil engineer testified during the inquest that his examination of the wreckage showed that all members were heavier than needed except for the top chord. He felt the Howe truss pattern was not well adapted for heavy iron bridges because it resulted in too many compression members which increased the dead load.

Mr. Tomlinson stated that he had found that some of the diagonal bracings were not in their correct position for which they were designed. Some of them were out of position by as much as three inches. His feeling about beefing up of the compression members was also felt by others. Some of the diagonal members had their ends chipped to make room for tension rods which also could have weakened the structure. The erection of the bridge was very faulty in that the person responsible and in charge was very inexperienced in bridge work. The bridge was under the supervision of Mr. Tomlinson until he was dismissed. Mr. Rogers replaced him and had never before erected a bridge. The erection was faulty in that modifications were made in the field. Mr. Rogers had never seen the plans for the bridge.

Testimony was given by an experienced machinist who had worked on the fabrication of the bridge in the shop. He said the bridge was assembled on supports and as the formwork under the center of the span was removed the bridge began to deflect under its own weight and would have collapsed had all the formwork been removed. This defect was kept a secret. One reason for the defect was that the beams in the chords were placed in the structure so that bending occurred about the weak axis. This error was corrected and some additional members added which were not included in the original design. This also added to the dead weight.

Another error in its construction was that the structure was not long enough to span the distance between abutments which resulted in modifications which had not been accounted for in the design.

Some facts submitted by three civil engineers employed by the Legislative Committee to investigate the Ashtabula disaster were:

1. All the tension members had very large factors of safety and were well able to sustain all loads that could possibly come upon them.

2. All the compression members were deficient in capacity and had very low factors of safety.

3. The top chord as well as the diagonal braces took part in the failure. All had very low factors of safety and failure was inevitable.

The material itself was of superior quality and no weakness was found which could not have been discovered and prevented.

It was quite apparent in the erection of this bridge that members were cut, chipped, filed and their lengths changed to make members come together. This could result in much different stresses than those for which the bridge was designed. This freedom of manipulating members seemed to come about by the belief by workmen that a very large factor of safety was involved, such as 4 to 6. The actual factor of safety was 1 1/2 to 2 but with the manner by which members were modified it may have been as little as 1 in some cases. By appreciating a low factor of safety, the necessity of good workmanship would be quite apparent to those in charge.

There are various reasons for the collapse of the Ashtabula Bridge. The primary one is faulty construction although it was probably strong enough for the time at which it was constructed. Trains at that time were of considerably less weight.

Second Quebec Bridge

One of the most dramatic bridge failures was the failure of the Second Quebec Bridge on September 12, 1916 (3, 11, 12, 22, 38, 97). It is believed that only 13 lives were lost in this accident. The failure consisted of the center suspended span (see Figure 9 shown on page 54) falling into the St. Lawrence River during its hoisting operation.

The truss type suspended span was erected at a shallow point of the river about three miles below the bridge site called Sillery Cove. At low tide the bed of the river was exposed which permitted better working conditions. The span, which was 640 ft. long, was erected on piling. When the span was ready for transporting to the bridge site, floating scows were placed under the span and as the tide rose the span was supported by the scows. The trip to the site was free of any incidents. Meanwhile at the site the cantilevered portion at each end was ready to receive the center span. A system of hangar chains was attached to the span and as the tide lowered the weight of the span was transferred to the cantilevers.

There were thousands of spectators at the site, many of whom were prominent engineers. The jacking process began and proceeded without incident for about 15 ft. when suddenly there was a loud report and the span fell into the river. The south end of the span seemed to drop first as was evidenced by a photograph taken by a newspaper reporter.

The span had been supported at each of its four corners on rockers which had rocker pins both longitudinally and transversely, thus performing as a universal joint between the span and the lifting girder (see Figure 10). The lifting girder was then connected to the lifting hangars as shown in Figure 11. There seemed to be a possible unstable condition here because the point at the connection to the hangar chains was at about the same elevation as the universal joint. With the truss supported about 4 ft. above this point a small amount of eccentricity would tend to tip the girder. It was first thought that fracture of the rocker casting was the cause. However the dropping of the corner of the truss could have caused a sudden thrust and a possible rotation of the girder, allowing the one corner of the span to slip off and into the river. Every precaution had been taken to keep the span level during the jacking. The lifting chains and girders were all in good condition

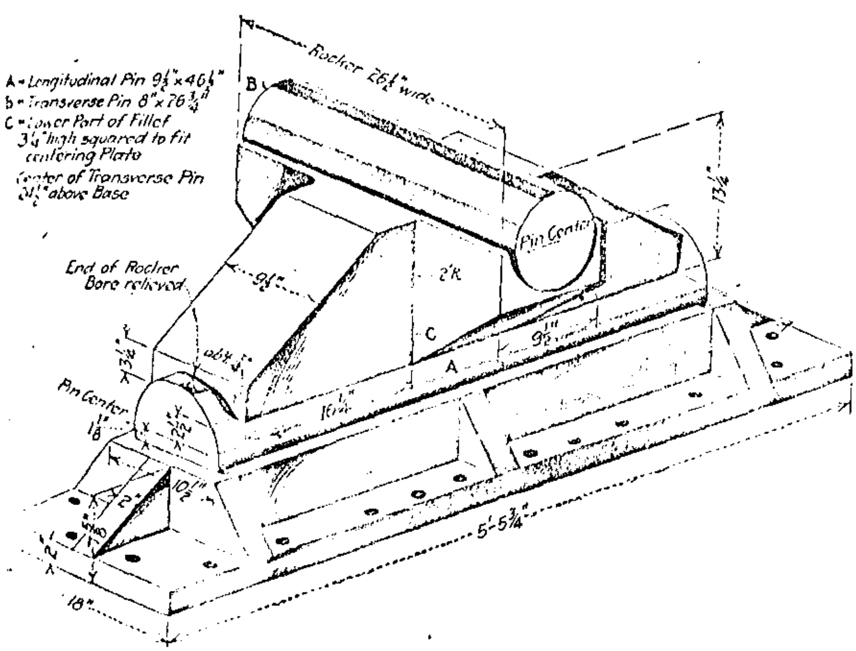



Figure 10. Diagram of rocker used for hoisting suspended span of the Second Quebec Bridge (12)

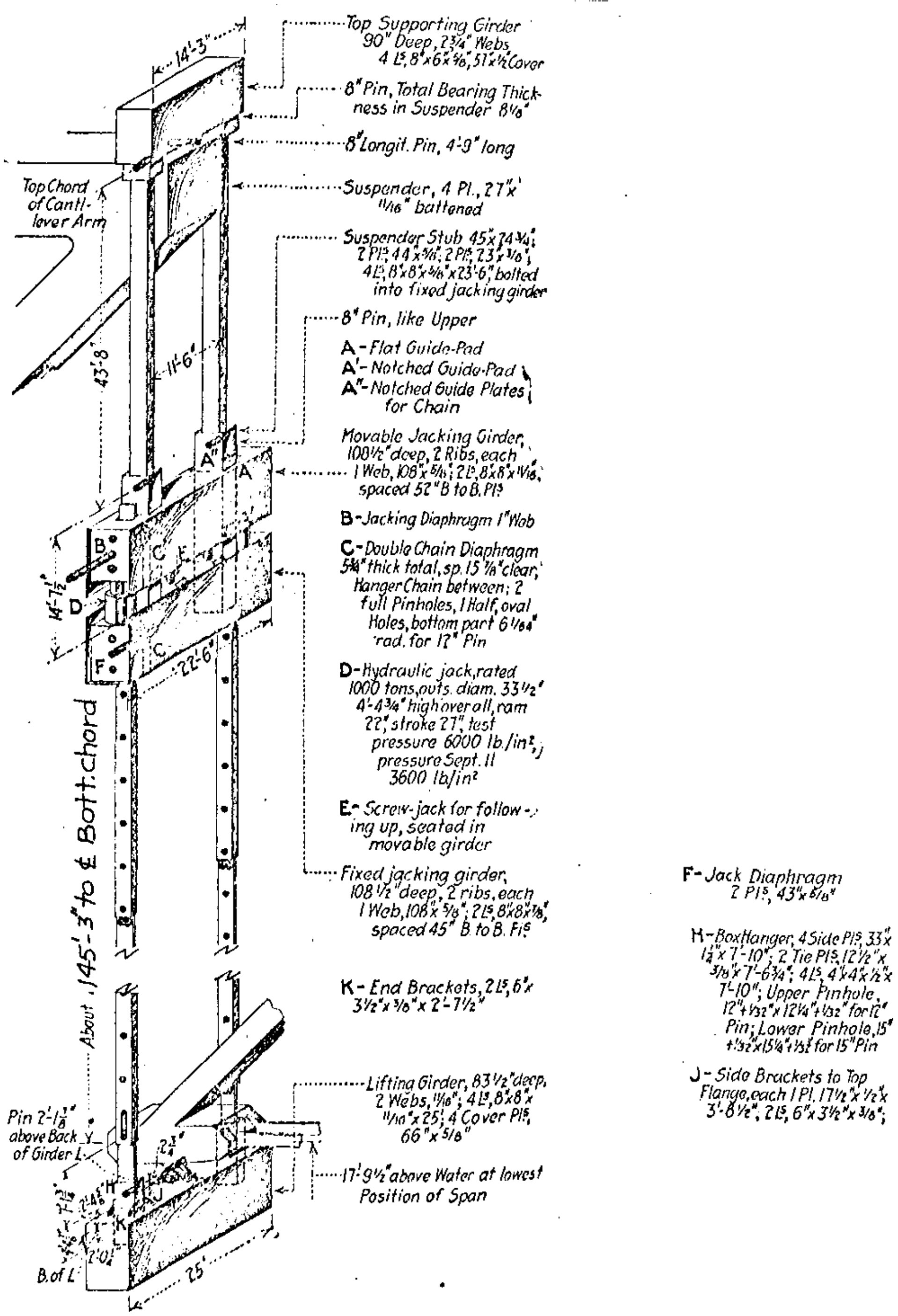



Figure 11. Diagram of hanger used for hoisting suspended span of the Second Quebec Bridge (12)

following the accident. The load on each of the rockers was 1,300 tons and this great load could account for the loud report preceding the collapse.

Measurements were taken of the hangar chain at the south end of the span and some stretching was observed. The observed stretch in the hangar opposite the one near the corner of the truss that first collapsed showed that the other rocker carried a load of 60 per cent to 100 per cent in excess of its load before collapse.

The question arose as to the design stresses in the rocker. The St. Lawrence Bridge Company, designers of the bridge, furnished the board of engineers with calculations used in design of the rockers. The design reaction at each corner of the truss was 3,000,000 lb. which included 20 per cent impact. There certainly was no impact on the rocker at the time of failure. The allowable stress was 20,000 psi in both the rockers and the hangars. The question arose as to why the allowable stress in the rocker castings could be as great as that in the hangars as the factor of safety for the hangars could definitely be established. The rockers were not tested prior to erection as the hangars had been. The rockers should have had a larger factor of safety due to possibly more variable material and a much more irregular stress pattern. The hangars had been tested up to failure at a stress of 60,000 lb.

The rotation of the lifting girder of an amount sufficient to permit the span to slip off the rocker had been seen as probable. The rotation could have been as little as 5 per cent which could be the angle of friction of the rocker on the pin. Many conditions would have had to be accurate to account for no rotation of the lifting girder -- the workmanship would have had to be accurate, the lower casting would have had to be bolted to the girder correctly, and the girders would have had to receive the load of the span correctly at beginning of raising. If the span had been moored by guy wires at a location not directly below its final position in the bridge and the hoisting chain attached, an eccentric moment may have resulted as the tide lowered and the span moved horizontally to its correct position between the cantilevers.

Other forces could have been introduced to cause rotation of the girder as the south end of the span was 2 ft. higher than the north end. Also, a temperature change took place which resulted in about a 2 1/2 in. expansion. This would tilt the girder slightly. The two conditions combined would tend to aggrevate the moments in the south girder and improve that in the north girder as the span was being raised.

The St. Lawrence Bridge Company accepted full responsibility for the accident and began almost immediately to replace the span. There was full confidence in the original design and a few minor alterations were made to assure that the bridge could not again slip off the rocker unto the lifting girder. A lead cushion was substituted for the second pin in the rocker casting thus resulting in a much more stable condition.

The erection progressed very rapidly and smoothly and the first crossing was made on October 17, 1917 with the bridge being fully completed and accepted by the Canadian government on August 21, 1918.

Germany's Frankenthal Bridge

In August of 1950 the construction was completed on the largest welded plate girder bridge in the world at that time (64). The bridge site was at Manneheim, Germany across the Rhine River. The construction was started in 1939 and the uncompleted bridge failed in December of 1940 when erection of the main girders was nearly complete. For erection sequence see Figure 12. Thirty-six construction workers lost their lives in the collapse which was initiated by the carelessness of the erection superintendent.

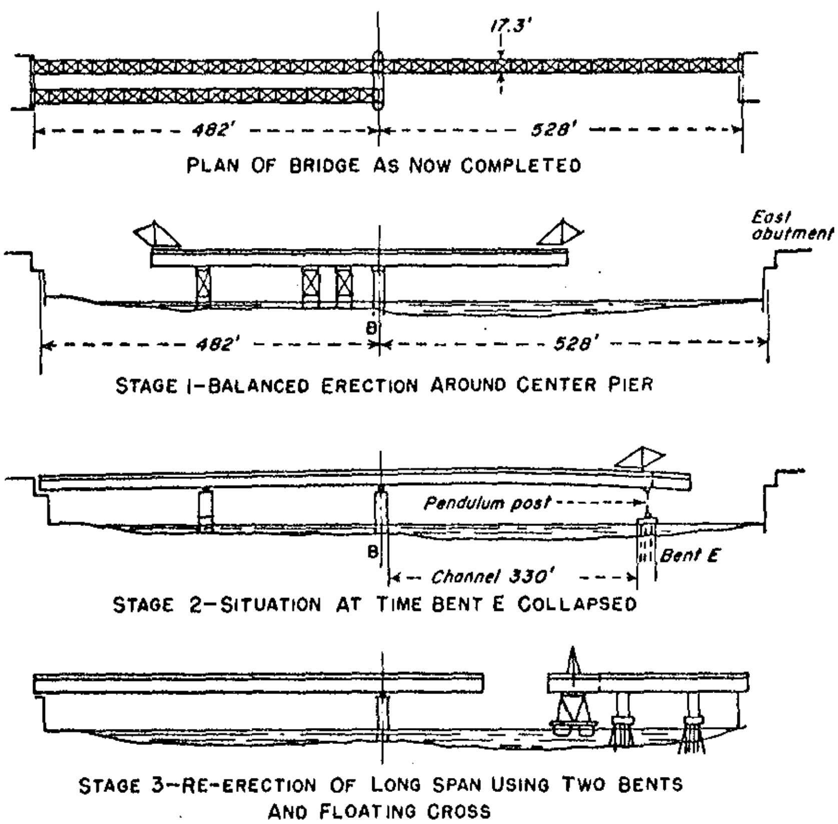
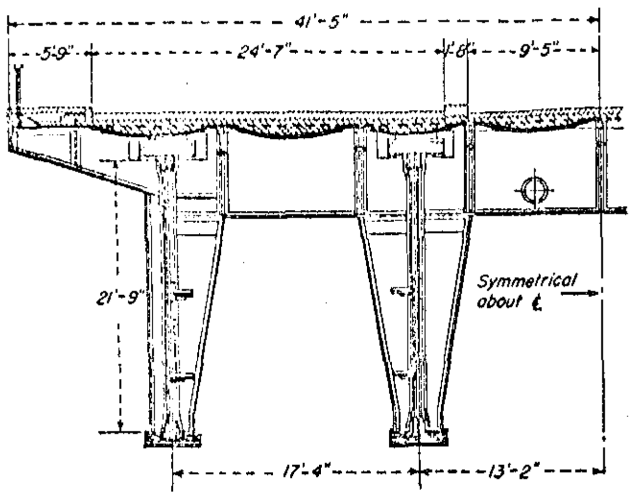



Figure 12. Erection sequence of Frankenthal Bridge (64)

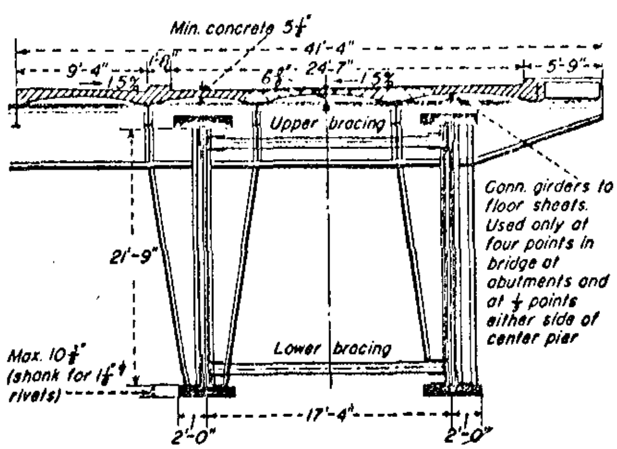
The original design of the bridge consisted of four lines of continuous welded plate girders with spans of 482 ft. and 528 feet. The girders were 21'-9" deep with webs of from 3/4 to 7/8 in. thick and flanges up to 4 ft. wide and 10 1/2 in. thick. The 528 ft. span was over the navigational channel and a clear width of 330 ft. was required by the navigation authorities which necessitated a cantilever method of construction. A balanced section was constructed over the center pier as stage 1, this being supported as necessary by temporary piers. The 482 ft. span had been completed and the other span was cantilevered a distance of 370 ft. to a temporary pier. The supporting structure was not complete at this time so erection was continued for another 46 feet. At this point the deflection of the free end of the girder was about 6 feet. A jacking process was planned to place the girders on the supporting mechanism of the temporary pier. About 300 tons of dead load were added by the 46 ft. extension which had not been anticipated in the original design. During the jacking process a large horizontal force was created by uneven raising of the jacks. A further eccentricity was created by a new angle at which the girders were deflecting. Also adding to the eccentricity was the deflection of the pile group of the temporary pier. The combination of these horizontal forces caused the collapse of the temporary pier and the span yielded at the point of contraflexure.

The wrecked span had to be cleared from the channel with very little salvage value. World War II delayed completion of the bridge for nine years.

Reconstruction of the east span began in 1950 and it was decided to use only two girders instead of the four girders in the original design (see Figure 13). This resulted in a very complicated and costly transition and it was decided to cut the floor beams of the western span at the centerline. This alteration resulted in a more slender span and thereby required lateral bracing between the two girders.




HALF CROSS SECTION—ORIGINAL DESIGN (4 GIRDERS)




TYPICAL CROSS SECTION NEW SPAN (2 GIRDERS)

Figure 13. Diagrams of the original cross-section and the revised cross-section of the Frankenthal Bridge (64)

The method of erection was at this time considerably different from that at the time of collapse. Floating cranes and a sufficient number of temporary piers were used which resulted in much shorter cantilever portions.

Spokane River Bridge

A 250 ft. twin rib concrete arch bridge over the Spokane River in Spokane, Washington collapsed during construction of the concrete arch ribs on February 6, 1917 (20, 90, 102). Three workmen lost their lives. The arch ribs were 6 ft. square in cross-section at the top of the arch tapering to 6 ft. x 8 ft. at the abutments. The collapse took place without any warning while concrete was being placed. The arches were being erected on timber falsework, this being supported on timber piles driven to rock in the river bottom. The progress of the construction along with falsework details are shown in Figure 14. The point to which it was completed before its collapse is also shown.

The construction was under the supervision of the designer of the bridge who, just one week before the collapse, fell off one of the arches and drowned. The construction was then supervised by the city engineer.

The falsework consisted of 18 bents of 7 piles. The bents were spaced at 12 feet 4 inches. The falsework was as shown in Figure 14 with four lines of longitudinal sway bracing. The sway bracing was supposed to be bolted to the piles but instead only one to three spikes were used. Many of the piles had not had the bark removed and at least an 8 in. spike was necessary to reach through the sway bracing, the bark of the

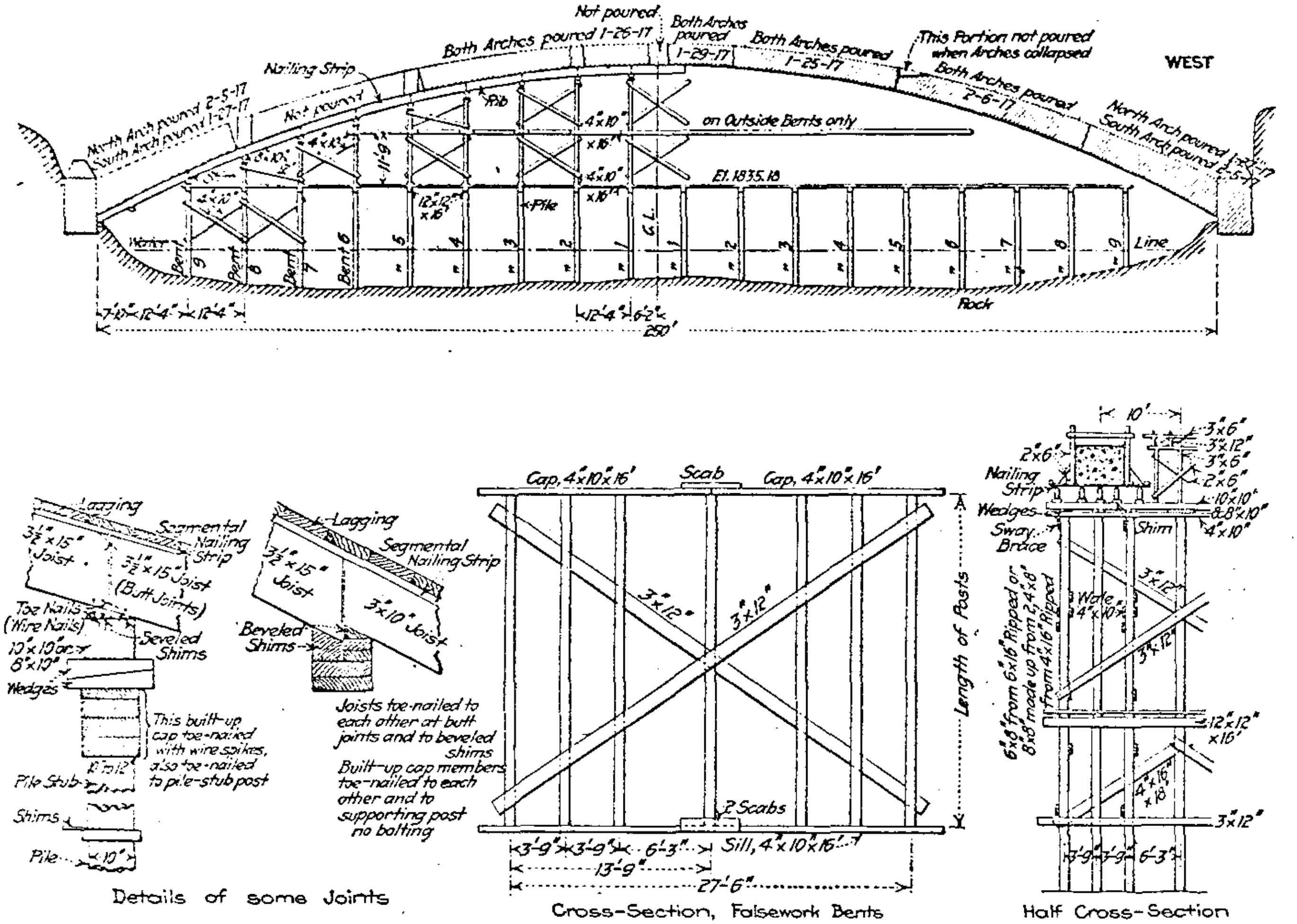



Figure 14. Details of Spokane River Bridge showing progress of construction and falsework (90)

pile, and 1 to 2 in. of soft sapwood. This would be a very insecure connection. The piles were said to be driven to solid rock but in some cases it was thought maybe small boulders caused refusal of the pile. Where boulders were known the piles were driven off center sometimes as much as 3 feet.

About 30 minutes before the collapse a check was made as to points at which concrete drops were made which might cause distortion of the falsework. Some concrete drops were as much as 15 ft. vertical. There were apparently no points along the arch which had been disturbed. It was estimated that the continuous impacts caused by dropping of concrete caused the failure. Also adding to the cause, was the weakness in the falsework due to insecure connections and very little pile penetration.

Tacoma, Washington Bridge

A rather unusual type of failure took place in a rather unusual type of bridge in the city of Tacoma, Washington in the fall of 1923 (39, 122). The failure consisted in a crack necessitating the closing of the bridge to traffic until repairs could be made. The failure was investigated by a committee of the local chapter of the American Association of Engineers and consisted of four men.

The bridge was an unusual type as shown in Figure 15. The shape of the arch was elliptical and carried a considerable amount of earth load as well as a railroad and two highway lanes. The width of the arch was 52 ft. which was made up of three separate arches of 18 ft., 16 ft., and 18 feet. These arches acted completely independent of each other.

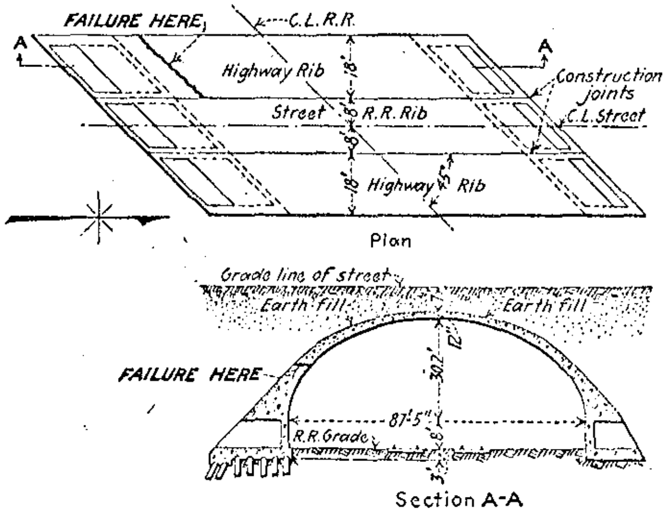



Figure 15. Sketch showing crack in Tacoma, Washington Bridge (39)

The failure did not result in a collapse but required immediate attention. The crack occurred at about the third point in the south end of the west rib. The nature of the crack was of a type which would require special attention and this is why a discussion of it is being included in this summary of bridge failures. Even though it was of a minor nature it could have resulted in a tragedy if a sudden collapse had taken place.

The crack was very distinct and consisted of separations of as much as 2 in. at places and the lower end was being pushed in due to the great force of the earth load. The plane of the crack was horizontal which permitted no resistance to lateral movement of the abutment. A number of the reinforcing bars had been broken and many showed signs of being stretched, which showed that the structure was nearing complete collapse.

A close examination of the crack showed that it had been a construction joint which had been incorrectly placed. There had been no bond with the rest of the arch. The joint should have been placed in such a way as to resist the thrust caused by the earth fill, in other words, it should have been more normal to the face of the arch.

There was laitance of from 1/2 to 1 in. at places along the crack. The laitance consisted of a fine whitish scum which formed on the surface of the green concrete because too much water was used in the cement mixture. Most of the laitance which had been in the joint had been washed out by seeping ground water and resulted in large openings along the crack. Also, there were found scraps of wood and rubbish, all of which showed great carelessness on the part of the contractor and the inspector. The laitance should have been removed before continuation of the pour but was allowed to set and was probably jarred loose by the vibrations caused by traffic. The concrete on the remainder of the structure was found to be of very good quality.

The cause of the failure, as described by the contractor, was that the concrete at the joint had received its initial set before resumption of pouring and that the workmen sent down into the forms to clean out the debris broke up the green concrete by walking on it and destroying its bond value. This was not the conclusion of the investigating committee. Their conclusion was that the cause of failure was due primarily to lack of bond due to laitance left in the construction joint, augmented by the fact that the joint was not made normal to the thrust of the arch ring.

The lesson of most value here is probably the dangers of allowing laitance to remain in a joint. Also, laitance will not occur if the cement mixture is correct. This is done by keeping the water-cement ratio to a minimum. This is probably not as much of a problem today because of the vibration methods used in placing concrete. The vibration permits the use of concrete with very low slumps, thereby keeping the amount of water used to a minimum.

## Summary

A discussion of bridge failures in this chapter indicates the importance of proper construction methods. The causes of failure are: modification of field methods without calculation of stresses, lack of knowledge and experience of the person in charge, instability of supporting members, a contractor attempting procedures not stated in specifications or design plans, use of faulty concrete with noncontinuous pour, and inadequate formwork.

### Ashtabula Bridge:

This bridge failure was a major disaster in that 92 lives were lost when a 156 ft. truss railroad bridge collapsed in 1876 (4, 5, 6).

The truss was the first attempt to change from the usual wood to wrought iron, increasing the weight considerably.

The erection was faulty in that many modifications were made in the field by adding members, cutting, chipping and forcing of members to fit. There was much disagreement of persons in charge in the methods of construction used. This, along with very low factors of safety, was the cause of failure.

Second Quebec Bridge:

This was a second attempt to span the St. Lawrence River where the First Quebec Bridge failed during construction in 1907 (3, 11, 12, 22, 38, 97). This second bridge was of the same type and the failure resulted in the loss of 13 workmen. It occurred in 1916 when the center span was being hoisted between the end cantilever sections. The center span fell from its chains into the river.

Modifications were made in the lifting apparatus to provide a more stable condition and the lifting procedure was then successful.

Germany's Frankenthal Bridge:

The construction of this bridge over the Rhine River at Manneheim, Germany began in 1939 (64). The construction procedure was by the cantilever method which was necessary to provide channel clearance for navigational purposes. The cantilever was 370 ft. beyond a pier at the time of collapse. Thirty-six workers lost their lives.

The cross-section of the bridge consisted of four welded girders but a transition was made to two girders when construction was again started in 1950.

The cause of the collapse was due to carelessness on the part of the construction superintendent in extending the cantilever beyond that for which it was designed before a temporary pier was built to adequately support it.

Spokane River Bridge:

The collapse of this bridge took place in 1917 during the construction operation and just shortly before completion of the concrete arches (20, 90, 102).

It was determined that failure was due to incorrect concrete pours which were from great heights and too many being concentrated at one location.

Another factor which contributed to the failure was the weakness of the falsework. Many of the connections were very insecure and pile penetration was very inadequate.

Tacoma, Washington Bridge:

This was not a collapse but it was impending (39, 122). The bridge was in the form of an arch and failure was taking place along a horizontal construction joint. Faulty construction in allowing laitance in the construction joint resulted in large openings. Large earth pressure began to make the condition more serious along with traffic loads.

# FAILURES DUE TO BRTTTLE FRACTURE

## General

There are three bridge failures summarized in this chapter which were mainly a result of brittle fracture.

The increasing use of high strength steels has increased attention in the phenomenon of the failure of a ductile material in a brittle manner. It is not so much the material being brittle, but rather it is the susceptibility to brittle failure that can be built into structures with improper design.

It is more economical to design with the use of high strength steels, but at the same time there is more of a chance for improper design than in conventional steel. Proper design is taking advantage of ordinary and economical grades of steel and using them to their fullest potential.

The following summary will discuss various ways by which steel bridges have failed under conditions making them more susceptible to brittle fracture.

### Kings Bridge

The collapse of the Kings Bridge at Melbourne, Australia on July 10, 1962 was of a somewhat unique character and one from which many lessons can be learned (15, 45, 53).

The failure was a partial collapse of a 100 ft. span and took place when a truck transporting a 30 ton crane passed over the bridge. The partial collapse resulted from brittle fracture of the steel and was brought about by improper steel for welding, unsatisfactory design details and low temperatures.

The Kings Bridge consisted of two 2300 ft. long parallel one-way bridges over the Yarra River. The bridge consisted of four lines of welded girders made up of high tensile steel with a reinforced concrete deck. The bottom flange was strengthened by the use of cover plates which terminated about 16 ft. from the end of the girder. The web was strengthened by use of vertical intermediate stiffeners welded to the web.

The failure was investigated by a Royal Commission of Enquiry and was reported to Parliament, the main part of the report being that there was insufficient notch ductibility of the steel used in the girders. Cracks occurred at seven different places within the one span. Three of the girders had cracks at both ends of the cover plate and the fourth girder had a crack at one end of its cover plate. Some of the cracks were completely through the flange and almost through the web.

The Royal Commission report stated that the cracks were caused by the unfamiliarity of the fabricator with the problems of welding low-alloy steel, and the quality of the steel, much of which was so high in carbon and so unexpectedly variable, that even an experienced fabricator would have had difficulties in welding it.

The specifications were to conform to the British Standard Specification 968: 1941. The minimum required yield point for the low-alloy high tensile strength steel was 55,000 psi. The chemical composition was required to be as follows: carbon 0.23 per cent; manganese 1.8 per cent; chromium 1.0 per cent; and manganese plus chromium 2.0 per cent. The report showed the analysis of the actual failed girders to be as follows: carbon 0.28 per cent; manganese 1.80 per cent and chromium 0.25 per cent. This analysis shows that the carbon content was too high resulting in a more brittle steel.

Another factor which was thought to be partly responsible for the failure was the fact that the design was done as a joint venture by four consulting firms under a subcontract to the general contractor. This made it unsuitable to provide the necessary supervision which should have been required normally. If a firm had been hired directly by the agency who was to own the bridge, there would have been inspectors directly responsible on the job at all times.

One major event which was critical occurred when the fabricators placed an order with a producer for steel in accordance with the specifications but statements requiring certain additional tests were omitted. Somehow they were bound to accept the steel without these tests and at the same time, were bound to the contractor to supply girders of steel which had been subjected to these additional tests. As a result, the fabricators failed to make sufficient tests to ensure the notch ductility and correct chemical analysis.

The method of welding was a large factor also. It is necessary to preheat any low-alloy high tensile steel before welding. It was concluded that there was an inadequate amount of preheating.

A brittle fracture takes place more readily in a high carbon steel with a decreasing temperature. There is also more of a tendency for this type of failure with an increase in material thickness. A brittle failure may take place very easily if a sharp notch is present in the steel at a point of high stress. Brittle fractures have occurred at lower than permissible working stresses. This is only possible when there are high residual stresses present. This area of residual stress usually occurs at welded connections with yielding at the welded area. If the fracture is arrested before complete failure the only way to re-initiate the fracture is for the remaining cross-section to be stressed to the yield point.

There must be three conditions to allow brittle fractures to take place. First there must be a notch or flaw. Second the temperature of the metal must be below the transition temperature which is the temperature at which the steel changes from ductile to brittle state. This transition temperature varies with the chemical composition, thickness and the rolling temperature. Third there must be yield stresses present. These could be partly caused by loading and partly by residual stresses.

From the inspection of the fractures in the Kings Bridge it was evident that they were two-stage or three-stage fractures. It is quite clear that small surface cracks occurred at the toe of the cover plate caused by improper welding in this area. These small cracks acted as notches and probably took place in the fabricating shop. Even small surface cracks would decrease the section modulus a great deal and, as a result, the remaining cross-section could become overstressed. This would re-initiate the fracture.

Welding of high tensile strength steel can be done, but the steel must be preheated. Before any welding takes place a sample of the same steel used in the structure should be welded and checked under a microscope for cracks. Its chemical composition must be satisfactory. It must be certain that the same chemical composition exists in the material for the actual structure.

The steel used in the Kings Bridge had various chemical compositions. The British Standard Specifications 968: 1941 required the chemical composition to be specified by ladle analysis. The amount of carbon was in some cases considerably higher than that specified by the ladle analysis. The manganese and chromium were also at their upper limits. Therefore the tensile strength was much higher than specified which made welding even more difficult. This fact along with a lack of knowledge as to how to weld this type of steel, variable composition and poor inspection made failure almost certain to take place. The fabricator should obtain information on the actual chemical analysis of the plates shipped from the manufacturer.

## Hasselt Bridge

Another bridge failure due to brittle fracture was the complete collapse of a 245 ft. Vierendeel truss bridge near Hasselt, Belgium on March 14, 1938 (24, 25, 32, 68).

The bridge was supported only at the abutments and was made up of a curved top chord, twelve vertical rigid frame panels without diagonals, and a horizontal lower chord which carried a 32 ft. roadway and two 5 ft.

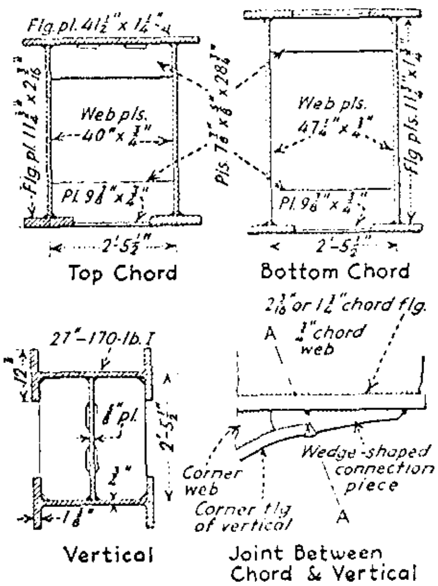



Figure 16. Structural details of Hasselt Bridge members; numerous fractures occurred along line AA (25)

sidewalks. Figure 16 shows the make-up of all the members. All splices in the bridge were of the butt weld type with no splice plates. There were twelve panels and panels 2, 4, 7, 9 and 11 contained field welds in the top and bottom chords. The curved gusset plates were shop welded which allowed for the straight vertical portion of the panels to be welded in the field.

The collapse was gradual and time was allowed for the bridge to be cleared of any traffic. The collapse began with a loud report and a crack appeared in the lower chord between the third and fourth verticals. The bridge did not fall as it was temporarily being supported by the top arch chord. It took about six minutes for a horizontal shear at the abutments to take place and then the bridge broke in two places with the center portion lying on the river bottom (see Figure 17). This bridge had only been in operation 14 months and its cost was $125,000.

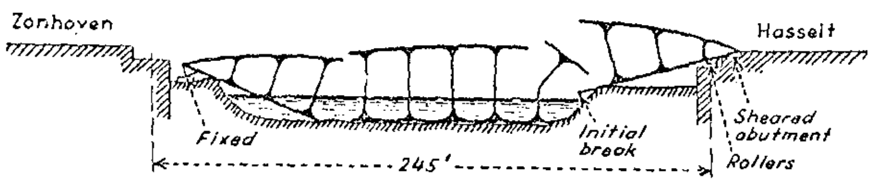



Figure 17. The Hasselt Bridge after collapse (25)

There should have been sufficient experience gained in the construction of this type of bridge as 50 had been built in Belgium in the previous six years. The design of this type of bridge originally consisted of shop welds and field riveting but had recently been changed to field welding only.

The cause of the collapse, as reported by the Belgian Commission appointed to investigate, was poor quality steel along with poor welding. The steel which was required in the specifications was to have been Belgian St. 42 with a tensile strength of from 52,000 psi to 64,000 psi.

One factor which appeared very questionable was the welding of the 3/4 in. web plates to 2 1/8 in. thick flange plates with only a 3/8 in. weld. Also the homogeneity of such thick plates was a matter of concern.

The carbon content was to be 0.15 per cent and the sulphur and phosphorus not more than 0.03 per cent. This chemical analysis would almost certainly result in a metal very satisfactory for welding. The primary fractures occurred at points of weld and it was believed that the heat used in welding had made the steel brittle. Other fractures occurred a distance away from welds but these were a result of the impact during collapse.

There was believed to be stress concentrations during welding at the splice between the curved gusset plates and the top and bottom chords. This welding had to be done with both ends being quite rigidly restrained. Many fractures occurred at these points although they probably occurred during the fall of the bridge.

The erection was a problem due to the poor alignment of the adjacent members in the shop. This resulted in additional stresses caused by forcing of members to fit and also additional welds being used to make up the difference in alignment.

The quality of the steel was undoubtedly poor as the appearance of the fractures were quite similar to that of cast-iron. The quality of the weld was also poor and would not have been accepted as complying with any of the structural weld specifications of today. There was lack of penetration, undercutting and very irregular surfaces. It appeared that very large electrodes were used which also could cause additional stresses. Also many field welds were made in danger zones which resulted in internal stresses being built up.

The effects of welding on a steel structure is an area which requires a large amount of study and experience. Shrinkage stresses also can be introduced by welding.

Although about fifty bridges of similar design had been built in Belgium, this bridge was probably a victim of an unfortunate combination of factors contributing to its failure.

Duplessis Bridge

The Duplessis Bridge spanning the St. Maurice River at Three Rivers, Quebec collapsed at 3:00 a.m. on January 31, 1951 and four lives were lost (16, 24, 29, 116, 123). The temperature was 30° below zero and fortunately the traffic was very light at the time or possibly more lives would have been lost.

The bridge was completed in 1947 and was of the welded girder type with two lines of girders, a composite reinforced concrete deck and transverse floor beams. The cost of the bridge was $3,000,000. There actually were two bridges, one referred to as the East Crossing and the other the West Crossing. The two were joined by a small island at the center of the river. The failure took place in the West Crossing which consisted of eight spans of continuous steel girders. This west structure was 1380 ft. long having six 180 ft. spans and two 150 ft. spans. The deck was made up of an 8 in. thick reinforced concrete deck which acted compositely with the top flanges of the girders. The roadway was 42 ft. wide and carried four lanes of traffic. It was designed for alternate loading of one 20 ton truck per lane with a 30 per cent impact loading and a 100 psf uniform load on the floor beams and 70 psf on the girders. The depth of the girders was 12 ft. at the piers and 8 ft. at midspan.

The bridge was designed by the engineering staff of the Department of Public Works of the Province of Quebec and constructed by Dufresne Engineering Company, Ltd. of Montreal. The structural steel was fabricated and erected by the Dominion Bridge Company, Ltd., Lackine, Province of Quebec.

The four west spans of the West Crossing fell into the ice covered river without warning. There were partial failures prior to this collapse which seem to indicate the cause of the January 31 collapse. The first failure took place in February of 1950 with cracking of the top flange taking place in a negative moment section at the end weld of the cover plates in the heat affected zone. These fractures were discovered at various places but one had been located in one of the spans that failed. Some of the fractures extended down through the web and partially into the lower flange. The reason there was no collapse at that time was because of the composite action and the deck reinforcement resisting the tension forces. The concrete deck was serving as the top tension flange until repair.

The board appointed to investigate the bridge failure consisted of a lawyer and a civil engineer, both of Montreal. It was at first reasonable to assume that the collapse was caused by a brittle fracture of the steel resulting from faulty material with a notch sensitivity very abnormal. This was later observed in laboratory tests. The plates which made up the girders were of poor quality rimmed steel which means there were gas bubbles entrapped in the steel during solidification. A high percentage of carbon and sulfur was discovered in the chemical analysis. The structural steel was supplied by a Canadian mill that normally produced sheets and strips of steel for which rimmed steel was suitable. The structural steel with the above characteristics has a great tendency for brittle fracture at low temperatures. Analyses of the steel showed carbon content ranging from 0.23 to 0.40 per cent; the sulphur from 0.04 to 0.116 per cent; and manganese from 0.30 to 0.33 per cent. This indicated extensive segregation in the chemical analyses. Also yield strengths had been reported varying from 27,800 psi to 57,800 psi with an average tensile strength of 58,000 psi. The high carbon portions of the plates were undesirable for welding. Although the weld appeared to be of good quality, there may have been some undetected cracks. Authorities on welding consider use of such a steel inadvisable in a welded structure because high carbon and impurities lead to poor notch toughness especially in cold weather.

## Summary

It is hoped the reader has become more aware of the conditions that lead to brittleness of otherwise ductile steel. It is of utmost importance that a designer becomes aware that these conditions exist. With this knowledge he then can obtain the necessary design criteria to minimize the chances of brittle fracture.

The following factors increase the susceptibility of steel to brittle fracture: improper welds, low temperatures, high carbon content of the steel, and stress concentrations due to field welding of very rigid joints. These were all found in the bridge failures summarized below.

Kings Bridge:

The collapse of a portion of the Kings Bridge at Melbourne, Australia in 1962 was a lesson in the use of high strength steels along with lack of knowledge in its use and weldability (15, 45, 53). Also the common aspect of inadequate inspection was an important factor in the collapse.

The cause of the failure was low notch ductility along with variable carbon content of the steel and unfamiliarity of the fabricator with the welding of a steel of such high tensile strength.

There were several cracks in the flanges and webs along the bridge length. The most common occurrence of cracking was at the toe of the cover plates. These cracks originated in the welded portion and were caused by residual stresses near the weld.

Hasselt Bridge:

Another example of brittle fracture was the collapse in 1938 in Hasselt, Belgium of a 245 ft. single span Vierendeel truss type bridge (24, 25, 32, 68). The truss consisted of an arch upper chord and vertical panels which were rigidly welded to the top and bottom chords. The connections were welded at the construction site rather than riveted which had been done previously in many other bridges of similar type.

Cause of the collapse was poor weldability of unsatisfactory material. The carbon content was unusually high, resulting in brittle steel. Also, some connections had been forced into alignment and welded thus causing additional residual stresses over and above those caused by welding.

Duplessis Bridge:

This bridge was located at Three Rivers, Quebec and a portion of it collapsed in 1951 on a cold winter day with a loss of four lives (16, 24, 29, 116, 123).

This bridge was of the welded girder type and failure was caused by brittle fracture. The very low temperature at the time accounted for its low ductility along with a carbon content much higher than specifications would allow. This also resulted in poor weldability in the girders.

# FAILURES DUE TO INADEQUATE FOUNDATIONS

## General

Four bridge failures are discussed in this report which were caused by inadequate foundation material. This type of failure, however, does not seem to belong in the same category with structural failures. The structure may be very sound structurally but with the failure of the supporting material the structure must yield. This yielding, of course, results in stresses which cannot possibly be resisted.

Foundation material should be investigated by persons well qualified in that specific field. There are many variables with which to deal. Some sound judgment, along with past experience, is necessary. Therefore, it is a very special field and the designer must rely on the results of their study. With a maximum allowable bearing value obtained the designer may then proceed with the design of the structure.

It is the object of this section to give the reader a better understanding of the importance of a satisfactory foundation. In subsequent paragraphs, a brief summary is given of the failures of the four bridge failures resulting from foundation problems.

### Custer Creek Railroad Bridge

There were 47 lives lost when a passenger train of the Chicago, Milwaukee, St. Paul and Pacific Railroad plunged into Custer Creek on June 19, 1938 (87, 101). The foundation of the piers had been undermined by flash flooding due to a cloudburst about 25 miles upstream from the site of the accident. Custer Creek is normally a dry run creek about nine months of the year. The engine, along with two coaches, two tourist sleepers, a baggage car and a mail car plunged into the stream without any warning.

The bridge was made up of eight spans, three of which were continuous plate girders and five which were concrete trestle spans. All were supported on concrete piers which had spread footings on gravel 10 ft. below stream bed.

The bridge had been inspected only two months prior to the accident and appeared to be in excellent condition. The Milwaukee bridges were routinely inspected four times a year. This bridge was 25 years old and had received no trouble due to flooding during its lifetime.

Today it is probably unheard of to build a pier foundation on gravel. Piling are normally used to support a pier and spread footings are only used when solid rock can be found near to the surface of the ground. Usually if there is solid rock which can be reached with a reasonable length of pile the pile will be driven to bear on the rock. As in the case of gravel, pile bearing by friction can be built up very rapidly, thus requiring a rather short pile.

Peace River Bridge

A foundation failure was the cause of a collapse on October 16, 1957 of the Peace River Bridge on the Alcan Highway in British Columbia (19, 78, 79). The bridge had a total length of 2,130 feet. This was made up of a main suspended span of 930 ft, suspended end spans of 465 ft.

each and simple supported truss spans of 135 feet. The truss spans connected the anchorage blocks with the first piers. The main suspension cables were made up of 1 7/8 in. strands in rectangular shape. There had been no movement of the anchorage blocks since it was built in 1942 as a part of the rush war program to complete the Alcan Highway. Slipping of the anchorage block at the north shore was noticed twelve hours before the collapse and was evidenced by the breaking of a water supply line passing in front of the block. The supply line was for the Pacific Petroleum Company which is a part of great industrial activity in the two years preceding the failure. It was thought that possibly the disturbance of the bank was brought about by considerable construction in the area adjacent to the bridge.

The anchorage was founded on shale which may have deteriorated into clay. No piling had been used under the anchorage blocks or under the piers because of great amount of haste in its construction. Any movement was to be resisted by keying into what was considered very firm shale.

When there was noticeable movement of the anchorage, traffic was removed and a large crowd gathered to watch the collapse. At the time of the collapse the anchorage blocks had slipped a distance of 12 ft. at the north shore. This slipping caused the first pier to topple, which dropped the truss span and also the adjacent suspended span into the river. The other spans remained intact but there was a large deflection in the main span.

According to a consulting engineer this failure was the end to many partial failures that had occurred since its construction. Soon after its opening to traffic there was observed to be a great amount of vibration in the suspension cables which resulted in breaking of strands. Seventy-one broken strands were reported and repaired at various times during 1947. It was reported by the Advisory Board on the Investigation of Suspension Bridges in 1952 that there was serious oscillation in the wind and in 1955 serious scouring of one pier took place.

The loss of this bridge created problems in how to handle the traffic which normally passed over the bridge. A ferry was used as a temporary measure and a railroad bridge near the site was decked over and used for highway vehicles.

A study of the remaining portion of the bridge took place with the result that repair was not feasible and that the bridge should be replaced.

New River Bridge

The failure of a reinforced concrete bridge across the New River in the Imperial Valley in southern California took place in the early twenties (40). This was a progressive failure and was the subject of attention of the California State Highway Department for some time.

In 1905 and 1906 the Colorado River overflowed its banks and spread out over a strip of land about six miles wide and extended from the Salton Sea to the Mexican border. As the waters receded a new channel began to be formed. At first the channel was approximately 800 ft. wide and was eroded to a final depth below the original ground level of about 100 feet. The final cutting resulted in a narrow gorge about 60 ft. deep at the center of the newly formed river. A cross-section of the river at the location of the bridge is shown in Figure 18.

The soil consisted of loose loam and appeared to be the same type throughout the depth of the pier piling.

The bridge was built in 1918 by Imperial County on the State Highway from San Diego to Imperial Valley and spans the New River. The bridge consisted of eleven bents of 30 ft. each making the length of the bridge 330 feet. Three of the bents were in the deep cut of the channel which had been cut even deeper since the construction of the bridge thus leaving a considerable amount of the piles exposed in two of the bents. There were two clusters of concrete piles for each bent, above which was formed a reinforced concrete footing and a square concrete pier column. There was great difficulty in driving the piles which would seem to indicate a stable material.

In early 1920 there appeared to be a lateral movement of the center of the bridge indicated by cracks forming with apparent tilting of the

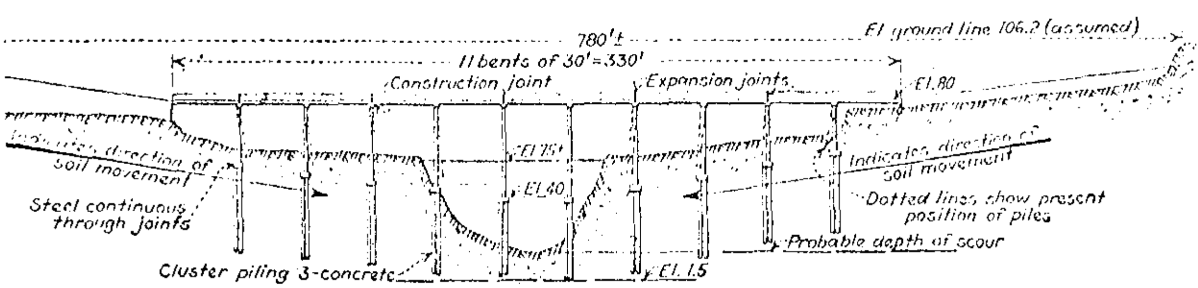



Figure 18. Reinforced concrete trestle over New River in Imperial Valley, showing movement of bents (40)

pier columns in the downstream direction. Cracks were appearing at or near the ground line and at the connection between the pier caps and the concrete girders. The abutments also were moving towards the center of the stream. The first bent from the west abutment showed a movement of the bottom of the column relative to the top of about 10 in. towards the center of the river.

The apparent cause of the failure which became increasingly more serious was the lateral movement of the soil, this movement being permitted by the release of pressure due to the cutting of the deep channel. There were also settlements of as much as 1 ft. at the approaches to the bridge.

Big Sioux River Bridge

One of the twin bridges carrying Interstate 29 over the Big Sioux River between Iowa and South Dakota collapsed in April of 1962 as a result of high flood waters (47, 91). The flood waters were about 25 ft. above normal at the time of the collapse. There was no traffic on the bridge at the time of the collapse.

The bridge was completed in 1959 at a cost of about $200,000. It was designed to withstand the greatest flood which may occur in a 50 year period. The cost of the bridge was shared by the federal government at a rate of 90 per cent, the remainder being shared between South Dakota State Highway Department and Iowa State Highway Department. The federal government's share in any repair is only 50 per cent of the repair cost. The bridge was a continuous welded plate girder with a total length of

556 ft. with a reinforced concrete deck and supported on four piers. The piers were supported by timber piles and the apparent undermining of one of these piers by flood waters was the primary cause of the collapse.

The bridge was located just upstream from the junction of the Big Sioux with the Missouri River. At the time of the very high flows in the Big Sioux the Corps of Engineers had cut the flow in the Missouri from 15,000 cfs to 5,000 cfs at Garvin's Point Dam, 75 miles upstream to prevent flood damage and the flow in the Big Sioux was measured at 55,000 cfs. This great differential was thought to have caused a very high velocity at the bridge site thus causing scouring at the vicinity of the piers. The piers were built at an angle of about $30^{\circ}$ with the direction of flow evidently with the thought of a channel change sometime in the future. This could have caused some eddying at the piers.

The conclusion arrived at by the Bureau of Public Roads was as follows (47):

1. The location of the bridge was not considered the best from the engineering point of view but seemed to be a result of a right-of-way problem

2. The combination of the very high flooding in both 1960 and 1962 along with the skewed piers which reduced the effective waterway by 35 to 40 per cent were large factors in the failure

3. Insufficient study was given by both state departments and the Bureau of Public Roads to the effect of obstructions in the channel

4. It was noticed that scouring of the banks was taking place and attention was given to that and scouring under the piers went unnoticed

5. The recently constructed Fort Randall and Gavin's Point Dam changed the flood-flow conditions in the Missouri River.

This failure points out the fact that more study is needed of the bridge site as to its hydraulic and foundation characteristics. Also, there is need for more thorough inspection following any flooding in the area.

## Summary

The foundation failures discussed indicate to the reader what type of problems may result from improper foundation material. It is difficult to determine which type of foundation failure is the most common.

Shale is a satisfactory foundation material but care must be taken as deterioration sometimes takes place when subjected to air or water. Exposure of the shale during excavating to place foundations many times results in difficulty in the future. It is quite common for foundations keyed into shale to eventually have some lateral movement.

Foundations placed on sand or gravel are obviously susceptible to failure in the event of rapidly moving water. The placing of piers within a stream in a manner which acts as an obstruction to the normal flow of water can cause serious problems. This must be given considerable thought and study prior to the design and must be inspected regularly following completion of the bridge construction.

The following bridge failures will illustrate the problems involved with various foundation materials.

Custer Creek Railroad Bridge:

As a result of a flash flood this railroad bridge collapsed in 1938 with the tragic loss of 47 lives (87, 101).

The bridge was made up of eight spans. The piers were supported by spread footings on gravel 10 ft. below stream bed which was normally a dry run creek. The undermining of the piers caused a sudden collapse of the bridge.

Peace River Bridge:

This was a bridge failure which was a result of the failure of foundation material (19, 78, 79). The bridge was a suspension bridge with a main span of 930 feet. Anchorage was made by large concrete deadmen which were keyed into shale. This shale apparently had deteriorated and allowed the deadman to move at one end of the bridge. A movement of 12 ft. caused the first pier to collapse and followed by the collapse of the main suspended span.

New River Bridge:

The failure of this bridge was unusual in that the foundation material moved laterally from both abutments toward the center of the stream (40). The piers moved toward the center of the stream also.

The movement of the soil was due to a deep channel being formed by flooding of the Colorado River. This channel caused the release of passive pressure and slipping began.

Big Sioux River Bridge:

This bridge failure took place in 1962 where Interstate 29 crosses the Big Sioux River between Iowa and South Dakota (47, 91).

The failure was a result of a large flood undermining one of the piers but the conditions at the time were aggravated by insufficient hydraulic study, skewed piers with future channel change proposed, improper maintenance inspection, and a high flow gradient caused by lowering of the flow in the Missouri River into which the Big Sioux flows.

# FAILURES DUE TO UNDETERMINED CAUSE

## General

There are four bridge failures discussed in this report where the cause of failure is undetermined. In three of the failures the courts were unable to place the responsibility of the failure on anyone. In the most recent failure, the Point Pleasant Bridge, the cause is still under investigation.

Quite often it is difficult or impossible to determine where the responsibility for a failure lies. In the event of the collapse of a bridge pier it is difficult to determine whether the failure was due to scour or to poor foundation material. If scour could be determined as the cause it would possibly indicate lack of maintenance or lack of design in providing adequate protection. If poor foundation material is definitely the cause then the responsibility lies with the designer. The designer must obtain adequate preliminary information regarding the foundation material and hydraulics. An example of this is the failure of the Claverack Bridge in the following summary.

It is hoped that the reader may become more aware of the unusual circumstances which may take place. Every significant step in the design of a structure must be given the greatest amount of consideration. The engineer must be able to keep an "open mind" and not let the design become so routine that he fails to consider significant factors.

The following summary of bridge failures will discuss the types of failures where the cause was undetermined.

Point Pleasant Bridge

The collapse of the suspension bridge over the Ohio River between Point Pleasant, West Virginia and Gallipolis, Ohio on December 15, 1967 is probably the bridge failure of most interest to engineers and to the general public at the present time (31, 49, 82). This is due to the tragic loss of life and to the fact that investigations into the cause are being carried on at the present time.

First of all it would seem proper before discussing and trying to analyze the failure to go back to 1929 and describe the construction of the bridge. This bridge was of a unique design and still is quite unique today. This bridge was the first of its type to be built in the United States. The unique features of the bridge were: unusual type of anchorages, the use of heat-treated eyebar chains which were used in part to take the place of the top chord of the stiffening truss, and inclusion of methods to make any adjustments in the chains, hangers, or trusses after erection was complete.

The total length of the bridge was 2,235 ft., which included a center span of 700 ft. and spans of 380 ft. with the remaining 775 ft. divided between the two anchorages. The roadway width was 22 ft. with a 5 ft. sidewalk and designed for an American Society of Civil Engineers H-15 loading.

The original design consisted of the usual type of continuous suspension cables and a stiffening truss. An alternative design was later presented for bids, this alternative being the one that was built. This design consisted of approximately half of the top chord of the stiffening truss as a part of the suspension system which was made up of eyebars. The bridge was owned by the West Virginia-Ohio River Bridge Company and was operated as a toll bridge replacing ferry service in the past.

The unique feature of the anchorage was the fact that it did not consist of the usual gravity type anchorages on solid rock. The solid rock was too far down in this case and would have been much too expensive to reach. The horizontal pull of the suspension cables on the anchorage was about 4,500,000 lbs. The anchorage design was as shown in Figure 19, with the horizontal force of the cables being resisted by the dead load of the anchorage and the shear resistance of the concrete piles. There were 405 octagonal reinforced concrete piles, 16 in. diameter, driven to about 12 ft. with the top 3 ft. embedded in the bottom of the concrete trough which was 32 ft. wide. This trough was filled with earth fill and the approach roadway was poured on it.

The eyebars of the chain were heat-treated carbon steel with an ultimate strength of 105,000 psi and had an elastic limit of 75,000 psi. The design working stress was 50,000 psi. The chains formed the top chord of the stiffening truss for 12 of the 28 panels of the center span and the first 7 panels of the end spans.

The towers were 103'-10 1/4" high and were designed to allow for change in length of the spans due to temperature and to permit movement due to moving live loads. This was permitted by the design of a rocker at the base of the tower, the upper part of the shoe being curved and the lower part being flat.

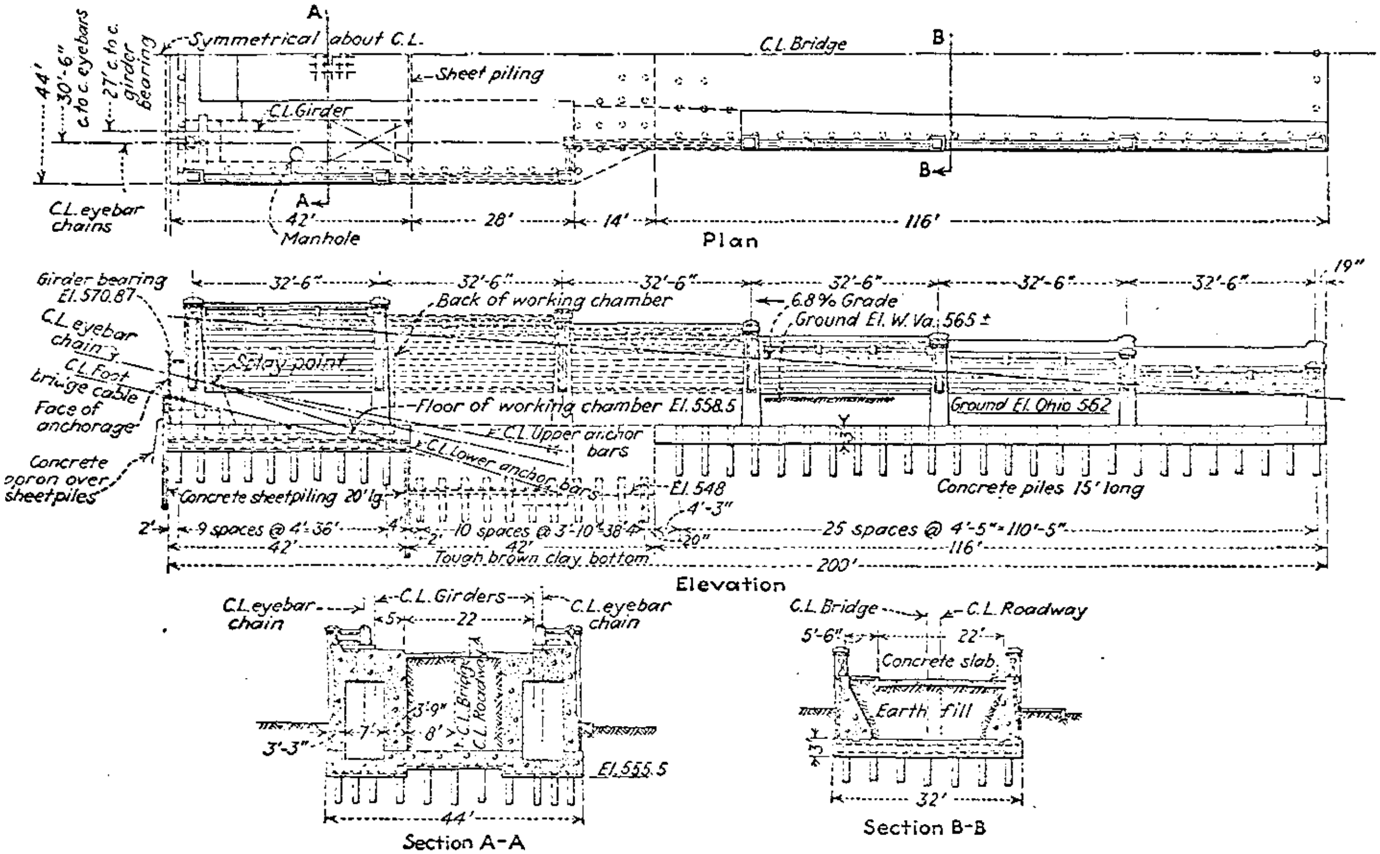



Figure 19. Details of anchorage of Point Pleasant Bridge (49)

The make-up of the stiffening truss can be seen in Figure 20 and were of the laced channel type. The fact that there was no allowance for any adjustments to be made when erection was complete was an interesting feature. The last panel to be placed, which was the panel at the center of the bridge, could not be fabricated until final measurements could be made. The only error to be found was 5/16 in. in placing of the anchor bolts which was very remarkable.

The eyebar chains were erected by use of a traveler supported by an erection cable as shown in Figure 20. The cables were adjusted to a sag of 46 ft. in the main span under no load and a temperature of $68^{\circ}$ F. With any other temperature the necessary adjustments had to be made. Erection of the eyebars proceeded from the center of the main span toward the tower and at the same time from the cable bents toward the towers, with the final link placed at the tower. This was done to maintain balance throughout the bridge.

The stiffening truss was erected in the same sequence as the eyebar chain. When the top chord was replaced by the eyebar chain the gusset plate which was to receive the diagonals was not drilled until all members and dead load were in place. In order to complete the truss before the roadway was in place, drums filled with water were used to supply the equivalent dead load.

The bridge was designed by J. E. Greiner Company and built under its supervision. The General Contracting Corporation of Pittsburgh, Pa., was the general contractor of the piers, anchorages, and approaches. The American Bridge Company furnished and erected the steel superstructure.

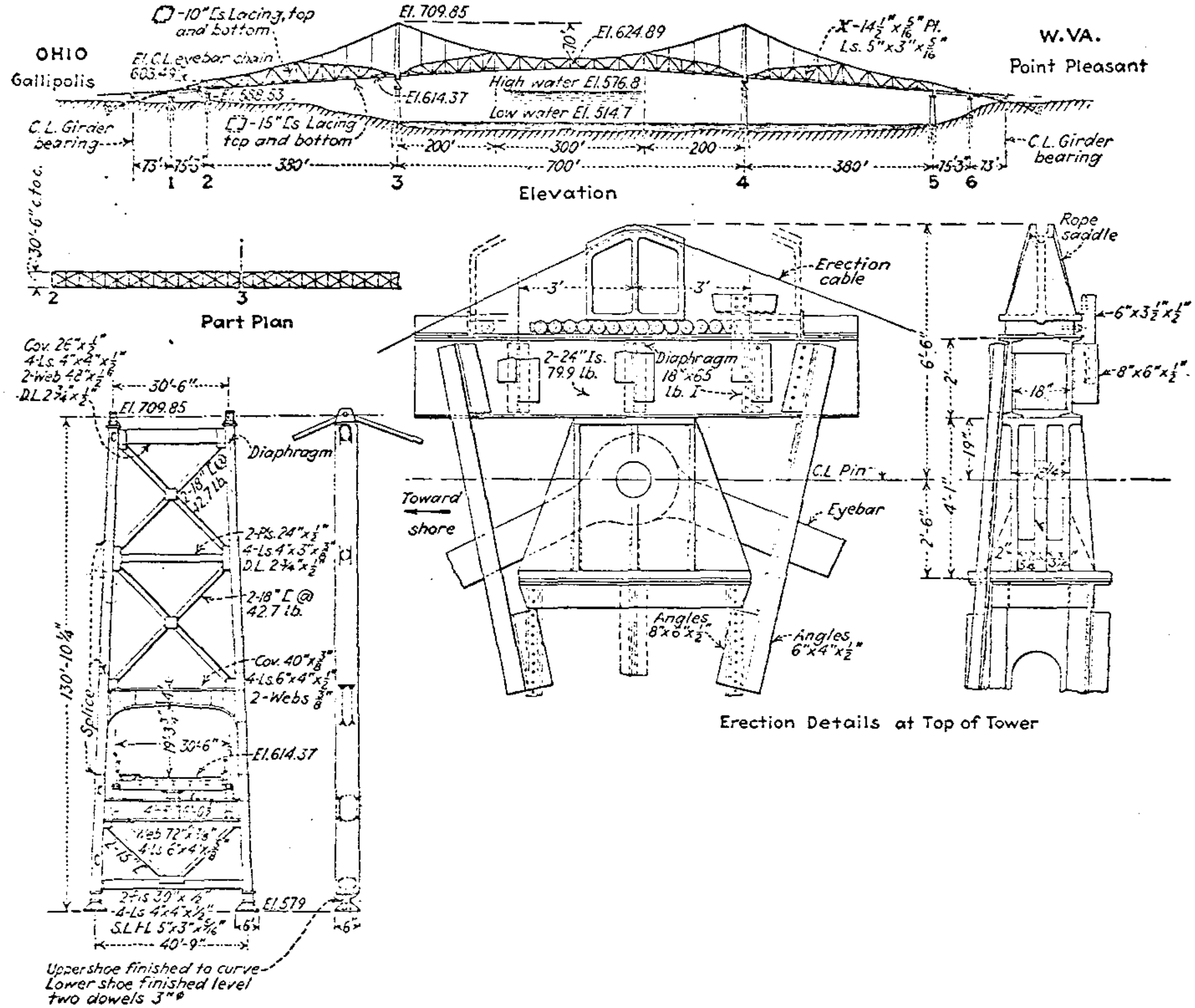



Figure 20. Details of Point Pleasant Bridge (49)

It took eleven months to complete the bridge.

The bridge collapsed on December 15, 1967 during rush hour traffic at which time the traffic was almost motionless. Twenty-four vehicles were removed from the river with the loss of 46 lives.

The bridge apparently failed on one side first and dumped the vehicles into about 70 ft. of water, then with the bridge twisting like a ribbon it eventually collapsed from anchorage to anchorage. The towers then collapsed on top of the wreckage and vehicles.

There have been many ideas as to the cause but the exact cause has not yet been determined. One suspension bridge expert explained that the ratio of dead load to live load usually ranges between 5 to 1 and 8 to 1 but in this bridge the ratio was less and the live load increasing all the time. He felt that the failure may have been due to fatigue.

A partner of the design firm explained how the design of a bridge is much different today than it was then. The engineers would prepare stress sheets and the steel and bridge contractors would design to meet the needs. Today the engineer designs the complete bridge from abutment to abutment.

It had been reported that some crystallization existed in some of the breaks, many of which occurred due to the impact of the falling bridge. The head of the Department of Civil Engineering at Carnegie-Mellon University in Pittsburgh had said there were no signs of fatigue as ductility apparently existed in the broken areas. He felt the bridge was overloaded with the live loading criteria much less than that for which we design today. However, it was quite certain that the bridge was not overloaded. The total number of vehicles removed from the river was somewhat less than previously reported. Heavy concentrated loadings over a period of time could have caused fatigue of the members.

It had been reported that there had been a flattening of the main span which may be of some interest in analyzing the collapse. The end spans were designed and erected with a 6.8 per cent grade and the main span was parabolic and tangent to the end spans. Over the years the main span had flattened which resulted in a noticeable "bump" at both towers.

The accident is being investigated quite extensively by federal authorities which include the National Transportation Safety Board, and the Bureau of Public Roads. The bridge was to be reassembled in two dimensions as the pieces could be recovered from the river.

The exact cause has not been found at the present time but a new bridge has been designed and construction is under way at the time of this writing. The National Transportation Safety Board of Washington, D.C., has published an interim report on the failure dated October 4, 1968. This report covers the facts and circumstances surrounding this bridge failure along with recommendations and conclusions. The final report as to the cause of failure will be drawn up when the examination and analysis are complete.

<u>Bluestone River Bridge</u>

An erection failure which took the lives of 5 workmen occurred on March 31, 1949 (9, 10, 21, 84). The bridge was located near Hinton, West Virginia and was being built to span the Bluestone River.

The method of erection was the cantilever process. At the time of the collapse, span 3 was being extended to reach pier 2 and was cantilevered a distance of 231 feet.

The bridge consisted of five spans as shown in Figure 21 and was to be a total length of 1,158 ft. between abutments. The span in which failure took place was the longest span of the bridge. At the time of the collapse a 7 ton guy derrick was being used to place an erection truss between the end of the cantilevered truss and pier 2. This was to take some of the load off during the construction of the remaining panels. It would seem to be a much more reasonable method of erection to cantilever equal distances out from each of piers 2 and 3 and meet at the center of the span. The truss span was in the position shown in plan in Figure 21 due to the snapping of the downstream top chord. The workmen reported hearing a loud report just at the time of failure so there was no warning of failure taking place.

At the time of the collapse there were attempts being made to beef up some of the main truss members to resist some of the erection stresses. The top chord which broke had been increased from a design section of two 13 in. channels of 31.8 lbs. per ft., and one 22 in. by 1/2 in. cover plate to two 13 in. ship channels at 50 lbs. per ft. and one 22 in. by 1/2 in. cover plate. The erection truss which was being placed in position at the time of the accident was to be under the downstream truss. The other erection truss was not in position at the time. The weight of the erection truss was about 5 tons and was being boomed out by the derrick to be suspended beneath the main truss. It

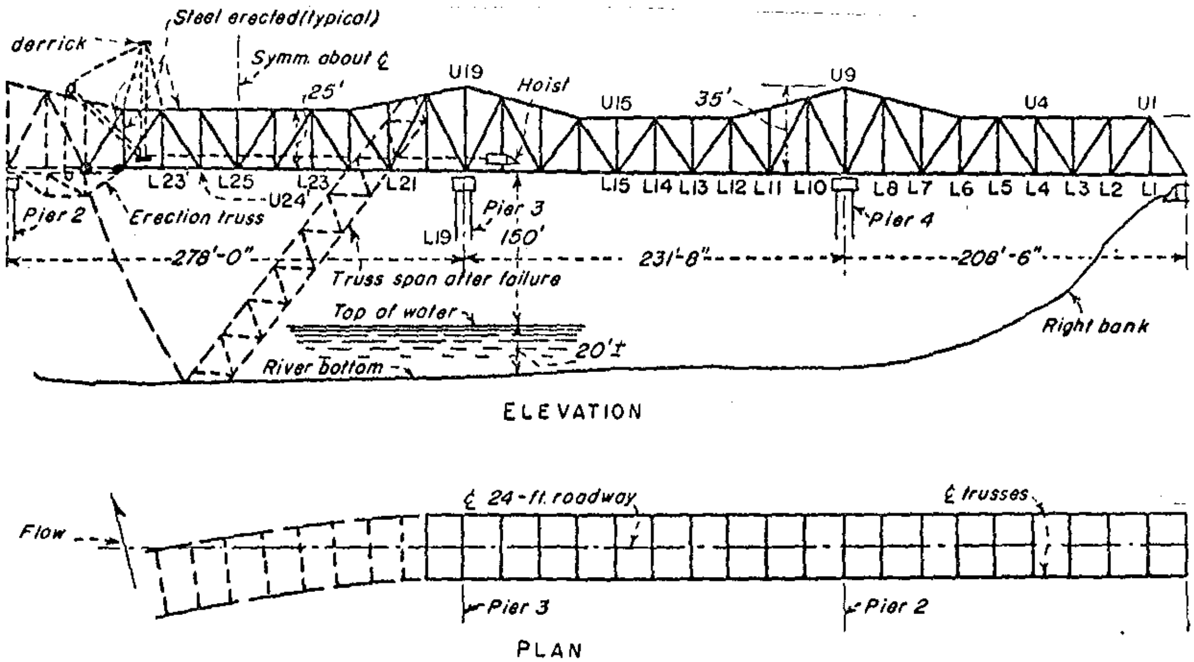



Figure 21. Details of Bluestone River Bridge showing collapsed center span (84)

was to be pinned at the end of the cantilevered position and the other end was to be placed on top of pier 2.

There had been no unusual circumstances at the time, such as any appreciable wind or any superimposed loads which were not included in the design of the structure. The guy derrick was in motion at the time and this was considered the only reasonable answer to the collapse.

There was, however, no known cause for the record. The same type of erection had been used successfully on a number of bridges previously and the material and stresses were determined to be satisfactory and in accordance with the West Virginia State Highway Department specifications.

The completion of the bridge was delayed six months by the accident. A change in erection was made when replacing span 3. The cantilever portion of span 3 was extended to within nine panels of pier 2, where before the cantilever was six panels longer. There were no design changes made.

Hartford Bridge

One of the longest welded plate girder spans to be built previous to 1941 was the 300 ft. span of the 270-300-270 ft. continuous girder bridge (59, 81). With only 48 ft. remaining to be erected to reach the first river pier there was a sudden collapse into the Connecticut River at Hartford, Connecticut. Fifteen lives were lost in this accident and several were seriously injured. The bridge was designed by the Connecticut State Highway Department. Its length was 3,950 ft., which included long approach spans on both sides of the river.

The method of erection was by cantilever, being supported at some point between the piers by falsework bents. The span which collapsed was being supported by a falsework bent 120 ft. from the first river pier. It is this bent which seemed to just "kick out" allowing the two cantilevered girders to drop to the river. The first girders to be laid from the pier were 126 ft. in length and weighed 101 tons. These extended 6 ft. beyond the falsework bent. A traveler erection derrick which was the largest ever built and weighed 176 tons had been used to add 96 ft. of additional girders. This had been completed and still another 96 ft. section was in the process of erection when the collapse took place.

The steel falsework bent was supported on two groups of 28 wood piles in each group. The piles were driven to rock at a distance of about 65 ft. before refusal. Steel grillages were placed on top of the pile groups to support the steel bent which reached about 65 ft. to the bottom of the girders.

There were no other spans damaged in any way nor were any piers damaged. The end of the girders at the first pier were tied to the pier by steel plates bolted to anchors set in the concrete to prevent any uplift during the cantilever erection procedure. These plates were broken leaving the pier in undamaged condition.

The collapse dropped 430 tons of structural steel into the river along with the 176 ton traveler and 32 workmen. The water was about 15 ft. deep and the steel was a total loss.

The cause of the failure was due to the movement of the falsework bent although it is not known what caused it to move. It could have been the instability of the material through which the piles were driven to reach rock. There had been known shifting of soil at the location of a dike some distance upstream.

The reconstruction was started again in May of 1942 and the bridge was opened to traffic in September of 1942. The method of erection had been changed to the extent of using two falsework bents during the erection of each span and the wood piles were replaced by steel piles.

Claverack Bridge

The failure of a reinforced concrete bridge over Claverack Creek at Hudson, N.Y., on February 26, 1918 was a rather minor type of failure but, nevertheless, is quite interesting in the way in which it failed and the lessons that can be learned from a failure of this type (14, 30, 121).

The bridge consisted of eight 30 ft. spans made up of a 22 in. thick slab reinforced with 1 in. square steel bars in the bottom of the slab. The same type of bars were placed over the piers to resist the negative moments. The slab was continuous and supported by floor beams on 2 ft. square concrete posts. Each post rested on a 6 ft. square by 2 ft. thick base and each base had four wooden piles under it.

On the night of February 26, just seconds before a car went down with two of the end spans, a heavy truck had passed over the bridge with no apparent failure. All the spans were in the same condition after the failure as before. A clean break was apparent at the second pier with the slab apparently just shearing off.

The failure is of interest also because of the sudden settlement of a portland cement plant on August 2, 1915 just a hundred yards downstream along the river bank. The plant sank into the ground so suddenly that 5 men were buried alive. Only a chimney was left showing above the ground. The stream bed bulged upward so suddenly that water spilled over its banks and nearly covered the bridge.

A study of the soil revealed a surface layer of cemented sand and gravel from 7 to 20 ft. thick. Under this layer was a soft, wet, unstable clay of 100 to 120 ft. thick. It was believed that a movement of the apparently stable layer of gravel did permit the structure to sink into the soft clay.

The collapse of the two spans was obviously the result of failure of the pier foundation material. The piles which were driven were reported to have sunk, after being driven through the top gravel layer, due to only the static load of the 2,000 lb. hammer. Therefore, the pier columns were probably supported only by the foundation slab.

It was found after the failure that the gravel around the foundation, and even under it, had been washed away and that there was only a soft unstable clay beneath the foundation. Whether the gravel had been scoured away or had shifted due to a movement of the top strata was never determined. It was very possible the same type of accident had taken place as occurred to the cement plant a few years before. If this is the case, the responsibility would seem to lie with the designer who did not take due consideration of the type of soil which was to support the bridge.

If the loss had been caused by scour, it points to the fact that maintenance authorities sometimes ignore the possibilities of pier undermining. The destructive action of water cannot be taken lightly.

## Summary

A discussion of the bridge failures in this chapter are examples of failures which have occurred but no legal responsibility was placed on anyone. The Point Pleasant Bridge failure is still under investigation.

Three of the failures were of undetermined cause.

Following is a short summary of the bridge failures in this category.

Point Pleasant Bridge:

This bridge failure resulted in a tragic loss of 46 lives on December 15, 1967 during peak traffic (31, 49, 82). The bridge crossed the Ohio River between Point Pleasant, West Virginia and Kanauga, Ohio.

The bridge was a suspended type with a 700 ft. main span. A unique feature of this bridge was a portion of the top chord of the stiffening truss being an integral part of the eyebar suspension chains.

There were two bridges of the same design built in 1929, the other being closed to traffic at the present time for a large scale inspection. The Point Pleasant Bridge is being replaced and under construction at the time of this writing.

The cause of the failure has not yet been determined although there are many theories as to the cause.

Bluestone River Bridge:

This failure took place during erection with the collapse of a cantilever span as final preparations were being made to connect with a pier (9, 10, 21, 84).

There were 5 workmen killed as there was no warning of failure. A loud report was heard just at the instant of failure with the snapping of one top chord member.

It was thought possible that the motion of a 7 ton derrick placing additional members could have been the cause but this was not established for the record.

Hartford Bridge:

This was an 840 ft. welded steel girder bridge of three spans. The collapse of one span took place in 1941 as it was being cantilevered toward another pier (59, 81). There were temporary piers used but for some unknown reason one of the temporary piers collapsed. There were 15 workmen killed as a result of the failure.

The probable cause was instability of the foundation material on which the temporary pier was placed. Shifting of soil had been known to have taken place at a dike some distance upstream.

Additional temporary piers were used along with steel piles and the reconstruction took place without incident.

Claverack Bridge:

This failure involved a relatively small concrete slab bridge and there were no lives lost (14, 30, 121). The failure of one span took place in 1918 with the shearing of a 22 in. thick slab at the pier.

The interesting part of this failure is the conditions surrounding the bridge and which give a clue as to the cause. It was determined that a pier had settled either from lateral movement of the gravel layer which supported the pier or settlement. Which one actually occurred was not determined.

A clue which would support the possibility of settlement was records of the complete disappearance of a cement plant three years previously just a few hundred yards away from the bridge.

## SUMMARY AND CONCLUSIONS

Failure of bridges is often associated with considerable inconvenience and tragedy. However, it is a way of learning and a way of teaching. That is the general purpose of this report. It is very essential that failures are made known and are not kept hidden. It is very possible that through knowledge of a past failure, some possible future failure may be averted. It is as important that we learn from the failures of others as from their successes.

In Table 3 the 26 bridge failures reported herein are summarized. The failures began with the Ashtabula Bridge failure in 1876 and continue up to the Point Pleasant Bridge collapse in 1967. Some of these failures are minor and some are major, but all contain valuable lessons. Recommendations are given for each failure which, if they had been observed, would most certainly have contributed to the success of the structure.

We are aware that many failures would be impossible to eliminate. However, many seem to be so unnecessary and possibly could have been prevented had sufficient precautions been taken. The public must be allowed to rely on the responsibility of the professional people who design and build the bridges.

It can be seen in the table that a large number of failures were a result of lack of final checks by competent personnel. It is common practice at the present time for a bridge to be designed by a professional design engineer and also to be completely checked by another

Table 3. Summary of bridge failures

| Bridge | Year of failure | Failure | Cause | General[a] classification of cause | Recommendations |
|---|---|---|---|---|---|
| Ashtabula | 1876 | Collapse of a truss span under weight of passenger train | Severe modification during construction with inexperienced personnel | C | Construction must be supervised by experienced personnel and constant inspection essential |
| Tay | 1879 | Simple spans collapsed during wind storm | Defects in piers and modification of original design without adequate checks | D | Higher standards for design and inspection |
| Tardes Viaduct | 1884 | Superstructure lifted off support by wind force during erection | Inadequate lateral support for launching mechanism | D | Greater consideration for wind forces on superstructure during erection |
| Moenchenstein | 1891 | Truss span collapsed under heavily loaded passenger train | Poor quality material and underdesign of truss members | D | More emphasis to be placed on increasing live loads |
| First Quebec | 1907 | Collapse of cantilever and anchor section during construction | Erroneous stresses in members by using assumed weights in design | I | Closer inspection and consideration of all significant factors |

[a] D = design error; I = inadequate inspection; C = inadequate construction procedures, B = brittle fracture; F = inadequate foundation; U = undetermined cause.

Table 3. (Continued)

| Bridge | Year of failure | Failure | Cause | General classification of cause | Recommendations |
|---|---|---|---|---|---|
| Attica | 1914 | Collapse of truss span under weight of passenger train | Damaged end post of truss with inadequate inspection | I | Bridge should have been analyzed after accident, while traffic suspended |
| Second Quebec | 1916 | Suspended span fell while being hoisted to its final position between two cantilevered spans | Instability of lifting apparatus | C | Details which may appear to be insignificant must not be overlooked |
| Spokane River | 1917 | Collapse of a concrete arch during construction | Improper procedure of concrete pour and faulty falsework | C | Consider impact caused by concrete pour and importance of adequate falsework |
| Claverack | 1918 | Collapse of concrete span following movement of pier | Undetermined; foundation material settled or was washed away | U | More consideration to be given to foundation stability |
| New River | 1920 | Bottom of piers being moved by soil movement | Slipping of embankment which was due to cutting of deep, narrow channel by flood waters | F | Sufficient preliminary investigations into soil condition prior to design |

Table 3. (Continued)

| Bridge | Year of failure | Failure | Cause | General classification of cause | Recommendations |
|---|---|---|---|---|---|
| Chester River | 1921 | Collapse of cantilever sidewalk | Apparent old fracture of supporting bracket | I | Maintenance inspection must be given greater consideration |
| Palm Beach Arch | 1922 | Three barrel arch spans collapsed due to pier failures | Very poor concrete and insufficient reinforcing steel in piers, inadequate rock foundation | I | Inspection must require adequate construction procedures |
| Tacoma, Washington | 1923 | Open crack appeared at construction joint of arch bridge; there was no collapse | Loss of bond at construction joint due to foreign material | C | Proper bond must be acquired and precautions taken in regard to foreign matter in joint |
| Lincoln Highway Bascule | 1928 | Counterweight dropped into river | Overstressed counterweight tower | D | Must consider dynamic stresses during movable bridge operation |
| Hasselt | 1938 | Collapse of Vierendeel truss bridge | Brittle fracture as result of high carbon content of steel and residual stresses resulting from poor quality welds | B | Designer must have sufficient knowledge in design of structures of high strength steel |

Table 3. (Continued)

| Bridge | Year of failure | Failure | Cause | General classification of cause | Recommendations |
|---|---|---|---|---|---|
| Custer Creek | 1938 | Bridge collapsed along with passenger train during flash-flood | Undermining of pier footing which was supported on gravel | F | Pier footings should not be supported by gravel; piles should be driven as support for the structure when required |
| Tacoma Narrows | 1940 | Collapse of suspension bridge span under severe oscillation | Inadequate design for aerodynamic forces | D | Sufficient wind tunnel tests must be required in flexible structures |
| Germany's Frankenthal | 1940 | Collapse of cantilevered steel girder during construction | Erection of cantilever longer than specified in design plans | C | Alteration of design plans must not be permitted without approval of design engineer |
| Hartford | 1941 | Collapse of steel girder spans during construction; failure of temporary pier | Undetermined; possibly caused by shifting of foundation material | U | Consideration should be given to supporting medium for temporary as well as permanent piers |
| Bluestone River | 1949 | Collapse of cantilevered truss during construction | Undetermined; possibly caused by motion of traveler derrick | U | Erect cantilever from both piers simultaneously to eliminate excessive loading |

Table 3. (Continued)

| Bridge | Year of failure | Failure | Cause | General classification of cause | Recommendations |
|---|---|---|---|---|---|
| Duplessis | 1951 | Collapse of welded steel girder bridge | Brittle fracture as a result of very cold temperatures and faulty material | B | Designer must consider temperature of the area and design to keep brittleness to a minimum |
| Second Narrows | 1956 | Collapse of cantilever span during construction | Faulty grillage of temporary pier | D | Adequate design checks required |
| Peace River | 1957 | Collapse of suspension bridge | Deterioration of shale at one abutment into which anchorage block was keyed | F | Ample consideration to deterioration must be given when shale is supporting material; deterioration of shale often takes place when exposed to air or water |
| Big Sioux River | 1962 | Bridge on Interstate 29 over Big Sioux River collapsed during high flooding | Undermining of a pier by flood waters; sufficient preliminary study was not obtained | F | Adequate maintenance inspection of pier foundation, especially during flooding |

Table 3. (Continued)

| Bridge | Year of failure | Failure | Cause | General classification of cause | Recommendations |
|---|---|---|---|---|---|
| Kings | 1962 | Partial collapse of welded girder bridge | Brittle fracture as a result of low notch ductility, variable carbon content in the steel and lack of knowledge in welding of high strength steel | B | Laboratory tests must be made to determine properties; sufficient knowledge of brittle fracture criteria is essential |
| Point Pleasant | 1967 | Collapse of suspension bridge | Undetermined; under investigation at time of this writing | U | An assessment should be made regarding inspection and maintenance practices throughout United States |

design engineer. A check is also made of the shop drawings which are made up by the steel fabricators. This check is necessary to prevent costly delays during erection or possibly faulty construction. The designer often has competent personnel make inspections at the fabricators before it is shipped to the construction site. This often results in a savings to the owner.

This procedure is a great improvement over that used prior to about 1940. During that period the designer only made stress diagrams of the bridge and the remainder of the project was the responsibility of the fabricator, the inspector, and the bridge contractor. Much of the inspector's time was used in calculating actual stresses in the completed portion of the structure as the erection progressed. The inspector today has much more of an opportunity to view each operation during construction.

It would be ideal if a bridge design engineer could be permitted to spend some time as an inspector during the construction period. Also, it would be advantageous for the inspector to gain knowledge of the design procedures in the office. This would appear to be of great value, as it would possibly reveal significant factors which otherwise would be hidden by routine procedures.

It is quite certain that many failures are never made known, even today, to the general public; especially those which appear to be of minor significance and probably little or no loss of life. However, the bridge failures discussed should permit the reader to become more aware of the many factors which may lead to failure. Bridge failures today, when compared with the number of bridges constructed, are definitely fewer percentagewise, and probably numerically, than in previous years. It should be noted that although the number of failures has been reduced in recent years, several factors which have not been too critical in the past may not become significant. These are brittle fracture and fatigue. With the increased use of high strength steels and the resulting increases in allowable stresses, both of these factors will need to be considered more critically. In fact, brittle fracture has already been a problem and was the cause of three of the failures reported herein. Fatigue will certainly become more of a problem in the future as loads increase and higher truck traffic densities occur. Bridges will approach critical fatigue much sooner.

Probably the most valuable information that may be obtained from this thesis is the circumstances surrounding the failures and the fact that these various types of failures do occur. This knowledge is certainly of great value as the first major item of design should be a summary of the factors affecting design which, if not considered, may have tragic results.

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# APPENDIX

## Brittle Fracture and its Relationship to Bridge Failures

Several failures have resulted from causes about which substantial research data and knowledge are now known. One of these failure types is brittle fracture. The failures which have been discussed show us that lessons can be learned from actual brittle fractures. Although mistakes are frequently very costly they are probably the best lessons learned and are remembered. Combining the information learned from all these sources, it is hoped that failures by brittle fracture can be significantly reduced.

The commonly measured properties of structural steels, such as ultimate strength, yield strength and elongation do not always assure freedom from failure at low temperatures, especially in large steel structures. Failure sometimes occurs under these conditions at stresses below the design stresses. Resistance to initiation of cracks called "notch toughness" is a characteristic very important in design, fabrication and construction.

The opposite of brittle steel is ductile steel. A ductile material has the characteristic of yielding plastically under high stress. A structure may behave in a ductile or in a brittle manner depending upon the temperature, mechanical condition and resulting stress conditions under which it is loaded. The factors contributing to the relative ductility or brittleness are: 1) material; 2) temperature; 3) velocity of deformation and 4) state of stress (111).

Ductility and brittleness are important characteristics also in other metals such as molybdenum, tungsten, and zinc and in non-metals such as tar and glass. Tar is a good example of brittle fracture under the conditions of temperature and velocity of deformation. Tar will flow plastically when a load is applied to it, if at a relatively high temperature and if the load is applied slowly. But, if the temperature is low or load applied rapidly, a brittle fracture may occur. Whether a material will flow plastically or fracture in a brittle manner depends upon the ratio of maximum shear stress to the maximum tensile stress and the flow and fracture characteristics of the material. An increase in the carbon content of steel will decrease its ductility and the yield strength is also increased. As the yield strength approaches the fracture strength, the steel becomes more and more brittle. Brittle behavior can take place by retarding plastic flow. This is done by increasing the carbon content, loading at a rapid rate or introducing notches.

A steel structure may be made stronger by using higher strength steel rather than to increase the size of the members, but when higher strength steels are used the structure becomes much more susceptible to failure if any notches appear. It has been shown that an increase of 0.10 per cent of carbon will raise the temperature at which brittle fracture can take place by $25^{\circ}$ to $50^{\circ}$ F. When conditions are present which could cause brittle failure, steels of lower carbon content are usually used.

Any condition that will cause a low ratio of maximum shear stress to maximum tensile stress should be avoided. These conditions can occur in design by introducing unfavorable stresses in a structure and also by notches introduced during fabrication or construction. Some examples are: incompletely penetrated weld joints, weld cracks, base metal or under bead cracks and accidental notches.

An explanation of the effects of notches on brittle steel may be useful to the reader. When a tensile stress is applied to a plate without any notches the stress is uniform across the width of the plate. When a notch is present as shown in Figure 22 a high stress is concentrated at the apex of the notch and drops instantly to zero. Experiments show

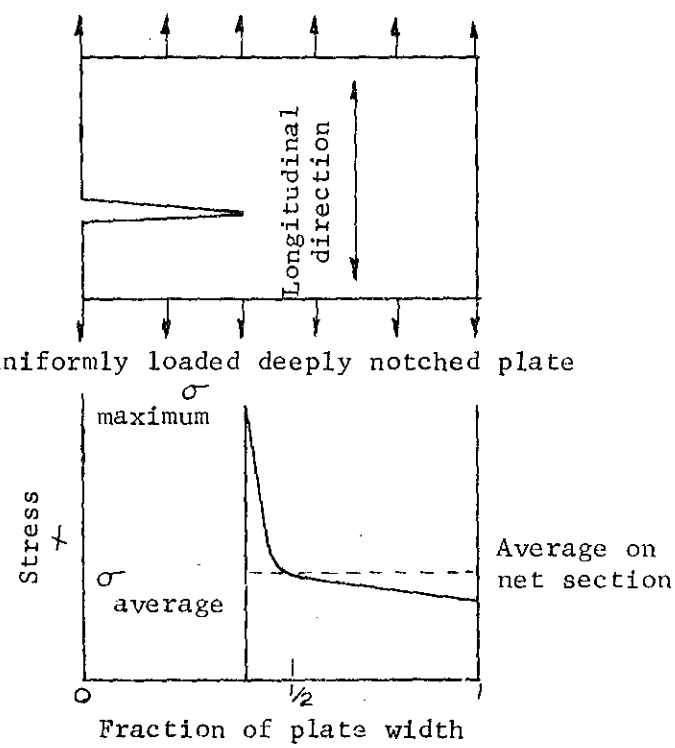



Figure 22. Diagram of notched plate showing longitudinal stress distribution (111)

that thicker plates with notches will fracture at lower temperatures than the thinner plates when under the same stress conditions. This is because in the thicker plate, once a crack starts to form at the apex of the notch there is a component of much higher stress in the thickness direction of the plate. At the same time the longitudinal tensile stress concentration factor is the same in the thicker plate as in the thinner plate. The reason for the component of higher stress in the thicker plate is due to the fact that the stress can reach a higher point before it begins to decrease and reaches zero at the edge (see Figure 23). The variation in the stress component along the width is also shown. The stress concentration at the apex of a notch is quite unfavorable and becomes more so with an increase in plate thickness and sharpness of notch.

The effect a notch has on a steel depends on many factors, one of which is temperature. When a steel is used for design of a structure extensive tests should be made at various temperatures because there is no exact answer for the notch effect on brittleness of a particular steel. A representative sample should be tested for notch toughness for each member in the structure which will be subjected to a large tensile stress.

With a given geometry and for a specific strain rate there is a temperature above which brittle fractures will not occur in mild steel. This is called the "transition temperature." This transition temperature occurs slowly in some steels and abruptly in others. This should also be considered in laboratory tests if the location of a particular

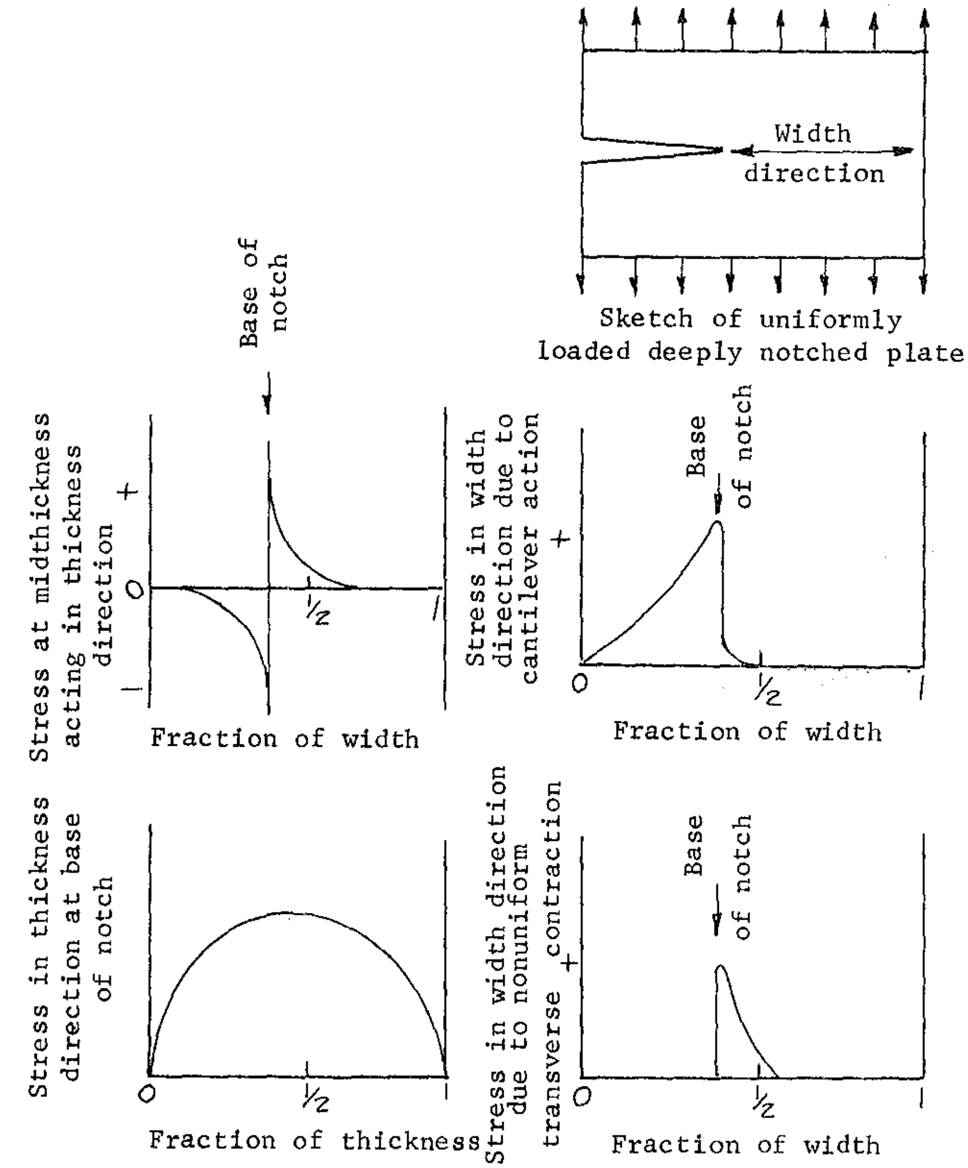



Figure 23. Diagrams showing stresses in the thickness and width directions (111)

structure is in an area of relatively low temperatures for any short period of time. This apparently was not a factor in the failure of the Kings Bridge in Melbourne, Australia. The transition temperature can be raised considerably with the addition of carbon. Phosphorous also is very effective in raising the transition temperature. The temperature effect played a part in the failure of the Duplessis Bridge at Three Rivers, Quebec and was aggravated by the use of poor quality steel with impurities and variable carbon content which would make laboratory tests quite meaningless.

Another factor which can change the brittle characteristic of a steel is the welding operation. Although failure of a structure made up of brittle material does not usually occur because of defects in weld it is important for the designer to know and understand the effects of poor weldability on brittle steel. Under quality control it is possible to apply a weld to a steel which is stronger than the base metal.

The problems of weldability are of two types -- one is joinability and the other is serviceability. The joinability is dependent on the ability of the construction welder and serviceability, which deals with the mechanical performance of the weldments, is governed by the chemical composition, microstructure, and the structural discontinuities. The chemical composition is controlled by selection of the electrode type and the welding procedure. The microstructure is a function of the cooling rate and the structural discontinuities are dependent upon the design of the joint and skill of the welder.

The elements of which the weld material is made can greatly affect the quality of the welded joint. Usually the carbon content of mild steel welds is only about 0.1 per cent and manganese is at a high level, which accounts for a ductile weld. Gases such as oxygen, nitrogen, and hydrogen are often the cause of a poor quality weld. The skill of the welder is very important in keeping the presence of gas bubbles to a minimum. The use of submerged arc welding is used today quite extensively and is quite effective in this respect.

The microstructure of a weld is controlled by the chemical composition and the cooling rate. An increase in the cooling rate decreases the quality of the weld and increases the brittleness and existence of defects in the weld. The importance of good quality welds can not be overlooked. There should be extensive inspection of welds both by sight and by magnetic particle inspection to detect any defects in the weld below the surface.

Whether or not residual stresses caused by welding are a contributing factor in brittle fractures is not fully known. Residual stresses, however, in general are sometimes a serious matter if the material at that point is already highly stressed. Therefore the possibility of residual stresses must be considered in design. The residual stresses can be relieved by several methods which will be included in the following discussion.

It is commonly thought by those who are not aware of what actually takes place during heating of a metal, that residual stresses are set up only by the shrinkage of the weld. A discussion as to what actually takes place when a metal is heated to a very high temperature and a high temperature gradient would be quite beneficial at this point. When a flat metal is heated at a small area to about 700$^{o}$ F the heated portion expands radially outward and is resisted by the cool portion of the plate. This results in biaxial compressive stresses in the heated portion with the stresses greatest in the latter portion of the plate. With the expansion of the metal, plastic flow takes place which can occur only in a direction perpendicular to the plane of the steel plate. This causes the thickness to increase with most of the added thickness still remaining after the steel is cooled. However, the volume of the steel must be the same after heating as before, with the result of shortening in radial directions of the heated spot. This causes tensile stresses to remain in the metal along radial lines from the heated spot. This same effect can be produced either by using a torch or by welding operation.

Quite often web plates for welded girder bridges are butt welded when a longer plate is required than that produced by a rolling mill. This butt joint weld is a good example of residual stresses and must not occur at a point in the span where large stresses may occur. A zone of tensile stress occurs along the weld and parallel to the weld to a distance into the plate several times the width of the weld. Beyond this there is compressive stress with the maximum value about one-fifth the yield strength of the steel being welded. The tensile stress in the vicinity of the weld may go as high as 60,000 psi.

The stresses which occur across a weld are not very serious in unrestrained plates and may induce stresses up to 5,000 psi although when a welded portion is restrained as in a pressure vessel the problem is somewhat more serious. In fact the residual stress may be as high as the yield strength of the metal. Patterns of these two types of residual stress problems are shown in Figure 24.

Residual stresses may be relieved by two methods. One method is by thermal means and the other is by mechanical means. The thermal method consists of heating the welded assembly to 1100-1200$^{o}$ F and holding that

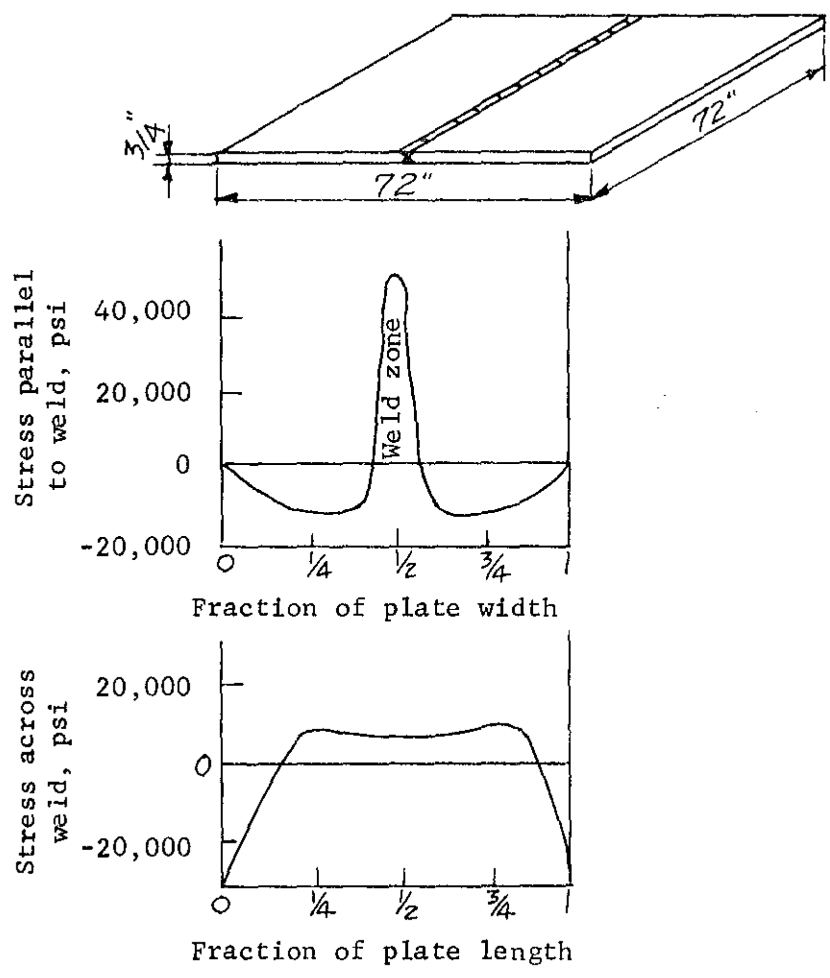



Figure 24. Diagram of butt welded steel plate and typical residual stress pattern (111)

temperature for two or three hours, then cooling very slowly. The residual stresses within the metal are relieved by plastic flow or creep with a uniform stress being obtained through the heated area. Also by this heat treatment the more brittle steel in the weld region is toughened by an alteration of the microstructure produced by exposure to very high temperature. Mechanical stress relieving is done by stretching plastically the regions containing tension stresses caused initially by shortening.

A welded structure is much more susceptible to failure than a riveted structure. There is not the problem of residual stresses caused by heat in a riveted structure. If a crack occurs in a riveted joint there is a good chance that the crack may be terminated at a rivet whereas in a welded joint the crack is almost sure to continue.

There is no sure factor of safety for design especially if the steel is of a brittle nature. The method of laboratory testing of a representative sample of the material would be the only means of obtaining a somewhat reasonable criteria for design purposes. The number of welded structures that fail are small but even so the great expense incurred by those very few may be large and seems unnecessary, nevertheless.

A further concept of brittle strength is given by use of the following graph which explains the transition range from ductile behavior at high temperatures to brittle behavior at lower temperatures. The three graphs shown in the following diagram (Figure 25) are yield strength "Y", brittle strength "B", and flow stress in the presence of a notch "3Y".

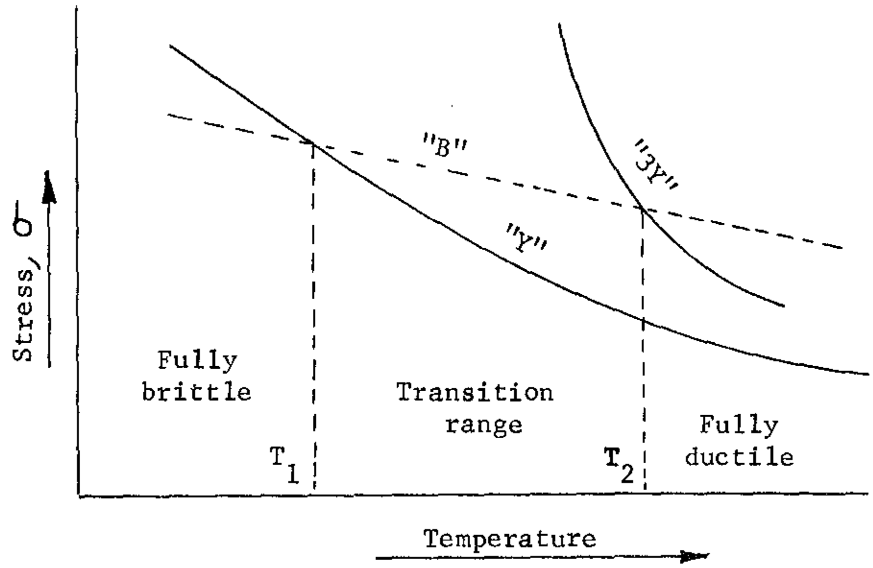



Figure 25. Transition from ductile behavior to brittle behavior (113)

"3Y" represents three times the yield strength "Y". These graphs are plotted as functions of temperatures. It is indicated that above the temperature $T_2$, the point when "B" intersects "3Y", the material will be fully ductile with or without a notch and below $T_1$ where "B" intersects "Y" the material will always fracture in a brittle manner. The transition zone is where the material will act in a ductile manner under uniaxial stress and in a brittle manner if a notch is present. Failure may take place before three times the yield strength has been reached if a notch is present.

In 1951 an important piece of research was published by Eldin and Collins (113) on the fracture and yield stress of 1020 steel at temperatures in the region near absolute zero (12° K, or -438° F) and up to 185° K (-126° F). All tests were made using uniaxial tension and no notches. It was found that from 12° K (-438° F) to 61.5° K (-346° F) all specimens fail as a brittle fracture. This fracture took place with no reduction in the area of the specimens. As the temperature was increased above 61.5° K reduction of area took place and a yield stress could be measured. When the temperature reached 104° K (-272° F) the fracture indicated the presence of ductile material and above 185° K (-126° F) all the fractures were shear.

There are two conditions requisite to a brittle failure. First, the rate of stress application and strain rate must be high enough to bring the yield stress of the material up to the brittle strength without introducing any triaxial stresses and second, sufficient plastic deformation is induced at the tip of a crack, so that approximately three times the yield strength exceeds the brittle strength of the material. These two conditions may occur together to accomplish a fracture where alone neither condition is sufficient.

It is shown in Figure 26 how the material varies between the ductile state and the brittle state with a transition range between. The energy absorbed is plotted against the three conditions which affect the relative change from one side of the graph to the other.

It is shown that the material may fail in a brittle manner with very little energy absorption if the temperature is decreased, the loading rate increased, and increasing transverse stresses.

In the design of structures there are many points of large stress

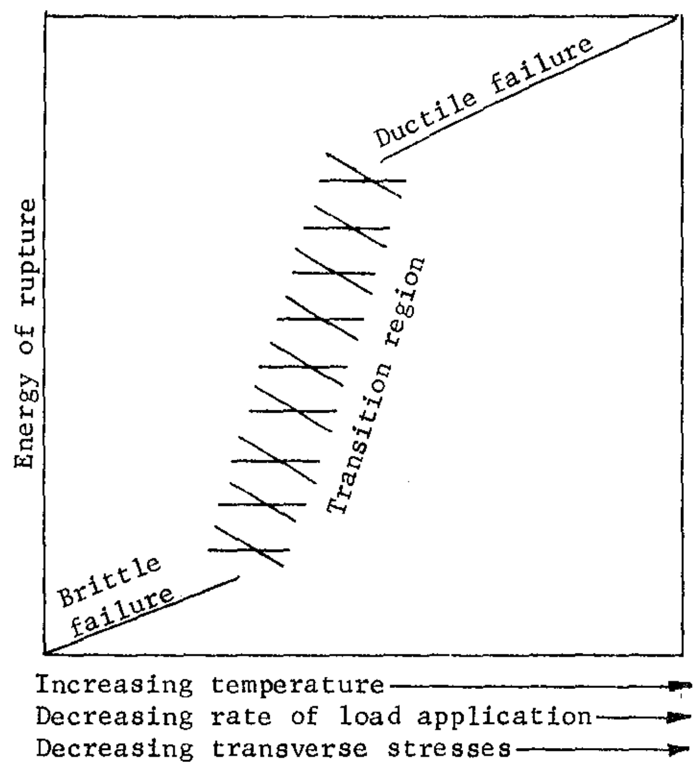



Figure 26. Diagram showing effect of temperature, rate of loading and transverse stresses on the type of failure (113)

concentration where the actual stresses may not be determined. This may create quite a serious problem in the event of brittle material. In order for reliable stress calculations to be made, the strains must be in the elastic range for a fairly large area. Dependence must be placed on some structure existing under somewhat the same conditions. It may be unknown whether that structure is performing well due to a precise knowledge of the stress condition or whether it is due to the material having the ability to yield properly in the zone in question. Much of the knowledge gained in the past regarding brittle failures can be used in design of a structure only as an approximation as to when brittle fractures may occur and what precautions should be taken to guard against brittle failure. Laboratory tests of the actual material or a representative sample should be made.

Structural steel may become brittle at low temperatures especially in the presence of a notch. Notches may occur during fabrication or erection. If a crack starts at a notch, greater toughness is necessary to stop the crack than is required to prevent one from starting. Therefore, a good designer would make the area of stress concentration lie within a tough material which could prevent the start of a crack.

It is the viewpoint of many designers that an area which may be overstressed due to a condition such as a notch, will relieve its stresses by yielding. This is not always true when viewed with the possibilities of brittleness under certain conditions.

A ductile material will always show signs of decreasing in cross-sectional area during yielding when subjected to an axial load. If the material is restrained from deforming and would remain at the original cross-section there could be no yielding as the volume must always remain the same. If an increment is stretched, the volume within the original increment must be the same after stretching as before, therefore the cross-section must decrease accordingly. If the material is under triaxial loading until the breaking point is reached the result will be a brittle fracture.

If in a steel structure there is a notch in an area under tension, the apex of the notch will probably reach the yield stress. If the material adjacent to the notch apex restricts any lateral deformation, the material will be in a brittle state even though the material may be ductile. The thicker plates are more susceptible to brittle fracture in the presence of a notch because of the same principle. There is more of a resistance to any yielding and there is little or no elongation of the material at the notch.

A material, therefore, may be ductile but if placed in a structure such that yielding is restrained brittleness may result. An example of lateral restraint is shown in Figure 27.

The narrow section tends to get smaller in cross-section but is restrained by the thicker portion adjacent to it. The results will be

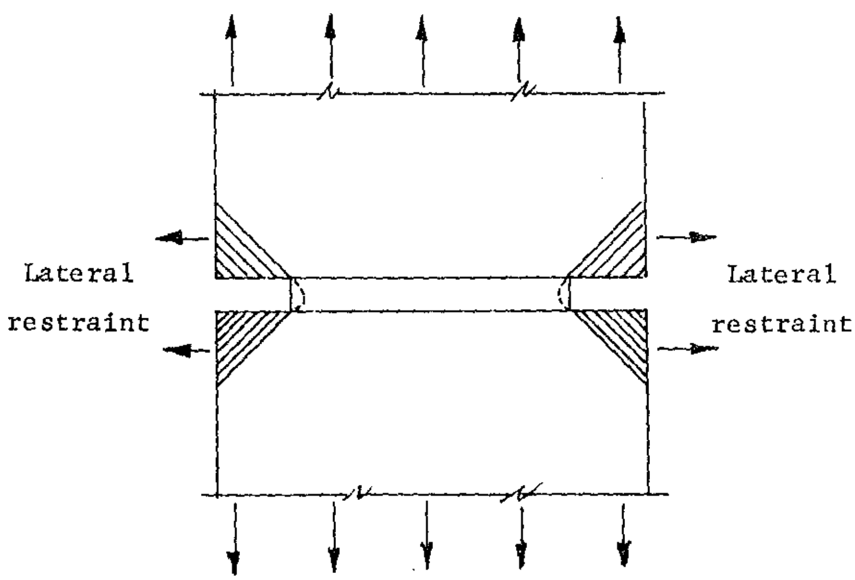



Figure 27. Diagram of notched test bar (23)

brittle fracture when the axial loading reaches the breaking point of the steel. Actually the steel could be under a relatively low stress.

Today many large structures of steel are welded rather than riveted. A riveted joint may act as shown in Figure 28. The plates are under uniaxial stress only due to the deforming of the angles. This differs from a similar welded joint as shown in the same figure. The tension force on each of the plates tends to create lateral forces within the through plates at the welded joint preventing the plate from narrowing. As a result there can be little or no yielding of the through plate and a brittle fracture may take place. Within a welded structure a three-dimensional continuity may be built in and form what may be called acute rigidity or absence of flexibility.

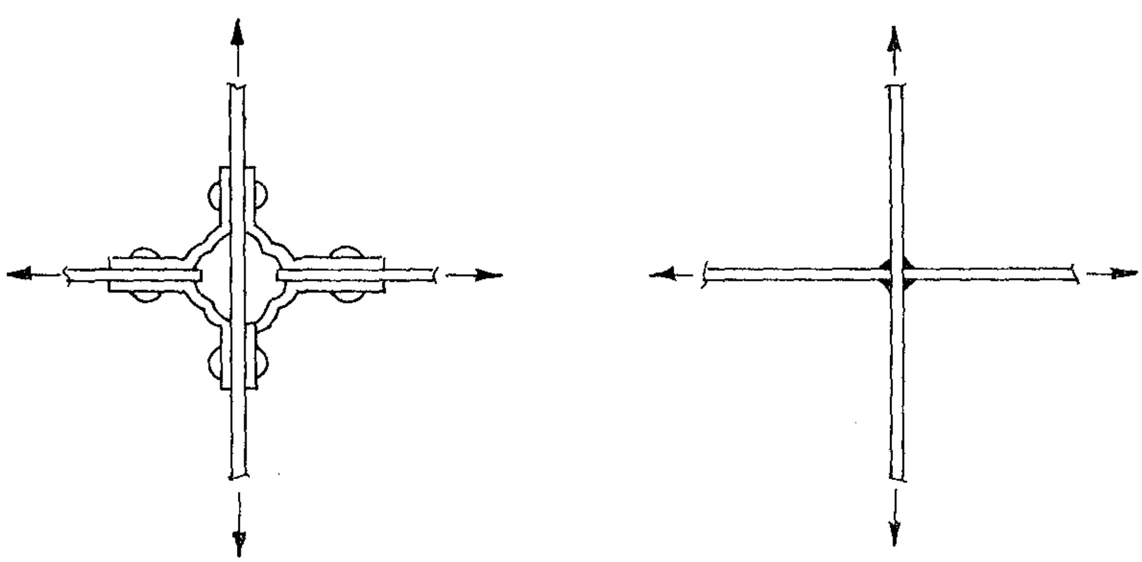


Figure 28. Diagram of riveted and welded joints (23)

Steel is a very versatile material and because of this brittleness is a problem. Steel can always be fabricated into a structure without any loss of its properties if done correctly and with care. It is quite simple to get 100 per cent joint efficiency with a simple welding operation but nevertheless care must also be taken. This sometimes leads us into the problem of brittleness. If 100 per cent joint efficiency could not be obtained this problem would not exist. Also if riveting were more efficient we would probably have no problem with brittleness. Therefore care must always be taken and all possibilities which could lead to brittleness must be considered very carefully because brittleness can be designed into a structure.

As discussed somewhat previously the matter of rapidly applying loads may cause the material to be brittle when otherwise it would be a ductile material. If a notch is present this problem is even more great. A sharp notch is said to have a stress concentration factor of three. This means that if we apply a 10,000 psi average stress to the notched structure, the stress at the notch apex will be 30,000 psi. If it took 3 seconds to apply this 10,000 psi stress, then it only took 1 second for the stress at the notch to reach 10,000 psi. Therefore 30,000 psi was being applied at the notch during the same time that the remainder of the structure was receiving 10,000 psi. Within the vicinity of a notch the strain rate is therefore increased.

All the previous points discussed which can lead to brittleness can be serious and must be given much thought by a designer and also the fabricator and erector. These points are all made even more serious if the temperature is low. A good explanation of what takes place with a variation in temperature is shown in Figure 29.

An important fact to be considered in the event of brittle material occurring is the crack initiation or crack propagation. A laboratory test which is used to determine a material's ability to resist cracking initiation and propagation is the Charpy Impact Test. Once a crack is started it sometimes takes a very low stress to affect the continuation of the crack. The Charpy Impact Test is accomplished by the use of a blow striking a sample of the material in which there is a standard notch. The force of the impact is measured in foot pounds. A graphical example

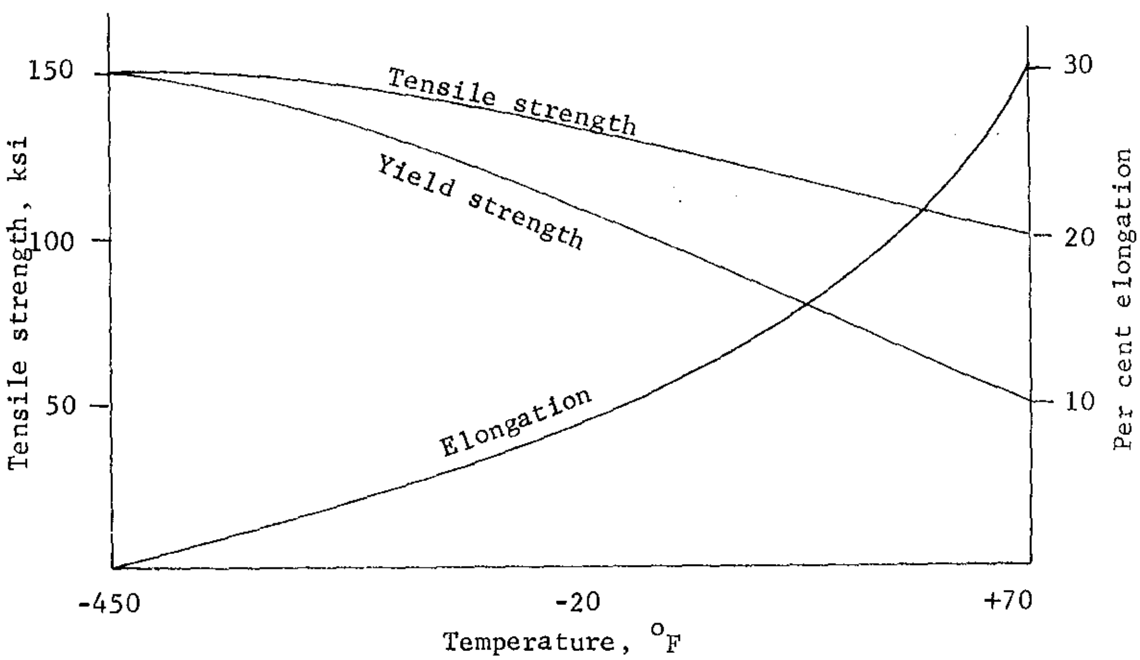



Figure 29. Diagram showing effect of temperature on mechanical properties (23)

of what takes place within a material under varying temperatures and loads is shown in Figure 30.

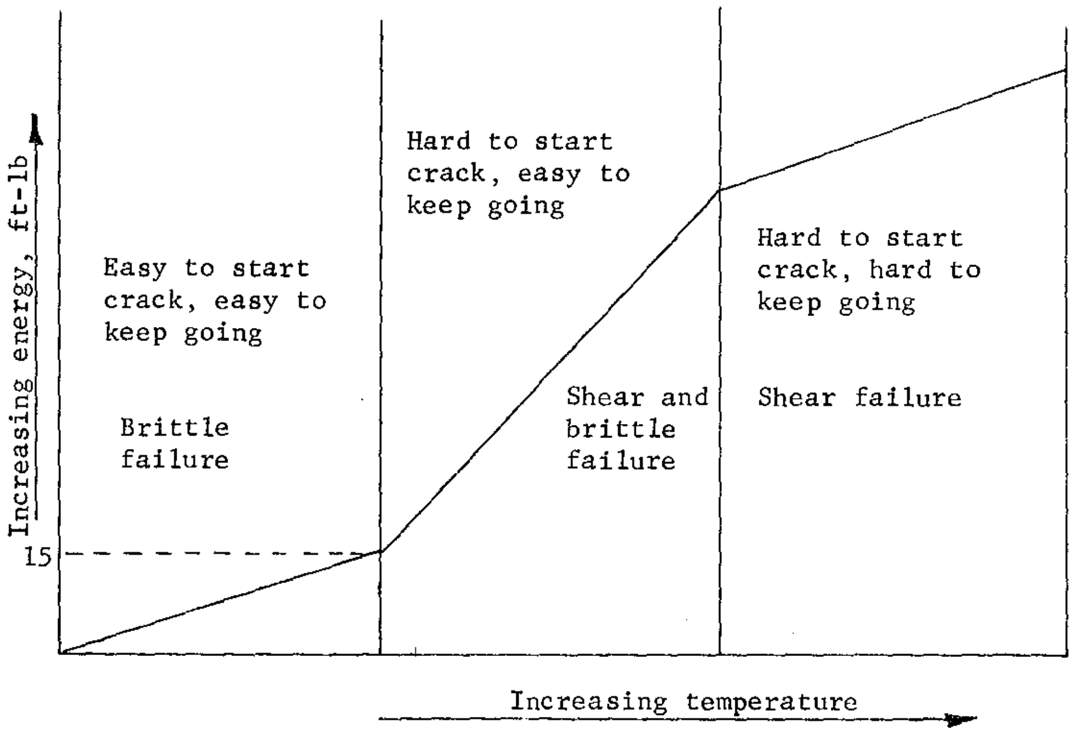



Figure 30. Diagram showing a typical impact-transition temperature curve (23)

The specimens under the conditions at the far right do not break easily nor in a clean manner. They more or less bend and tear. The ones at the left break clean and suddenly. The specimens in the center are within the transition range where a crack is hard to start but once started it is hard to stop.

In the design of a steel structure it must be considered whether the conditions leading to brittleness may be present. A designer should ask himself certain questions to determine whether or not the risk of brittle fracture is serious and requires special design consideration. The following questions and answers are very important and must be given considerable amount of thought (23).

1. What is the minimum anticipated service temperature?

   The lower the temperature, the greater the susceptibility to brittle fracture.

2. Are tension stresses involved?

   Brittle fracture can occur only under condition of tensile stress.

3. How thick is the material?

   The thicker the steel, the greater the susceptibility to brittle fracture.

4. Is there three-dimensional continuity?

   Three-dimensional continuity tends to restrain the steel from yielding and increases suceptibility to brittle fracture.

5. Are notches present?

   The presence of a notch increases susceptibility to brittle fracture.

6. Are multiaxial stress conditions likely to occur?

   Multiaxial stresses will tend to restrain yielding and increase susceptibility to brittle fracture.

7. Is loading applied at a high rate?

   The higher the rate of loading, the greater susceptibility to brittle fracture.

8. Is there a changing rate of stress?

   Brittle fracture occurs only under conditions of increasing rate of stress.

9. Is welding involved?

   Weld cracks can act as severe notches.

10. Are there riveted or bolted seams that would stop a crack and prevent a disastrous failure?

A characteristic of ductility is its capability of the material to flow plastically. This plastic flow is dependent only on the shear stresses, and the maximum shear stress is equal to one-half of the difference between the maximum and minimum principal stresses. In a triaxial stress system the ratio of maximum shear stress to maximum tensile stress can range from zero to greater than one. This ratio determines whether or not a steel will behave in a brittle or a ductile manner. If the ratio is low the tendency will be towards brittleness.

When an unnotched specimen of mild steel is tested at liquid nitrogen temperature, it will be as brittle as cast-iron if subjected to a tensile loading and as ductile as copper if loaded in torsion. The ratio of maximum shear stress to maximum tensile stress is one-half for the tensile test and one for the torsion test.

The only way this ratio can be made smaller than one-half is by use of a notch and the ratio depends upon the sharpness of the notch and the plate thickness. In laboratory tests the easiest and most common means of determining the brittleness characteristics of a steel is by use of a notched specimen. The evaluation of results and selection of tests are to compare with the service performance of the structure.

The appearance of the fracture surface is quite different from a brittle (cleavage) fracture to a ductile (shear) fracture. Shear is accompanied by plastic flow. This forms voids in the atomic structure of the steel and increases as the plastic flow increases. The voids increase to a point where a fracture takes place. A shear failure follows a diagonal path across the crystals which make up the material.

A brittle fracture or cleavage fracture follows the plane of the crystals. This fracture does not involve plastic flow. A very small amount is sometimes present -- usually only a fraction of one percent.

At high temperatures the cleavage strength is high relative to shear fracture strength, and the reverse is true at low temperatures. The transition from one to the other depends on the ratio of maximum shear stress to maximum tensile stress. When the ratio is one, as in torsion, the transition is below $-300^{\circ}$ F for mild steel; when it is one-half, transition occurs at about $-250^{\circ}$ F, and the transition for notched specimens occurs in the normal ambient temperature range.

The appearance of the two types of fractures is important for identification. Shear failures appear gray and silky while cleavage failures appear bright and granular. A cleavage failure usually leaves a herringbone fracture on the surface of a steel plate with points in the direction of the source of failure.

As previously discussed, the rate of loading contributes to brittleness. This is explained by the fact that it takes time for plastic flow to take place. Plastic flow must take place in order to yield. If the load comes on the member at a rapid enough rate, there is not time for yielding and cleavage fracture results.

Welding is known to have an effect on brittle behavior of a metal. This effect is usually caused by defective welds which leave notches permitting brittle fractures to start. Also welds provide three dimensional continuity and plate-to-plate continuity necessary for continuous crack propagation.

Notch ductility is decreased by a rapid rate of cooling of the weld. Therefore preheating of the material to be welded is sometimes done and also light fillet welds are to be avoided if possible as cooling is much faster than in a heavier weld. Post heating is sometimes used to retard the cooling process and this works quite well in most cases. Usually a temperature of 1100-1200$^{\circ}$ F is obtained.

A weld specimen to be tested for relative brittleness should consist of a portion of the two metals welded for the full length. A notch should be placed across the weld, the heat affected zone, and the base plate.

A further complication of welding is the residual stresses which exist due to the high temperature to which the metal was exposed. The reason for the residual stresses and how they take place has been discussed previously. The actual effect that residual stresses have on brittle behavior is not certain. Some feel that a very slight amount of plastic flow will relieve any residual stress. Welding stresses can be effectively relieved by furnace heating to 1100-1200$^{\circ}$ F. Also some stress relieving has been found to take place by low temperature.

The brittle behavior of steel will continue to plague engineers for years to come but the knowledge which has been gained through past failures and laboratory experiments will greatly help to reduce failures in the future if it can be brought to the attention of the designer, the fabricator and the construction workers. All have a part to play in the completion of sound structures, such as steel bridges, pipelines, and buildings. Brittle failures may never be entirely eliminated, but proper precaution based on proper knowledge of the causes can help to reduce the failures significantly.

## Analysis of Suspension Bridge Failures

It was concluded by the board investigating the failure of the Tacoma Narrows Suspension Bridge that there was no faulty material in the bridge nor had there been faulty workmanship (56). The design was adequate to carry the loads which were used in the design of other previous suspension bridges. The problem in this bridge was that the stiffening girders along with the narrow roadway were, for the size of the span, too flexible to withstand the aerodynamic forces. There was insufficient capacity for damping and for dissipating energy built up by wind forces. The initial failure took place due to the slipping of a cable band on the north side of the bridge. This magnified the torsional action to the point of failure of one of the ties at the center of the span. An out-of-phase oscillating motion took place which increased to extreme amounts with subsequent breaking of suspenders.

The question of aerodynamic action on suspension bridges was considered quite extensively in the study of this bridge failure by the investigating board. It had not been realized that the aerodynamic forces would affect a structure of such magnitude, although its flexibility was far in excess of that of comparable suspension bridges. The bridge was well designed for static forces but the criteria for static forces does not always apply to dynamic forces.

Several methods were used to dampen the oscillation due to wind forces but they were quite ineffective. The means used for damping were: hydraulic jacks at the towers placed at the ends of the floor system; stay ropes placed diagonally from the center of the main span cable to the stiffening girder; and stay ropes placed from the top of the towers to the floor system. The flexibility of the span was so great that the above systems for damping were quite ineffective.

A study was made of similar suspension bridges before the Tacoma Narrows Bridge was designed and the need for damping was considered necessary. Models also were built in the laboratory and the need for damping was discovered. The methods had been used in similar but smaller suspension bridges, but apparently with less relative flexibility than the Tacoma Narrows Bridge. Details of this study are given in Reference 56, but are summarized very briefly in the following paragraphs.

In a wide and more rigid suspension bridge the transverse tilting of the floor and the resulting torsional stresses in the floor caused by non-uniform distribution of live load and eccentrically applied wind forces would be negligible. In this narrow and flexible bridge the problem became a major one and required investigating. Under normal loading the tilting of the floor system was considerably greater than in other bridges which were studied. See Figure 31 for comparative tilting of the floors.

The investigating board made comparisons of the twisting or tilting of the bridge floors of several designs under two different loading conditions. The first loading condition was the application of a torsional moment of 10,000 ft-lbs per lineal ft. of bridge from the tower to mid-span. The results of the comparison of the Tacoma Narrows Bridge with other suspension bridges is shown in Table 4.

The second comparison is based on the floor tilt resulting from a loading of 100 lb. per ft. on the near cable from the tower to mid-span and the same load on the far cable from mid-span to the other tower. These results are shown in Table 5 and Figures 32 and 33.

The ability of a suspension bridge to resist irregular and turbulent wind forces is reflected by their torsional deformations under these assumed loads. The application of 10,000 ft-lbs per ft. of bridge was applied to all the suspension bridges listed regardless of width. The George Washington Bridge, for example, required a load of 94.3 lbs. per ft. up on one cable and down on the other cable. These forces with a distance of 106 ft. between produced a torsional moment of 10,000 ft-lbs per foot. The Tacoma Bridge with a width between cables of only 39 ft. required 256 lb. per ft. applied to the cables to produce a torsional moment of 10,000 ft-lbs per foot. This comparison resulted in a tilt of 38 times that of the George Washington Bridge. If the moment had been proportioned according to width the result would only have been 14 times as great instead of 38.

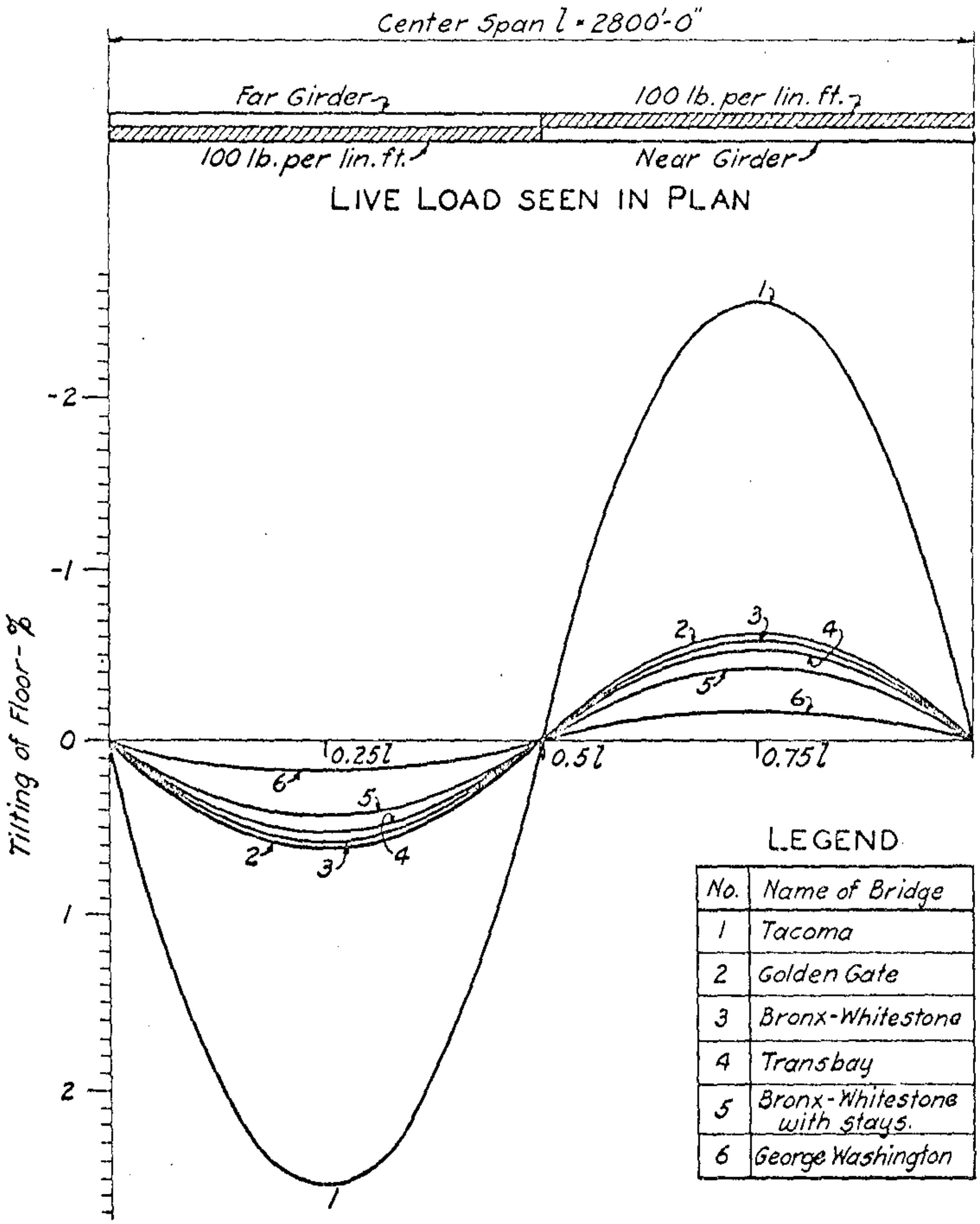



Figure 31. Comparative tilting of floor of the five longest suspension bridges (56)

Table 4. Comparison of torsional deformation of Tacoma Narrows Bridge[a] with others (56)

| Bridge | b | c | d | e | Rotation | Ratio of rotation of Tacoma as built, to others |
|---|---|---|---|---|---|---|
| Tacoma, as built | 39 | 2.564 | 3.18 | 8.154 | .2091 | 1.0 to 1 |
| Tacoma, fully braced design | 39 | 2.564 | .296 | .760 | .0195 | 10.7 to 1 |
| Toledo | 59 | 1.695 | .26 | .441 | .0075 | 27.9 to 1[f] |
| George Washington | 106 | .943 | .62 | .585 | .0055 | 38.0 to 1[g] |
| Bronx-Whitestone with stays | 74 | 1.351 | 1.08 | 1.459 | .0196 | 10.7 to 1[g] |
| Bronx-Whitestone without stays | 74 | 1.351 | 1.22 | 1.648 | .0223 | 9.4 to 1[g] |
| Transbay | 66 | 1.515 | .80 | 1.212 | .0184 | 11.4 to 1[g] |
| Golden Gate | 90 | 1.111 | 1.36 | 1.511 | .0168 | 12.4 to 1[h] |
| American Crossing Thousand Islands | 30.5 | 3.279 | 1.54 | 5.050 | .1656 | 1.3 to 1[i] |
| Deer Isle | 23.5 | 4.255 | 2.92 | 12.424 | .5287 | .4 to 1[i] |

[a]Based on deflections at quarter point of loaded half of span due to a torsional moment of 10,000 ft-lb per ft. of bridge from tower to mid-span.

[b]Width c. to c. cables, in feet.

[c]Load per lin. ft. of cable, up on near cable and down on far cable, in units of 100 lb.

[d]Difference in deflection, in feet, at quarter point of loaded half of span due to loads of 100 lb. per lin. ft. of cable from tower to mid-span up on near cable and down on far cable.

[e]Difference in deflection of two cables.

[f]Based on data furnished by Waddell and Hardesty.

[g]Based on data given in Ammann, von Karman, Woodruff report.

[h]Based on data furnished by Clifford E. Paine.

[i]Based on data furnished by Robinson and Steinman.

Table 5. Comparison of torsional flexibility of Tacoma Narrows Bridge[a] with others (56)

| Bridge | Deflections in feet at cable Down | Up | Diff. | Cable spacing in ft. | Rota-tion | Ratio of rotation of Tacoma as built, to others |
|---|---|---|---|---|---|---|
| Tacoma, as built | 1.59 | .39 | 1.98 | 39 | .0508 | 1.0 to 1 |
| Tacoma, fully braced design | .690 | .612[b] | .078 | 39 | .0020 | 25.4 to 1 |
| Toledo | .13 | .06 | .19 | 59 | .0032 | 15.9 to 1[c] |
| George Washington | .31 | .19 | .50 | 106 | .0047 | 10.8 to 1[d] |
| Bronx-Whitestone with stays | .54 | .22 | .76 | 74 | .0103 | 4.9 to 1[d] |
| Bronx-Whitestone without stays | .61 | .24 | .85 | 74 | .0115 | 4.4 to 1[d] |
| Transbay | .40 | .29 | .69 | 66 | .0105 | 4.8 to 1[d] |
| Golden Gate | .68 | .36 | 1.04 | 90 | .0116 | 4.4 to 1[e] |
| American Crossing Thousand Islands | .77 | .25 | 1.02 | 30.5 | .0334 | 1.5 to 1[f] |
| Deer Isle | 1.46 | .45 | 1.91 | 23.5 | .0813 | .6 to 1[f] |

[a]Based on tilting of floor in main span at quarter points from load of 100 lb. per ft. on near cable from tower to mid-span and the same load on far cable from mid-span to other tower.

[b]Down.

[c]Based on data furnished by Waddell and Hardesty.

[d]Based on data given in Ammann, von Karman, Woodruff report.

[e]Based on data furnished by Clifford E. Paine.

[f]Based on data furnished by Robinson and Steinman.

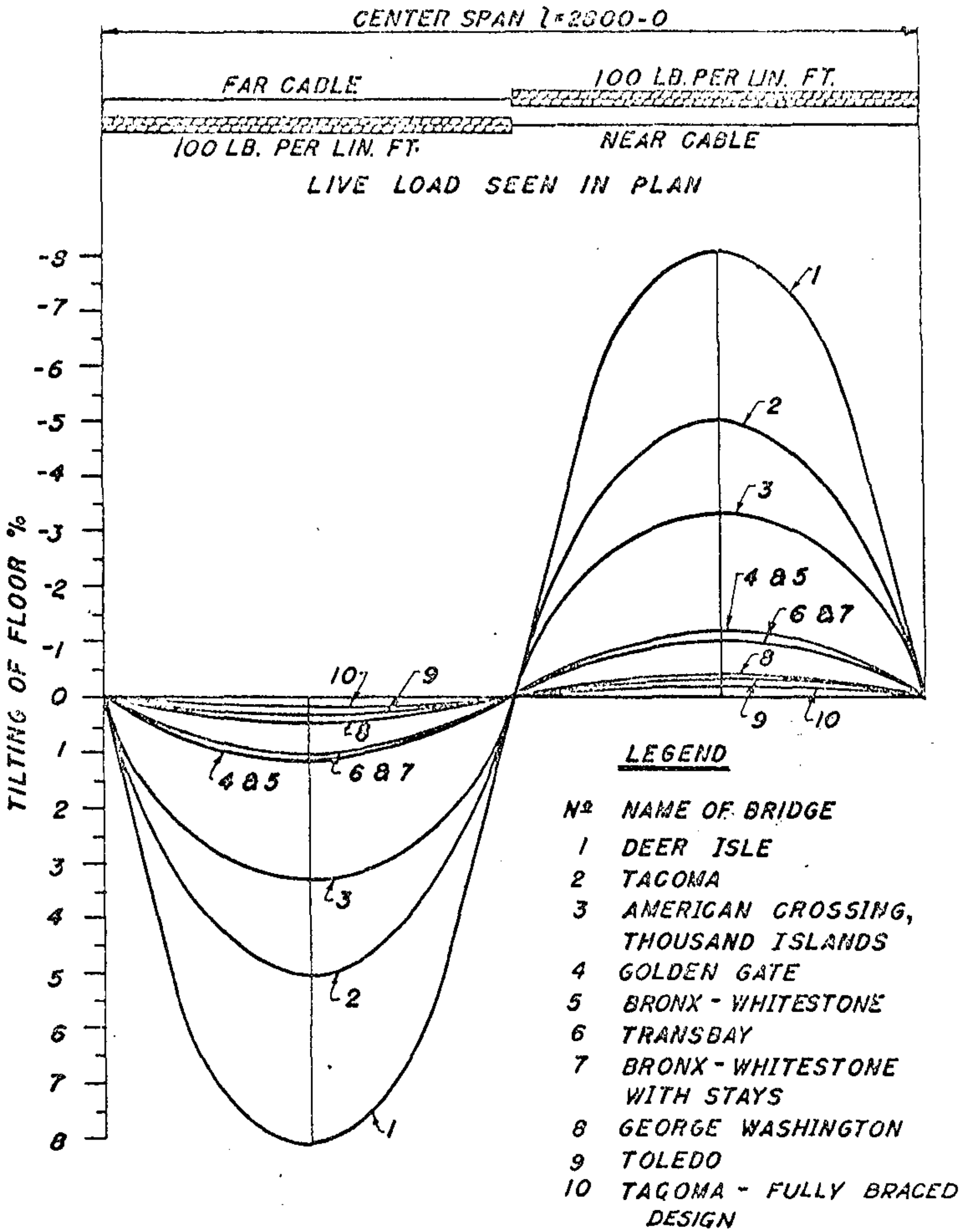



Figure 32. Comparative torsional deformations of ten bridges (56)

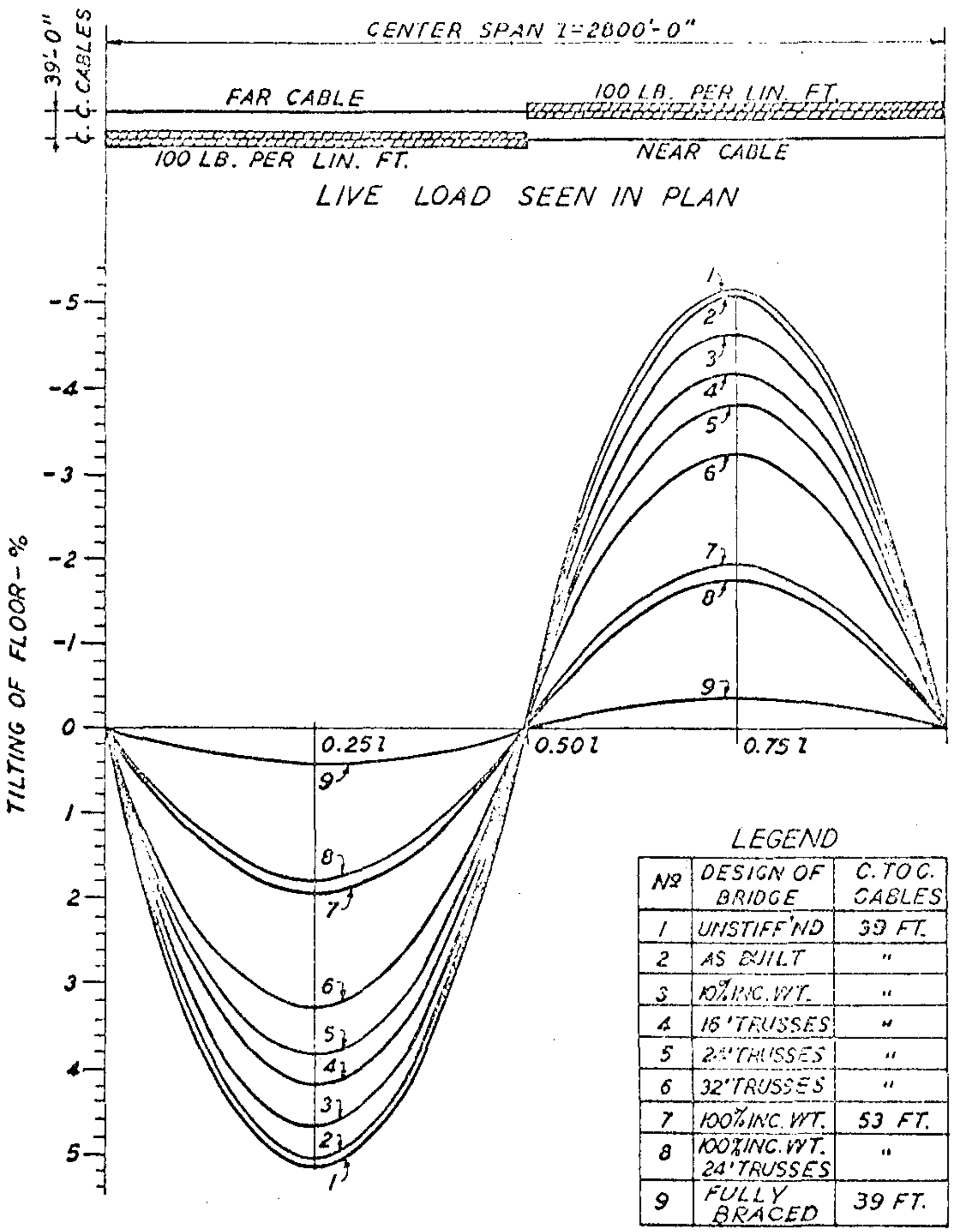



Figure 33. Comparative torsional deformations of nine Tacoma Bridge designs (56)

By use of the second type of loading the results were as shown in Table 5 with the tilting of the Tacoma Bridge being 10.8 times that of the George Washington Bridge. Also compared is the Tacoma Bridge with a cross-section as built and a cross-section fully braced (see Figures 34 and 35).

Torsional resistance which is very critical in the design of suspension bridges can be increased by: an increase in weight of the span, decreasing the cable sag, increasing the moment of inertia of the stiffening trusses, and increasing the width. The effects of these are shown in Figure 33. Comparison No. 9 gives the most resistance to torsion which is accounted for by its being fully braced. The fully braced design consisted of an open grid deck instead of concrete as used in the as-built bridge. It is interesting to note the great increase in No. 8 with the width increased to 53 ft. and 100 per cent more weight.

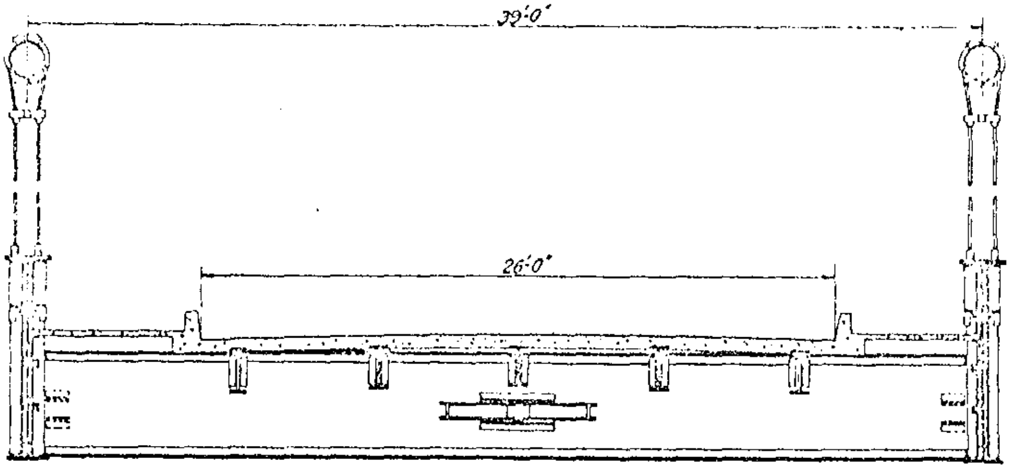



Figure 34. Section through suspended structure of Tacoma Narrows Bridge; as built (56)

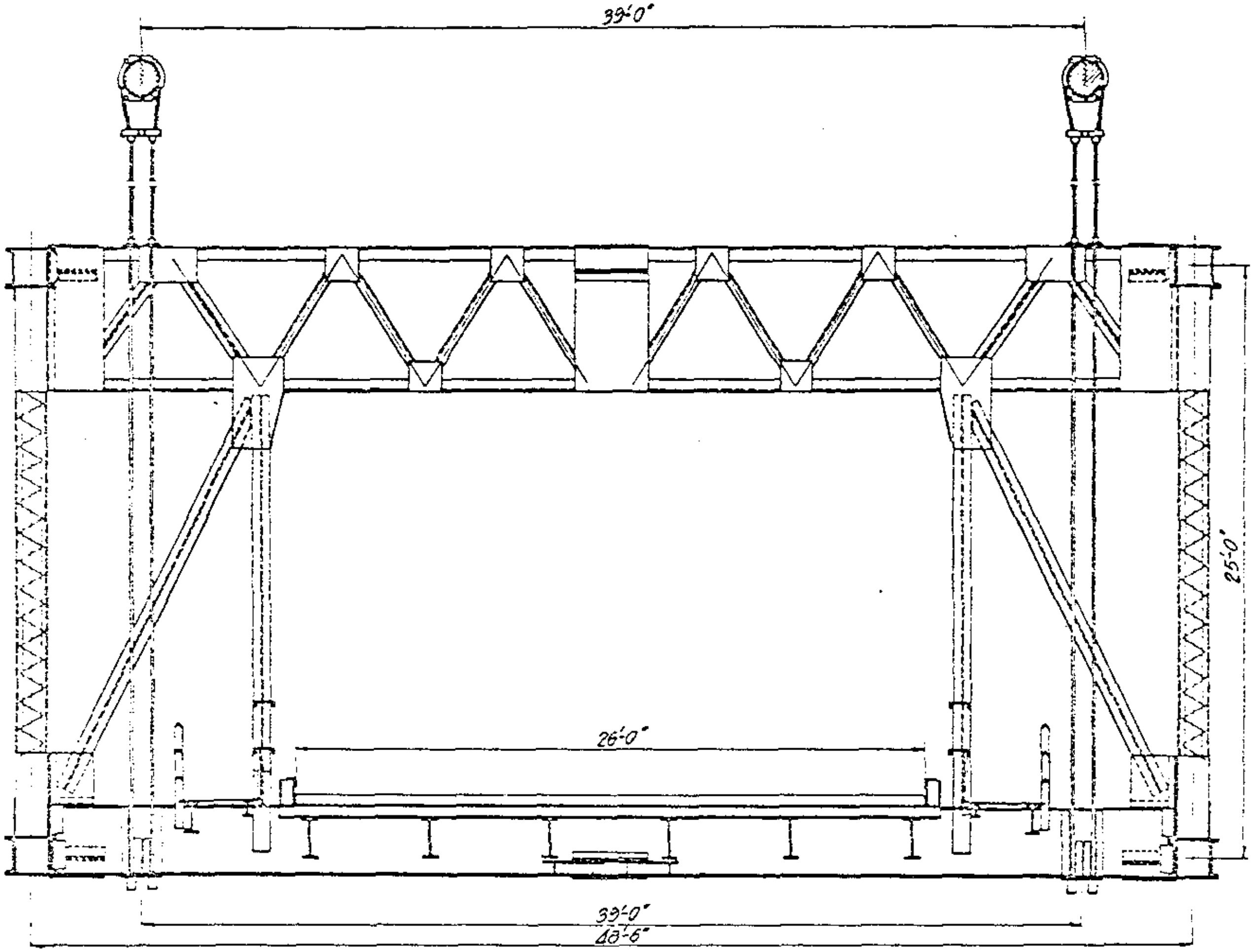



Figure 35. Section through suspended structure of Tacoma Narrows Bridge; fully braced design (56)

A suspension bridge is much more flexible than other types of bridges and the design is therefore very different. A movement of any part of the bridge causes a noticeable movement at other points throughout the entire bridge. The stiffening girders are usually shallow in comparison to their length and the rigidity of the structure is dependent largely on the dead load for its stabilizing effect. The term rigidity is not clearly defined to the structural engineer but it is very important to have rigidity in a structure to maintain a stable condition. Rigidity in a suspension bridge can refer to resistance to an increase in amplitude, increase in tilting of the deck, as to the acceleration of any movement. The amount of rigidity required may depend on the effect of motion on those using the bridge and excessive tilting may affect the operation of vehicles. Lack of rigidity may cause excessive wear on moving parts of the bridge. All these facts must be given consideration by the designer.

Wind is probably the most common cause of vibration in a suspension bridge and is found to have had a great effect on the Tacoma Bridge due to its lack of rigidity. The action of wind on a suspension bridge is very complex. Winds are usually treated as uniform static loads on a structure but this is not the case as gusts of wind cause very large forces and cause turbulent motion. Complex eddies and pressures are caused by wind on a fixed bridge but the situation becomes much more complicated when the bridge itself moves also. If the movement of the bridge is periodic, the pressures from the wind must also be periodic. If the pressures are in phase with the periodic movement of the bridge, a condition of self-induced resonance may result which is termed "negative damping."

Vibrations can be somewhat predicted if the conditions causing them can be predicted. Vibrations can be computed mathematically or by use of models. In the case of vibrations being caused by wind there is very unpredictable results from any calculations. If repeated impulses from wind throw energy into a vibrating system this vibration must be damped as fast as it is produced or resonance results and a build-up may result in failure of the structure. Damping may be supplied by the natural damping capacity of the structural steel itself, by friction, or by some mechanical means. In the Tacoma Narrows Bridge there was insufficient damping either in the steel itself or in the mechanical means employed. Many cycles of vibration occurred before the amplitude became objectionable.

In the Tacoma Bridge only 1 or 2 per cent of the total energy was stored in the girders. If the girders had been able to store more energy the failure may have been eliminated. The larger and stiffer the girders and cross bracing, the more energy will be absorbed and also damped by the structural steel in the girders.

There is a great deal to be learned from past experience and observations of other suspension bridges. Quite a number of suspension bridges have been observed and records made. Some of these along with some statistics are shown in Table 6. Much can be learned from observing structures already in existence, especially with regard to variable and unpredictable forces as those caused by wind which really has no

Table 6. Dimensions of suspension bridges with spans of 1200 feet or over (56)

| Name and location | Year completed | Center span ft. | Side spans ft. | Cable sag ft. | Width c.c. cable ft. | Depth of stiff'g girder ft. | Dead load center span lbs. per ft. of bridge | Design live load lbs. per ft. of bridge |
|---|---|---|---|---|---|---|---|---|
| Golden Gate San Francisco, Cal. | 1937 | 4200 | 1125 | 475 | 90 | 25 | 21,600 | 4,000 |
| George Washington New York, N.Y. | 1931 | 3500 | 610-650 | 325 | 106 | Ultimate double deck | | |
| | | | | | | 29 | 39,000 | 8,000 |
| | | | | | | Present single deck | | |
| | | | | | | -- | 31,500 | ----- |
| Tacoma Narrows Tacoma, Wash. | 1940 | 2800 | 1100 | 232 | 39 | 8 | 5,700 | 1,000 |
| Transbay San Francisco, Cal. | 1936 | 2310 | 1160 | 231 | 66 | 30 | 18,800 | 8,000 |
| Bronx-Whitestone New York, N.Y. | 1939 | 2300 | 735 | 200 | 74 | 11 | 11,000 | 3,000 |
| Ambassador Detroit, Mich. | 1929 | 1850 | 817-973 Unloaded backstays | 209 | 67 | 22 | 12,400 | 3,300 |
| Delaware River Philadelphia, Pa. | 1926 | 1750 | 716.7 | 197 | 89 | 28 | 26,000 | 6,000 |

Table 6. (Continued)

| Name and location | Year completed | Center span ft. | Side spans ft. | Cable sag ft. | Width c.c. cable ft. | Depth of stiff'g girder ft. | Dead load center span lbs. per ft. of bridge | Design live load lbs. per ft. of bridge |
|---|---|---|---|---|---|---|---|---|
| Bear Mountain West Point, N.Y. | 1924 | 1632 | Unloaded backstays | 200 | 61.3 | 30 | 11,540 | 3,160 |
| Williamsburg New York, N.Y. | 1903 | 1600 | 596 Unloaded backstays | 178 | 67 | 40 | 17,200 | 5,700 |
| Lions Gate Vancouver, B.C. | 1938 | 1550 | 615 | 150 | 40 | 15 | 4,600 | 1,230 |
| Mid Hudson Poughkeepsie, N.Y. | 1930 | 1500 | 750 | 150 | 42 | 20 | 8,800 | 3,000 |
| Manhattan New York, N.Y. | 1909 | 1470 | 725 | 148.5 | 96 | 24 | 23,280 | 8,000 |
| Triborough New York, N.Y. | 1936 | 1380 | 705 | 138 | 98 | 20 | 20,000 | 4,000 |
| St. Johns Portland, Ore. | 1930 | 1207 | 430 | 121 | 52 | 18 | ----- | ----- |
| Mount Hope Providence, R.I. | 1928 | 1200 | 504 | 120 | 34 | 18 | 5,300 | 1,500 |

exact solution. A discussion will be made of several suspension bridges with the hope that some designers may be made aware of the critical areas in design of suspension bridges.

The failure of the Tacoma Narrows Bridge was caused by oscillations being built up by dynamic wind forces and was aggravated by slipping of the stay ropes on the cable. The bridge was well designed for the dead load and live loads for which it was to carry but its flexibility was much too great when acted upon by variable wind forces. Its behavior can only be explained by its response to aerodynamic forces. Extensive laboratory experiments were made following the Tacoma Bridge failure along with theoretical studies.

The behavior of comparably large suspension bridges may partially explain the action of the Tacoma Bridge. Of the many similar bridges probably the Golden Gate Bridge and the Bronx-Whitestone Bridge have the most flexible stiffening girders. They have experienced some effects caused by aerodynamic forces. A discussion will be made of five of the longest suspension bridges with a comparison of their elastic characteristics and actual behavior.

The early history of suspension bridges goes back about 150 years and it seems as though the design of suspension bridges should have almost been mastered if the past designs and their actions had been studied. Most of the suspension bridges built in the first half of the 19th century were comparatively flexible. Oscillations of large degree were not uncommon in those days. Two accounts of interest follow (56).

"A highway bridge of about 400 ft. span over the Elk River, near Charleston, West Virginia, is suspended from wire cables without any kind of stiffening construction. The bridge accommodates a heavy traffic from the adjoining lumber region, and the oscillations and undulations of the flow, under the moving load of a four-ox lumber wagon, are enough to make a person seasick, but without causing an apparent inconvenience to the travel of vehicles, and certainly without detriment to the strength and durability of the structure. The bridge, as far as the speaker could ascertain, was built long before the Civil War; therefore, at the time it came under his observation, it had seen at least thirty years service, and it was still in an excellent state of preservations."

A suspension bridge over the Merrimac River near Newburyport, Mass., built in 1810 was described in the same article (56) as follows:

"When, some twelve to fifteen years ago, trolley cars were invented, the community owning this bridge did not hesitate to permit the crossing of electric cars, without making any change or addition to the structure, except to provide it with the necessary girder rails. The effect of a trolley car, weighing, perhaps 12 to 14 tons, on the unstiffened floor was more appalling to the speaker than anything he had ever experienced. Standing at one end of the bridge and seeing a car enter at the other end, it seemed suddenly to vanish from sight, giving the impression that the bridge was breaking down, but in a few moments the car emerged again, and after it had reached the opposite end of the bridge assumed its usual shape and appearance, as though it had never been disturbed. Actual measurement showed that the car caused a local deflection of about 20 inches in one quarter of the span, and a corresponding rise in the opposite quarter. The combination of the depression and rise projected on the car produced the impression that the car was gradually vanishing from sight. The speaker made a careful examination of all parts of the bridge, and found them in perfect condition and of ample strength for sustaining the loads."

There are many reports of failures in early suspension bridges. These failures were caused by wind, or moving loads. There is one failure due to marching of troops which produced synchronized oscillation of the structure.

Most of the early bridges were built of timber and were of very light construction. Very little was known about the effects of wind forces on the structure and causes were usually attributed to structural failure, material defects, or poor maintenance. The only means of stiffening was the weight of the floor and the railings. Stay ropes were quite common at that time. Some of the failures of early suspension bridges were discussed previously in the article describing the failure of the Tacoma Narrows Bridge (62).

The action of the Tacoma Narrows Bridge cannot be compared with any of these early bridges because they were of much shorter spans and the weights much less. Nevertheless, it is worthwhile to know that the problem of flexibility is not one brought about in the most recent bridges.

The flexibility became objectionable in the second half of the 19th century from the traffic point of view and this led to more stiffening. This was the major purpose for stiffening even into the 20th century. Oscillation and excessive deformations became a matter of concern but still very little was known about oscillations due to dynamic wind action. The only design criteria regarding wind was an assumed amount of static pressure.

As the elastic theory began to develop the emphasis was being placed on deep girders or stiffening trusses. The Williamsburg Bridge across the East River in New York,which was completed in 1904, had a stiffening truss of 40 ft. deep. This was to comply with the design specification of the depth being not less than 1/40 of the span length, its span being 1600 feet.

The theory of added weight increasing rigidity came about with use of concrete decks instead of wooden decks. This resulted in a decrease in girder depth as spans increased. This is shown in Table 5. The importance of weight as a stiffening factor was realized in the George Washington Bridge. It had a center span of 3,500 ft. and a suspended weight of 68,000 tons. It had a depth of 29 ft. or 1/120 of the span length. It was ultimately planned to add a second deck for railway transportation and the stiffening was decreased assuming that the double deck would add to the rigidity of the bridge. Even before the second deck was added the bridge behaved very well under live loads and wind loads due to its weight.

Engineers began to favor a greater amount of flexibility in their design and a decrease in weight was also taking place. This was the case in the design of the Golden Gate Bridge with a 4,200 ft. span and weight of 45,000 tons in the center span. The Bronx-Whitestone Bridge had a span of 2,300 ft. and weight of 13,000 tons and the Tacoma Narrows Bridge had a span of 2,800 ft. and weight of 8,000 tons. All these bridges experienced some oscillations but effective damping has been used.

There has been some motion in all suspended bridges either vertically or laterally but none of any unusual character in spans up to 1,850 ft. nor in the San Francisco-Transbay Bridge of 2,310 ft. span. All of these have relatively rigid stiffening trusses and ample width which is unlike the Tacoma Bridge.

The Golden Gate Bridge, completed in 1937, has experienced some torsional motions under moderate winds, but only slightly noticeable.

In 1938 and in 1941 there had been winds up to 75 and 62 m.p.h., respectively. Vertical oscillations were then observed with amplitudes of approximately 2 feet. At the same time lateral movement of 8 and 5 ft. were observed. Weight is an important factor in the rigidity of the Golden Gate Bridge.

The Bronx-Whitestone Bridge was subjected to wind forces which caused oscillation of large amounts but was corrected by damping installations as shown in Figure 36. Double amplitudes of 24 in. had been observed with the most severe motion when the wind was at an angle less than $45^{\circ}$ to the longitudinal axis of the bridge. The most severe motion also occurred when a single node was at the center of the bridge span. The bridge was more stable when exposed to winds of higher velocities. The weight of the bridge was the contributing factor in stability of the Bronx-Whitestone Bridge.

Weight not needed to meet the design requirements is an expensive way to supply additional rigidity in long span suspension bridges, because even when added to the floor slab, which is relatively cheap, the weight involves considerable additional expense in increase of suspender size, cable size, towers and anchorage. Addition of weight to the floor slab increases the moment of inertia with a resulting decrease in deflections.

The weight of the Tacoma Narrows Bridge was 6,270 lb per ft. An increase of 10 per cent by adding 2 in. to the thickness of the roadway slab would decrease the quarter point deflection by only about 5 per cent as seen in Figure 37. Increasing the weight by 100 per cent which could

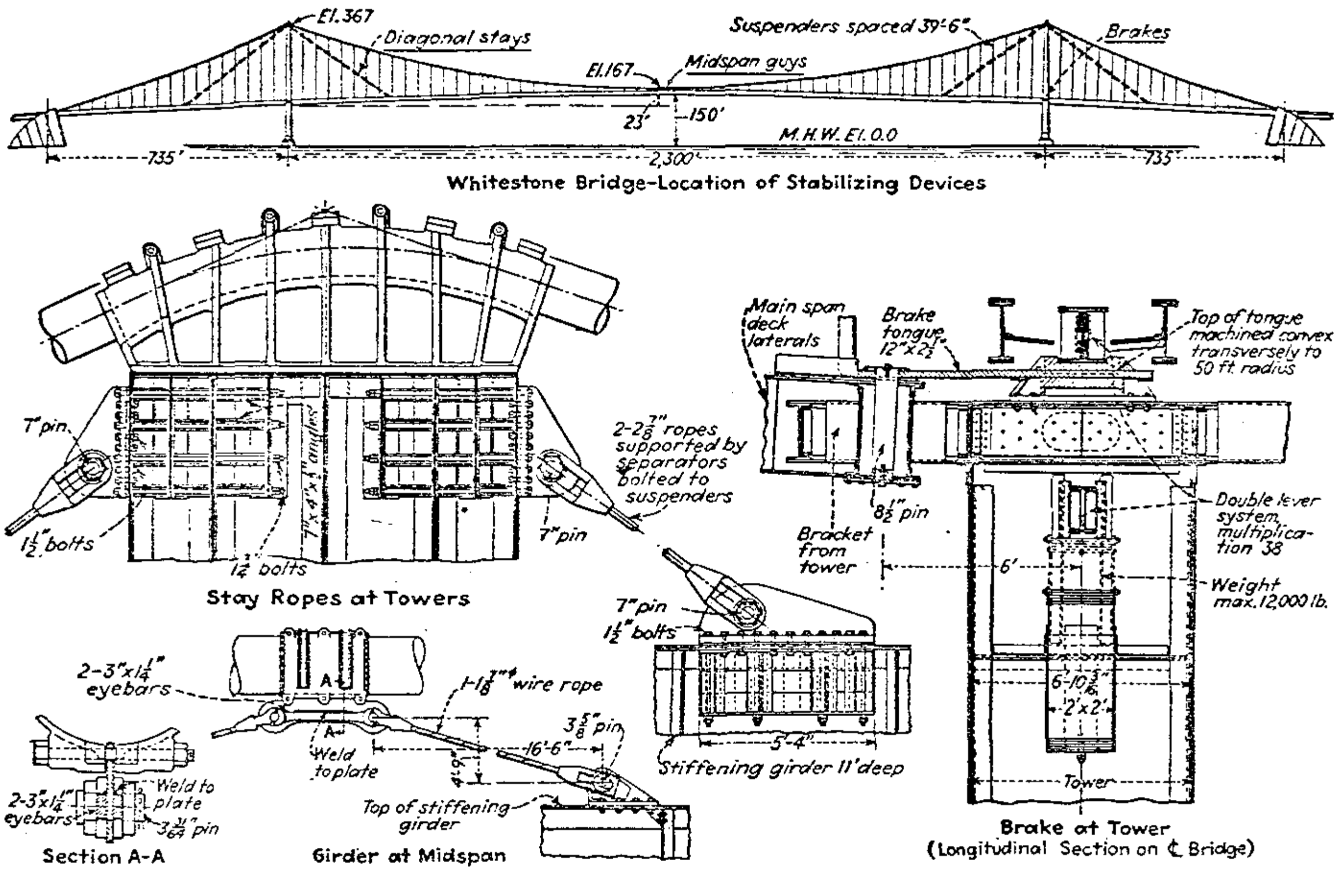
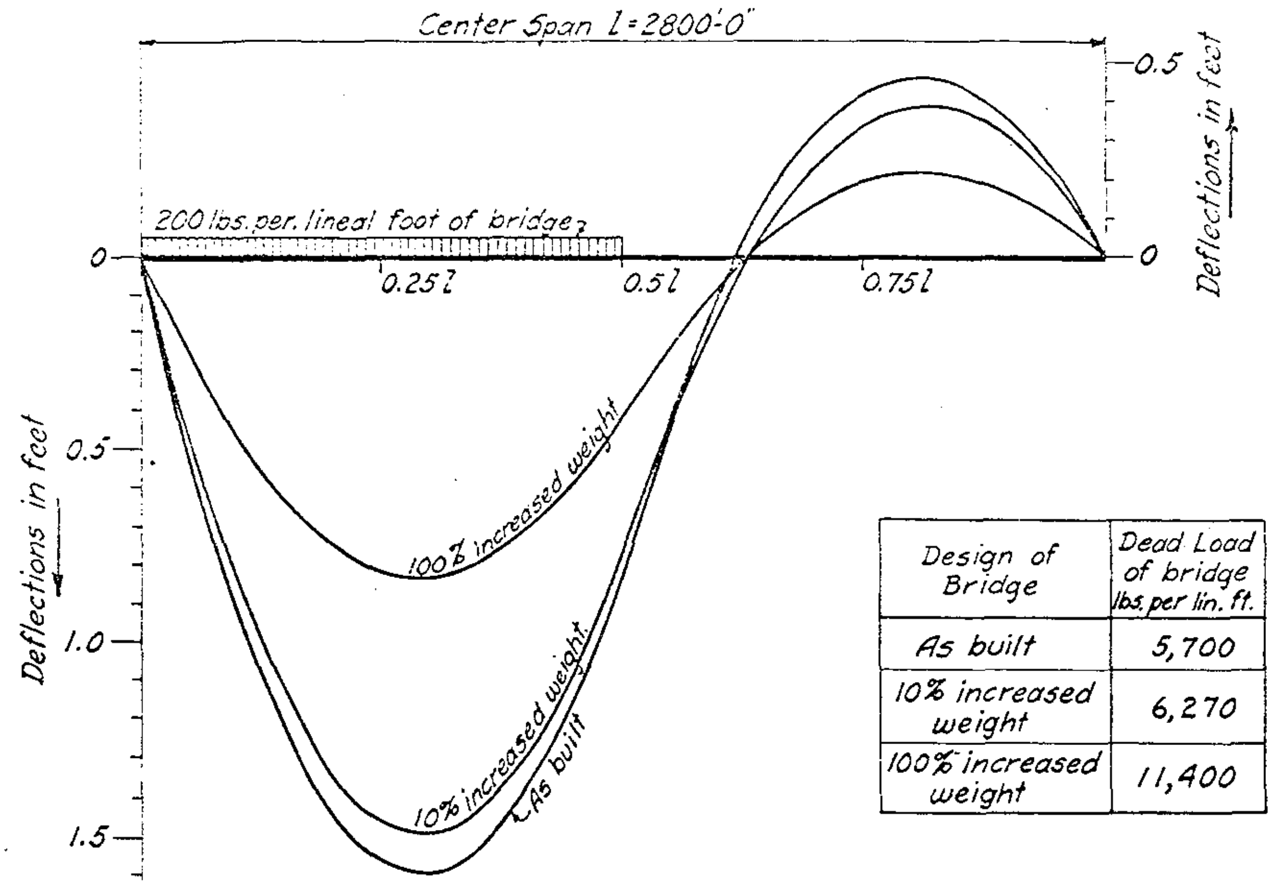



Figure 36. Damping installations on the Bronx-Whitestone Bridge (56)




| Design of Bridge | Dead Load of bridge lbs. per lin. ft. |
|---|---|
| As built | 5,700 |
| 10% increased weight | 6,270 |
| 100% increased weight | 11,400 |

Figure 37. Effect of weight on comparative vertical deflections of the Tacoma Narrows Bridge (56)

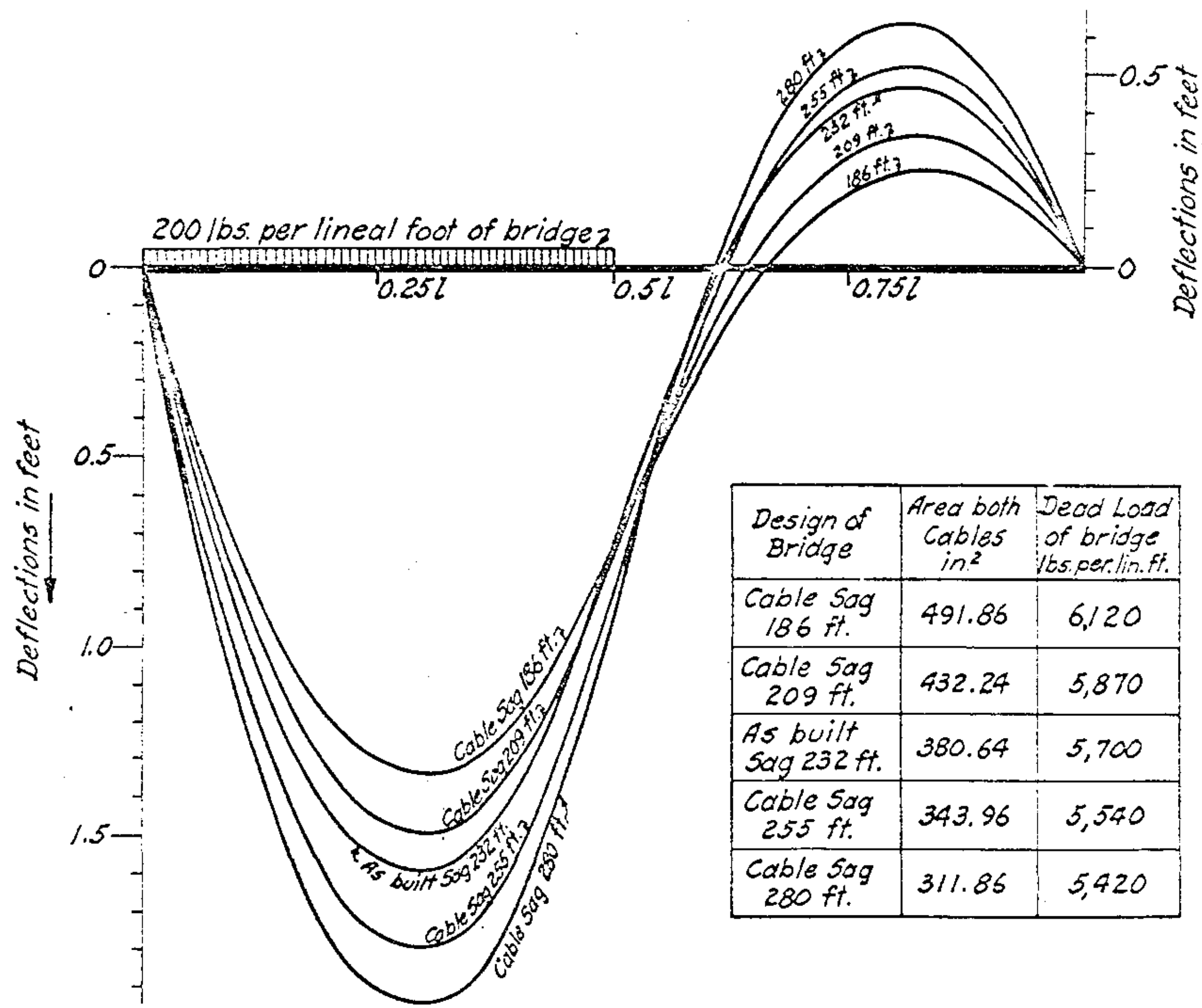
be accomplished by doubling the traffic capacity, would decrease the static deflection near the quarter point from 1.59 ft. to 0.83 ft. which is a 48 per cent decrease. To make the bridge with rigidity like the Golden Gate and the Bronx-Whitestone the weight would have to be made 2 1/2 times greater with the other proportions remaining the same.

If the weight cannot be made to comply with the design requirements a decrease in the cable sag can accomplish the same result up to a point where the added expense becomes too great to make it economically feasible. The added cost would be in the increase in cable size and in anchorages. This was a factor in the design of the Tacoma Narrows Bridge. The cable sag was decreased as much as economically feasible.

The cable sag of the Tacoma Narrows Bridge was 232 ft. giving a ratio of cable sag to span of 1:12. A decrease of 10 per cent or a sag of 209 ft. would decrease the deflection at the quarter point from 1.59 to 1.49 which is only 6.3 per cent. This could not supply the necessary rigidity alone. The effect of cable sag on deflection is shown in Figure 38.

A comparison of vertical deflections under static loads on the various suspension bridges reflects their susceptibility to oscillations. The effect of weight as well as the stiffness of the suspended structure are reflected in their ability to resist vertical deformation. The dynamic rigidity is not considered, however, since damping effects are not reflected nor are other dynamic characteristics of the structure.

The oscillations caused by wind forces are usually kept to a minimum which are acceptable by the public as they pass over the bridge. These




| Design of Bridge | Area both Cables in.² | Dead Load of bridge lbs. per lin. ft. |
|---|---|---|
| Cable Sag 186 ft. | 491.86 | 6,120 |
| Cable Sag 209 ft. | 432.24 | 5,870 |
| As built Sag 232 ft. | 380.64 | 5,700 |
| Cable Sag 255 ft. | 343.96 | 5,540 |
| Cable Sag 280 ft. | 311.86 | 5,420 |

Figure 38. Effect of cable sag on comparative vertical deflections of the Tacoma Narrows Bridge (56)

oscillations are usually not critical in the structural stability of the bridge. However, a rather small wind can cause large effects by acting longitudinally on the suspenders, cables and towers and also when acting directly at right angles to the structure.

The regularity and frequency of the motion of the bridge is not necessarily due to the frequency and intensity of the wind forces but is largely due to the behavior of the suspension system of the bridge when acted upon by dynamic forces. The action of a suspension bridge is sometimes much more severe in a mild wind than in very large winds. The action of suspenders under the influence of wind forces greatly affects the cables. Freely suspended cables are very sensitive to displacements under forces acting in their vertical planes. Large oscillations are caused by the large amount of energy to which they are subjected and by the very little amount of energy absorbed by elastic deformation. The result is movement of the system and if not damped in some way the motion continues to build up. Model tests show that, after the dynamic impulses have stopped, the flexible suspension system continues to oscillate for a great length of time.

The more flexible the system, the more the whole system tends to act as a free catenary and the motion will continue to increase as the capacity of the system to absorb energy decreases. Also the more flexible the system the more easily it oscillates in increasing numbers of waves or nodes at higher frequencies. The action of the cables will greatly control the action of the whole system if the stiffening girders are very flexible. The influence of the stiffening girders on the action of the cable system is 1.3 per cent for the Tacoma Narrows Bridge, 2.5 per cent for the Bronx-Whitestone Bridge, and 7.5 per cent for the Golden Gate Bridge. This means that the vertical deflections of the cables is decreased a very small amount in the Tacoma Narrows Bridge. The vertical flexibility is far in excess of that of any of the other bridges used in the comparison. These facts are given in the following table.

Table 7. Comparative vertical deflections (56)

| | For 1/4 point | For center of span |
|---|---|---|
| Golden Gate Bridge | 2.3 to 1 | 3.9 to 1 |
| Bronx-Whitestone Bridge | | |
| without stays | 2.6 to 1 | 3.3 to 1 |
| with stays | 3.0 to 1 | 3.7 to 1 |
| San Francisco-Transbay Bridge | 4.0 to 1 | 3.5 to 1 |
| George Washington Bridge | 5.1 to 1 | 10.4 to 1 |

These same results can be seen graphically in Figure 39. It is apparent that the vertical deflection of the Tacoma Narrows Bridge was comparatively very large.

Lateral motion of long span suspension bridges is not a matter of as much concern as that of vertical deflection. None of the bridges studied have had lateral deflections that became a threat to the structural stability of the bridge. The Golden Gate Bridge in a 75 m.p.h. wind storm had a lateral deflection of 8 ft., but this much of a

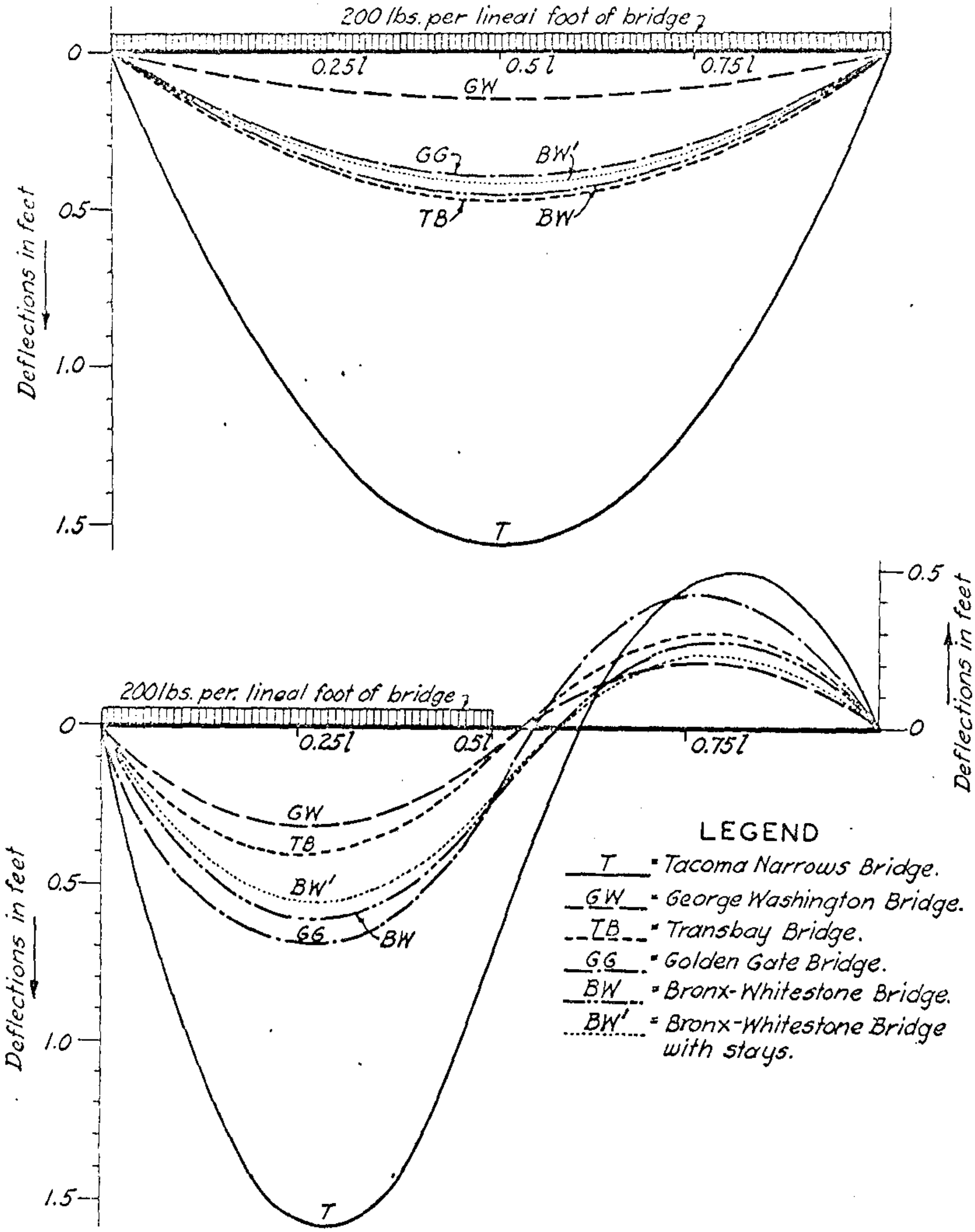



Figure 39. Comparative vertical deflections of the five longest suspension bridges due to a load of 200 lbs. per lin. foot of bridge (56)

deflection was permitted in the design. This movement was not objectionable to the motorists on the bridge. Lateral deflection is a result of average wind pressure on the exposed area of the structure. The rigidity in the lateral direction of the bridge with most flexibility in the vertical direction has been sufficient to resist any large oscillations in the lateral direction. The damping effect in the lateral direction is also great compared to that in the vertical direction. The moment of inertia is much greater about a vertical axis than about a horizontal axis. The Tacoma Narrows Bridge had a moment of inertia in the lateral direction of 35 times that in the vertical direction. The dynamic resistance is even much greater than the increase in lateral moment of inertia.

The distribution of wind forces to the floor and cables for the Tacoma Narrows Bridge is shown in Figure 40 and illustrates the supporting effect of the cables on the wind truss. If the wind truss is relatively flexible in the transverse direction much of the wind force acting on the suspended structure is transmitted to the cables and then to the towers. Even the most flexible and narrow long span suspended bridges have a relatively small amount of lateral motion. The lateral stability and deflection are largely independent of the rigidity and width of the wind truss. The rigidity of the wind truss in the lateral direction is a more important factor in resisting torsion when the vertical rigidity is insufficient.

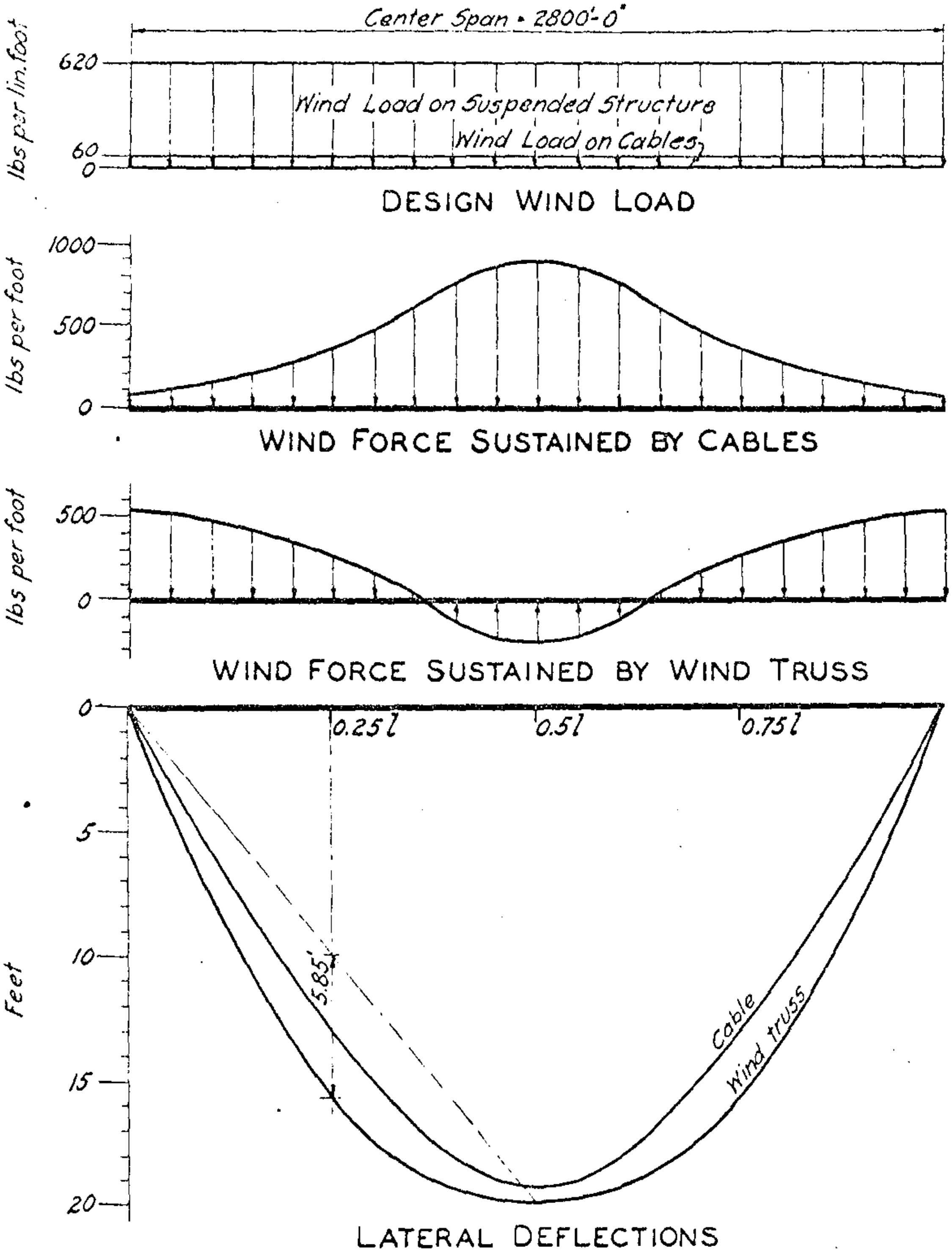



Figure 40. Distribution of 620 lbs. per lineal foot design wind load and resulting lateral deflections of the Tacoma Narrows Bridge (56)