A REVIEW OF MODERN PRACTICES IN THE DESIGN AND CONSTRUCTION OF LONG SPAN BRIDGES WITH PARTICULAR ATTENTION TO THE RELATION OF INFLUENCE DIAGRAMS OF STRESS TO THE DISTRIBUTION OF METAL IN STRUCTURES OF THE CENTILEVER TYPE

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FOREWORD

Bridges have been important and have made an active claim upon human ingenuity since the dawn of civilization. Nature first demonstrated the possibility of a crossing over streams or other openings without obstructing the opening itself. Waddell has, as a frontispiece in his work "Bridge Engineering," a rather imaginative portrayal of a band of monkeys crossing a stream by means of an animated chain. One monkey has attached himself to a tree and holds to another monkey who in turn holds to another and so on. This animate chain first acts as a pendulum until the head monkey can, at the end of the swing, grasp a tree trunk on the opposite side of the stream. A suspension bridge is thus completed. Human chains have been recorded in connection with certain rescue work. Most of our lessons come from natural manifestations and it is not surprising that man contrived to fall trees across space after having crossed on those felled by nature. Swinging across a stream on a grape vine was the classical manner in which to elude one's pursuers during the conquering of our own wilderness. The Hellespont was bridged by a system of pontoons which permitted the crossing of an army. Every student of Latin must be somewhat familiar with the genius of the Roman military engineers. Man has closely observed and well learned the lessons of nature, and chasms, torrents, or mighty rivers are no longer serious obstacles in the path of the march of progress.

The building of a bridge was considered in the light of a glorious achievement in the earliest of times and during the Middle Ages there was a monastic brotherhood--The Brethren of the Bridge--who considered it a sacred privilege to be permitted to participate in such undertakings. Today the completion of a new structure is the occasion of public celebration and the dedication ceremonies are conducted in all dignity. In more recent times we have seen, and benefitted by, the great advances made in bridge engineering due to greater information concerning the various problems involved and the ability to fabricate finer materials into the completed structure. The layman of today rather takes scientific developments for granted, but to those who have been confronted with the problems involved in the building of a bridge, for instance, cannot but marvel at the success which crowned the efforts of those ancient builders who could only build upon inspiration and experience. The open spandrel arched masonry bridge was developed by an ingenious builder after his solid arch or filled spandrel had twice failed--the open spandrel was devised to lessen the load at the haunches, experience necessitating a new idea in design.

The greatest advance in the science of bridge engineering has been made well within the past century. Our stupendous structures of today are the product of a thorough knowledge of weights and loads, the stresses caused by the application of such forces, the behaviour of the members under such stress, skill in the making of materials of construction, the science of using these materials to the best advantage, and fabricating them into the completed structure. Most institutions have, at some time during their histories, a golden age, and it would appear that bridge building, as a science, is enjoying, or is soon to enjoy, this glorious page in its history.

Stone was the first material of importance and most of the famous bridges of the past were constructed of this material. Timber has always had its place but is not mechanically applicable to major structures and does not have the permanence demanded in minor structures. Its importance as a structural material must not be belittled, however. Wrought iron and cast iron were tried and enjoyed the structural lime-light for a while and then gave way to steel. It is with steel that the wonders of today have been effected and it is with this material that we shall concern ourselves in its application to long span bridges. With the economic production of special alloys and with the effective heat treatments to which such alloys are subjected, a structural material has been developed which is specially suited to our needs. Even the engineer is somewhat awe-stricken at the proportions of some of our modern structures, the greatest of which is now under construction over the Hudson River. This stupendous structure is of the suspension type and is to have a clear span of 3,500 feet--nearly three-quarters of a mile.

LONG SPAN BRIDGES

Long span bridges have been mentioned but no definite limit has, as yet, been set beyond which a span shall be long and below which a span shall be short. This line of division is not definitely drawn but can be approached from a consideration of the various types of structures.

The most simple structures are those composed of a simple, or a system of, simple beams suitably supported and upon which decking is placed to permit the passage of vehicles or to receive certain loads. The limiting span for a structure of this type is generally specified as 30 feet. For spans ranging between 30 and 80 feet the plate girder is usually specified. The plate girder being a built-up member of steel which acts as a beam, but due to the concentration of metal at the flanges it is more economical than an ordinary beam. Beyond 80 feet simple trusses are usually the most economical structure to use, although architectual or aesthetic demands may decide the use of trusses of special configuation or pure arches. Simple trusses, structures which are statically determinate, have been built to bridge spans as great as 750 feet.

However, the economy of building a simple truss with this span is in question, and it is apparent that other types could be built for the same span with an economy of metal and erection cost. While it is mechanically and structurally possible to build simple trusses of greater span, it is not the type of least cost. There will be some particular span which will mark the economic limit of the simple truss and beyond this limit it is economically desirable to employ some other type of structure. This economic limit of the simple truss may be taken as the end of the short span and the beginning of the long span. This span is, according to modern practice, about 600 feet.

In most cases of simple span construction it is customary to erect the structure upon false work which is removed after the erection and connection of the essential members of the truss. This false work is, of course, an item of additional expense and will interfere with commerce in case of a navigable stream being bridged. Long span construction does away with most of this falsework and it is entirely eliminated in the main spans. This fact of erection often determines the type of bridge to use in any particular instance. If a gorge is to be crossed, of such depth as to make the use of falsework impracticable, it is very desirable--and often essential--to be able to build without falsework. It is, of course, possible to erect a simple span upon the cantilever principle, but due to the fact that the erection stresses will be different in nature to the stresses occurring in the complete structure it becomes necessary to design primarily tension members to withstand compression erection stresses, and such procedure is not economical of metal. This is quite frequently done in very short spans. One of the distinguishing features of the long span construction is the erection, of the main spans at least, without the use of falsework, and the members of the completed structure have stresses of like nature as those experienced during erection.

TYPES OF STRUCTURES SUITABLE TO LONG SPANS

The unstiffened suspension bridge was the first type of structure to be applied to long spans, followed by the continuous truss. Out of the development of the continuous truss came the cantilever type which for a while almost totally eclipsed the continuous type. Besides these three types must be mentioned the braced arch whether three-hinged, two-hinged, or hingeless. This latter type has been used in some very notable present-day examples, but the economical limit does not greatly exceed that of the simple truss. The types of structures most readily applicable to long spans are brought down to the following list:

Suspension

Cantilever

Continuous

Arched.

THE CONTINUOUS TRUSS

If a beam be supported at its ends in such a fashion as to be free to rotate about both points of support and free to move longitudinally at one support, it is said to be simply supported and statically determinate. If a third support is introduced between the end supports the beam becomes continuous and is no longer statically determinate. The magnitude of the intermediate reaction will depend upon the stiffness of the beam and must be determined from a consideration of the deflection of the system under any particular loading. Since trusses act in a manner similar to beams, this same method of analysis is applicable to this type of structure as well.

As previously mentioned the continuous truss was an early structural development and was frequently used in the first half of the last century. Shortly after 1860 this type went into disfavor and was given very little attention in this country, and prior to 1917 there was but one major example of this type in America. This was the Lachine Bridge over the St. Lawrence River near Montreal. It was built as a cantilever and then converted to a continuous type for the live loads. In

1917 was completed the Sciotoville Bridge across the Ohio River. This bridge has two continuous spans of 775 feet each. Since this time the type has been given more attention and should certainly be included as a standard type in American practice.

The first and most common objection raised against the continuous truss is its static indeterminateness. It is true that the analysis of an indeterminate structure is much more involved than in the case of simple structures, but methods have been developed by which the analysis can be made with as great a certainty as that of the simple type and the increase in time and expense due to such analysis is lost in the decrease in cost of the continuous type.

It is true that an unequal settlement in the piers would cause a change in the magnitude of the stresses in the members and might even cause a reversal of stress. This objection is frequently raised, but what engineer is going to place a major structure upon piers subject to settlement without also including arrangements to compensate for such possible settlement?

In nearly all specifications concerning the design of bridge members, special notice is given to those members subject to a reversal of stress. Steinman looks upon the more stringent of these specifications as the relics of the old fatigue theories which have been exploded, and contends that there is no place for such requirements in specifications for long span bridges. It is true that in a continuous structure there are more members subject to stress reversal than in the simple type and if these stringent specifications are enforced one of the most potent sources of economy of the continuous over the simple type is severely reduced.

Modern practice indicates that the continuous type of bridge is sound in every way and it is especially applicable to cases or sites having the following conditions:

Long span
Moderate truss depth
Piers of moderate height
Good foundations
Spans approximately equal
Cantilever erection.

THE CANTILEVER TYPE

A beam supported at an end and at some intermediate point constitutes a cantilever system and the portion of the beam which projects beyond the intermediate point of support, and whose end is free, is called the cantilever arm--the remaining portion of the beam being the anchor arm. If the beam is so loaded that the moment of the cantilever arm about the support at the intermediate point is greater than the moment of the anchor arm about the same support, then the system will tend to rotate about this point of support and the reaction at the end support will become negative.

If, in the case of a continuous beam or structure the elastic curve be plotted, it will be seen that this curve has points of contraflexure. It may be shown that for a particular system the travel of this point of contraflexure is confined to relatively small limits as the nature of the loading is changed over a very considerable range. It is also known that the moment in the truss or beam is zero at these points of contraflexure. These facts led Ritter, in 1860, to propose to cut the continuous structure at the points of contraflexure and introduce rockers or hangers incapable of transmitting moment at the points of severence. Such a system would become statically determinate, but the condition must be imposed that the system is also stable.

From these considerations the cantilever bridge was developed and since it is statically determinate the continuous type, from which it developed, was discarded and particular attention given to the cantilever type. The stamp of public and professional approval became so firm that an era of cantilever bridges was entered with the result that a great many of these structures were erected without reference to economic suitability. A large number of these were not economically justified since a series of simple spans or a continuous structure could have been built at less cost.

The first great bridge of this type was that across the Firth of Forth and which was completed in 1890. This structure has a clear span of 1,700 feet and remains as one of the outstanding engineering achievements of modern times. After the completion of this bridge there were numerous others erected but of lesser spans. The notable examples were the Hudson River bridge at Poughkeepsie, the Monongahela cantilever, the Beaver, and the Carquinez. The greatest cantilever bridge yet built is that across the St. Lawrence River at Quebec. This structure has a clear span of 1,800 feet and was completed in 1917 after two major disasters, but the structure as completed remains as one of the greatest examples of the type. The Carquinez cantilever has a clear span of 1,100 feet; it was completed in 1927 and is a splendid example of the type as applied to highway traffic. For a study of details and connections as applied to this type of bridge construction the St. Lawrence, Carquinez, and Beaver cantilevers are outstanding examples.

The cantilever bridge is a proven type and will undoubtedly remain as a standard type in American practice. Its particular limitations are a lessor degree of rigidity, and more steel per unit live load carried--both of which must be waived in sites peculiarly adapted to this type of structure.

THE SUSPENSION BRIDGE

The suspension bridge consists, essentially, of a passageway attached to a cable, or to a system of cables, which has been placed across and opening and suitably anchored. The vertical loads imposed upon the structure cause horizontal pulls at the points of support and at the anchorages. In its simple adaptations, at least, it may be used at almost any site where there is sufficient clearance for the sag or deflection of the cables and suitable provisions for anchorage. Aside from the simple beam bridges and the masonry arch, this type of construction enjoys the greatest antiquity. It is known that suspension bridges existed in China nearly 2,000 years ago. Historians believe that China and India produced the first suspension bridges in which iron chains were used. One of these in the Province of Yunnan is described by Kirchen, and is said to have been built by order of the Emperor Ming in 65 A.D. The length of this structure is given as 300 feet and like all others in that country prior to the sixteenth century it had the plank floor laying directly on the chains. Many others are reported in China, among them being one over the River Pei with a span of "several hundred" feet. A remarkable one is in Hindustan with a span of 600 feet over the Sampoo River, and is described by Major Ramel. In 1802 Humbolt found a suspension bridge in Peru crossing the Chambo River, with a span of only 40 feet, the cables of which were 3 feet in diameter and made up of twisted roots. He found another with a span of 131 feet with cables 4 inches in diameter, supported on timber frames, the cables being attached to posts driven into the ground. Others in South America had cables made of cow-hide. One of the earliest known suspension bridges in Europe was built by soldiers in 1515 to transport artillery over the Padus River in Italy. The first suspension bridge recorded in America was built by Finley in 1706.

The cables may be built up from wrought iron or forged steel links, either forged together or pin connected. This forms one of two distinct types of cable, the other being units of stranded or parallel wire units. In any event, the cable is more or less flexible and free to assume various positions of equilibrium when subjected to various load conditions. For light loadings and unimportant structures a distortion under moving loads may not be intolerable, but for major structures such distortion is to be avoided and for this reason suspension bridges may be either simple and unstiffened, or stiffened and comparatively rigid structures.

In the case of the unstiffened bridge a concentration imposed upon the structure is transmitted directly to the cable at not less than one nor more than two points and this condition is conducive to large local deflections. If, however, the structure upon which the passage way is placed is made stiff and somewhat rigid there will be a distribution of concentrated loads over the entire length of the cable, depending upon the degree of stiffness or rigidity of the frame suspended from the cables. The introduction of this somewhat rigid stiffening structure is the distinguishing feature of the type of structure known as the stiffened suspension bridge. This latter type is the one most frequently encountered in modern practice and also most adapted to modern traffic. It is also the best solution which has as yet been offered for the case of very long spans. The exceptional features of this type of bridge which makes it so especially suitable to the long spans are, its inherent latteral rigidity, comparative ease of erection without the use of falsework, and the marked predominance of members carrying tensile stresses.

Continuous, cantilever, and arched bridges have been fairly well developed and standardized within the past quarter-century--the improvements being made more in the matter of proportions and details and less in general principles of design. The suspension type, on the other hand, still offers a diversity of major principles in design from which to select and apply to any particular structure.

In 1826 the Seguin Brothers, in France, made a series of experiments with stranded wire cables and their results were accepted with the subsequent erection of a number of suspension bridges in which this type of cable was used. There are, however, inherent disadvantages to be noted in the consideration of a stranded cable, the principal objection being the uncertainty of any mathematical dealings with the problem of the ratio of stress to strain and the further uncertainty of elastic recovery upon the successive application and removal of loads. The removal of this objectionable feature of wire cables must be attributed to Roebling who first introduced the parallel wire cable, in which the individual wires which compose the cable are all parallel to the longitudinal axis of the completed cable. Cables of this type have come to be one of the most important adjuncts to modern suspension bridge practice. They are almost universally spun in place, with very little uncertainty concerning the behaviour of the cable under stress and the distribution of stress among the individual wires which compose the cable as a whole.

The search for materials having high tensile strength has led to the use of heat-treated wires in some of the present-day structures, but there is still much to be done in this respect, as is very forcefully brought home in two structures being erected at the time of this writing. In the spinning of the cables for the Mount Hope suspension enough of the individual wires broke during the spinning operations to excite concern which eventually led to the condemnation of the cables. A bridge under construction at Detroit had its cables already in place and much of the suspended structure attached, but the cables had been fabricated under the same specification as those of the Mount Hope bridge. There had also been a few failures of individual wires during the spinning of the Detroit cables but not enough to warrant a condemnation. However, after the condemnation of the Mount Hope bridge it was decided to dismantle the Detroit bridge and replace the cables. These operations are very expensive both in time and money and forcefully illustrate the care which must be exercised in these things and the field for further experimentation and perfection.

The eye-bar chain is the other alternative in the matter of cables, and is gaining favor after having been practically discarded in favor of the parallel wire cable. The pin-connected eye-bar chain has a number of desirable features. The links may be made, treated, and tested before being put in place, and due to the apparent uncertainty of heat-treated materials this possibility of tests upon the full-sized member is desirable. There is also a variation of stress along the length of the cable increasing as a function of the slope from the center to the point of support. Since the eye-bar chain is made up of links it is possible to vary the cross-section in proportion to the stress, and such an arrangement leads to an economy of material.

The method of arranging the cables in their saddles at the tops of the towers has received a great deal of consideration. As the main and side spans are subjected to various conditions of loading there is a tendency to destroy the equilibrium which must exist at the top of the tower. If it were possible to devise a frictionless sheave as a cable support at the top of the tower, equilibrium would readily be restored by a travel of the cable over the sheave. Such an arrangement is not feasible, so the first development was to mount the saddle on rollers so that equilibrium could be maintained by a travel of the saddle, which travel would change the distance between supports in accordance with load and temperature variations. However, due to the fact that the vertical reaction at this point is of rather large proportions, it has been found that this readjustment was not smooth and instantaneous but lagged the load and temperature variation and made jerky readjustments. These jerks, aside from terrifying people who happen to be on the bridge at the time, cause rather severe and indeterminate stresses in the members of the structure. The practice of rigidly fixing the cable to the saddles and the saddles to the towers was evolved from these considerations.

Even when the cables are fixed to the towers there must still be some provision to permit readjustment of span under load and temperature variation in order that equilibrium be maintained. This must be accomplished by the towers themselves. In the earlier developments the towers were built of masonry and were comparatively rigid and their behaviour under latteral forces in the vertical plane of the cable would be questionable. However, since the cable is fixed to the tower, this essential displacement in the vertical plane of the cable must be provided by a deflection of the tower. This requirement led to the introduction of the steel tower, the first notable example of which being those used in the Williamsburg bridge, which was completed in 1904. Any considerable deflection of the top of the tower would cause as much eccentricity and resultant bending moment. Since the vertical reaction is large, a small eccentricity would produce a large bending moment. In order to eliminate this undesirable feature it was proposed to hinge the towers at their bases and the Manhattan bridge was the first major application of this idea. The failing of this idea lies in the low and indeterminate efficiency of the rockers and the entrance of a bending moment due to rocker friction.

Most of the outstanding examples of suspension bridges in the past ten years have shown a tendency towards the fixed-base tower, although there have been enough hinged-base towers placed in operation to leave the final decision somewhat in doubt. It is probable that the fixed-base will eventually predominate. The Fort Lee bridge now under construction is to have parallel wire cables and fixed-base towers.

The configuration of the suspension bridge has recently received some attention. The conventional design is the parallel wire cable from which is suspended a parallel chord stiffening truss, usually of the Warren type. At the center of the main span the cables may be very close to the stiffening truss and such a condition has caused certain designers to remark upon the proximity of two major members, one acting in tension and the other acting in compression. If one member could be placed which would perform the duty of both cable and truss chord, there would be a tendency towards cancellation of stress and a resultant saving in metal. Also a study of the bending moments as they occur in the stiffening truss reveals a maximum near the quarter points. These considerations led Steinman to propose, design, and ultimately build a bridge at Florianapolis, Brazil, which he has designated as a suspension bridge of the Florianapolis type. This type combines the cable and upper chord of stiffening truss in the central portion of the main span and further gives a stiffening truss of varying depth, with its greatest depth in the region of the maximum bending moments. The writer has proposed a suspension bridge of the Florianapolis type, the configuration of which is shown in the accompanying sketch.

NATURAL AND IMPOSED LIMITATIONS AFFECTING THE SELECTION AND ADAPTATION OF THE TYPE OF STRUCTURE TO PARTICULAR SITE

Leon Moisseiff was Engineer of Design on the Delaware River bridge. In an address to a meeting of the Franklin Institute Mr. Moisseiff remarked, "The planning of a municipal bridge is the art of coordinating into a well-balanced system the demands of commerce and travel with land values and allowable finances, and of the available materials and means of construction with economic efficiency and monumental beauty. Some of these factors are determined by laws, some are directed by public opinion, and others are left to the knowledge and judgment of engineers and architects." Although Mr. Moisseiff limits his remarks to municipal bridges, his observations are applicable to bridges in general. He clearly indicates that the problem must be approached from many different angles.

The people for whom bridges are built are interested not only in a safe, substantial structure, but primarily in the investment involved. Bridges are built not merely for the purpose of crossing a river but because by investing a certain amount of capital in the enterprise, a substantial return is to be realized from the investment by being able to cross the river. In public works this return may make itself manifest in the convenience to the public, the opening up of a better and more spacious residential district, or in facilitating access to recreational areas. Any reasonable investment which will contribute to the general well-being of the people is worthy of favorable consideration--a bridge strategically or conveniently located may, under many circumstances represent such an investment.

In the beginning of our history many bridges were desirable but the state had not the wealth to invest in many worthy projects. At this time private capital was induced to erect and operate under government franchise a great many toll bridges. In the majority of instances these toll bridges became sources of very satisfactory returns on invested capital and an era of toll bridges entered which flourished for many years but eventually the toll bridge went into disfavor. The idea of toll bridges lay dormant for many years. The remarkable increase in the radius of our activities within the past few years, due to the increased use of motorized traffic, has created a demand for roads and bridges which the state has not been able to meet. The result has been the revival of the toll bridges and it would seem that America was now in another toll bridge era.

The Saturday Evening Post gave editorial approval of this idea in December of 1927. This statement is reproduced:

"There is a persistence today in overcoming distance and conquering mere physical obstacles that is a new expression of man's struggle with Nature. High standards of life, the all-pervading automobile, and beyond that the urge for commerce among all sections of the country--these have ushered in an era of bridge building that should command attention. Great natural obstacles, such as the larger rivers, bays and estuaries, are being overcome on a scale that has largely escaped public notice.

"The completion last year, 1926, of the bridge between Philadelphia and Camden helps to open the empty spaces of southern New Jersey to the teeming population of the country's third largest city. There is no need of moralizing on the International Peace bridge, which was opened last summer at Buffalo. The satisfaction which comes from any closer linking of Canada and the United States is obvious.

"Less is known generally of the Carquinez Strait span or that bearing the name Chowan in North Carolina. The former crosses the upper portion of San Francisco Bay waters and greatly shortens the trip from Sacramento and inland regions to such East Bay cities as Oakland and Berkeley, and to San Francisco. The various portions of San Francisco Bay extend far inland, north, east, and south, the only direct land approach to the city by the Golden Gate being along a narrow peninsula. It has been a city with a bottle neck, but the grip of what is to this extent an adverse geography is now partly broken.

"The Chowan Bridge, which has been opened within the year, restores to North Carolina its six lost counties. Looking at a map of the state, one sees in the upper right-hand corner a group of counties cut off from the rest of the state by Albemarle Sound and the Chowan River on the south and west, with the Atlantic Ocean on the east, and the Virginia line on the north.

"A few months ago ground was broken for the great Hudson River Bridge, which will cross, it is hoped in five years' time, from Fort Washington to Fort Lee. There is also now the Bear Mountain Bridge in the mountainous section up the river; another is under way, and in time it may be necessary to link lower Manhattan with the Jersey shore.

"Manhattan Island is far more closely riveted by bridges and tunnels to that portion of New York State which lies to its east than with equally distant sections of New Jersey to the west. More bridges across the Hudson will help to restore the balance of this somewhat lopsided development. The metropolis cannot continue to grow unless the rivers and channels which surround its heart are spanned in many places. Circulation is its imperative necessity.

"Many of the newer bridges are owned and operated by private corporations, but with a limited or terminable franchise. In this way the immediate users pay for the construction, and at the end of, say forty years, the state or contiguous counties become the owners of the property. In the meantime tolls pay off the capital which has been invested and a sufficient income upon the same.

"We see no reason why the building of great bridges should not go on, for there are many more physical obstacles to be overcome. New York needs several more spans, and San Francisco is not satisfied as yet. Private capital is willing to do much of the work, and can be induced to accept many a commission on a limited rather than a perpetual franchise. In this way the state secures a bridge in course of time without a penny's worth of cost to the taxpayers. The taxpayers seem to have plenty of other uses for their money, and as long as private capital can and will provide facilities, it should be encouraged to do so."

The Hudson River Bridge will represent an investment of some sixty millions of dollars, but a very careful economic study gives assurance that this tremendous investment in a toll bridge will be justified. The analysis of the Mount Hope Bridge indicated that the initial investment would be about $6,000,000 and that the immediate volume of traffic would be about one million vehicles per year. This volume of traffic is expected to gradually increase. An average toll of 50 cents per vehicle would retire the bonds in nine years besides paying interest and operating costs (operation and maintenance estimated at $25,000 to $30,000 per year). A 30-cent rate would retire the bonds in 14 years. A rate of 50 cents for the first five years and a 25-cent rate thereafter would retire the bonds in 12 years.

There is, of course, considerable counter-action in the instance of toll-bridge projects, and it is often declared that such projects are but schemes of the powerful steel corporations to insure a market for their wares. It is true that there is opportunity for mismanagement in this as in most other projects in which the taxpayer may ultimately become the proverbial goat, but as long as toll projects are inaugurated upon sound economic demand and with effective governmental supervision and franchise, there is no valid argument apparent to the writer against toll bridges. A demand is met and those who create the demand pay the bill.

A series of simple spans wherein a careful balance has been made between the cost of piers and spans, represents a safe, rigid, and economically satisfactory crossing. The item of finding the project of minimum cost by adjusting longer spans and fewer piers against shorter spans and more piers must be taken into consideration. Series of simple trusses have been erected upon the cantilever principle but as previously mentioned, this method is not economical and is used only in special cases.

However, it is often necessary or desirable to cross openings which are so deep that the cost of false work and piers would become prohibitive, or a channel may be used for nagivation and must therefore be kept open. In such cases the cantilever or suspension type must be considered. Again, the nature of soil and depth of overburden to solid bedrock must be taken into account and these conditions will influence the choice of pier locations. Pneumatic caissons are frequently used, but it is well to investigate the added cost of such work and balance it against the cost of greater span length and more economical pier locations. There are tremendous reactions required at the points of support of the larger structures and this question of pier location must be considered not only from an economic standpoint but from that of permanence and stability as well. These points are self-evident.

In the cantilever structure there may be a tendency for the anchor arm to lift from its support, in which case it must be anchored. This will necessitate the carrying of an anchor line to bedrock or the erection of an anchorage of such inherent weight as to counteract the tendency of the anchor arm to lift. Under different conditions of loading this reaction may be reversed and the anchorage must then sustain compressive forces.

The suspension bridge requires peculiar pier and anchorage conditions. The horizontal pull of the cables reaches enormous proportions and these members must be securely anchored. The anchorage of this type of bridge is a critical point and many sites which would otherwise be favorable to this type of structure are made unfavorable by the inadequacy of anchorage facilities. Anchorages may be built which have weight enough in themselves to be rigid but a bedrock connection is to be preferred. In the case of the Delaware River Bridge monumental beauty was considered worth while and the anchorages were therefore made heavy enough to serve the purpose and at the same time were designed along pleasing architectual lines.

The custom of erecting memorials in commemoration of a citizen's, a community's, or a nation's participation in a great act often finds expression in the erection of a bridge. This idea seems sensible because the monument is achieved and the people who remain have the use of the monument in their daily endeavors. In such cases limitations in selection of type may be imposed upon the designer in order that the sentiment be expressed in the finished structure and that the structure will also harmonize with its surroundings. There have been cases where an Art Committee had a great deal to say about appearances of proposed structures and when too powerful such committees may become a serious thorn in the side of the designer. An instance of this is found in the city of Pittsburgh--the art committee decided that a suspension bridge would best harmonize with the surroundings of a certain site but there was no reasonable possibility of anchorage for such a structure. The designers therefore devised a self-anchored suspension bridge and erected it upon the cantilever principle. It is very doubtful if this plan was economical, but the additional cost was deemed justified in the interests of art.

Right-of-way is often a serious problem and often the designer must provide for clearance over certain prior claims. The demands of the War Department that bridges over a navigable stream must have certain definite minimum clearances accounts for the otherwise unexplainable and unnecessary height of some of our structures above water level.

The habit of locating railways along river banks necessitates suitable provisions to keep inviolate the interests of the railway. The beauty of a suspension bridge, or of a cantilever for that matter, is greatly enhanced by perfectly symmetrical lines, the attainment of which is often a difficult task, due to desirable anchorage or pier sites being on property which cannot economically be condemned.

Strategic location in the interests of public service and the nature and volume of expected traffic is another item to be considered. Traffic census is often taken over a period of years in order to decide upon the economic advisability of a proposed structure. In case of proposed crossings now served by ferries, a careful census of ferry traffic combined with real estate or commercial developments will be considered. When the ferries are privately owned and operated and the proposed bridge is to be free, the ferry company may sensibly be damaged and entitled to an equitable settlement. If the bridge is to be a toll project, then the possible return upon investment and the possible competition with the ferry is certainly an item.

Bridge economics could well become a field of fruitful research in itself. The proposal of a bridge, and in particular a major structure, must be considered from a great variety of conditions, and the art of coordinating the public needs, the available capital and materials of construction, the natural and imposed features, and the aesthetic and monumental tastes, is a problem to try the ingenuity of any designer or board of engineers.

ECONOMIC LIMITATIONS OF SPANS OF BRIDGES OF PARTICULAR TYPES

In proposing the definition of a long span, mention was made of the possibility of building simple trusses for spans as great as 750 feet, but it was pointed out that such an adaptation of the simple truss was not economical. It is also found that there are limits beyond which it is not economical to construct bridges of the types applicable to long-span construction. The physical limitations would permit spans of much greater length than does the economic limit. Stienman states, "The maximum span possible to erect may be defined as the length at which the ratio of intrinsic weight to applied weight becomes infinite."

In investigating this problem it becomes evident that there are apparently certain ranges of span length through which certain types of construction will represent the project of least cost. Also there will appear a particular span length at which there will be no economic choice between two particular types of structure, and this span may be termed as the span of equal cost for the two types concerned.

Due to the fact that the continuous type and arched type of bridges have received so little attention and application in the field of long-span construction, there is very little reliable data concerning the economic limitation of these two types. From such meager information as has become available due to actual erection of structures of these types it appears that 1,000 feet is about the limit of their usefulness. A hingeless arch has been recently completed in Australia, the span of which is 1,640 feet.

Dr. Steinman studied this problem and after exhaustive investigation concluded that the span of equal cost for cantilever and suspension bridges was 1,670 feet. Beyond this span of 1,670 feet the suspension was best adapted and offered economic advantages. Several years later Dr. Waddell took up the problem and after minute examinations, which included a review of Dr. Stienman's work, decided that the span of equal cost was 2,190 feet. This surprising disagreement of 520 feet rather forcefully emphasizes the fact that the problem is difficult to deal with and that specialists, even, will not always agree upon the necessary assumptions. The fact must also be borne in mind that due to changes and improvements in materials and methods, the span of equal cost for one period may not be so for a subsequent period.

The Portsmouth Bridge across the Ohio was completed in 1927 and has a clear span of only 700 feet. This structure was designed for modern highway traffic, having three traffic lanes and two sidewalks. The project was opened to competitive bids and three proposals to erect a cantilever structure were received. A single proposal to erect a suspension structure was received and this latter bid was 10 per cent lower than the lowest cantilever proposal.

It thus appears that the most satisfactory manner in which to compare the costs of various types for any particular application is to make tentative designs for each. This is certainly an item of expense but until it is possible to attain a closer agreement, by mathematical predetermination, than now exists, it seems advisable to make comparative designs for any particular case. The fixing of spans of equal cost should receive more attention.

LOADS AND LOAD SYSTEMS USED IN DESIGN

With a few adjustments and certain minor assumptions it is possible to determine very reliably the stresses which occur in the members of a structure due to the intrinsic weight of the structure. Other dead or static loads may be dealt with in equal certainty. On the other hand, due to the, oftentime, extreme variation in nature, magnitude, and point of application of the live or moving loads, the stresses due to such loadings cannot be dealt with in such certainty. Even if such moving loads were entirely regular there would still remain the stresses caused by their kinetic nature and which are largely indeterminate. These conditions have led to the general adoption of certain conventional load systems which are expected to represent, within reasonable limits of error, the live loads which will actually be brought upon the bridge. Cooper's E-50 loading was the first attempt at the standardization of railroad loadings and is still used to a great extent, although certain railway companies prefer to use the system in some modified form which is supposed to more nearly represent the loadings as represented by their own rolling stock. Within the past few years Steinman has published a system of conventional loadings which are somewhat more simple in application than the Copper system and which is said to give satisfactory representation of actual loads.

If the maximum moment at any point in a span due to a particular system of concentrated loads be known it is possible to compute the load uniformly distributed over the entire span which would produce the same moment at this same point. Now if this be done for as many as, say, the 1/10 span points and for a range of spans including all those ordinarily dealt with it is possible to plot from this information a system of curves showing the relation or variation of equivalent uniform load with span length and at particular points within the span. This system of equivalent uniform loads, although not widely used, has much to recommend it. The limitation of the equivalent uniform load is the application to short spans where the application is in question, but in long spans the results have the virtue of simplicity and yields results well within reasonable limits of error.

For highway loadings the 15 or 20-ton truck has come to be the conventional manner of designating concentrated loads. A single truck, or two trucks in tandem, is assumed to be preceded and followed by a uniformly distributed load. Since many of our modern bridges have three or more traffic lanes, and often sidewalks and electric car loadings, various combinations must be used in order to satisfactorily represent the actual loaded conditions.

A few loadings as applied in the design of the typical structures of the present time will be considered. The American Association of State Highway Officials has drawn up a system of loading the application of which is exemplified in the Crooked River steel arch in Oregon. This bridge has two traffic lanes and it was assumed that a concentration of 21,000 pounds was free to act at any point in the span and in each lane. Further, this concentrated load was superimposed upon a uniformly distributed load of 450 pounds per lineal foot and of such length as to cause maximum stress in any particular member. This is the class A loading as specified by the association already mentioned. The stresses occasioned by this static load was increased by an impact coefficient derived from the formula gr

$$I = \frac{36}{w + 18} \times \frac{L + 250}{10L + 500}$$

w = width of roadway

L = loaded length of span to produce maximum stress.

The loads contemplated in the design of the Carquinez cantilever are as follows: A conventional 20-ton truck was specified to occupy an area 10 feet wide and 32 feet long with wheel concentrations of 6,000 pounds on each front and 14,000 pounds on each rear wheel. The wheels are assumed to be spaced 6 feet apart on the front axle and 12 feet apart on the rear axle. X? The number of trucks per traffic lane was limited to three, to be followed and preceded by a uniformly distributed load of 600 pounds per lineal foot. The electric car loadings consist of a train of six 96,000-pound cars of standard gauge, each car 60 feet long with wheel concentrations of 12,000 pounds each. The axles are assumed to be spaced 6.5 feet in the trucks and the trucks spaced 33.5 feet center to center. In computations the car loading was considered to displace an equal length of one line of motor trucks and uniform load. The sidewalk loading consists of a uniformly distributed load of 50 pounds per square foot.

For the design of the floor system and hangers a train of 96,000 pound cars each 60 feet long, as above stated, was used, while for the trusses and piers a train load of 1,600 pounds per lineal foot and 500 feet in length was specified. The train weight upon the bridge is limited to a total of 800,000 pounds. The highway loading for the floor system and hangers consists of 20-ton trucks in three lanes without the railway loads or two lines of trucks with the railway loads.

The Beaver cantilever is a good example of railway loadings. The floor system of this bridge was proportioned for the carrying of two trains, one on each track, each weighing 6,000 pounds per lineal foot, and each preceded by two locomotives weighing 426,000 pounds. This is Cooper's E-60 loading. Two trains were assumed on each track wherever split loading produced greater stresses. The trusses were designed for loads 10 per cent less than these. The dead load averaged about 9 tons per lineal foot, of which .5-ton was for timber deck and rails. The dead load stresses were computed from weights figured from the stress sheets. This method was repeated until the sections, computed on the basis of actual dead loads, figured from shop drawings, came within 2 per cent of the sections used.

The wind loads were proportioned by the areas of exposed surfaces as finally developed, and were equivalent to about 1,100 pounds per lineal foot for the lower chord for anchor and cantilever arms and about 1,000 pounds per lineal foot for the suspended span. The equivalent uniform load for the top chord was about 600 pounds per lineal foot. These figures are derived from the assumption of a wind load of 300 pounds per lineal foot on the train and 30 pounds per square foot of exposed surfaces of the two trusses.

In the design of the Portsmouth suspension of 700-foot span over the Ohio River a 20-ton truck was assumed to act at any point, or 3, 15-ton trucks abreast. This loading for the floor system and suspenders. For the stiffening trusses, cables, and towers a uniformly distributed load of 1,400 pounds per lineal foot plus a superimposed concentration of 42,000 pounds at any point. The live load stresses in the stringers were increased 37.5 per cent for impact, while 30 per cent was added to the floor beams.

THE STRESS ANALYSIS OF A 1,600-FOOT CANTILEVER BRIDGE

In order to the better illustrate certain points within the scope of this article the writer has elected to include herein an independent analysis of a long-span bridge of the cantilever type. This part of the problem is entirely hypothetical in the instance of span length and contemplated loads. The principles of design are expected to be consistent with modern practice.

It is assumed that an opening of 1,600 feet is to be spanned and that physical conditions are such that the piers may be so located as to give a clear main span of 1,000 feet, leaving symmetrical anchor or side spans of 300 feet each. The loadings will be the same as those contemplated in the design of the Carquinez cantilever. In addition to the loads specified the live load stresses in the stringers will be increased 37.5 per cent as an impact factor. Similar stresses in the floor beams will be increased 30 per cent to allow for impact.

Before contemplating the configuration of the proposed structure an examination will be made of the general proportions of a number of representative structures of this same type. This examination is made in the following tabulation:

due.

STRUCTURE	Main Span	Suspended Span	Cantilever Arm	Anchor Arm	Sus. Span / Main Span	Tower / Main Span	Width / Main Span
Sewickley	750	350	200	300	.467	.153	
Beaver	769	285	242	320	.371	.181	.045
Wabash	700	307		298.	.438	.156	
Quebeck	1800	640	580	515	.355	.170	
Carquinez	1100	433	333.5	500	.394	.153	.0382
Firth of Forth	1700	350	680	685.	.206	.197	
Monongahela	812	360	226	346	.444	.160	.0395
Cincinnati-Newport	520	208	156	252	.400	.144	

A study of the above tabulation indicates somewhat of an agreement concerning the proportions as applied to this type of structure. The ratio of suspended span to main span ranges in the vicinity of .4 and this value is close to Waddell's recommendation of 3/8 or .375. The suspended span being a simple truss, it is desirable to keep its length within economic limits. Against this consideration, the matter of deflection in the cantilever arms must be taken into account. The deflection at the end of the cantilever arm will be due to the deformation of both the cantilever and anchor arms. From this consideration it is desirable to keep these sections comparatively short. For aesthetic reasons the anchor arms should be as nearly symmetrical with the cantilever arms as possible, although this ideal may not always be possible on account of pier locations and right-of-way considerations. The tower height may be conveniently determined from a ratio of tower height to main span. From the tabulation this ratio is seen to range in the vicinity of .16.

In long spans the wind loads become a very considerable factor and the structure must be wider than would ordinarily be required in order to provide economical and sufficient lateral bracing. Representative structures indicate that the ratio of width to main span should range in the vicinity of .04 and this is close to the specification that the width shall not be less than 1/20 of the main span. .05

In view of the foregoing considerations it is decided that the structure shall have the dimensions as listed:

	feet
Main span...............	1,000
Cantilever arm..........	300
Anchor arm..............	300
Suspended span..........	400
Height of towers........	155
Separation of trusses...	45

This selection gives the following ratios:

$$\frac{\text{suspended span}}{\text{main span}} = .40$$

$$\frac{\text{tower}}{\text{main span}} = .155$$

$$\frac{\text{width}}{\text{main span}} = .045$$

The width of 45 feet will permit 3, 10-foot traffic lanes and 2, 7.5-foot sidewalks, less allowances for width of truss members. Provision is made in the central traffic lane for street railway tracks.

The configuration, showing only essential members, is given on Plate I., while the complete diagram is shown on Plate IV.

STRESS ANALYSIS

To facilitate the determination of stresses due to imposed loads and to the weight of the structure itself, influence lines have been computed and drawn, and are shown on plates I., II., and III. These diagrams represent the direct stress in the various members due to a unit load as it moves from end to end of the structure. As a check upon these lines or diagrams a graphical solution is made upon the assumption of a uniformly distributed load of one pound per lineal foot of truss, which load is assumed to produce concentrations at the lower panel points.

The concrete floor is to rest upon steel stringers which are in turn supported by built-up plate girder floor-beams. The stringers are to be attached to the floor-beams in such a manner as to give the finished roadway a parabolic crown of 6 inches. The forms to support and mold the concrete floor will be so placed as to make the lower surface of the slab one inch below the top of the stringers--the stringers will then act as chairs for the reinforcing steel which is to be of the fabricated type. This arrangement insures very definite position of reinforcing steel within the slab. The concrete floor is to be 7 inches thick, exclusive of wearing surface, reinforced on the lower side with 1-inch square steel bars spaced 4 inches center to center, and on the upper side with 1/2-inch round steel bars spaced 6 inches center to center.

In the suspended span the intermediate stringers are selected as standard 20-inch, 65.4-pound I-beams, while the two central or inner stringers which carry street railway loads are selected as 20-inch, 100-pound I-beams.

The sidewalks are to be concrete 3 inches thick, reinforced with 3/8-inch round steel bars on 6-inch centers, placed 3/4-inch from the lower surface. The sidewalk stringers will be 10-inch, 25.4-pound I-beams.

In the design of floor beams it is assumed that 1/8 of the web area is available to resist flange stresses due to bending moment. In the suspended span the floor beams are selected as:

web plate	1-5/16 in. x 54 in. plate
angles	4-5x5x1/2
cover plates	6-11x1/2 in.
	2-11x5/8 in.

$\frac{45}{pcs}$ = too small

In the cantilever section and in that part of the anchor section having 30-foot panel lengths the following members are selected--the concrete floor and sidewalk being the same throughout the entire structure:

roadway stringers	20 in. - 85 lb. I-beams
	24 in. -105.4 do
sidewalk stringers	10 in. - 30 do
floor beams	1 - 54 x 5/16 web plate
	4 - 5 x 5 x 1/2 angles
	6 - 11 x 5/8 cover plates
	2 - 11 x 3/4 cover plates.

For the 22.5-foot panels, are selected:

roadway stringers	20 in. - 65.4 lb. I-beams
	24 in. - 79.9 do
sidewalk stringers	8 in. - 18.4 do
floor beams	1 - 54 x 5/16 web plate
	4 - 5 x 5 x 1/2 angles
	4 - 11 x 5/8 cover plates
	2 - 11 x 1/2 cover plates.

There are still two cases to be considered in the selection of floor beams, which are at the following points: the floor beam carrying the reactions from the 22.5-foot panel on one side and the 30-foot panel on the other. Also is to be considered the floor beam carrying the reactions from the 25-foot panel on one side and the 30-foot panel on the other. After consideration of these cases it is decided to use the same girder in both instances, and which has the following composition:

1 - 54 x 5/16 web plate

4 - 5 x 5 x 1/2 angles

8 - 11 x 5/8 cover plates.

as/pls = too small

In all cases the cover plates on the plate girders are to be cut off according to the formula:

$$L_n = L\sqrt{\frac{a_1 + a_2 + a_3 + \cdots a_n}{A}}, \text{ where}$$

a_n = area of any element of flange group

A = total gross area of flange (not including 1/8 of the web)

L = length of girder span

L_n = length of cover plate required.

The panel concentrations due to the dead weight of the floor system, not including the lower lateral system, are found to be:

spans	lbs.
22.5-foot.....	47,600
25-foot.......	54,800
30-foot.......	66,900
22.5-30-foot..	57,250
25-30-foot....	60,850

THE RELATION OF INFLUENCE DIAGRAMS OF STRESS TO THE DISTRIBUTION OF METAL IN A TRUSS

In short spans and simple structures there is no great variation in the distribution of metal throughout the truss and certain empirical formulae have been developed which give, somewhat reliably, the total weight of metal in terms of span and type of loading. After this total weight of metal is thus derived it is considered as a load uniformly distributed over the length of the span and the panel concentrations due to dead weight of structure found accordingly. For minor structures this method is apparently satisfactory, although in certain cases this initial assumption of total weight and distribution of weight may be investigated after tentative selection of sections and proper readjustments made.

Coming now to the case of the cantilever structure it is at once apparent that the assumption of uniform distribution of metal would be very materially in error due to the greater concentration of metal in the region of the towers. There are data available concerning the total weight of metal in various bridges of this type but to the writer's knowledge there is no published method of making a rational tentative distribution of metal; in fact, as was noted in the review of the Beaver cantilever, it was seen that the stresses due to dead load of structure were found by successive adjustments of sections and weights. In considering this situation the thought came to the writer's mind that if the total weight of metal in the structure could be adequately predetermined from comparison with other structures or from any source whatever, it should be possible to obtain a suitable panel distribution factor from a consideration of influence diagram areas and lengths of members.

It is at once apparent that the stress in any particular member is a function of the influence diagram area, that the stress is a measure of the section required, which is a measure of the weight of member per unit of length of member; hence, for any particular joint in the structure the summation of the products of "influence areas" by lengths of members will be a measure of the panel concentration.

Let A = effective influence area for any member

L = length of the same member; then,

A x L = the "member factor."

For any particular joint the panel or "joint factor" would be the summation of the "member factors," which contribute to the joint in question, and for any joint, j, let the joint factor be represented as:

$$\sum A_j L_j = \text{"joint factor."}$$

The summation of member factors over the entire structure is a measure of the total weight of truss metal, and let this be represented as:

$$\sum A \times L = \text{truss factor.}$$

It is now apparent that the "panel concentration factor" is the ratio of "joint factor" to "truss factor," or in the symbolism:

$$\frac{\sum A_j L_j}{\sum A \times L} = \text{panel concentration factor.}$$

Now a tabulation may be made of "member factors" from which the joint and truss summations can be made, and finally the "panel concentration factor or ratio" derived.

There still remains an influencing factor to be considered; the required sectional area for a tensile member carrying a certain stress will be materially less than that of a compression member carrying the same stress. It is, therefore, necessary to correct the "member factor" by some quantity, k, which will include this difference in metal required. In the final form the panel concentration factor appears as:

$$\frac{\sum k \times A_j \times L_j}{\sum k \times A \times L} = R = \text{the panel concentration factor.}$$

It is necessary to evaluate the quantity, k, and it is to be seen that only an approximation of this evaluation can be made, but it is believed that this approximation may be adequately expressed from the following consideration.

The allowable unit stress in a compression member is expressed in the conventional formula

$$S = 16{,}000 - 70\ L/r$$

with an upper limit of 12,500 pounds per square inch. The ratio

L/r is specified as 120 for main truss members and 160 for secondary members, and since most of the members in a truss of the proportions under consideration are relatively long, it is expected that the upper limit of 12,500 pounds per square inch will not be often realized. It is decided after these considerations that in the final analysis the average stress in compression members will be about 11,000 pounds per square inch, so the correction factor, k, may be evaluated as:

$$k = 16{,}000/11{,}000 = 1.45,$$

so in the following consideration the tension "member factor" is expressed as A x L, and the compression "member factor" is to be taken as 1.45 x A x L. The following tabulation is made from this consideration.

Joint	Member	Length	Inf-Area	K A L	$\frac{K A_i L_i}{E}$	R	Joint Load
0	L_0-U_1	34.478 x .5	462	8 020			
	L_0-L_1	22.5 x .5	301.5 K	4 920	12 940	.00907	22 300
1	L_0-U_1	34.478 x .5	462	8 020			
	L_0-L_1	22.5 x .5	301.5 K	4 920			
	U_1-L_1	26.25 x 1.	22.5	590			
	U_1-U_2	34.478 x .5	462	7 950			
	U_1-L_2	34.478 x .5	13.7 K	342			
	L_1-L_2	22.5 x .5	301.5 K	4 920	26 742	.0187	46 000
2	U_1-U_2	34.478 x .5	462	7 950			
	U_2-U_4	45.58 x .25	536	6 110			
	U_2-L_4	68.957 x .25	345.5 K	8 625			
	U_2-L_2	52.25 x 1.	33.8	1 765			
	U_1-L_2	34.478 x .5	13.7 K	342			
	L_1-L_2	22.5 x .5	301.5 K	4 920			
	L_2-L_3	22.5 x .5	301.5 K	4 920	34 632	.0243	59 800
3	U_2-U_4	45.58 x .5	536	12 200			
	U_2-L_4	68.957 x .5	345.5 K	17 250			
	L_3-3	26.125 x 1.	22.5	587			
	U_4-3	40.25 x .5	15.2	306			
	L_2-L_3	22.5 x .5	301.5 K	4 920			
	L_3-L_4	22.5 x .5	301.5 K	4 920	40 183	.0282	69 400
4	U_2-U_4	45.58 x .25	536	6 110			
	U_4-U_6	46.59 x .25	548	6 380			
	U_4-3	40.25 x .5	15.2	306			
	U_4-L_4	59.50 x 1	27.1 K	2 340			
	U_2-L_4	68.957 x .25	345.5 K	8 625			
	L_3-L_4	22.5 x .5	301.5 K	4 920			
	L_4-L_5	22.5 x .5	663 K	10 800			
	U_6-L_4	84.27 x .25	250	5 270	44 751	.0314	77 200
5	U_4-U_6	46.59 x .5	548	12 800 ~~6 380~~			
	U_6-L_4	84.27 x .5	250	10 500 ~~9 180~~			
	L_5-5	35.625 x 1.	22.5	800 ~~5 260~~			
	L_4-L_5	22.5 x .5	663 K	10 800 ~~4 940~~			
	L_5-L_6	22.5 x .5	663 K	10 800 ~~2 410~~			
	L_6-5	37.3 x .5	13.3 K	360	46 060	.0323	79 500
6	U_4-U_6	46.59 x .25	548	6 380			
	U_6-U_8	48.016 x .25	765	9 180			
	U_6-L_4	84.270 x .25	250	5 260			
	U_6-L_8	84.270 x .25	161.7 K	4 940	25 760		

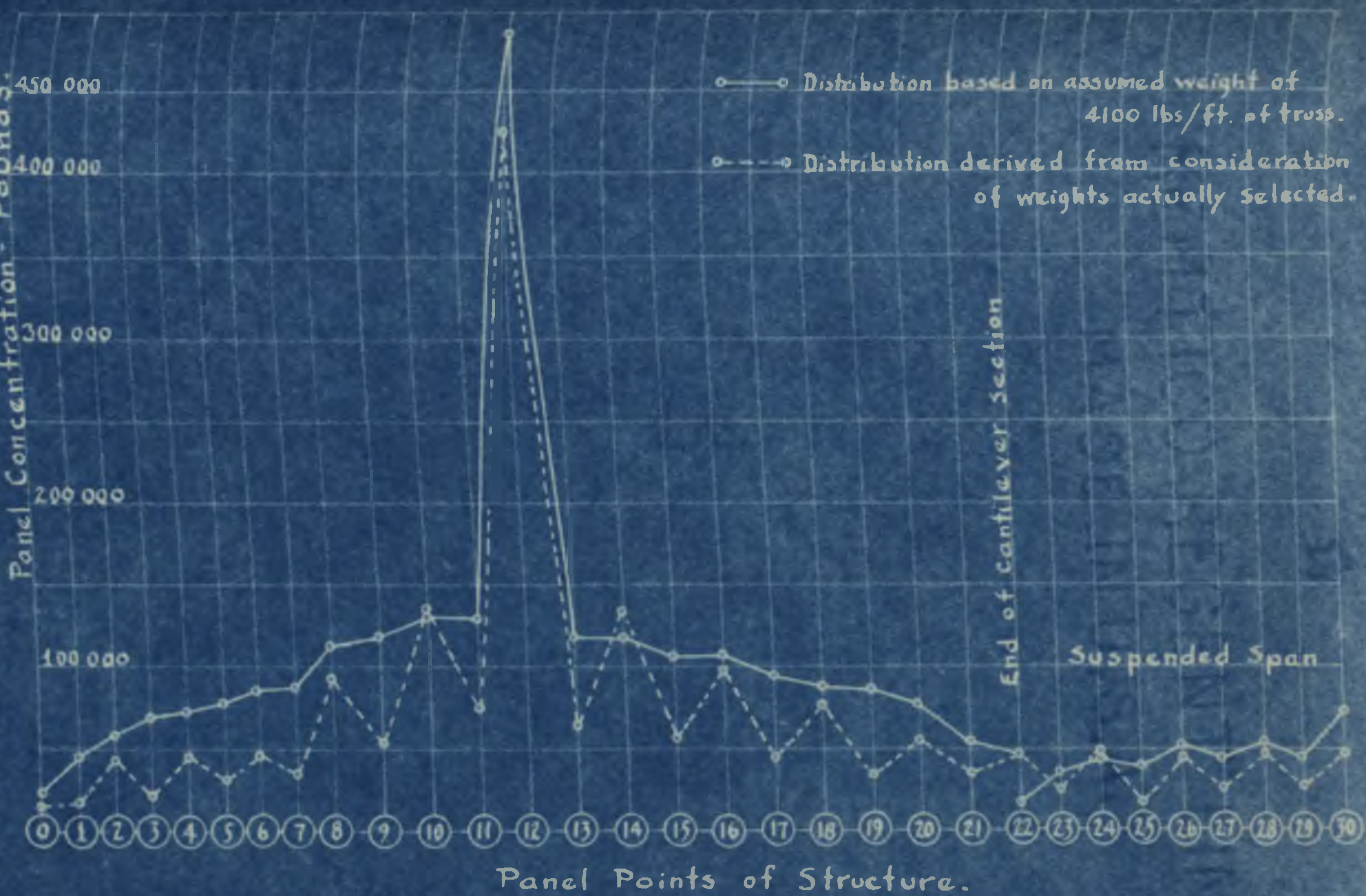
Joint	Member	Length	Infl-Area	$K A L$	$K A_j L_j$	R	Joint Load
6 (cont)	U_6-L_6	71.25 x 1	33.8	2410			
	L_6-5	37.30 x .5	13.3 K	360			
	L_5-L_6	22.5 x .5	663 K	10 800			
	L_6-L_7	22.5 x .5	663 K	10 800	50 130	.0352	86 600
7	U_6-U_8	48.016 x .5	765	18 350			
	U_6-L_8	84.270 x .5	161.7 K	9 880			
	U_8-7	57.003 x .5	14.6	416			
	L_7-7	35.625 x 1.	22.5	802			
	L_6-L_7	22.5 x .5	663 K	10 800			
	L_7-L_8	22.5 x .5	663 K	10 800	51 048	.0358	88 000
8	U_6-U_8	48.016 x .25	765	9 200			
	U_8-U_{10}	66.649 x .25	796	13 300			
	U_8-7	57.003 x .5	14.6	416			
	U_8-L_8	88.00 x 1.	79	6 950			
	U_6-L_8	84.27 x .25	161.7 K	4 930			
	$U_{10}-L_8$	131.491 x .25	138.8	4 560			
	L_7-L_8	22.5 x .5	663 K	10 800			
	L_8-L_9	30.0 x .5	717 K	15 600	65 756	.0461	113 200
9	U_8-U_{10}	66.649 x .5	796	26 500			
	$U_{10}-L_8$	131.491 x .5	138.8	9 120			
	L_8-L_9	30.0 x .5	717 K	15 400			
	L_9-L_{10}	30.0 x .5	717 K	15 400			
	L_9-9	58.5 x 1.	30	1 750			
	$L_{10}-9$	65.743 x .5	16.8 K	800	68 970	.0483	119 000
10	U_8-U_{10}	66.649 x .25	796	13 250			
	$U_{10}-U_{12}$	71.021 x .25	802	14 250			
	$U_{10}-L_8$	131.491 x .25	138.8	4 560			
	$U_{10}-L_{12}$	131.491 x .25	135 K	6 430			
	$U_{10}-L_{10}$	117.0 x 1.	45	5 270			
	$U_{10}-12$	60.0 x .25	7.7 K	167			
	$L_{10}-9$	65.743 x .5	16.8 K	800			
	L_9-L_{10}	30.0 x .5	717 K	15 600			
	$L_{10}-L_{11}$	30.0 x .5	717 K	15 600	75 927	.0532	130 100
11	$U_{10}-U_{12}$	71.021 x .5	802	28 500			
	$U_{10}-12$	60.0 x .5	7.7 K	335			
	$U_{10}-L_{12}$	131.491 x .5	135 K	12 900			
	$11-12$	65.743 x .5	16.8	553			
	$L_{11}-11$	58.50 x 1.	30	1 755			
	$L_{10}-L_{11}$	30.0 x .5	717 K	15 600			

Joint.	Member	Length	Infl-Area	K A L	K A, L j	R	Joint Load
11 (cont.)	$L_{11}-L_{12}$	30.0 x .5	717 K	15 600	75 243	.0527	129 800
12.	$U_{10}-U_{12}$	71.021 x .25	802	14 250			
	$U_{12}-U_{14}$	71.021 x .25	802	14 250			
	$U_{12}-L_{12}$	155.0 x 1.	912.41 K	205 000			
	11 - 12	60.0 x .5	16.8	505			
	12 - 13	60.0 x .25	16.8	252			
	$U_{10}-L_{12}$	131.491 x .25	135 K	6 430			
	$U_{14}-L_{12}$	131.491 x .25	91.5 K	4 360			
	$L_{11}-L_{12}$	30.0 x .5	717 K	15 600			
	$L_{12}-L_{13}$	30.0 x .5	656 K	14 250	~~27~~		
	U_{10} - 12	60.0 x .25	135 K	2 940			
	U_{14} - 12	60.0 x .25	7.7 K	167	283 004	.1975	485 000
13	$U_{12}-U_{14}$	71.021 x .5	802	28 500			
	U_{14} - 12	60.0 x .5	7.7 K	334			
	12 - 13	65.743 x .5	16.8	552			
	$U_{14}-L_{12}$	131.491 x .5	91.5 K	8 720			
	$L_{12}-L_{13}$	30.0 x .5	656 K	14 300			
	$L_{13}-L_{14}$	30.0 x .5	656 K	14 300			
	L_{13} - 13	58.50 x 1.	30	1 750	68 456	.0480	118 000
14	$U_{12}-U_{14}$	71.021 x .25	802	14 250			
	$U_{14}-U_{16}$	66.649 x .25	658	10 970			
	$U_{14}-L_{12}$	131.491 x .25	91.5 K	4 360			
	$U_{14}-L_{16}$	131.491 x .25	139.4	4 580			
	$U_{14}-L_{14}$	117.0 x 1.	45	5 250			
	$L_{13}-L_{14}$	30.0 x .5	656 K	14 250			
	$L_{14}-L_{15}$	30.0 x .5	648.3 K	14 100			
	L_{14} - 15	65.743 x .5	16.8 K	800	68 560	.0481	118 100
15	$U_{14}-U_{16}$	66.649 x .5	658	21 900			
	$U_{14}-L_{16}$	131.491 x .5	139.4	9 150			
	L_{14} - 15	65.743 x .5	16.8 K	800			
	$L_{14}-L_{15}$	30.0 x .5	648.3 K	14 100			
	$L_{15}-L_{16}$	30.0 x .5	648.3 K	14 100			
	L_{15} - 15	58.5 x 1.	30	1 755	61 805	.0433	106 500
16	$U_{14}-U_{16}$	66.649 x .25	658	11 000			
	U_{16} - 17	62.321 x .5	21.2	662			
	$U_{16}-L_{16}$	88.0 x 1.	73.8	6 490			
	$U_{14}-L_{16}$	131.491 x .25	139.4	4 580			
	$U_{18}-L_{16}$	89.752 x .25	186.6 K	6 080			
	$L_{15}-L_{16}$	30.0 x .5	648.3 K	14 100			

Joint.	Member	Length	Infl-Area	K A L	$K A_j L_j$	R	Joint Load
16 (cont)	$L_{16}-L_{17}$	30.0 x .5	468 K	10 200			
	$U_{16}-U_{18}$	63.652 x .25	618.1	9 820	62932	.0441	108 200
17	$U_{16}-U_{18}$	63.652 x .5	618.1	19 650			
	$U_{18}-L_{16}$	89.752 x .5	186.6 K	12 150			
	$U_{16}-17$	62.321 x .5	21.2	662			
	$L_{16}-L_{17}$	30.0 x .5	468 K	10 200			
	$L_{17}-L_{18}$	30.0 x .5	468 K	10 200			
	$L_{17}-17$	33.375 x 1.	30	1000	53 862	.0378	93 000
18	$U_{16}-U_{18}$	63.652 x .25	618.1	9840			
	$U_{18}-U_{20}$	61.188 x .25	257	3 940			
	$U_{18}-L_{16}$	89.752 x .25	186.6 K	6 070			
	$U_{18}-L_{20}$	89.752 x .25	322.5	7 250			
	$L_{17}-L_{18}$	30.0 x .5	468 K	10 200			
	$L_{18}-L_{19}$	30.0 x .5	454.5 K	9 850			
	$L_{18}-19$	44.887 x .5	20.2 K	658			
	$U_{18}-L_{18}$	66.75 x 1.	45	3 000	50 808	.0356	87 600
19	$U_{18}-U_{20}$	61.188 x .5	257	7 850			
	$U_{18}-L_{20}$	89.752 x .5	322.5 K	21 000			
	$L_{18}-19$	44.887 x .5	20.2 K	658			
	$L_{19}-19$	33.375 x 1.	30	1 000			
	$L_{18}-L_{19}$	30.0 x .5	454.5 K	9 870			
	$L_{19}-L_{20}$	30.0 x .5	454.5 K	9 870	50 248	.0352	86 500
20	$U_{18}-U_{20}$	61.188 x .25	257	3 930			
	$U_{20}-U_{22}$	60.187 x .25	236.5	3 550			
	$U_{20}-21$	42.250 x .5	23.2	492			
	$U_{20}-L_{20}$	54.750 x 1.	15.4 K	1 225			
	$U_{18}-L_{20}$	89.752 x .25	322.5	7 250			
	$U_{22}-L_{20}$	78.102 x .25	328.4 K	9 300			
	$L_{19}-L_{20}$	30.0 x .5	454.5 K	9 880			
	$L_{20}-L_{21}$	30.0 x .5	454.5 K	9 880	45 507	.0320	78 700
21	$U_{20}-U_{22}$	60.187 x .5	236.5	7 120			
	$U_{20}-21$	36.836 x .5	23.2	428			
	$U_{22}-L_{20}$	78.102 x .5	328.4 K	18 600			
	$L_{20}-L_{21}$	30.0 x .5	100 K	2 200			
	$L_{21}-L_{22}$	30.0 x .5	100 K	2 200			
	$L_{21}-21$	25.0 x 1.	30	750	31 298	.0219	53 900
	See	following	Sheet.				

Joint	Member	Length	Infl-Area	k A L	K $A_j L_j$	R	Joint Load
22.	U20-U22	60.187x.25	236.5	3 560			
	U22-U24	50.0 x.25	100	1 250			
	U22-L20	78.102 x.25	328.4K	9 300			
	U22-L22	50.0 x1.	215	10 750			
	L21-L22	30.0 x.5	100 K	2 200	27 060	.0190	46 700
	Summation for Cantilever, ΣKAL:				1427 032		
	Suspended Span						
22.	U24-L22	70.711x.25	265K	6 800			
	~~U24-~~ L21-L22	30.0 x.5	100K	2 200			
	L22-L23	25.0 x.5	187.5	2340	11 340	.0211	16 900
23	U24-L22	70.711x.5	265 K	13 550			
	U22-U24	50.0x.5	100	2500			
	L22-L23	25.0x.5	187.5	2340			
	L23-L24	25.0 x.5	187.5	2 340			
	L23-23	25.0 x1.	25	625			
	L24-23	35.355x.5	17.7 K	453	19 308	.0445	35 600
24	U22-U24	50.0x.25	100	1250			
	U24-U26	50.301x.25	282.9K	5150			
	U24-L22	70.711x.25	265 K	6 800			
	U24-L26	70.711x.25	157	2780			
	U24-L24	50.0 x1.	37.5	1875			
	L23-L24	25.0 x.5	187.5	2 340			
	L24-L25	25.0 x.5	175	2190			
	L24-23	35.355x.5	17.7K	456	22 641	.0523	41800
25	U24-U26	50.301 x.5	282.9K	10600			
	U24-L26	70.711 x.5	157	5 550			
	U26-25	35.355x.5	17.8	314			
	L24-L25	25.0 x.5	175	2 180			
	L25-L26	25.0 x.5	175	2180			
	L25-25	25.0 x1.	25	62.5	21449	.0495	39 600
26	U24-U26	50.311x.25	282.9K	5 170			
	U26-U28	50.172x.25	329 K	5 970			
	U26-25	39.437x.5	17.8	351			
	U26-L28	74.701x.25	111	2080			
	U26-L26	55.5 x1.	81.7K	6 570			
	U24-L26	70.711x.25	157	2 780			
	L25-L26	25.0 x.5	175	2 190			

Joint	Member	Length	Infl-Area	K A L	KA_jL_j	R	Joint Load
26 (cont)	$L_{26}-L_{27}$	25.0 x .5	270	3 370	28 481	.0658	52 700
27	$U_{26}-U_{28}$	50.122 x .5	329 K	11 900			
	$U_{26}-L_{28}$	74.701 x .5	111	2 080			
	$U_{28}-27$	40.019 x .5	16	320			
	$L_{26}-L_{27}$	25.0 x .5	270	3 370			
	$L_{27}-L_{28}$	25.0 x .5	270	3 370			
	$L_{27}-27$	27.75 x 1.	25	694	21 734	.0502	40 200
28	$U_{26}-U_{28}$	50.122 x .25	329 K	6 000			
	$U_{28}-U_{30}$	50.010 x .25	343 K	6 240			
	$U_{28}-L_{30}$	77.336 x .25	82.4	1 590			
	$U_{28}-27$	27.75 x .5	16.	220			
	$U_{28}-L_{28}$	59.0 x 1.	55.3 K	4 730			
	$U_{26}-L_{28}$	74.701 x .25	111	2 070			
	$L_{27}-L_{28}$	25.0 x .5	270	3 370			
	$L_{28}-L_{29}$	25.0 x .5	318	3 980	28 200	.0651	52 100
29	$U_{28}-U_{30}$	50.010 x .5	343 K	12 450			
	$U_{28}-L_{30}$	77.336 x .5	82.4	3 180			
	$U_{30}-29$	39.437 x .5	16.5	325			
	$L_{28}-L_{29}$	25.0 x .5	318	3 980			
	$L_{29}-L_{30}$	25.0 x .5	318	3 980			
	$L_{29}-29$	29.5 x 1.	25	740	24 655	.0569	45 500
30	$U_{28}-U_{30}$	50.01 x .25	343 K	12 450			
	$U_{30}-U_{32}$	50.01 x .25	343 K	12 450			
	$U_{30}-29$	39.437 x .5	16.5	325			
	$U_{30}-31$	39.437 x .5	16.5	325			
	$U_{30}-L_{30}$	60.0 x 1.	20.2 K	1 760			
	$U_{28}-L_{30}$	77.336 x .25	82.4	1 595			
	$U_{32}-L_{30}$	77.336 x .25	82.4	1 595			
	$L_{29}-L_{30}$	25.0 x .5	318	3 980			
	$L_{30}-L_{31}$	25.0 x .5	318	3 980	38 460	.0888	71 100
	Summation for Sus. Span ΣKAL				433 136		



Assumed total wt. of cantilever 2,460,000 lbs.

Actual " " " " 1,653,430 " .

A study of the tabulations and chart just presented shows a theoretical distribution of metal which is at least encouraging. The assumption of the total weight of truss as 2,460,000 pounds is apparently not good, but the general agreement between the forms of the two curves would indicate that the proposed method is worthy of further consideration. The panels are subdivided in this structure and this fact would probably account for the smaller concentrations at the sub-panel points as given by the actual recapitulation of weight. This factor could be adjusted in the preliminary investigation by assuming less weight at the sub-panel points and more weight at the panel points. By reducing the values given on the full graph line in the ratio of 1,653,430/2,460,000 or .67 per cent, a much closer agreement between the two curves will be found.

SECONDARY STRESSES

In all of the considerations to this point notice has been taken of primary stresses only. These stresses are assumed to be due to direct axial loads which do not produce bending movement with subsequent flexural stresses. It is known, however, that a framed structure is subjected to variable loading with corresponding deformation there are produced within the members bending moments and accompanying flexural stresses. These stresses occasioned by the deformation of the structure are known as secondary stresses. If all the joints of the structure were made with frictionless pins, secondary stresses would not be present, but due to pin friction in such joints and the inherent rigidity of riveted joints these secondary stresses do exist and may assume proportions as great as fifty per cent of the primary stresses.

The calculation of secondary stresses is not an easy matter--the theory is comparatively simple in derivation but decidedly lengthy and tedious in application. Fortunately, it is not necessary to consider these stresses in minor structures. In the design at hand the lengths of members are given as actual length of member when stressed under the entire dead load of the structure, so that when carrying no live load the structure will be in its normal position, which does not produce secondary stresses. The deformation will be due to live loads and temperature variation and since the ratio of live load to dead load in a structure of this sort is small, the deformation will not be great. This arrangement keeps the secondary stresses due to deformation down to a minimum, and in this analysis only the primary stresses are considered.

REVIEW AND CONCLUSIONS

Long span bridges are distinguished by type rather than actual span. A simple truss may be adapted to the same span as would ordinarily demand a long span type of structure, but economy will dictate the use of cantilever suspension, continuous, or arched types. The economic limit of the simple truss is about 600 feet.

Spans of equal cost are not clearly defined and for any particular case tentative plans should be drawn including all types applicable. For spans above 1,000 feet the suspension bridge has preference if suitable anchorage is readily available.

Available data concerning bridge economics are inadequate.

Toll bridges are apparently economically justified.

There is apparently a rational relation between the effective areas of influence diagrams of stress, lengths of members, and distribution of metal. This proposition is not completely worked out in this paper, but the writer hopes to carry it to a more satisfactory conclusion in a subsequent paper.

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use

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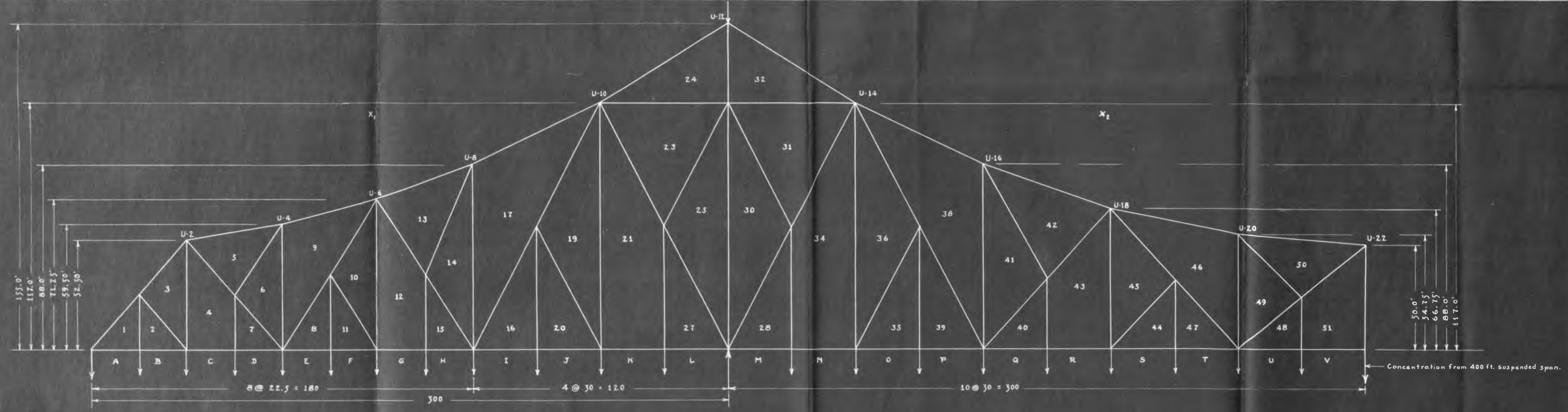
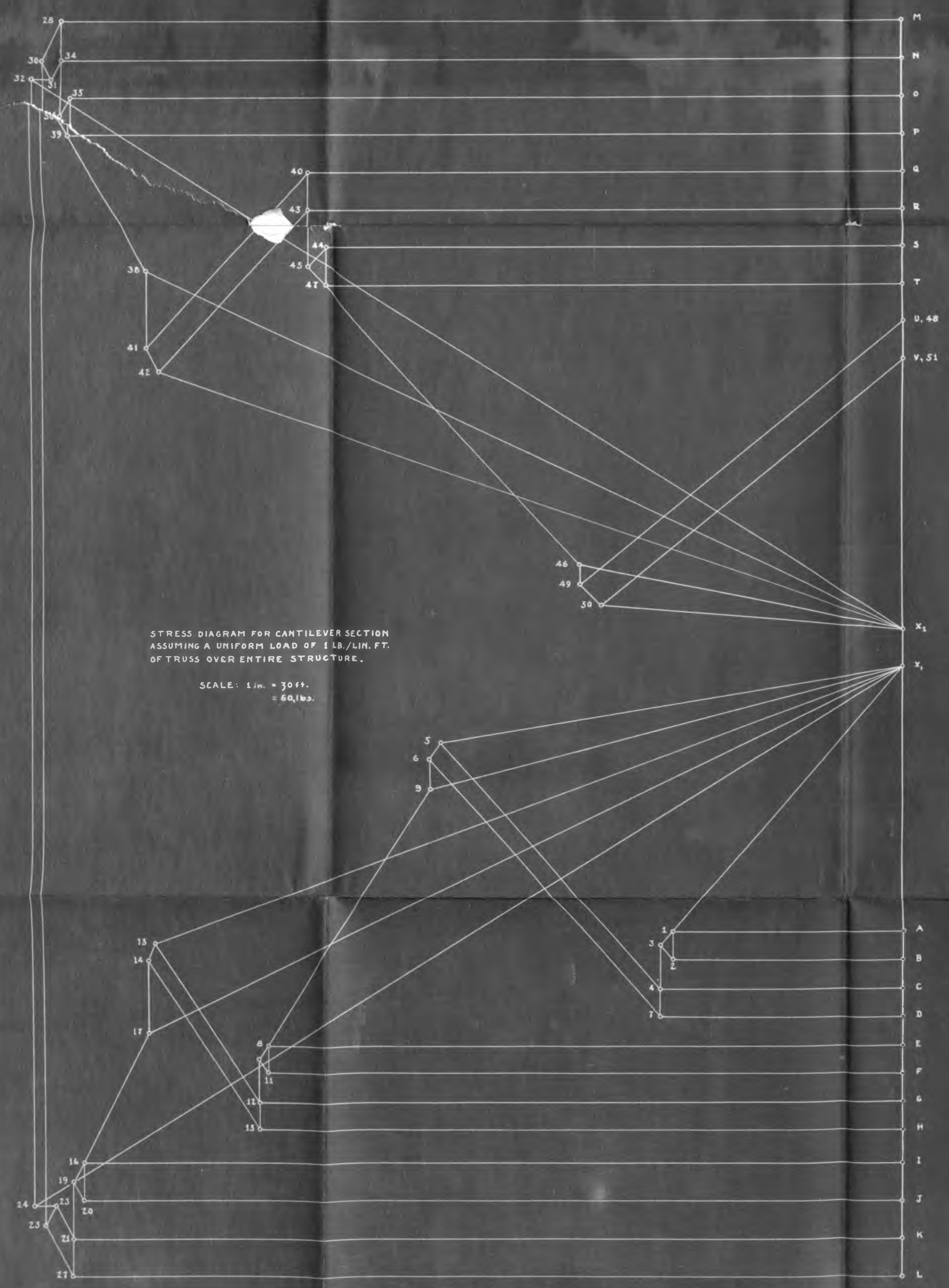
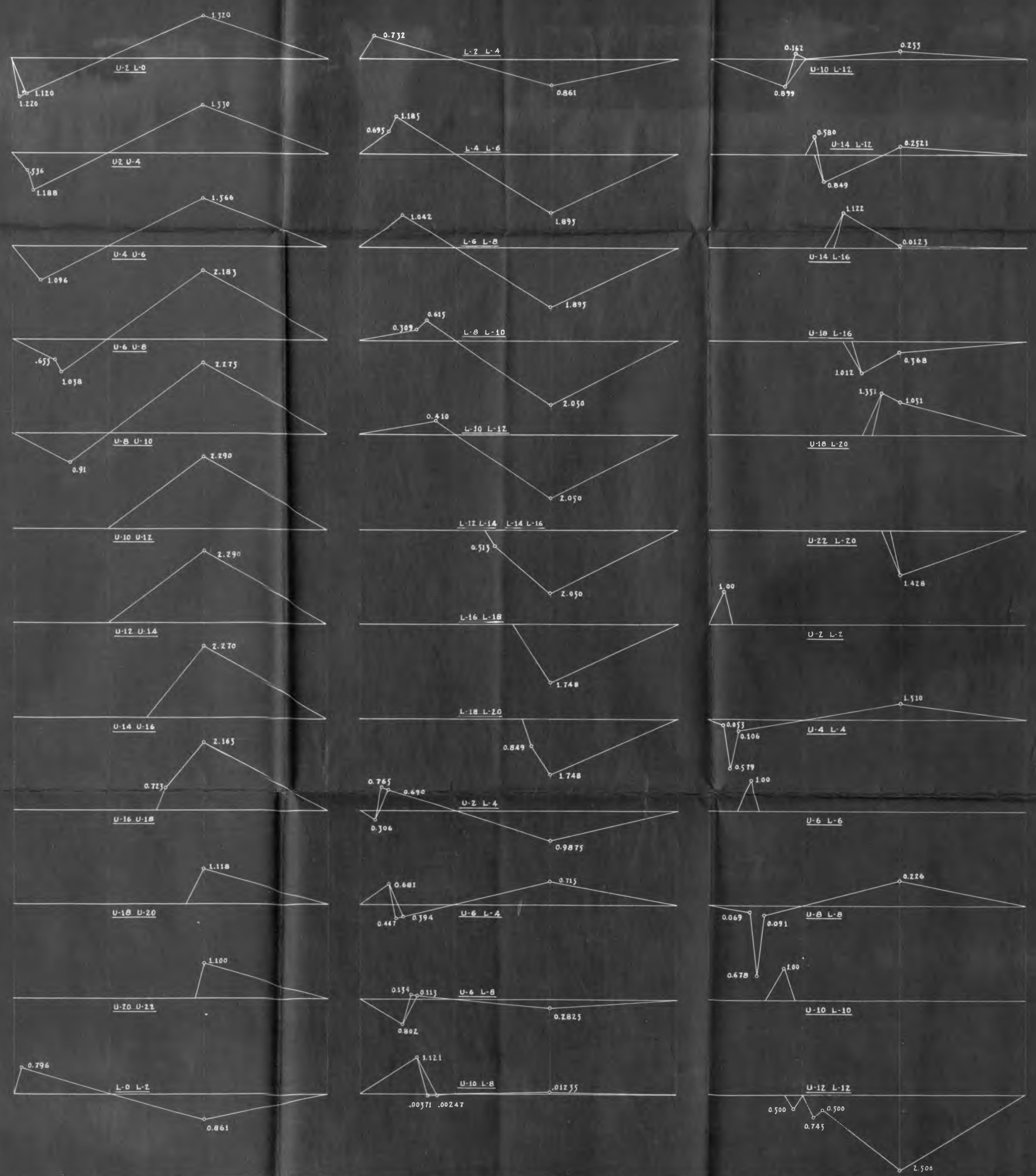


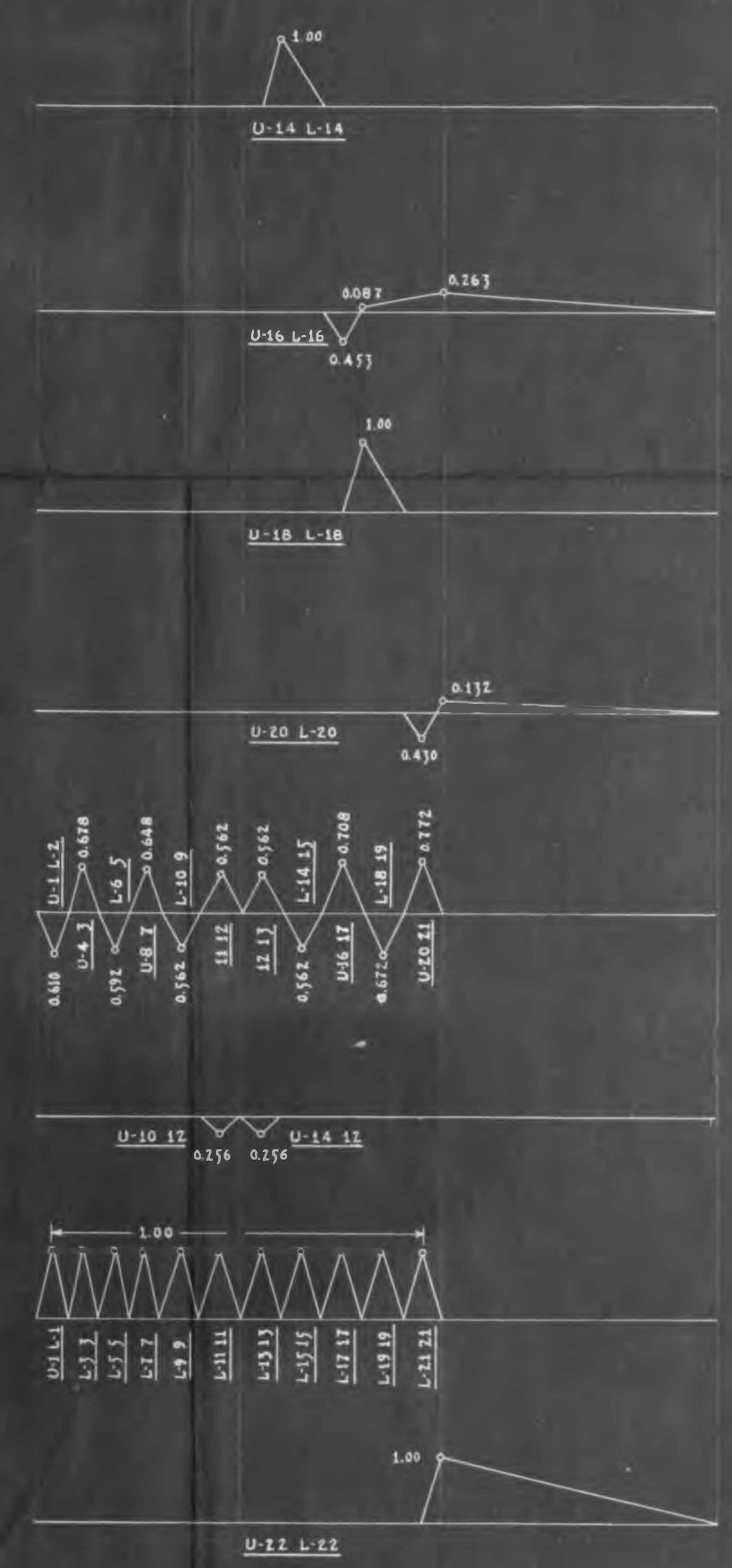
Note: The stress diagram for uniform unit load is supplemtary to the influence diagrams of stress. The stress in any particular member as taken from the stress diagram must equal the algebraic sum of the influence diagram areas for the same member. An immediate check is thus obtained.



STRESS DIAGRAM FOR CANTILEVER SECTION ASSUMING A UNIFORM LOAD OF 1 LB./LIN. FT. OF TRUSS OVER ENTIRE STRUCTURE.

SCALE: 1 in. = 30 ft.
= 60 lbs.





INFLUENCE DIAGRAMS FOR STRESS IN MEMBERS DUE TO A 1 LB. LOAD MOVING ACROSS ENTIRE STRUCTURE. AREAS ABOVE BASE LINE REPRESENT TENSION.

SCALE: HOR. 1 in. = 20 ft.
VERT. = Variable.

1/5

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DEPARTMENT of CIVIL ENGINEERING.

A 1600 FT. CANTILEVER BRIDGE.

SCALES: INDICATED.

OCTOBER 1928.

H.B. Blodgett.

LATERAL BRACING, CANTILEVER SECTION.

1 IN. = 20 FT.

STRESS DIAGRAM, UNIFORMLY DISTRIBUTED LOAD OF 1 LB. PER LIN. FT.

1 IN. = 100 LBS.

INFLUENCE DIAGRAMS FOR STRESS.

1 IN = 300 FT. HORIZONTAL.
" VARIABLE VERTICAL.

2/5

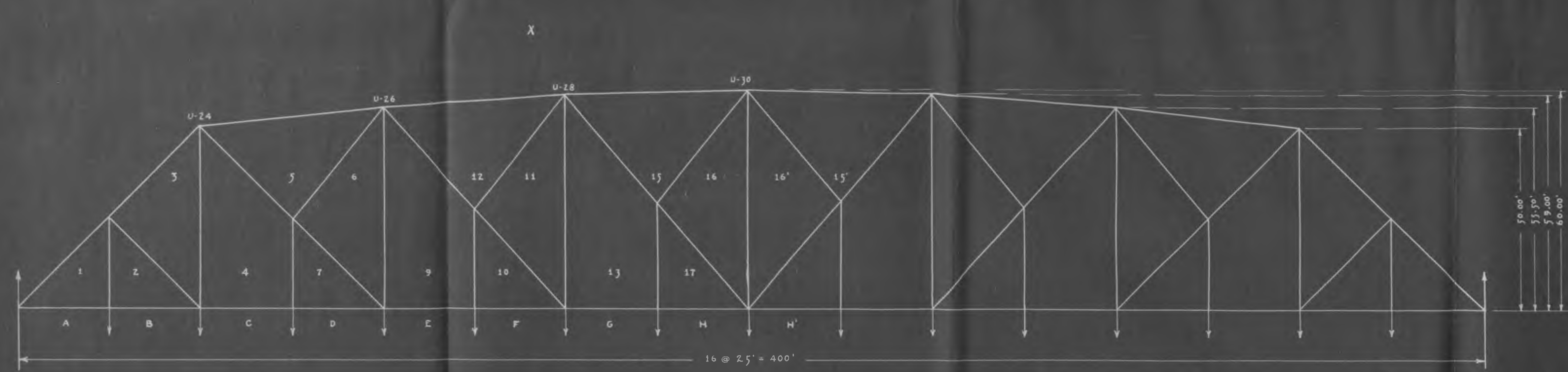
UNIVERSITY OF ARIZONA
DEPARTMENT OF CIVIL ENGINEERING.

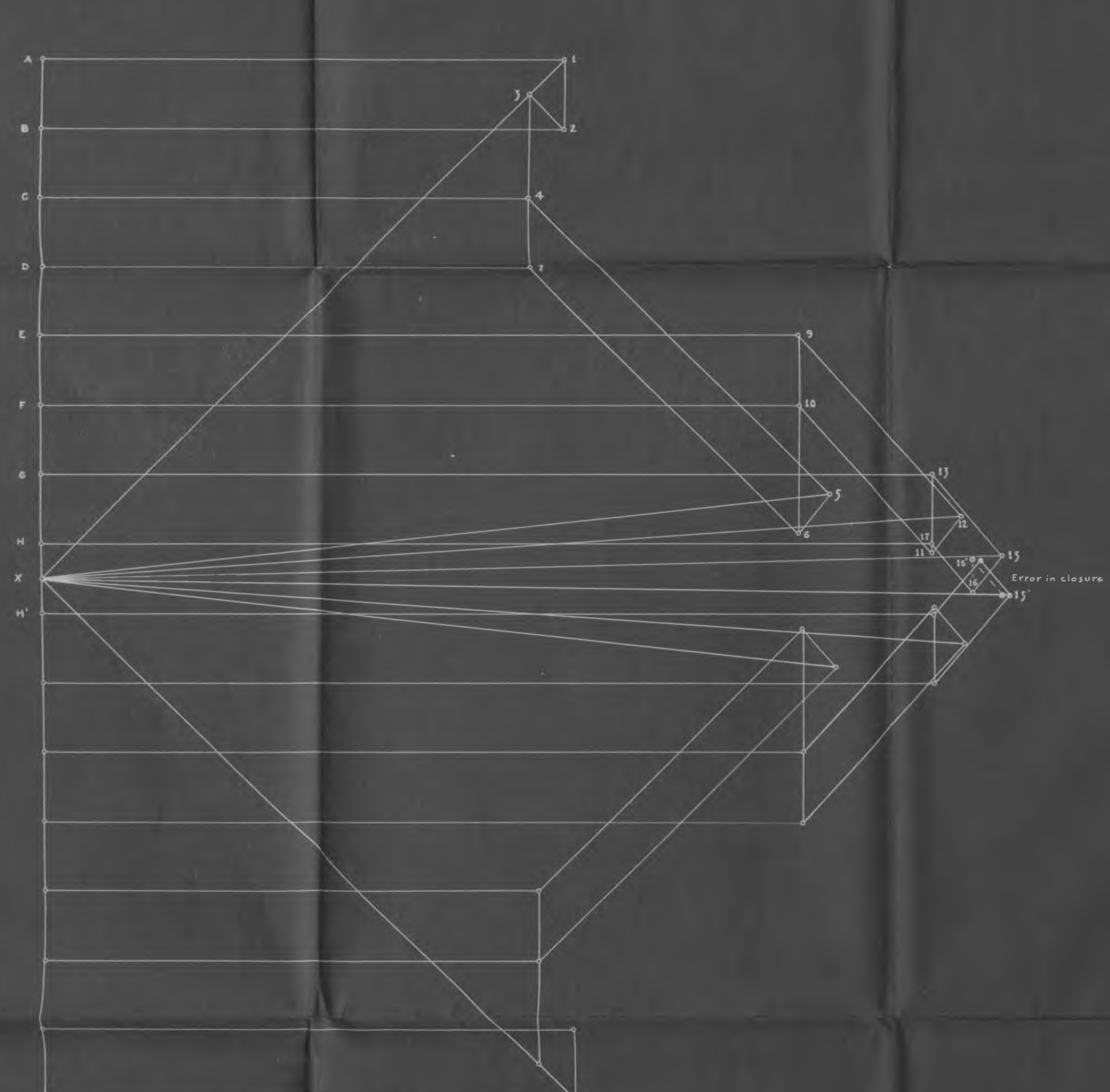
A 1600 FT. CANTILEVER BRIDGE

SCALES: INDICATED.

NOV. 1928.

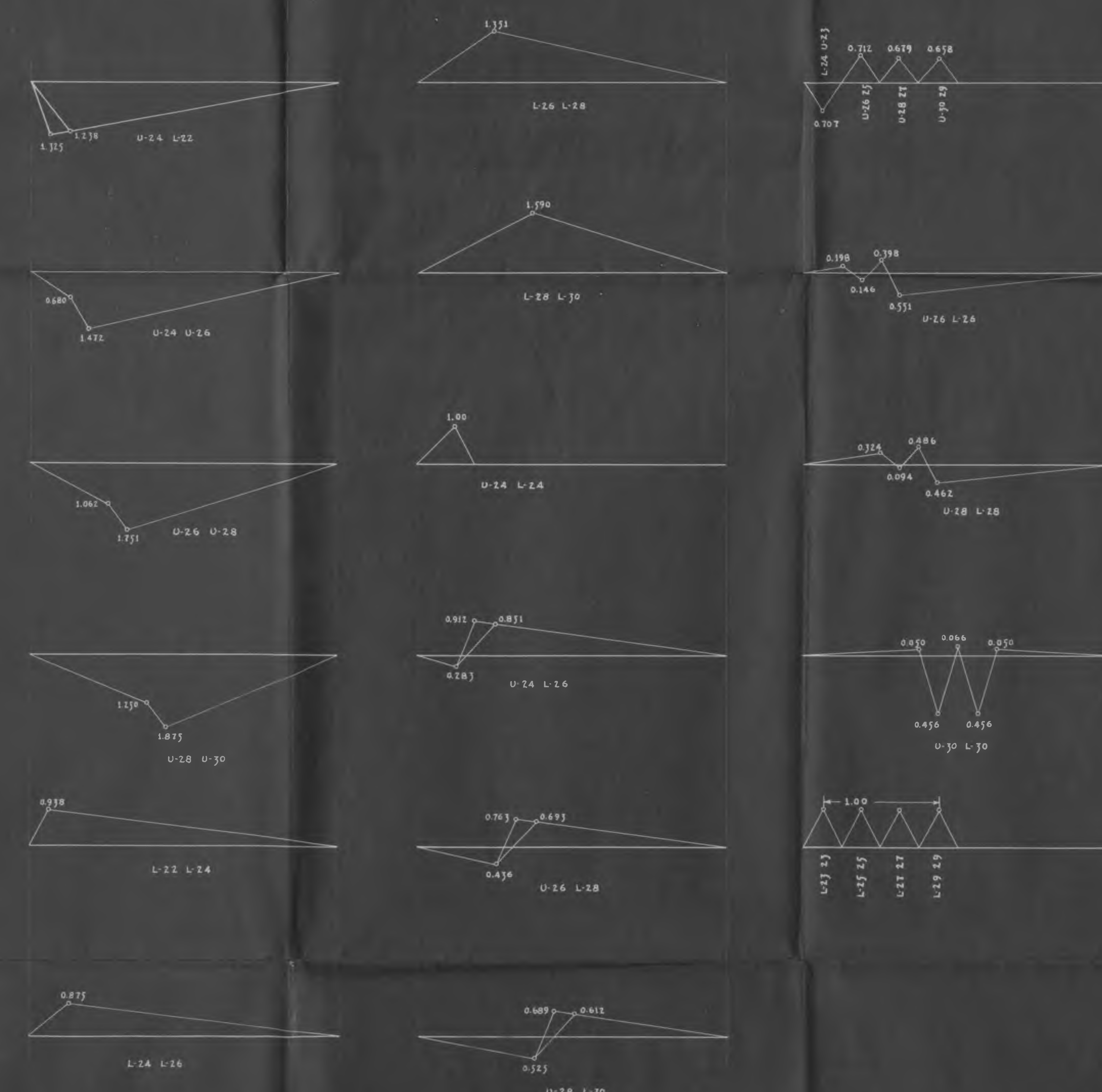
H. B. Blodgett.





STRESS DIAGRAM FOR UNIFORM LOAD OF 1-LB PER FOOT OF TRUSS.

SCALES: 1" = 20' = 30 LBS.



INFLUENCE DIAGRAMS FOR STRESSES IN MEMBERS.

SCALES: HORIZONTAL 1" = 100'
VERTICAL - VARIABLE.

3/3

UNIVERSITY OF ARIZONA
DEPARTMENT OF CIVIL ENGINEERING.

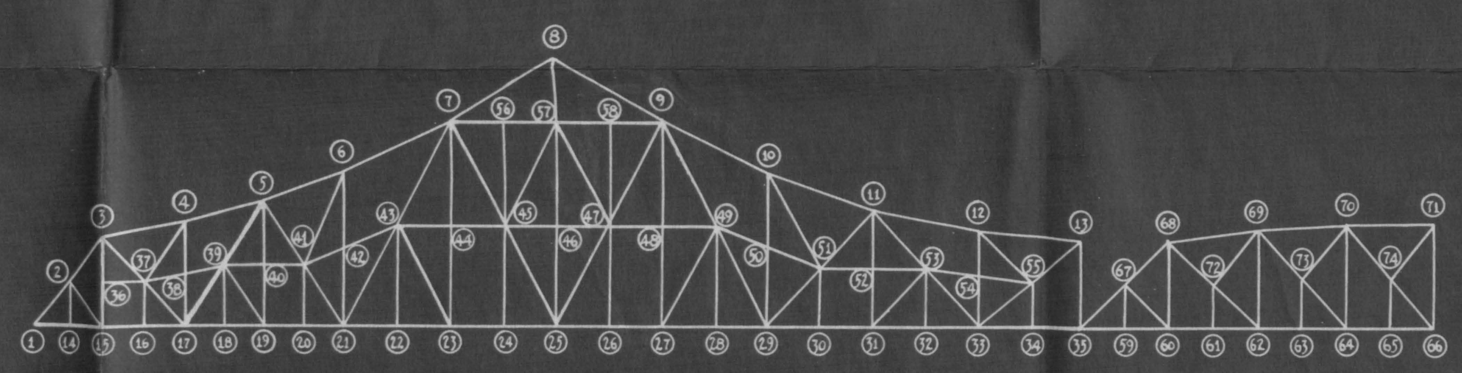
A 1600 F-T. CANTILEVER BRIDGE.

SCALES INDICATED

OCTOBER 1928.

① MEMBER	② LENGTH STRESSED	③ LENGTH UNSUPPORTED	④ SECTION AREA (in²)	⑤ SECTION MOM.-INERT-	⑥ SECTION RAD.-GYRA-	⑦ STRESSES FROM STRUCT-	⑧ STRESSES FLOOR	⑨ STRESSES LIVE	⑩ STRESSES IMPACT	⑪ STRESSES WIND	⑫ STRESSES COMBINED	⑬ STRESSES SELECTED	⑭ DEFLECTION DATA SL/EA	⑮ DEFLECTION DATA U	⑯ DEFLECTION DATA SUL/EA	⑰ DEFLECTION DATA	⑱ DEFLECTION DATA	⑲ WEIGHT
UPPER CHORD MEMBERS																		
L0-U1 1-2	34.478		79.50	3571.51	6.70	+ 358 950	+ 647 000	+ 665 000 / - 284 000	∓ 31 650	∓ 24 700	+ 1 647 900	+ 585 600	+ .286	+ 1.320	+ .755	+ 641 950	+ .147	9 025
U1-U2 2-3	34.478		"	"	"								"	"	"	"	+ .147	9 025
U2-U4 3-4	45.580		96.00	512.	2.31	+ 532 250	+ 806 000	- 262,000 / + 780 000	- 29 100 / + 37 200	∓ 38 000	+ 2 092 550	+ 679 100	+ .398	+ 1.530	+ .609	+ 755 300	+ .220	14 900
U4-U6 4-5	46.590		110.00	1109.	3.16	+ 553 300	+ 844 000	- 188,000 / + 828 700	- 21 000 / + 39 500	∓ 76 700	+ 2 187 700	+ 694 600	+ .370	+ 1.566	+ .580	+ 810 800	+ .215	17 100
U6-U8 5-6	48.016		196.00	3 200	4.04	+ 917 810	+ 1 380 000	- 213 000 / + 1 162 000	- 23 600 / + 55 500	∓ 116 800	+ 3 439 310	+ 969 200	+ .337	+ 2.183	+ .736	+ 1 141 500	+ .244	32 000
U8-U10 6-7	66.649		192.00	2 305	3.46	+ 963 320	+ 1 450 000	- 211 500 / + 1 206 800	- 23 500 / + 57 500	∓ 172 000	+ 3 651,520	+ 1 008 700	+ .507	+ 2.275	+ 1.154	+ 1 238 200	+ .391	44 400
U10-U12 7-8	71.021		252.00	4 120	4.04	+ 1 286 400	+ 1 763 000	+ 1 215 000	+ 58 000	∓ 237 000	+ 4 360 700	+ 1 016 300	+ .493	+ 2.290	+ 1.130	+ 1 311 300	+ .340	61 000
U12-U14 8-9	71.021		252.00	4 120	4.04	+ 1 286 600	+ 1 763 000	+ 1 215 000	+ 58 000	∓ 141 800	+ 4 265 700	+ 1 016 300	+ .481	+ 2.290	+ 1.101	+ 1 216 100	+ .314	61 000
U14-U16 9-10	66.649		192.00	2 305	3.47	+ 954 700	+ 1 448 000	+ 1 014 600	+ 58 300	∓ 76 000	+ 3 387 600	+ 850 600	+ .470	+ 2.270	+ 1.068	+ 984 900	+ .310	44 400
U16-U18 10-11	63.652		159.25	2 240	3.75	+ 911 400	+ 1 360 000	+ 960 000	+ 55 200	∓ 33 200	+ 3 162 800	+ 803 000	+ .506	+ 2.165	+ 1.098	+ 891 400	+ .309	33 800
U18-U20 11-12	61.188		128.00	729	2.39	+ 359 250	+ 565 000	+ 398 000	+ 28 800	∓ 8 200	+ 1 305 250	+ 344 000	+ .249	+ 1.118	+ .278	+ 381 000	+ .081	22 700
U20-U22 12-13	60.187		80.00			+ 334 000	+ 521 000	+ 366 000	+ 28 300	∓ 8 000	+ 1 198 800	+ 307,500	+ .361	+ 1.100	+ .397	+ 343 800	+ .114	
LOWER CHORD MEMBERS																		
L0-L1 1-14	22.500		59.00	1641.77	5.27	- 270 100	- 401 000	+ 185 000 / - 457 300	+ 20 600 / - 21 800	#	- 1 074 950	- 382 050	- .164	- .861	+ .141	- 403 850	+ .053	4 520
L1-L2 14-15	"		65.00	1724.47	5.15	"	"			∓ 70 000	- 1 144 950	"	- .159	- .861	+ .137	- 473 850	+ .056	4 980
L2-L3 15-16	"		76.00	1940.26	5.06	- 277 980	- 421 000	+ 185 000 / - 457 300	+ 14 000 / - 21 800	∓ 140 000	- 1 242 830	- 382 050	- .147	- .861	+ .127	- 543 850	+ .055	5 820
L3-L4 16-17	"		81.00	2 111.83	5.12	"	"	"	"	∓ 210 000	- 1 312 830	"	- .146	- .861	+ .126	- 613 850	+ .059	6 200
L4-L5 17-18	"		177.76	42 702.3	15.5	- 722 540	- 1 095 000	+ 188 000 / - 1 000 500	+ 20 900 / - 47 800	∓ 280 000	- 2 995 640	- 850 300	- .152	- 1.895	+ .288	- 1 178 100	+ .113	13 600
L5-L6 18-19	"		"	"	"	"	"	"	"	∓ 350 000	- 3 065 640	"	- .155	- 1.895	+ .294	- 1 248 100	+ .119	13 600
L6-L7 19-20	"		201.76	46 594.8	15.2	- 732 140	- 1 113 000	+ 180 000 / - 1 000 500	+ 20 000 / - 47 800	∓ 420 000	- 3 163 240	- 850 300	- .142	- 1.895	+ .268	- 1 318 100	+ .111	15 400
L7-L8 20-21	"		210.76	48 488.8	15.2	"	"	"	"	∓ 490 000	- 3 233 240	"	- .138	- 1.895	+ .261	- 1 388 100	+ .112	16 100
L8-L9 21-22	30.000		285.76	60 138.8	14.52	- 980 980	- 1423 000	+ 107 000 / - 1 092 500	+ 12 000 / - 52 000	∓ 560 000	- 3 924 780	- 908 800	- .165	- 2.050	+ .338	- 1 520 800	+ .131	29 100
L9-L10 22-23	"		"	"	"	"	"	"	"	∓ 654 000	- 4 018 780	"	- .169	- 2.050	+ .346	- 1 614 800	+ .139	29 100
L10-L11 23-24	"		294.76	69 876.8	13.40	- 994 670	- 1441 000	+ 95 500 / - 1 092 500	+ 10 600 / - 52 000	∓ 746 000	- 4 142 470	- 908 800	- .1685	- 2.050	+ .346	- 1 706 800	+ .143	30 000
L11-L12 24-25	"		321.76	66 346.8	14.38	"	"	"	"	∓ 840 000	- 4 236 470	"	- .158	- 2.050	+ .324	- 1 800 800	+ .138	32 800
L12-L13 25-26	"		285.76	47 986.8	12.98	- 964 000	- 1441 000	- 968 000	- 50 300	∓ 800 000	- 4 095 500	- 840 200	- .172	- 2.050	+ .353	- 1 690 500	+ .146	29 100
L13-L14 26-27	"		"	"	"	"	"	"	"	∓ 680 000	- 3 975 500	"	- .167	- 2.050	+ .343	- 1 570 500	+ .135	29 100
L14-L15 27-28	"		285.76	"	"	- 948 700	- 1 425 000	- 965 000	- 50 500	∓ 568 000	- 3 822 870	- 830 670	- .161	- 2.050	+ .330	- 1 449 170	+ .125	29 100
L15-L16 28-29	"		261.76	44 821.3	13.10	"	"	"	"	∓ 465 000	- 3 925 870	"	- .180	- 2.050	+ .370	- 1 346 170	+ .127	26 650
L16-L17 29-30	"		176.00	29 923.1	13.05	- 658 870	- 1 030 000	- 725 000	- 46 500	∓ 366 000	- 2 709 370	- 608 000	- .185	- 1.798	+ .332	- 1 120 500	+ .137	17 950
L17-L18 30-31	"		163.00	12 991.2	8.92	- "	"	"	"	∓ 280 000	- 2 635 370	"	- .194	- 1.798	+ .348	- 1 034 500	+ .137	16 600
L18-L19 31-32	"		145.00	11 158.3	8.78	- 643 600	- 1 000 000	- 705 000	- 45 500	∓ 196 000	- 2 476 100	- 591 000	- .204	- 1.798	+ .368	- 832 500	+ .124	14 800
L19-L20 32-33	"		136.00	10485.3	8.79	"	"	"	"	∓ 122 000	- 2 402 100	"	- .212	- 1.798	+ .381	- 758 500	+ .120	13 850
L20-L21 33-34	"		"	"	"	No Stress	No Stress	No Stress	No Stress	∓ 57 200	∓ 57 200	No Stress					#	13 850
L21-L22 34-35	"		"	"	"			"	"	No Stress	No Stress	"					#	13 850
VERTICALS																		
U1-L1 2-14	26.125		8.94	133.8	3.87	+ 18 300	+ 49 500	+ 35 000	+ 10 500	#	+ 78 300	#					#	795
U2-2 3-36	26.125		12.06	"	"	+ 50 425	+ 74 300	+ 52 500	+ 26 000	#	+ 150 725	#					#	1 070
2-L2 36-15	26.125		"	"	"	"	"	"	"	"	"	"					#	1 070
3-L3 37-16	26.125		11.72	157.0	3.66	+ 20425	+ 49 500	+ 35 000	+ 10 500	#	+ 80425	#					#	1 040
U4-4 4-38	33.375		8.94	133.8	3.87	+ 43 260	+ 56 500	- 42 000 / + 80 000	- 4 700 / + 3 800	#	+ 170 500	+ 66 940	+ .227	+ .151	+ .0342	+ 70 740	+ .014	905
4-L4 38-17	26.125		"	"	"	"	"	"	"	"	"	"	+ .227	+ .151	+ .0343	+ 70 740	+ .014	905
5-L5 39-18	35.625		11.72	157.0	3.66	+ 30 155	+ 49 500	+ 35 000	+ 10 500	#	+ 80 155	#					#	1420
U6-6 5-40	35.625		17.58	322.4	4.28	+ 60 915	+ 74 300	+ 52 500	+ 26 200	#	+ 161 415	#					#	2180
6-L6 40-19	35.625		"	"	"	"	"	"	"	"	"	"					#	2180
7-L7 41-20	35.625		12.06	256.2	4.61	+ 34 875	+ 49 500	+ 35 000	+ 10 500	#	+ 94 875	#					#	1460
U8-8 6-42	42.572		14.64	287.0	4.43	+ 96 030	+ 114 000	- 42 000 / + 119 700	- 4 700 / + 3 800	#	+ 313 990	+ 100 160	+ .377	+ .226	+ .085	+ 103 960	+ .028	2020
8-L8 42-21	45.428		"	"	"	"	"	"	"	"	"	"	+ .377	+ .226	+ .085	+ 103 960	+ .028	2020
9-L9 43-22	58.500		17.58	322.4	4.28	+ 53 465	+ 66 000	+ 46 500	+ 14 000	#	+ 133 465	#					#	3490
U10-10 7-44	58.500		29.28	802.8	5.24	+ 162 620	+ 99 000	+ 70 000	+ 26 000	#	+ 287620	#					#	5 375
10-L10 44-23	58.500		"	"	"	"	"	"	"	"	"	"					#	5 375
11'-11 56-45	58.500		~~17.58~~	~~322.4~~	~~4.28~~	+ ~~73 895~~	+ ~~66000~~	+ ~~46 500~~	+ ~~14 000~~	~~#~~	+ ~~153 895~~	#					#	
11-L11 45-24	58.500		17.58	322.4	4.28	+ 73 895	+ 66000	+ 46 500	+ 14 000	#	+ 153 895	#					#	3490
U12-12' 8-57	38.000		509.76	234 330.85	23.28	- 1 445 200	- 2 000 000	- 1 369 000	- 65 500	∓ 6 150	- 4 655 920	- 1 139 070	- .139	- 2.45	+ .341	- 1 210 720	+ .0916	65 900
12'-12 57-46	58.500		"	"	"	"	"	"	"	"	"	"	- .214	- 2.45	+ .525	- 1 210 720	+ .140	101 400
12-L12 46-25	58.500		"	"	"	"	"	"	"	"	"	"	- .214	- 2.45	+ .525	- 1 210 720	+ .140	101 400
13'-13 58-47	58.500		17.58	322.4	4.28	+ ~~62 015~~	+ ~~66 000~~	+ ~~46 500~~	+ ~~14 000~~	#	No Stress + ~~181 015~~	+ ~~39 000~~					#	
13-L13 47-26	58.500		"	"	"	+ 62 015	+ 66 000	+ 46 500	+ 14 000	#	+ 181 015	+ 39 000		.00			#	3 490
U14-14 9-48	58.500		29.28	802.8	5.24	+ 163 170	+ 99 000	+ 70 000	+ 26 000	#	+ 346 670	+ 58 500		.00			#	5 375
14-L14 48-27	58.500		"	"	"	"	"	"	"	"	"	"		.00			#	5 375
15-L15 49-28	58.500		14.64	287.0	4.43	+ 59 880	+ 66000	+ 46 500	+ 14 000	#	+ 178 880	+ 39 000		.00			#	2 920
U16-16 10-50	42.063		19.80	625.2	5.62	+ 88 790	+ 134 800	- 19 500 / + 114 300	- 10 800 / + 7 300	#	+ 310 590	+ 79 700	+ .264	+ .263	+ .0697	+ 87 000	+ .0196	2 575
16-L16 50-29	45.937		"	"	"	"	"	"	"	"	"	"	+ .289	+ .263	+ .0763	+ 87 000	+ .0214	2 810
17-L17 51-30	33.375		14.64	287.0	4.43	+ 42 675	+ 66 000	+ 46 500	+ 14 000	#	+ 161 675	+ 39 000		.00			#	1 660
U18-18 11-52	33.375		26.34	747.8	5.33	+ 95 195	+ 99 000	+ 70 000	+ 26000	#	+ 278 695	+ 58 500		.00			#	2 675
18-L18 52-31	33.375		"	"	"	"	"	"	"	"	"	"		.00			#	2 675
19-L19 53-32	33.375		14.64	287.0	4.43	+ 33 950	+ 66 000	+ 46 500	+ 14 000	#	+ 152 950	+ 39 000		.00			#	1 660
U20-20 12-54	29.750		8.94	133.8	3.87	+ 24 800	+ 34 100	- 24 000 / + 41 500	- 14 500 / + 3 500	#	+ 82 400	+ 20 000	+ .030	+ .132	+ .004	+ 23 500	+ .001	902
20-L20 54-33	25.000		"	"	"	"	"	"	"	"	"	"	+ .031	+ .132	+ .004	+ 23 500	+ .001	758
21-L21 55-34	25.000		14.64	287.0	4.43	+ 35 395	+ 66 000	+ 46 500	+ 14 000	#	+ 154 395	+ 39 000	+ .515	+ 1.000	+ .515	+ 305 900	+ .145	1025
WEB LONGITUDINALS																		
2-3 36-37	22.500		11.00	1 120.8	10.10	No Stress	No Stress	No Stress	No Stress	No Stress	No Stress	No Stress					#	1200
3-4 37-38	22.996		"	"	"	"	"	"	"	"	"	"					#	1260
4-5 38-39	22.996		"	"	"	"	"	"	"	"	"	"					#	1260
5-6 39-40	22.500		"	"	"	"	"	"	"	"	"	"					#	1200
6-7 40-41	22.500		"	"	"	"	"	"	"	"	"	"					#	1200
7-8 41-42	24.543		"	"	"	"	"	"	"	"	"	"					#	1350
8-9 42-43	32.724		"	"	"	"	"	"	"	"	"	"					#	1800
9-10 43-44	30.000		"	"	"	"	"	"	"	"	"	"					#	1650
10-11 44-45	"		"	"	"	"	"	"	"	"	"	"					#	1650
11-12 45-46	"		"	"	"	"	"	"	"	"	"	"					#	1650
12-13 46-47	"		"	"	"	"	"	"	"	"	"	"					#	1650
13-14 47-48	"		"	"	"	"	"	"	"	"	"	"					#	1650
14-15 48-49	"		"	"	"	"	"	"	"	"	"	"					#	1650
15-16 49-50	32.525		"	"	"	"	"	"	"	"	"	"					#	1800
16-17 50-51	32.525		"	"	"	"	"	"	"	"	"	"					#	1800
17-18 51-52	30.000		"	"	"	"	"	"	"	"	"	"					#	1650
18-19 52-53	30.000		"	"	"	"	"	"	"	"	"	"					#	1650
19-20 53-54	30.291		"	"	"	"	"	"	"	"	"	"					#	1665
20-21 54-55	30.291		"	"	"	"	"	"	"	"	"	"					#	1665
U10-11' 7-56	30.000		"	"	"	- 18 900	- 16 950	- 12 000	- 3 600	#	- 39 450	#		.00			#	1650
11'-12' 56-57	"		"	"	"	"	"	"	"	"	"	"		.00			#	1650
12'-13' 57-58	"		"	"	"	- 15 900	- 16 950	- 12 000	- 3 600	#	- 46 550	- 10 000		.00			#	1650
13'-U14 58-9	"		"	"	"	"	"	"	"	"	"	"		.00			#	1650
DIAGONALS																		
U1-L2 2-15	34.478		12.06	256.2	4.61	- 11 700	- 30 200	- 21 200	- 15 700	#	- 57 600	#		.00			#	1410
2-L3 36-16	34.478		Imaginary														#	#
U2-3 3-37	34.478		92.00	3 128.13	5.84	- 376 730	- 596 500	+ 148 000 / - 547 000	+ 16 400 / - 26 000	#	- 1 437 130	- 437 130	- .215	- .9875	+ .213	- 463 900	+ .068	8 900

MEMBER	LENGTH STRESSED	LENGTH UNSUPPORT-D	SECTION AREA (in²)	SECTION MOM-INERTIA	SECTION RAD-GYRA-N	STRESSES FROM STRCTR	STRESSES FLOOR	STRESSES LIVE	STRESSES IMPACT	STRESSES WIND	STRESSES COMBINED	STRESSES SELECTED	DEFLECTION DATA SL/EA	DEFLECTION DATA U	DEFLECTION DATA SUL/EA	DEFLECTION DATA	DEFLECTION DATA	WEIGHT
DIAGONALS																		
3-L4 37-17	34.478		92.00	3 128.13	5.84	- 376 730	- 596 500	+ 148 000 / - 547 000	+ 16 400 / - 26'000	#	- 1 437 130	- 437 900	- .215	- .9875	+ .425	- 463 900	+ .069	8 900
3-U4 37-4	40.250		7.00	37.35	2.31	+ 13 850	+ 33 500	+ 23 400	+ 7 000	#	+ 54 350	#		.00			#	960
5-L4 39-17	42.135		76.00	2 485.02	5.71	+ 356 550	+ 561 000	- 70 000 / + 458 200	- 7 800 / + 21 800	#	+ 1 256 300	+ 316 950	+ .279	+ .715	+ .199	+ 338 750	+ .054	10 900
5-U6 39-5	42.135		"	"	"	"	"	"	"	"	"	"	+ .279	+ .715	+ .200	+ 338 750	+ .054	10 900
5-L6 39-19	42.135		14.64	247.0	4.43	- 17 830	- 29 250	20 600	- 6 200	#	- 53 280	#		.00			#	1 860
6-L7 40-20	42.135		Imaginary.														#	#
7-U6 41-5	42.135		92.00	3 128.13	5.84	- 232 960	- 355 500	+ 12 500 / - 262 500	+ 3 000 / - 12 500	#	- 726 210	- 125 250	- .133	- .2825	+ .037	- 137 750	+ .007	13 200
7-U8 41-6	57.003		8.00	43.7	2.31	+ 22600	+ 32 100	+ 22 600	+ 6 800	#	+ 61 500	#		.00			#	1 500
7-L8 41-21	42.135		92.00	3 128.13	5.84	- 232 960	- 335 500	+ 12 500 / - 262 500	+ 3 000 / - 12 500	#	- 726 210	- 125 250	- .133	- .2825	+ .038	- 137 750	+ .007	13 200
9-U8 43-6	42.074		Imaginary														#	#
9-U10 43-7	65.745		76.00	2 485.02	5.71	+ 282 200	+ 305 000	+ 215 000	+ 10 000	#	+ 601 700	+ 4 440	+ .208	+ .0123	+ .002	+ 14 440	+ .0005	17 000
9-L8 43-21	"		"	"	"	"	"	"	"	"	"	"	+ .208	+ .0123	+ .003	+ 14 440	+ .0005	17 000
9-L10 43-23	"		26.34	747.8	5.33	- 30050	- 36 950	- 26 000	- 7 800	#	- 74 800	#		.00			#	5890
10-L11 44-24	"		Imaginary														#	#
11-U10 45-7	"		142.50	4 991.80	5.92	- 215 530	- 100 300	+ 140 000 / - 210 000	+ 6 700 / - 23 400	#	- 242 460	+ 96 770	- .046	+ .255	- .011	+ 73 370	+ .0035	31 800
11-12' 45-57	"		9.00	48.1	2.31	+ 41 500	+ 36 950	+ 26 000	+ 7 800	#	+ 86 250	#		.00			#	2 010
11-L12 45-25	"		142.50	4 991.80	5.92	- 215 530	- 100 300	+ 140 000 / - 210 000	+ 6 700 / - 23 400	#	- 242 460	+ 96 770	- .046	+ .255	- .012	+ 73 370	+ .0035	31 800
13-12' 47-57	"		9.00	48.1	2.31	+ 34 850	+ 36 950	+ 26 000	+ 7 800	#	+ 101 450	+ 21 850		.00			#	2 010
13-U14 47-9	"		92.00	3 171.13	5.87	- 191, 215	- 75 500	+ 107 800 / - 142 000	+ 8 000 / - 19 300	#	- 314 765	- 28 750	- .092	+ .2521	- .023	- 48 050	- .0035	20 550
13-L12 47-25	"		"	"	"	"	"	"	"	"	"	"	- .093	+ .2521	- .024	- 48 050	- .0035	20 550
14-L13 48-26	"		Imaginary														#	#
15-U14 49-9	"		76.00	2 476.2	5.70	+ 205 370	+ 307 000	+ 216 000	+ 11 300	#	+ 741 800	+ 18 130	+ .257	+ .0123	+ .003	+ 29 430	+ .0005	17 000
15-U16 49-10	42.074		Imaginary														#	#
15-L14 49-27	65.745		26.34	747.8	5.33	- 33 600	- 36 950	- 26 000	- 7 800	#	- 100 200	- 21 850		.00			#	5890
15-L16 49-29	65.745		76.00	2 476.2	5.70	+ 205 370	+ 307 000	+ 216 000	+ 11 300	#	+ 741 800	+ 18 130	+ .257	+ .0123	+ .003	+ 29 430	+ .0005	17 000
17-U16 51-10	62.321		10.00	53.4	2.31	+ 30 200	+ 46 700	+ 32 900	+ 10 000	#	+ 114 500	+ 27 600		.00			#	2 120
17-L16 51-29	44.876		120.00	4164.2	5.89	- 297 350	- 410 000	- 289 000	- 16 600	#	- 966 450	- 242 500	- .144	- .368	+ .053	- 259 100	+ .014	18 300
17-U18 51-11	"		"	"	"	"	"	"	"	"	"	"	- .145	- .368	+ .053	- 259 100	+ .014	18 300
17-L18 51-31	"		Imaginary														#	#
19-U18 53-11	"		92.00	3 171.13	5.87	+ 459 300	+ 710 000	+ 500 000	+ 40 000	#	+ 1 629 300	+ 420 000	+ .317	+ 1.050	+ .333	+ 460 000	+ .094	14 050
19-L18 53-31	"		20.52	357.6	4.18	- 23 150	- 44 500	- 31 300	- 9 000	#	- 102 900	- 26 250		.00			#	3 120
19-L20 53-33	"		92.00	3 171.13	5.87	+ 459 300	+ 710 000	+ 500 000	+ 40 000	#	+ 1 629 300	+ 420 000	+ .318	+ 1.050	+ .334	+ 460 000	+ .095	14 050
21-U20 55-12	42.250		9.00	48.0	2.32	+ 27 300	+ 51 000	+ 36 000	+ 10 800	#	+ 119 300	+ 30 200		.00			#	1 290
21-L20 55-33	39.051		120.00	4 164.2	5.89	- 458 200	- 723 000	- 510 000	- 37 000	#	- 1 645 200	- 427 000	- .214	- 1.428	+ .305	- 464 000	+ .086	15950
21-U22 55-13	39.051		"	"	"	"	"	"	"	"	"	"	- .215	- 1.428	+ .306	- 464 000	+ .086	15950
21-L22 55-35	39.051		Imaginary														#	#
U22-U24 13-68	50.000		Idle Member															28400
L22-U24 35-68	70.711	35.355	117.72			- 291 000	- 583 000	- 410 000	- 31 100	∓ 40 500	- 1 355 600							28400
U24-U26 68-69	50.301		134.72			- 322 000	- 622 000	- 437 000	- 33 200	∓ 27 800	- 1 442 000							23 100
U26-U28 69-70	50.122		149.22			- 380 000	- 724 000	- 510 000	- 38 700	∓ 44 500	- 1 697 200							25 400
U28-U30 70-71	50.010		153.76	6 436.8	6.48	- 402 000	- 755 000	- 532 000	- 40 500	∓ 100 000	- 1 829 500							26 100
L22-L23 35-59	25.000		60.00			+ 208 000	+ 413 000	+ 291 000	+ 22 000	#	+ 934 000							5 100
L23-L24 59-60	"		64.00			"	"	"	"	∓ 41 600	+ 975 600							5450
L24-L25 60-61	"		60.00			+ 197 000	+ 385 000	+ 272 000	+ 20 600	∓ 77 800	+ 952 400							5 100
L25-L26 61-62	"		64.00			"	"	"	"	∓ 108 500	+ 983 100							5 450
L26-L27 62-63	"		92.5			+ 309 000	+ 594 000	+ 419 000	+ 31 700	∓ 134 500	+ 1 488 200							7 850
L27-L28 63-64	"		95.0			"	"	"	"	∓ 153 000	+ 1 506 700							8 100
L28-L29 64-65	"		112.5			+ 368 000	+ 700 000	+ 493 000	+ 37 300	∓ 166 500	+ 1 764 800							9 600
L29-L30 65-66	"		"			"	"	"	"	∓ 175 000	+ 1 773 300							9600
L23-23 59-67	25.000		7.72			+ 21400	+ 55 000	+ 38 800	+ 6 700	#	+ 121 900							660
U24-L24 60-68	50.000	25.000	12.00			+ 35 800	+ 82 500	+ 58 000	+ 8 600	#	+ 184 900							2 100
U26-L26 62-69	55.500		41.75	446	3.27	+ 81500	+ 154 000	+ 18 000 / - 126 500	* - 10 800	#	+ 372 800							8 000
L25-25 61-72	25.000		7.72			+ 23 800	+ 55 000	+ 38 800	+ 6 700	#	+ 124 300							660
L27-27 63-73	27.750		9.2			+ 31 600	+ 55 000	+ 38 800	+ 6 700	#	+ 131 100							870
U28-L28 64-70	59.000		17.58	322.4	4.28	- 34 600	- 61 500	+ 42 200 / - 85 900	- 7800	#	- 189 800							3 600
L29-29 65-74	29.500		9.2			+ 27 300	+ 55 000	+ 38 800	+ 6 700	#	+ 127 800							930
U30-L30 66-71	60.000	30.000	8.94	133.8	3.87	- 12 700	- 27 300	+ 12 000 / - 31 200	- 4 200	#	- 75 400							1900
L24-23 60-67	35.356		11.72	157.0	3.66	- 15 000	- 39 000	- 27 400	- 4 800	#	- 86 200							1 500
U24-25 68-72	35.355		47.5			+ 170 000	+ 328 000	+ 243 000 / - 12 700	+ 21 000	#	+ 762 000							5 750
25-L26 62-72	35.355		"			"	"	"	"	"	"							5 750
U26-25 69-72	39.437		6.00			+ 16 900	+ 39 200	+ 27 600	+ 4800	#	+ 88 500							800
U26-27 69-73	37.350		32.5			+ 104 800	+ 191 500	+ 172 000 / - 40 300	+ 15 600	#	+ 483 900							4 100
27-L28 64-73	37.350		"			"	"	"	"	"	"							4 100
U28-27 70-73	40.019		6.00			+ 16 400	+ 37 200	+ 26 200	+ 4 500	#	+ 84 300							800
U28-29 70-74	38.668		18.00			+ 53 300	+ 88 500	+ 127 800 / - 70 500	+ 12 400	#	+ 282 000							2,400
29-L30 66-74	38.668		"			"	"	"	"	"	"							2400
U30-29 71-74	39.437		6.00			+ 18 000	+ 36 300	+ 25 500	+ 4 400	#	+ 84 300							800



MEMBER KEY DIAGRAM.

NOTE: Members designated in these tabulations in two ways as, U8-8, 6-42. This occasioned by the use of the first method in preliminary investigations when only members necessary to the truss were considered. The second made desireable after addition of auxilliary and imaginary members in subsequent considerations. Imaginary members used in secondary stress investigation.
Under "Deflection Data" column 16 is deflection increments due to all loads, including weight of structure. Column 18 is deflection increments due to kinetic conditions only. Column 13 shows stresses in members due to live load placed to give maximum deflection at point 35.

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UNIVERSITY of ARIZONA
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A 1600 FT. CANTILEVER BRIDGE

MAY 1929.

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A 1600 FT. CANTILEVER BRIDGE

DETAIL SCALE: 1" = 1'-0"

MAY 1929.