PROBABLE FUTURE TRENDS IN SUSPENSION BRIDGE DESIGN *

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LAST summer, when Professor Castleman extended me the invitation to speak before this Society at this Convention, he gave me as a subject, "What Kind of Bridges Were Developed by the Army During the War."

Now, any of you gentlemen, and I see a number in the audience, who have been connected with the United States Army in either civilian or military capacities, will know that it is quite a proposition to be able to get any article or any speech authorized by the different departments of the Army or the Navy. It has to go through a lot of channels, and by the time the paper is actually approved, the writer would not recognize it. Therefore, I asked that the title be changed so as to read, "Probable Future Trends in Suspension Bridge Design," without any reference whatsoever to the Army.

At that time, we were at war. We are now at peace. A number of articles and newsreels have appeared showing types of construction that would not have been permitted one year ago or two years ago. So, while I have no authorization, I am going to take the liberty of saying something about the development of a type of suspension bridge with which I was intimately connected, during my two-year stay with the United States Engineer Board of the Army at Fort Belvoir.

This is a controversial subject. There is a great difference of opinion among bridge engineers in regard to the proper type of suspension spans for lengths of one thousand feet and above. So I am going to ask you all to keep an open mind and not let any preconceived ideas of what have been the developments in suspension bridges up to the Tacoma Narrows failure in 1940 influence your ideas as to the probable future trends from 1945-46 on.

I assume that everyone in this audience has seen a suspension bridge. If anyone here has not seen a suspension bridge, kindly raise your hand. Everybody has seen one.

At the start, I am going to take a few minutes and run over some of the elementary principles of suspension bridge design because they have a direct bearing upon the studies that we made.

All suspension bridges have two parts; the first or main part consisting of the cable, the towers, and the anchorages, and the second or auxiliary

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part, consisting of the suspenders, stiffening trusses or girders, floor beams, stringers, lateral system, sidewalk brackets, floor deck, and so on. It so happens that in a suspension bridge, if the three main members hold, that is the towers, the cables and the anchorages, it does not make very much difference what happens to the rest of the members, as the structure will stand up, and the outstanding example of that fact is the failure of the Tacoma Narrows Bridge. Those who saw the motion pictures of that failure realize that that structure took a terrific beating, but when it was all over, the towers and the anchorages and the cables were still intact.

After some three years on war construction work in various parts of the United States, I went to the office of the Chief of Engineers in November, 1943. At that time, an analysis was being made of various types of American military bridges. There were, I should say, a dozen or more different types, floating and fixed, but the maximum length of a single span was about 140 feet. There did not seem to be any American designs of span-length greater than 150 feet.

At that time, November, 1943, there was a rather critical situation in the CBI theatre—China, Burma, India. You will recall that the Japs had taken the City of Singapore and had overrun the Malay Peninsula, had also extended north into Burma and cut the Burma Road, eliminating the possibilties of the Allies getting supplies into Western China over that road.

The next best method seemed to be to have the supplies delivered to Calcutta and then transported by rail to New Delhi and then, in some way, transported from New Delhi across the northern part of Burma into the western part of China. And you have seen in the papers since then the most excellent piece of American engineering military construction, the Ledo Road.

On this route there were some five or six rivers of fairly large magnitude that had to be crossed, the Irrawaddy, the Mogaung, and the Shweli in Burma and the Salween and the Mekong in China. The Japs had done a pretty good job of taking out some of the suspension spans across those rivers, although they did not take out the towers or the anchorages, and in only one case did they take out part of one of the cables. In that case they had made a fairly good hit on one of the cables and destroyed nine of the nineteen ropes. The Chinese had repaired that bridge, eliminating the stiffening trusses, but using the vertical structural steel members of the stiffening trusses, thus converting it into an unstiffened bridge. This bridge was a single lane and it was possible to get vehicles across using nineteen original ropes on one side and ten ropes on the other.

So from a military standpoint it seemed the logical thing to make a series of long span bridges which would be of the suspension type in order to eliminate the necessity of putting any false work or piers in the large rivers. A directive was prepared in the office of the Chief of Engineers requesting that designs be made for spans ranging in length from 150 up to 600 feet with a number of military characteristics, of which two parts were essential; first, that the bridge had to be two-lane, and, second, that they should be strong enough to carry (1) two lanes of 25 ton tanks, spaced 100 feet centers, or (2) one line of 50 ton tanks, spaced 100 feet centers, but anywhere across the width of the bridge, either on the center line or over toward either curb; or (3) one 80 ton tank spaced anywhere along the center line of the bridge, but deviating not more than one foot from that center line.

I went down to the Engineer Board, as Bridge Consultant, in February, 1944. There was a Bridge Designing Department about half the size of ours in the State Highway Department. This department already had prepared designs of a 300 foot, a 460 foot, and later on a 400 foot span using the ordinary conventional type of stiffening trusses, so after the directive arrived at the Engineer Board, designs were started for span designs from 150 up to 600 feet using the conventional type, and rather complete designs of a 160 foot, a 400 foot and a 600 foot span were made. When I came to analyze the weights, I found that whether the span was 160, 400 or 600 feet, the weights of the stiffening trusses alone were about fifty per cent of the total steel work in the bridge, exclusive of the two towers.

When you consider that the stiffening truss does not even carry its own weight, and its sole function, or rather its major function, is to distribute any local live loads in such a way that the live load stresses are distributed through the truss and then through the vertical suspenders and applied to the cable in approximately equal amounts so as to keep the cable in its approximate parabolic form and maintain the roadway level so that vehicles can go across the bridge without much of a travelling wave.

That objective is very important as far as commercial bridges are concerned, but it is not of so much importance as far as military bridges are concerned because any vehicle, and particularly the 50 ton, 60 ton and 65 ton tanks that go across a ponton bridge, as you have probably seen in the motion pictures, produce a very definite travelling wave as they go across these ponton bridges.

Therefore, looking at the problem from a military angle, we were wasting a considerable amount of steel that could be used to better advantage than simply going into stiffening trusses.

Furthermore, in the design of the stiffening trusses on the 600 foot span, there were some 50,000 field rivets that had to be driven in every span of that length, with, of course, corresponding less amounts for the shorter spans.

Furthermore, splice plates connected the different sections of the stiffening truss, and there were some 1,800 of these splice plates about four inches wide and two feet to two feet six long that had to be handled for each 600 foot span. Now, 600,000 pounds is three hundred tons, and this involved extra mill rolling, plant fabrication, rail transportation in the United States, ocean transportation, haulage over roads which were anywhere from bad to worse, and finally erection. Extra vehicles had to be used in the transportation which extra vehicles could serve more useful purposes, and furthermore this additional material involved additional time for erection.

About that time, Lt. Col. Jack Beretta arrived at the Engineer Board after being up in Newfoundland for twenty-eight months and was assigned to the Engineer Board as Assistant Director of Technical Division No. 4. Jack Beretta and I had been old friends in the American Bridge Company back around 1926 and 1927, and after his arrival we were discussing the various problems at the Engineer Board, including suspension spans, just as I am talking to you men, and Jack said, "Why not omit stiffening trusses and see whether we can a satisfactory bridge to transport the loads without the deflections being too great?"

The matter was assigned to the Bridge Design Section, but they did not even get to first base on any satisfactory results. So it was necessary for Jack Beretta and myself, to start off from scratch and see what could be done.

I might say that Jack Beretta is the son of one of the leading bankers down in Texas and he has a Consulting Engineering business of his own there. Furthermore, he and his father were the owners of several of the international bridges across the Rio Grande, some of which were unstiffened suspension spans. They are also stockholders in the International Bridge at Laredo, Texas, which is the gateway for the Pan-American Highway going into Mexico and points south.

Figure 1 shows the action of a flexible cord under various loads. If we have a flexible cord of uniform cross section and stretch it between two points, that cord goes into a curve which is called a catenary on account of the mathematics, as shown, Figure 1a. If we put a single load on that cord, the catenary immediately changes shape and goes into a form as shown in Figure 1b. This is exactly what happens when your good wives put your winter overcoats on the clothesline preparatory to boxing them up for the summer. It is a homely illustration, but it shows perfectly the action of a flexible cord under load. If we put the load at some other point, we get the condition as shown at Figure 1c; with two loads, the condition is shown at Figures 1a and 1e. This was our first model.

We then bought a quantity of chain of different lengths and tried out different combinations as shown in Figure 2 to see what could be done to get additional stiffness by reduced deflections of the cable and eliminate the condition of excessive deflection that you saw in Figure 1b when a

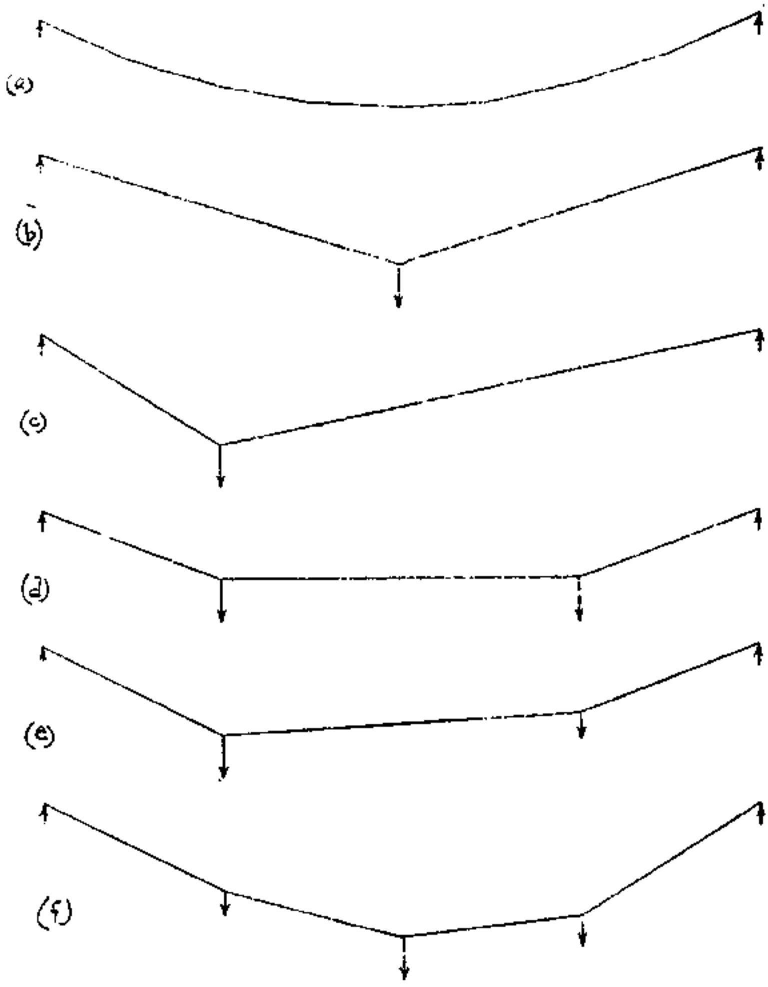


Fig. 1 Scale - None

single load was put on a cable. In Figure 2a we have the cable from which hung ordinary suspenders and with a load at X, the cable deflected in the same manner as in Figure 1b; with this load at X point 4 of the cable went down and the other points of the cable went up. When we rearranged the suspenders and brought the ends together as in Figure 2b and then put the load at X, that load, instead of acting at one point of the cable, was divided;

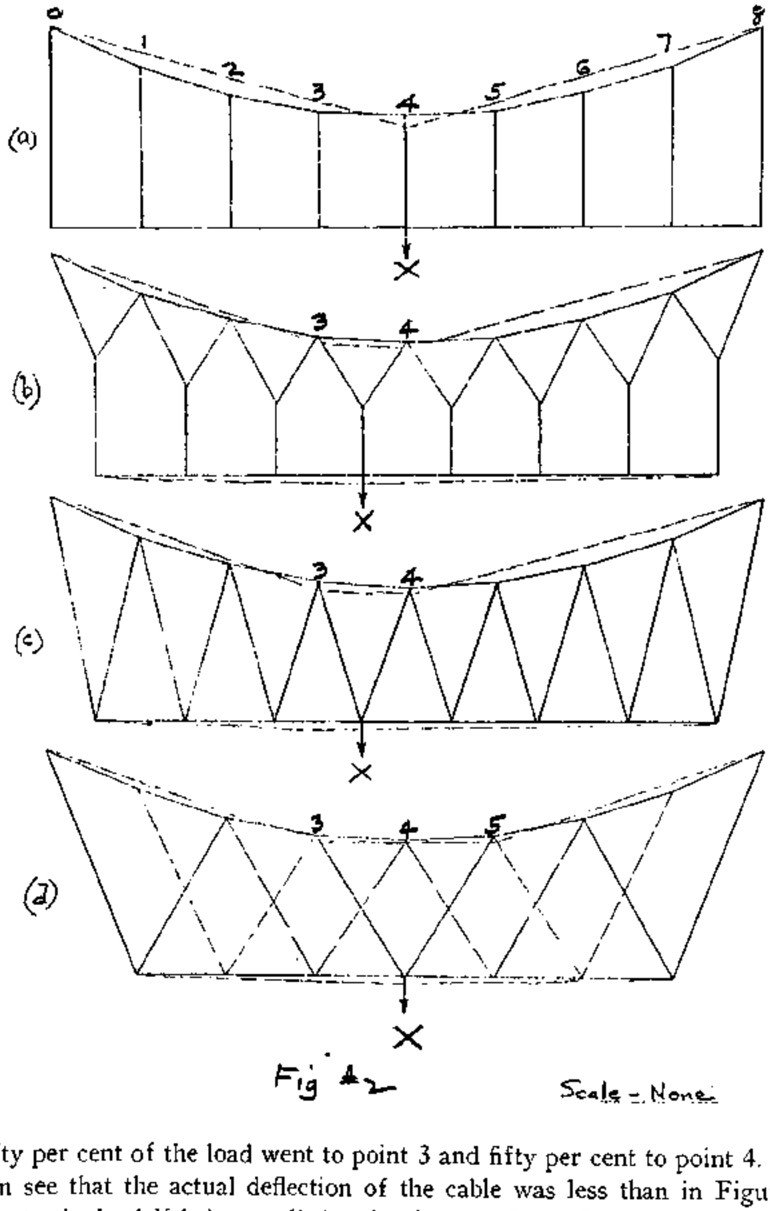


Fig #2

fifty per cent of the load went to point 3 and fifty per cent to point 4. You can see that the actual deflection of the cable was less than in Figure 2a due to the load X being applied at 2 points on the cable.

We then tried another arrangement eliminating the verticals as shown in Figure 2c causing the load to be applied at points 3 and 4, and the cable deflection was less, similar to Figure 2b.

Then we tried a third arrangement shown on Figure 2d where the diagonals applied the load X to the cable not at two adjacent parts, but at point 3 and point 5, with still less local deflection of the cable. By trying a number of these combinations, we came to the conclusion that the diagonals were a very important factor in reducing the deflections of the cable due to two things; first, on account of the fact that the intensity of the load at each of the two points on the cable was ½ of the total load, and secondly on account of the fact that these half loads were applied at different parts on the cable. In all cases the loads X used were steel washers.

Our next test was to make a model of a 600 foot span, six feet long on a scale of 1 to 100, as shown in Figure 3.

In this case we used plumber's chain for the cable, thin copper wire for the verticals, and for a pair of double diagonals in each panel, and in addition to that we had a hold-down cable also made of plumber's chain extending from tower to tower. We then shortened the length of the verticals and the diagonals in such an amount that there was a very definite upward curve to the hold-down cable which put additional downward loads on the main cable at each panel point.

That principle is not new because on suspension bridges, where the cables have been made of parallel wires, after the towers have been erected, the next operation is to put up the foot walk, to facilitate the spinning of the cables. If the foot walk was used alone and a fairly good-sized wind came up, there was a possibility of its blowing the footwalk as we say—inside out—just like an umbrella. So, in order to prevent that occurrence, storm cables, or hold-down cables, are used inverted under the footwalk cables and those storm cables eventually become the hand ropes on the completed bridge. And we used that principle here on the model as shown in Figure 3. When we put the weight on, at point i Figure 3, we had our first great surprise and also one of the indications that we were on the right track in our investigation by showing that not only were the two diagonals Hi and iJ in action, but the action extended four points on either side of point i as diagonals Ef, Fg, Gh, jK, kL, and lM were also in action shown by the heavy lines in Figure 3 and those particular diagonals became taut just like violin strings while the eight opposite diagonals in these 8 panels were slack. Thus the load X was distributed to 8 points on the cable by means of these diagonals.

But we found another interesting thing; there was some very definite action in tension by the diagonals bC, cD, No and Op in the panels near the ends of the bridge while diagonals Bc, Cd, nO, and oP were slack. The diagonals that were in action near the ends of the span sloped in opposite directions from the diagonals in action near the center of the span while the other diagonals in the same panels were slack, showing very definitely that when we put a load at X the central part of the cable went down and the two end parts tried to go up but the holddown cable was immediately put into action by the diagonal members near the ends of the span and in turn, prevented the end portions of the cable from going up, and thus made the whole span very definitely stiff.

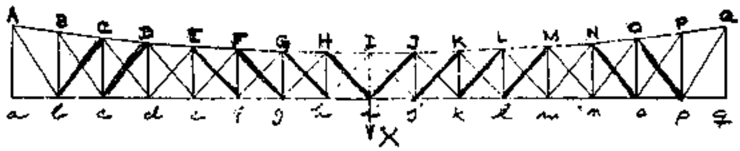


Fig. 3

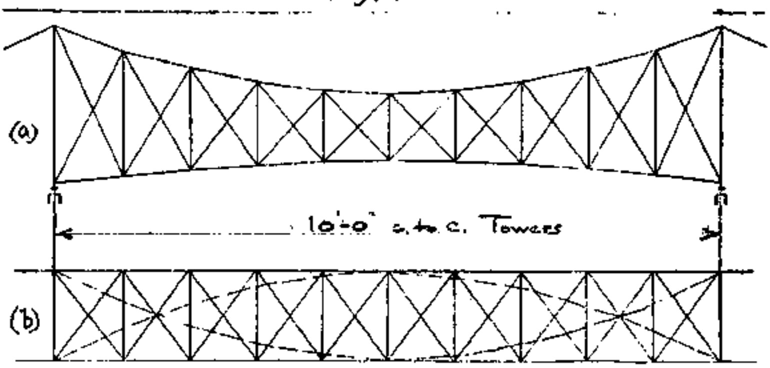


Fig 4

We required a period of two or three weeks on that study. About that time different people, civilian and military, going up and down the hall, saw me playing on our celotex board with all the washers, chains, weights, and copper wire of the several models, and it aroused a lot of interest. Finally, it got to Colonel Mattison, who was Executive Officer at the Engineer Board, and I told him that if he would give me about another week, I would put on a little show for him.

In the meantime the matter got to the ears of General Schultz, President of the Board, and the General said that he would like to attend, so I asked for fifteen minutes and, as a matter of fact, the discussion lasted for an hour and a quarter, following my explanation of the various steps on the different 2-dimension models shown in Figure 1, 2, and 3.

Thus far, all that we had been able to do was qualitative; we were not able to get from my models any results of a quantitative nature. Furthermore, we did not have the proper facilities at the Engineer Board to get that data. But I did want to get a three dimensional model and I had thought of asking for $1,500 to have our model shop down at the Engineer Board make this model in three dimensions so as to get some idea of lateral deflections on this type of bridge.

We wanted to make a 200 foot span model on a scale of one to twenty, which would be a ten foot model as shown in Figure 4a, in which the main cable would be one-eighth of an inch diameter airplane wire, the inverted holddown cable the same, and the verticals and the diagonals one-sixteenth inch diameter airplane wire. In the plan view Figure 4b, we would have 2-wire rope cable systems, one of the cable systems on each side of the bridge, floor beams and stringers which were made of wood, bottom laterals which were made of one-sixteenth inch airplane cords, and in this particular model we had some wind cables extending in parabolic form from the ends of the towers underneath the floor system, going across to the opposite truss at the center and back, shown by dotted lines in Figure 4b.

That is what I wanted to make, and when I saw all the interest that was taken, I immediately raised the $1,500 to $2,500 and received approval for same.

I will say that General Schultz backed me up straight through the investigation, and Colonel Mattison, also. But, even at that, I was not satisfied because I was not convinced that we had the necessary amount of engineering talent of the proper frame of mind in order to do the research investigation that was necessary on a project like this.

Lieutenant-Colonel Roebling happened to be in the audience that day, he being a great grandson of John A. Roebling, founder of the Roebling Company, and I told the General and the others that, while we were very glad to get the $2,500, we could not even produce any quantitative results from the 10 foot, 3 dimensional model of a 200 span, and recommended bringing an outside company, a regular contractor, into the picture and entering into a contract with them to make an extensive and an exhaustive study of the possibilities of a 600 foot span on a scale of one to ten with a model that would be sixty feet long, and I recommended the Roebling Company. That recommendation was approved.

I might mention the fact that it was somewhat unusual for me as a structural steel man to go to the wire rope industry to solve a problem in suspension bridges, but, after all, this was very definitely a wire rope problem because we did not have a single member in this entire cable system that was capable of taking compression, except the columns.

Every single member is a tension member, made of wire ropes, capable of taking tension only, furthermore made of material having an ultimate strength of 200,000 pounds per square inch as contrasted with ordinary 65,000 to 70,000 pounds structural carbon steel, so we did not have to worry about such things as the $^1/r$ reduction factor for compression.

Furthermore, with the elimination of the stiffening trusses, we had practically no wind area except the floor system because the entire cable system was just like a spider web. The nearest approach that I can think of is an ordinary bicycle wheel with tension spokes.

After we received the $2,500 our model shop made a model of a 200 foot span, scale of one to twenty shown in Figure 4, photographs of which are shown in Figure 5. We then made a little vehicle about 1 foot long to which were placed some lead weights underneath in order to represent a 25 ton tank as shown in Figure 5b; with additional weights placed on top of the vehicle, the weight could be brought up to 40 tons, 50 tons, 60 tons, 70 tons and 80 tons. We ran that tank as a 25 ton, 50 ton and 80 ton vehicle backwards and forwards over the model under various kinds of tests and found a number of remarkable results in the deflections at the various panel points.

It did not seem to make very much difference as far as the deflections at the individual points were concerned whether the verticals were left in or left out as the deflections were about the same. The panel loads in one case would be divided among three members, while if the verticals were left out, the same loads would be divided between the two diagonal members.

Therefore, the logical thing seemed to not to put the verticals in, or if the verticals were required for erection purposes, to make the ropes of smaller diameter in such a way that we could slacken them off and after erection leave them in place in case it was necessary to make some modifications later.

After we had erected the model we found that the cable system, and particularly the main cable at the top, was too strong proportionately for the rest of the structure, and in order to get the proper deflections, it was necessary to add something which would simulate the additional weight that would be necessary to bring the unit stresses in the cables about the same that they would be on the prototypes in actual bridges.

Therefore, we added the lead weights shown at the 9 panel points under the floorbeams and thus made the stress under dead load about 50,000 pounds per square inch in the cable. When the 80 ton tank was placed at any particular point there was a downward deflection at that point, and we had a stress of 80,000 pounds per square inch in the cable.

To take care of that deflection in the model we provided that the floor joints be articulated. One end of each stringer was connected to the floor-

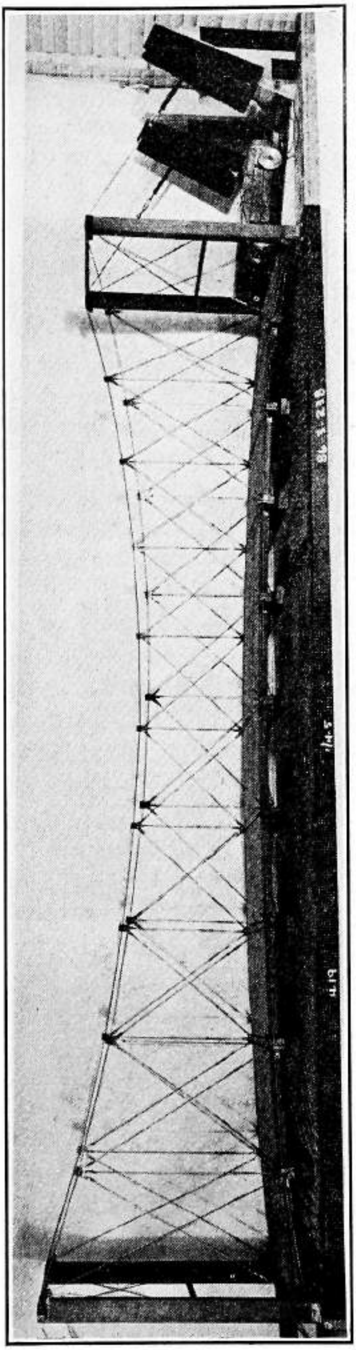


FIG. 5A

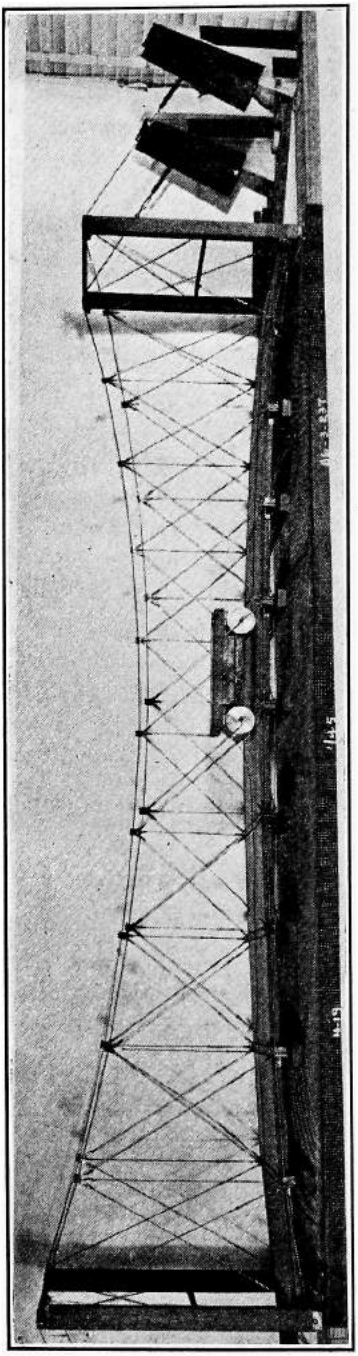


FIG. 5B

beam with two bolts, while at the opposite end the connection had only one bolt. So that articulation was provided at each panel point as that point went down, as the floor system would rotate around the single bolt.

When the 25 ton tank was out in the middle of the span there was no noticeable deflection that could be seen in Figure 5b and as far as wave action in the floor system was concerned it would not be noticeable to any person riding across a real bridge in the same proportions as this model.

As far as stiffness was concerned, we would stand at the center panel point and press laterally with sufficient force to lift the entire bridge off the supports on the near side, showing that as far as lateral stiffness was concerned we had ample rigidity in this model. And yet, outside of the towers, there was not a single compression member in the cable system.

In the meantime, the Roebling Company had been approached about Memorial Day, 1944, to construct a 60 foot model of a 600 foot span and when we submitted the proposition to them, we had another shock: that after all, we were not quite so good as we thought we were, because the Roebling organization had the same idea just about one year ahead of us. This came about in connection with three existing bridges that had been giving minor aerodynamic trouble. They never got to the point of failure as in the case of the Tacoma Narrows Bridge, but they did cut up sufficient capers so that they needed attention. One was Whitestone Bridge, another was the Thousand Islands Bridges, and the third was the Deer Isle Bridge.

The Roebling Company had been brought in as consultants on the strengthening of the Deer Isle Bridge and they started their studies in about the early part of 1943, so at the time that we contacted them in May, 1944, they had already completed their studies and practically all of the strengthening had been added to the bridge, except on one side span. When we came into the picture the particular model, that had been used for their studies in connection with the Deer Isle Bridge, was used for studies on the 60 foot model of the 600 foot span.

Roebling tried out various ratios of the sag of the cable to the length of the span on the pilot model. This model was about sixteen feet long, and had a depth at the center of the span from the main cables to the holddown cables, which at the start was about five and a half inches. They then ran their model vehicle across the span and measured the deflections. Then they reduced that center distance to five inches, measured them again and found that the deflections were less. Then they reduced the depth to four and a half inches, measured the deflection again and found that the deflections were still less.

Finally they came to a point where, if the depth was further reduced, the deflections began to increase. The result of those studies was used as a basis for the design of the 60 foot model of the 600 foot span.

The towers on the 1 to 10 model are about eight feet high; the cables are 33½ inches center to center; the span 60 feet. The tank that traveled across this bridge weighs 1,000 pounds corresponding to 50 tons; and by putting additional lead weights into the tank the weight was brought up to 1,600 pounds corresponding to an 80 ton tank, or 160,000 pounds, on the prototype 600 feet long. Photographs of this 60 foot model with the tank are shown in Figures 6 and 7.

That tank is moved across the span by a wire rope at a speed of one-half mile per hour, so that it takes just as long for this 1,600 pound tank to move across the 60 foot model as it would take an 80 ton tank to move across a 600 foot span at the rate of five miles an hour. We thus obtained a very comparable relationship not only in weight and deflections, but also in time.

In the 60 foot model, the main cable consists of four ropes, one-quarter inch diameter, the holddown cables are four ropes, one-eighth inch diameter and all of the verticals and the diagonals are one rope, one-eighth inch diameter. Roebling also discovered, as we did on our 10 foot model, that it did not make very much difference in the deflections at the panel points whether the verticals were left in or left out; therefore, the logical thing seemed to be to put the verticals in for erection purposes using smaller diameter ropes and then just before opening up the bridge to traffic, to loosen the turnbuckles and throw the vertical ropes out of commission.

One of the things that we found out, confirmed by Roebling on the 60 foot model, was that as the tank moved across the bridge the diagonals would go in and out of action depending upon the position of the tank on the bridge. As near as I can tell, the action is somewhat similar to an ordinary simple eyebar pin-connected span where, due to certain positions of the live load, the main diagonals go out of action and it is necessary to put in counters to take the stress, and I am assuming now that neither the main diagonals nor the counters will take compressive stress.

In the case of a riveted bridge where the diagonals are capable of taking compression as well as tension, it does not make very much difference. We simply make these diagonals strong enough. But in the case of our cable stiffened bridge, we had no members that would take compression, so that the diagonals would definitely be out of action. But, as far as the behavior of the bridge as a whole is concerned, and particularly in the roadway where the traveling wave certainly was not noticeable at all, the entire structure was very stiff.

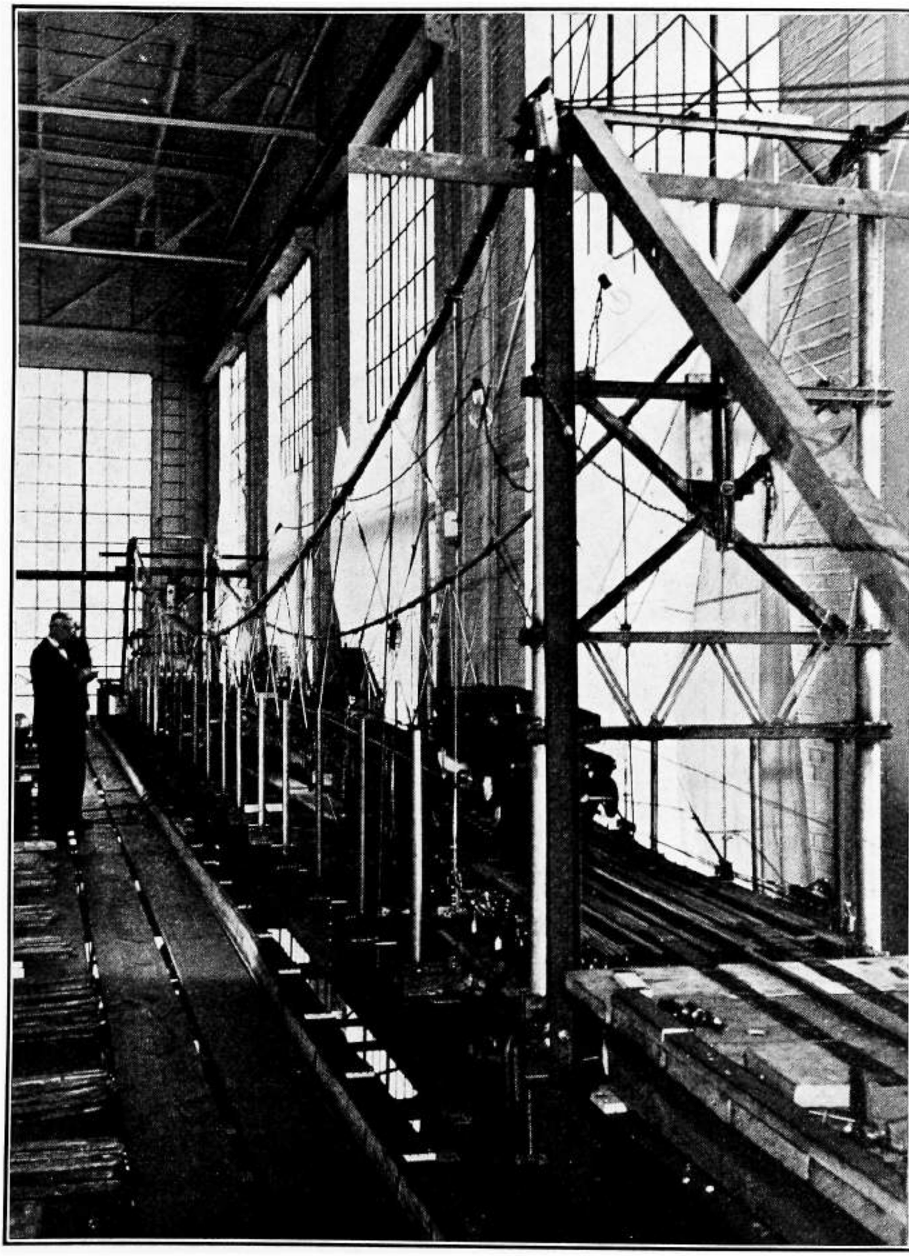
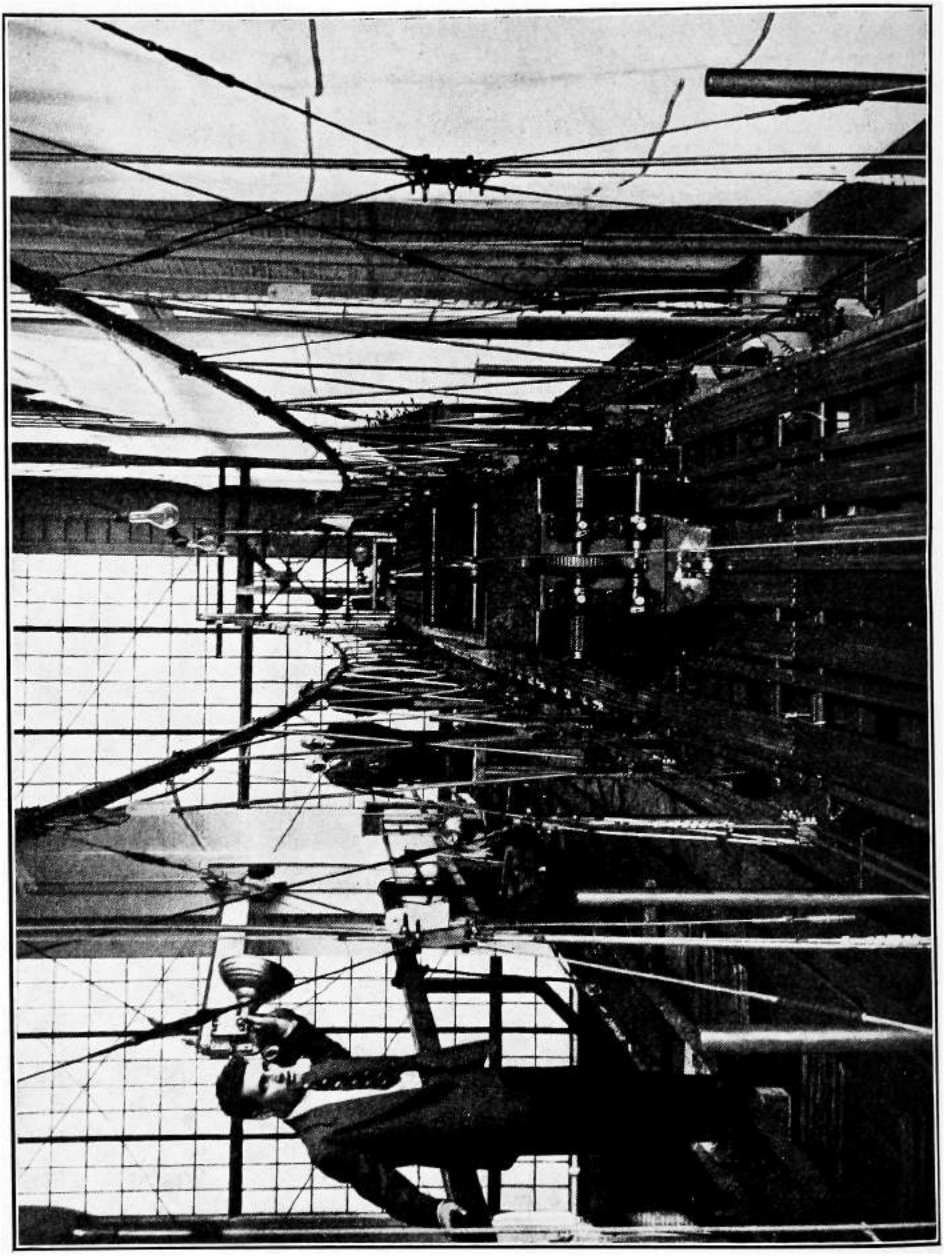


Fig. 6



At each of the panel points we had vertical pipes adjacent to the model and we attached ropes to the bottom chord, ran them out past these vertical pipes and over vertical sheaves, and to the ends of the ropes attached 25 pound weights, so that when 25 pound weights were placed at every panel point on the bridge, we had a horizontal force of 25 pounds acting at each panel point of the model. That represented a wind force of 80 pounds per linear foot on the 600 foot prototype.

We then attached a second 25 pound weight to the first, making 50 pounds, which meant that we had a horizontal wind force of 160 pounds per lineal foot on the prototype, and when we attached a third 25 pound weight there was the equivalent of a horizontal force of 240 pounds per lineal foot on the 600 foot prototype. From our design of the bridge itself, based upon actual areas, we did not see how we could get greater than 240 pounds per linear foot lateral force on any one of these spans from 150 feet up to 600 feet. So we felt safe in saying that, as far as the model of the 600 foot span is concerned, if that model would stand the equivalent of 240 pounds per linear foot on the 600 foot prototype, any span of length less than 600 feet would correspondingly stand 240 pounds per linear foot.

The lateral deflections at the center, on the model, were one inch for 25 pounds, two inches for 50 pounds, and three inches for 75 pounds at the panel points. Now, a three inch lateral deflection on a 60 foot model would be equivalent to a 30 inch lateral deflection on a 600 foot span, which is not excessive.

The principles that we discovered are applicable to commercial bridges, but what I am telling you now is based upon military loads, and on bridges for military use; however, even a lateral deflection of 30 inches on a 600 foot span would not be pronounced when you consider the fact that we are dealing with ropes.

On our 10 foot model mention was made about the articulated joints. On the 60 foot Roebling model the outside stringer at one end is connected to the floor beam with a straight riveted connection. At the other end, it is cut short and connected to an extension from the floor beam, as shown in Figure 8a, by means of a pin, and that arrangement provides for complete articulation of movement as there is a deflection at all the panel points during passage of the "80 ton" tank.

The holddown cables on the Roebling model were subjected to a prestress of 2,500 pounds which represents a stress of 250,00 pounds in a 600 foot prototype. In our model at Fort Belvoir the horizontal force was transmitted into the bottom of the columns by direct connections, but there we had a very definite rigid support. We did not know what condition we were going to meet out in the field, so that Roebling took the 250,000 pounds in the prestressed holddown ropes into the end panels of the outside stringers and connected them in such a way that the center of the pin was seven inches below the center of the stringers as shown in Figure 8a.

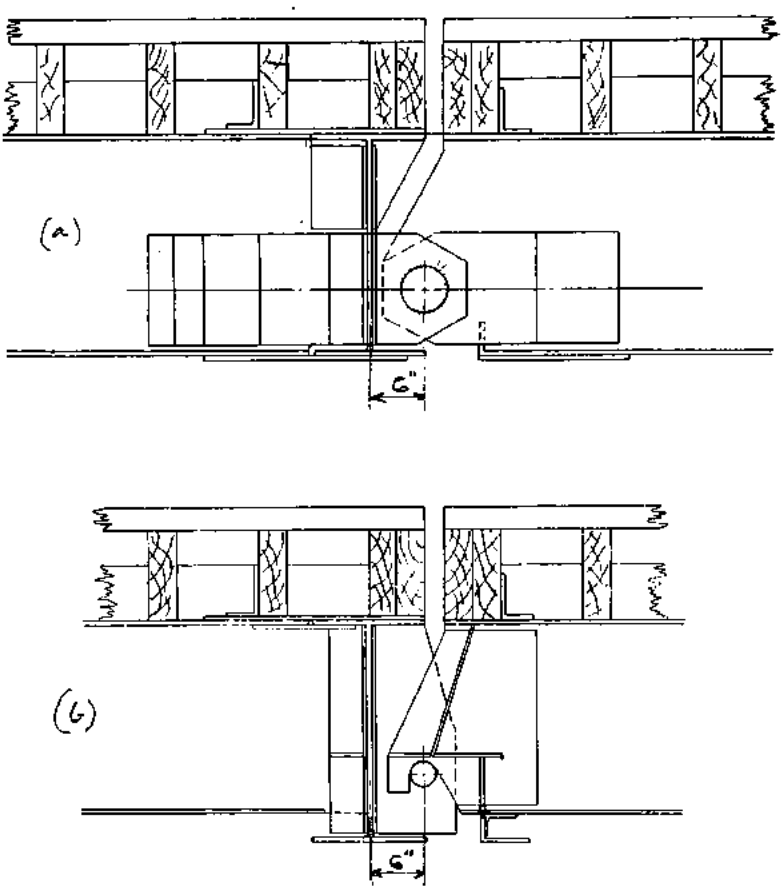


Fig 8 Scale 1"=1'-0"

Putting that force below the center line resulted in bending the stringers up, so that there was a little greater compression in the bottom flange than in the top flange of the stringer, from dead load only.

As soon as a live load went across the bridge, the effect from stringer action alone was the reverse. There would be compression in the top flange and tension in the bottom flange. So the result was that for the dead load stress in the outside stringers, we had a definite compression of 250,000 pounds on a 600 foot prototype, distributed in such a way that the unit compression was a little greater in the top flange than in the bottom flange. As soon as live load got in any particular panel, the tendency was to reverse that condition. In other words, the stresses were additive algebraically and Roebling took advantage of that principle in the design of the floor system.

With the intermediate stringers, there was no such horizontal force that had to be transmitted, so that all that was necessary to do was to have a pin of sufficient size to take the vertical reaction of the stringers, as seen in Figure 8b. Intermediate stringers however were not used on the Roebling model.

Down at the Engineer Board, after considerable experiments had been made with the models, there was a lot of interest in Washington. Therefore we gave demonstrations for any visitors from Washington who were particularly interested in bridges, when they visited the Engineer Board. There were Canadians, Russians, French, Australians and English, and we had some quite noted men there. Among them was Major General Steele of the Australian Army, who was Chief of the Engineering Section; and we had Major General Hughes of the English Army, who was very close to General Eisenhower. The cable stiffened idea created a lot of interest with bridge specialists.

In the meantime, after we had proceded to a certain length with our studies up at the Trenton Plant, we arranged for a general meeting of any military engineers who were interested to come up to Trenton and we put on a demonstration at the Roebling Plant in October, 1944. Among them were members of the General Staff of the United States Army, representatives of the New York District and the Philadelphia District, and representatives of the Canadian Army.

As a result of our studies at the Engineer Board supplemented by the investigation made by Roebling the most satisfactory profile to date is shown in Figure 9.

We were definitely trying to put the cable stiffened idea across because, thus far, we had only the advantage of the models. But the proof of the pudding is in the eating, and we were trying for a real bridge, since no matter what the models show, or no matter what the opinions of any individual engineers are, there is nothing like a real test on an actual bridge.

Therefore, we wanted to get as much information as possible disseminated throughout the bridge profession in the United States to see if we could smoke out any real objections to this type and find out if there were any flaws in it. Because, after all, we are human and it might be that there was some simple thing, that those of us who had been connected with it so intimately might have completely overlooked, that would have shown the whole idea to be unsound.

So we contacted engineers all over the country and among them was Professor Farquharson from the University of Washington. The Professor, in writing, commented upon this point: Suppose there was a load of 25 ton tanks in the near lane on the right half of the bridge, and that there was another load of 25 ton tanks on the far lane on the left half of the bridge. What kind of a warping or twisting action were we going to get at the center of the span?

We, therefore, put the equivalent of 25 ton tanks on the ten foot model at Fort Belvoir, and asked Robling to do the same thing with a series of distributed weights that they had on their model at Trenton. While it was

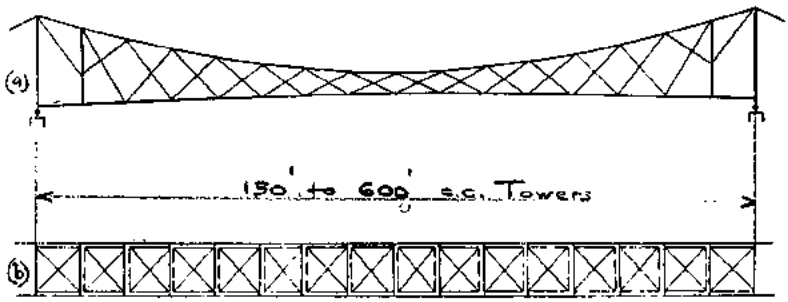


Fig. 9. Scale—None

true that the near right half of the bridge would deflect more than the far right half and the far left half would deflect more than the near left half; as far as the center is concerned, there was a difference in elevation of about a hundredth of an inch. You could hardly see it.

One of the reasons for that, I am satisfied, is the fact that we had articulated joints; that you could not really get enough rigidity in the floor to show up as far as twist was concerned. We had the movements, yes, but the articulated joints were such that the twist at the center of the span was eliminated.

That was just one of the questions raised by bridge engineers in different parts of the United States.

Thus far, I have not said a word about aerodynamic action. At the Roebling plant, they erected a wood frame over the bridge at one of the quarter points and we will call that panel point 5, there being twenty panels on the bridge. That frame was independent of the span itself. At the top of the frame, there was an inverted plunger, about three and a half or four inches in diameter, and we let air in the cylinder at fourteen pounds pressure. The bottom of that plunger was connected by a rope to the center part of the floor system at panel point 5 and, naturally, when the air was applied in the cylinder the plunger would go up and carry that part of the span up with it, while the opposite end of the span went down.

When the span went up to a certain distance, the air pressure was cut off electrically and, of course, the plunger went down and carried that part of the span down with it while the opposite end went up, and when the span got to the bottom of the stroke, the air was reapplied electrically. The result was that with a constant application of air and cutting off of the air, we had that part of the span vibrating up and down, while the opposite end also vibrated up and down with a node in the center of the span.

The remarkable part about it was that when we had all the verticals in and no diagonals, that span vibrated as a two-segment wave. In other words, the wave would go up at the left end and down at the right end and then go down at the left end and up at the right end. We had the Tacoma Narrows Bridge Action perfectly illustrated on the Roebling model. With all verticals and no diagonals the wave amplitude at each quarter point was about four and seven-tenths inches.

We then put in two diagonals at the two panels adjacent to the center and took out the center vertical and repeated the operation. Again we had a two segment wave, and the amplitude was a little greater, going up to about four and nine-tenths inches.

Then we put in two more diagonals, adjacent to the first two and had four panels of double diagonals at the center and took out two more verticals, when the wave amplitude was five inches and we still had the two-segment wave.

As we put in double diagonals in each panel and took out the verticals, working from the center towards the ends of the span, the wave amplitude began to decrease and went down from five to four and a half, four, etc., and finally dropped down to about two inches when we had all but the last panels of diagonals in, but we still had the span vibrating as a two segment wave.

As soon as we put in the end panels of diagonals and completed the system, the node disappeared and the span vibrated as a single-segment wave, over the entire length of the 60 foot model with an amplitude of one and seven-tenths inches, which was a very significant thing, proving conclusively the effectiveness of the double diagonals in each panel of the bridge. In other words, you cannot have a Tacoma Narrows failure with this type of bridge, as it seemed to be impossible to get this bridge to act as a two-segment wave. It just would not do it. We tried to hold it at the center and start it vibrating as a two segment wave but as soon as released it would immediately vibrate as a one-segment wave. It just seemed to be impossible to make it vibrate as a two segment wave with all the diagonals in place.

In contacting the engineers over the different parts of the country, we ran across an article by Professor George Maney of Northwestern University, which was written in April, 1941, about six months after the Tacoma Narrows failure; George Maney and I have been friends for thirty years. He tried out various models, and remember, this was done within six months of the failure of the Tacoma Narrows Bridge.

He used the ordinary suspension system with a stiffening girder, and with a load at the quarter point, one-half of the bridge went down and the other half went up. Then he suddenly released the load and the vibrations at this point were similar to those of the Roebling model with all verticals and no diagonals.

I do not know how much you gentlemen have done with harmonic motions, but it so happens that this curve represented by the magnitude of the vibrations is a logarithmic curve. So, we speak of the decrease of the vibrations as a logarithmic decrement. There were quite a number of vibrations in this particular model of the conventional suspension span, with a small logarithmic decrement.

When Professor Maney put in the system of double diagonals in each panel, he reduced the deflections to a very small amount. As far as the vibrations are concerned when the weight was suddenly released the magnitude of the vibrations was materially reduced, as was also the time required for the dampening out of the vibrations, proving conclusively that the introduction of a system of double diagonals very definitely had a dampening effect upon the vibration of the bridge if it started vibrating in the wind. In other words, it had a high logarithmic decrement and as I said Professor Maney did this three years before we got into the picture.

I wrote a letter to Professor Maney and asked his permission to reproduce part of his *Engineering News Record* article of April 24, 1941, here; I received his consent and also have the consent of the *Engineering News Record* to produce it.

Down at the Engineer Board, we did not have the facilities to produce any records like Professor Maney made, but, fortunately, Roebling did, and I have the records here. They are so light that I could not get lantern slides made of them, but after the talk is over, if any of you gentlemen are interested, you can come up to the platform and we will unroll those records so that you can see actual records of the vibrations of the Roebling model. However, the principle thing that I want to bring out is the fact that the logarithmic decrement is very high with the cable stiffened type. And, in the case of the Roebling model, no matter what the vibration was, it would dampen out in about forty seconds.

At this point I wish to read briefly a few of the conclusions that Professor Maney mentions in his article:

"The change in the structural action of the cable which is transformed into the top chord of a continuous stiffening truss for a part of the live load effect is almost as surprising as the tremendous dampening increment produced by the hanger system as compared with the ordinary dampening effects."

"The stiffening truss, as such, becomes no longer necessary, and for a small part of the saving, due to the elimination of the average size stiffening truss, a continuous truss is provided of an entirely different and a greater order of stiffness."

And I will say, gentlemen, that all of the experiments that we made at the Roebling plant over a period of one year have confirmed the statements that George Maney put in the *Engineering News Record* back in April, 1941. We really do not have a real suspension bridge but the structure is really a variable truss where the main cable is the top chord, the hold-down cable is the bottom chord and the diagonals and verticals comprise the web system and these go in and out of action for different positions of the live loads.

But we still have not got a bridge. I mentioned sometime ago that I tried to have a real bridge built and among the places that I visited was the New York City District Office. It so happened that the Chief Engineer, Lieutenant Colonel Panish, was one of my former students at the Evening Graduate School of the Brooklyn Polytechnic Institute back in 1930. So Colonel Panish said to me, "Bill, I will go to bat for you and see if we can find a place where we can build one." This was in May, 1945.

He called me up two or three weeks later and said, "I have got it. I plan to build a bridge on the military reservation at West Point. And, I will make you a proposition. I can build a 600 foot bridge there consisting of 60 foot spans, but I will build one of your bridges provided you folks (meaning the Engineer Board) will pay the difference in cost between whatever the cable suspension bridge would be and the cost of a bridge that I can build." That was fair enough. After all, we were going to spend the money to build a sample bridge, so we accepted his proposition.

He agreed to take care of the foundations, where, fortunately, we had rock, which was perfect. The Engineer Board agreed to furnish the steel work and the cables for the superstructure, and we entered into an agreement with the Commandant at the U. S. Military Academy to have the bridge erected by troops. It was intended that the contract would be signed around the end of June, 1945, but the Army moves with becoming dignity and poise, so we were just a month late and the contract was finally signed on the 30th of July.

Fourteen days later came V-J Day. Everything was suspended, including the West Point Bridge, which bridge was finally cancelled, so we still have not got a real cable stiffened bridge.

I have worked a year and a half on a theoretical investigations, and I am frank to confess, gentlemen, that I just do not know how to figure this bridge analytically. At the same time, if we are going to build these bridges, we cannot afford to build a pilot model every time we build a bridge, particularly in spans of short length.

I have, however, a lot of confidence in the possibilities. Our tests have proven conclusively that this type will satisfactorily carry the loads without excessive deflections.

In order to stimulate interest, I am going to offer a prize of $100 to be awarded at the next Annual Meeting of the Connecticut Society of Civil Engineers in 1947 for the best paper that is presented that will give a satisfactory analytical solution of the stresses of the cable stiffened type and I am going to make that competition open to four different groups.

First, it is open to any member of the Connecticut Society of Civil Engineers; second, to any member of the faculty or student body of the University of Connecticut; third, to any member of the faculty or student body of Yale University, and, fourth, to any regular member of the State Highway Department, except myself.

Thank you. (Applause)

DISCUSSION

Chairman Sumner: Get your slide rules early; the supply will shortly be exhausted.

I know Mr. Grove will welcome questions and general discussion of the paper to which you have just listened. Does anyone want to ask a question or two?

Professor Castleman: How do you measure your deflections on your model?

Mr. Grove: The vertical deflections were measured by rod and level. The movements of the tops of the towers were measured by Ames' dials. The stresses in the various members, cables, hold-down ropes, verticals, and diagonals were measured with the Baldwin-Southwalk electric strain gauges reading to micro inches so that we were able to get the change of stress, that is, the difference in the stresses due to the live load by the electric strain gauges. Does that answer your question?

Professor Castleman: Yes.

Chairman Sumner: Are there any others? Then I will take the liberty of asking one, if I may. I take it that this would effect a very appreciable economy in the construction of a suspension bridge?

Mr. Grove: Well, let us take the 600 foot span. If we use the ordinary conventional design for the military bridges, there was a weight of floor of 600 pounds per linear foot, and as I recall, a weight of 1,600 pounds per linear foot for the steel trusses that would enter into the calculations. That made 2,200 pounds per linear foot. Of course, the 600 pounds per linear foot in the floor was the same in either case, but in the cable stiffened type we cut that 1,600 pounds in the steel all the way down to 900 pounds per linear foot in the 600 foot span. In other words, we were designing for 1,500 instead of 2,200 pounds per linear foot dead load.

Now, I am glad you mentioned that because it is related to something that I forgot to mention in my talk. Generally conventional suspension bridges have a sag ratio of approximately one to ten. That is, for a six hundred foot span, there is a 60 foot sag. The length of the cable on a 600 foot span with a 60 foot sag works out very close to 616 feet. If we cut that sag in half and made a one to twenty ratio with a sag 30 feet on a 600 foot span, then the actual length of the cable is not 608 feet, but it is only 604 feet, which was a very important thing because the actual length of the cable is one of the main factors in determining the magnitude of the deflections. So if we cut that extra length from sixteen feet all the way down to four feet, you can see that the magnitude of the deflections would be materially reduced.

Now, in the designing, the moments, of course, are the same. If you divide the moment by a 60 foot depth, you get a certain horizontal component, and if you divide by a 30 foot depth you get twice that component. But we had eliminated enough of the dead load in the cable stiffened type to take care of that excess of horizontal component so that the same number of ropes could be used in the cable in either case. The West Point Bridge, by the way, was to be a span of 460 feet. For more complete data I refer you to my article in the *Engineering News Record*, March 7, 1946.