easy enough to provide a suitable section for heavy loads, but with light loads it is a different matter. In our building designs of to-day, for instance, the columns in the top story of any building, a 5-in. **H** section would support every load, but you cannot get it and what will you use in its place? As a matter of fact, I doubt if the manufacturers of steel have kept pace with the demand for economical sections along these particular lines. I understand that the Illinois Steel Company are developing these sections, and are making rolls for a new set of beams throughout; also that they are developing some very small, light sections, and are changing the moments of inertia and the weights of standard beams from 8 to 24 in.; so that very soon we shall have a new series of steel structural shapes to figure on. This is all right for floor construction, but what I am after is something that we can use for columns and struts, which is economical; where the steel can be actually stressed up to a working limit of, say, 10,000 lb. per sq. in. Heavy column sections may be built out of a number of structural shapes, but in the light sections we are compelled to waste material, no matter what we use.

Albert Smith: I am quite unequal to suggesting any section that would be thoroughly economical in the case which Mr. Davidson mentions. We use channels, light channels, with angles at the bottom in an endeavor to make the actual fiber stress come as near as possible to that permissible in such a case. It has sometimes occurred to me that it would be possible to use a pipe section in cases of that kind. If we could get pipes 18, 20, or 24 ft. in length of thin material, and could manufacture a standard detail for the end, to be threaded onto the pipe, we might be able to carry these very light loads economically. I think we are too much afraid of doing shop work, in general; we think a little too much of the cost of coping and of blacksmith work. With us, with the very excellent machinery that we use, he cost of shop work cannot be anywhere near as great as with English manufacturers. I refer especially to the small compression members of a bridge brace—those which we now make of single angles. English designers use T-bars almost exclusively for such struts, cutting off the outstanding leg, and I think such members could be used economically in our trusses. The cost of coping would not be great and we should avoid, in most cases, the gusset plate. Nine times out of ten two rivets are sufficient for a connection, and the two rivets could be driven through the remaining leg of the T-bar into the top chord.

In regard to large compression members—those to which Mr. Horton has referred—I wish that such tests could be made by the government, and that when they are made they would parallel the case of the members in the field—for instance, the vertical posts of railway bridges. It seems to me they would be better if tested in connection with the floor beam, and it does not seem impossible to put bending in that floor beam while the test is being carried on. It should be braced with a knee brace at the other end in just the way the bridge in actual service is braced, and then we might find an answer to a question which is as yet entirely unanswered in specifications or in practice: What constitutes free length? I have in mind a set of specifications which permits three definitions; fixed at both ends, fixed at one end and free at the other, and free at both ends and allows the free length to be made one-half, two-thirds, or the whole of the distance between connections on that basis. I do not know what constitutes a fixed-end connection in an actual structure; it seems to me it would be very hard to determine, and also very hard for anyone to say in a given case that he is willing to take one-half the length between connections as the free length of his member. There are so many things that enter into the stiffness of the member, in the connection, the yielding of the support, the rigidity of the attachment, that it seems as if a test, which would simulate actual conditions and show the actual value of the compression member under those conditions, would be very valuable. The difference in the permissive fiber stress is likely to run as high as 25% or 30%, according as you vary the free length—that is, supposing you use the ordinary formula, 16,000 minus 70 $\frac{l}{r}$, and 30% is a rather important amount in the design of your member.

I have no suggestion that bears directly upon the subject that the chairman has in mind.

Henry E. Vanderlip: In listening to Messrs. Horton and Smith, a few things have occurred to me. First, in regard to the question of wrinkling referred to by Mr. Horton. I have seen columns with that wrinkling between rivets, produced by heavy loading. There would seem to be a bulging out between the rivets, and yet the rivets had been spaced according to the so-called practice, which ordinarily is that no ¾-in. rivet—which is the standard rivet used in bridge and building construction—shall be placed at a greater distance than sixteen times the thickness of the thinnest outside piece of the member. This would mean that a ⅜-in. plate would be the minimum thickness where 6-in. spacing could be used, and a 5/16-in. plate, according to the rule, would permit not greater than 5-in. spacing. Yet, if you use that same 6-in. spacing with a ⅜-in. plate under some conditions of loading, you will get a bulging or wrinkling of the plate. According to the formula, it should not bulge. That is one of the peculiar things in the construction that nobody has ever seemed to explain.

The subject that the chairman brought up, about the advisable form of column for light loads and long lengths, is one which has not often been brought before the engineering profession. We might consider two forms: First, a post or column under four points supporting a water tank—a gravity tank—placed 25 ft. (approximately) above the roof of one of the standard warehouses as they are built in Chicago. There are all kinds of forms of columns used, and it does seem as if the engineers should agree on some standard form that seems to be the most economical for that purpose. You will see a dozen different forms to carry, say, a 25,000-gal. water tank. I have known of cast iron being used for that purpose, but personally I would not select it. I have seen two channels used so as to form, with cover plates, a box-shaped column. Then I have seen four angles with a single web plate, which we all know has unequal radii of gyration on the two axes. Then I have seen star-shaped forms, and the star may be formed by using two or four angles—two angles of sufficient area to take a light load—and increasing those two angles in thickness and in the size of the leg of the angle until we reach a point where good judgment would say you should use four angles as the load increases. I think this is a very good form of column for that purpose, for the reason that a water-tank tower is essentially a wind-braced structure. Therefore, since it is a wind-braced structure, one can arrange the position of the wind bracing horizontally and diagonally so that the question of the radius of gyration does not really enter into it. Another thing, the gusset plates can be arranged in between the angles, and the connections, from a bridge shop point of view, are about as good as can be obtained.

The other form I have in mind is a long column. Imagine a column, say 40 ft. long, in the middle of a large and high room, with an upper floor and roof of the building carried on that long column. According to the building ordinance, if the column is to be fireproofed with tile where