posts, while at the same time their flanges are riveted to the flanges of the arches. When the latter cross the posts obliquely, the cross-channels have to be heated and swedged in a block by pressure, so as to fit the oblique section. By using wedges of different shapes to correspond to the inclination of the arch, one press or one pair of blocks or dies will do for the whole. If the cost of such a press is objected to, the flanges of the cross-channels may be cut out and the stem only bolted to the post.

So far as the arches rise above the upper chords in the central span, the upper channels are connected by a top-plate of 14" thick riveted to the upper flanges. Laterally, between the posts in each panel, the arches are again connected by 4 channels of 50 inches length each, and also by stay-bolts passing through gas-tubes. It would appear that ample provisions have been made to insure the lateral as well as vertical stiffness of the arches.

Where the arches rise above the upper chords, the channels forming the latter are cut out and planed off to the proper level to make a good fit. Splicing-plates are then riveted over the joint, and the respective flanges are also connected by rivets. At the same time a plate of 4 feet long by 2 feet wide and 1 inch thick is riveted down on top, so as to cover all the joints. The depth of each arch is 41 inches, and its width 4 feet.

Considered as a straight girder freely spanning a distance of 60 feet, this combination would be stiff enough for railroad traffic. But its inherent stiffness is not calculated to serve as a supporting power, but to resist lateral deflection like a column. A round column is of course the best geometrical section for gaining lateral strength. But a columnar section in wrought-iron for the purposes of an arch, presents many difficulties and objections. What would be gained in section would be more than lost in reduced strength of framing.

The great facilities of construction which the plan before us offers, must not be underrated. And when we consider the small section of wrought-iron that is needed to make the arches effective, this plan will compare favorably with other plans heretofore attempted. The arches of the Coblenz bridge over the Elbe are 228 feet wide in the clear. Each rib is 10 feet 6 inches deep, with upper and lower flanges. The flanges of the centre rib are 3 feet 2 inches, and those of the outer ribs 2 feet 2 inches wide. These arched ribs have been treated as flexible trusses, consequently the upper and lower flanges have been graduated like chords, with the greatest section in the centre. The upper and lower chords are connected by posts and diagonal braces. This work is justly considered a model work; it is well proportioned and possesses ample strength. The stiffness is in the arches alone, and no attempt has been made to increase it by spandril-bracing. Its rigidity is also owing, in a great measure, to the placing of the floor below the crown of the arch.

The question arises, whether flanged arches with greater inherent stiffness would not be better adapted to the Parabolic Truss, than the arches here designed? I do not hesitate to answer in the negative. So far as simple supporting power is considered, under the action of uniformly distributed loads there would be no difference between the two plans. As regards stiffness, under the action of variable loads, however, the plan here proposed, with its great depth of trussing, will produce more rigidity, with the same amount of material expended, than would result from flanged arches. This view will apply with still greater force, when the arch descends below the lower chords. The depth of ribs in the Coblenz bridge is 10 6", and the clear span 306 feet [Prussian measure].—a proportion of about 1 in 50. This depth would evidently be entirely too small for a straight girder of 308 feet span, but it would be enough for a span of 100 ft.

In the Coblenz bridge, the floor intersects the girders or ribs at the lower chord, and we may therefore consider the whole arch, so far as stiffness is concerned, as divided in three equal parts. The central part is stiffened by the floor, and the spandrels are stiffened by the posts which support the floor. But no further bearing has been attempted in the spandrels, on account of contraction and expansion; but this appears to be a defect, because an expenditure of little additional material would have greatly added to the rigidity and strength of the work without materially interfering with contraction and expansion. It must be remarked, however, that the leading idea in the design of this celebrated viaduct was to interfere with the arches as little as possible, and to depend upon their own inherent stiffness, so that the ribs should be at liberty to accommodate themselves freely to variations of temperature. This same view led to the planning of the pivot skewbacks; but this defect has been corrected since by making the bearings of the skewbacks all solid.

When we omit the arches in our plan, considering the structure as a pure suspension-bridge, and compare its stiffness with that of the Niagara bridge, we discover that the truss here designed possesses ample stiffness for all railroad purposes. The Niagara trusses are only 18 feet high; those before us are 22 feet, with a span of 500 feet; while the Niagara span is over 800. In the Niagara bridge the truss-posts are 5 feet apart; here they are 20 feet. The parabolic trusses being double, there are four diagonal rods of 14 in. diameter in a panel of 20 feet, while there are 2 rods of 14 in. in a panel of 5 feet at Niagara. On the other hand again, the Parabolic Trusses are all of iron and have comparatively very strong chords, while the Niagara framing is of wood with light chords, which are, however, very materially assisted by the floors and the central girders, which latter distribute the weight of concentrated loads.