

(*Paper No. 3876.*)

“The Strengthening of the Roof of New Street Station,
Birmingham.”

By WILLIAM DAWSON, M. Inst. C.E.

THIS roof, erected in or about the year 1852 over the old portion of New Street Station, has a span ranging from 191 feet at the east or London end to 212 feet at the west or Crewe end.

Its construction has been described in the Proceedings,¹ and at the time of its erection its span was referred to as “the largest hitherto attempted,” the nearest to it being the roof over Lime Street Station, Liverpool, of 150 feet span. The latter, however, was replaced by a roof of 220 feet span in 1871. Both roofs belong to and are maintained by the London and North Western Railway Company.

Each principal of the Birmingham roof, of which there are thirty-six, is of the bowstring girder type, consisting of a rectangular arched rib, a tie-rod 4 inches in diameter, and vertical members with intermediate cross bracing (Fig. 1, Plate 3). The rib is splayed out at the bearings with cast-iron distance-pieces, giving a bearing-surface of 20 inches by 15 inches.

The principals are 24 feet apart from centre to centre (Fig. 2, Plate 3), with one end resting on the station-walls and the other on cast-iron columns. The height from rail-level to the crown of the arched rib (exclusive of the lantern or jack-roof) is 75 feet. The arch has a rise of 42 feet above the springing, and the tie-rod a versed sine

¹ J. Philips, “Description of the Iron Roof, in one Span, over the Joint Railway Station, New Street, Birmingham.” Minutes of Proceedings Inst. C.E., vol. xiv, p. 251.

THE STRENGTHENING OF THE ROOF OF NEW STREET STATION, BIRMINGHAM.

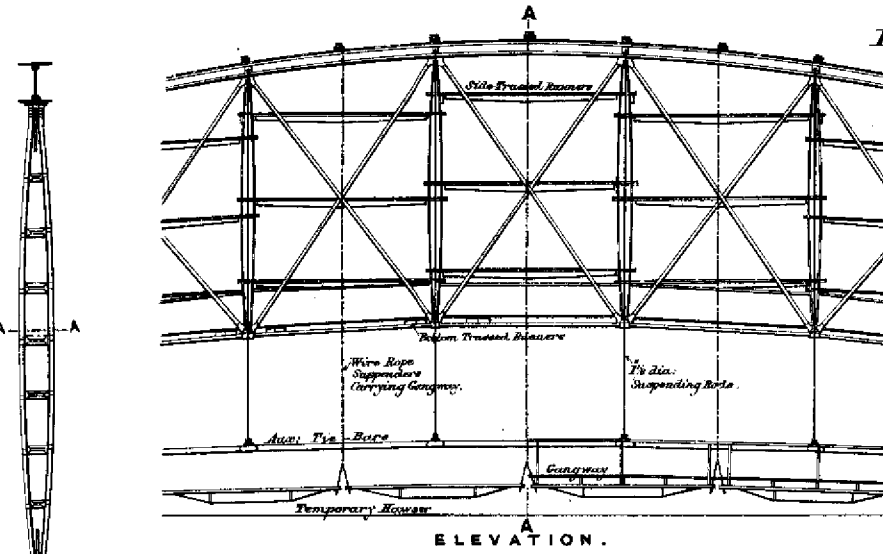
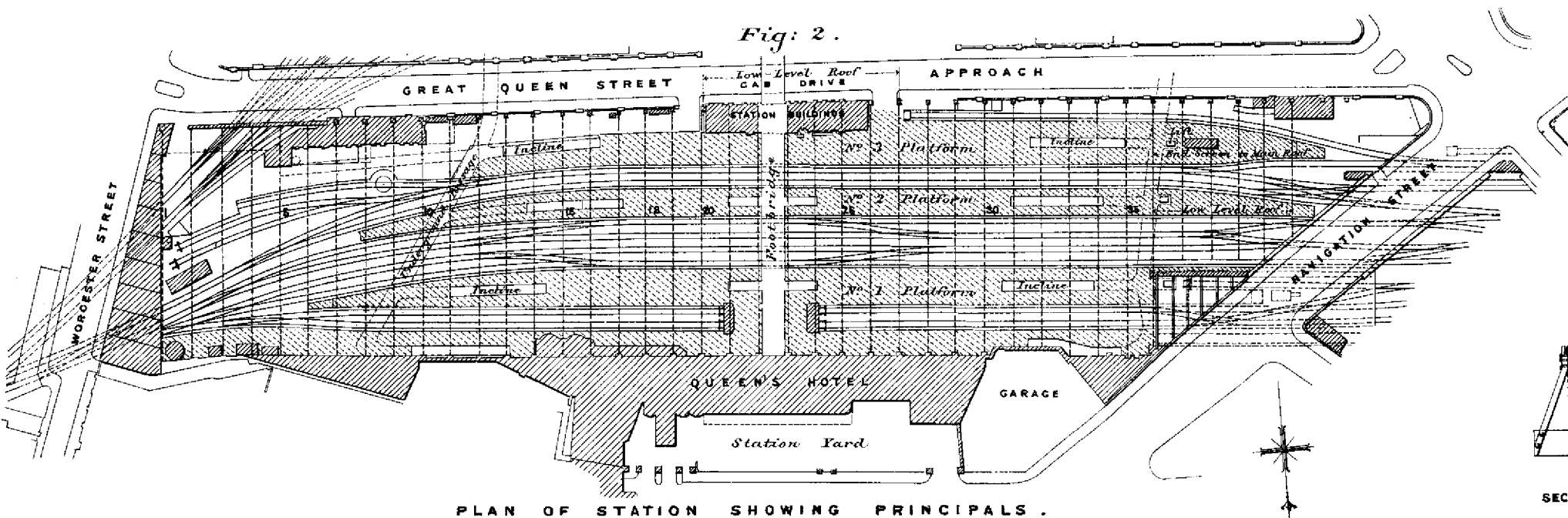
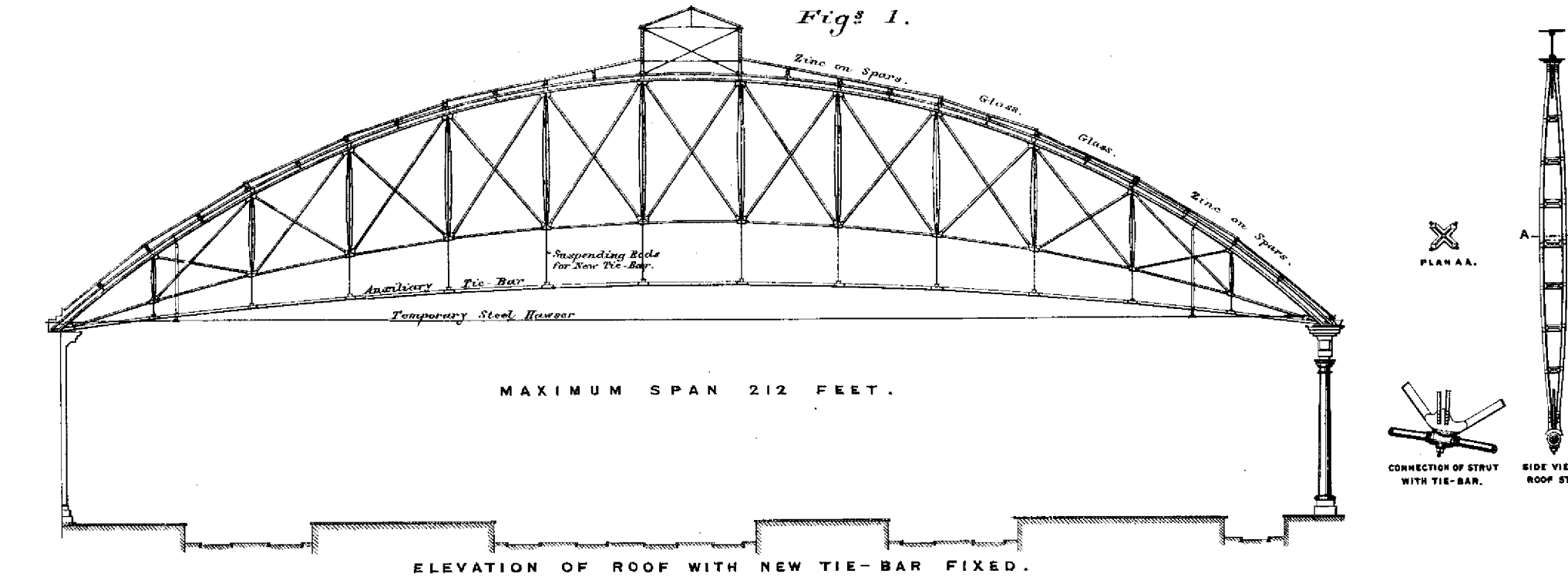
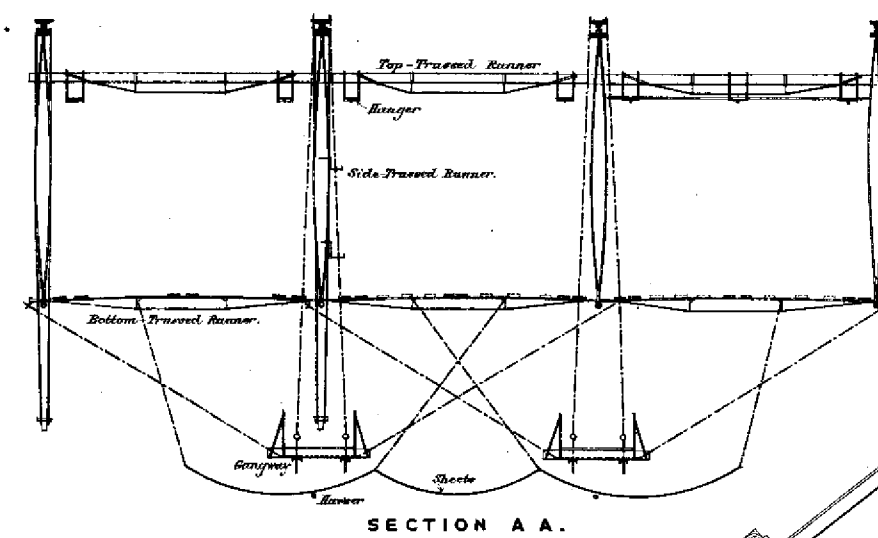


Fig. 4.



ARRANGEMENT OF SCAFFOLDING.

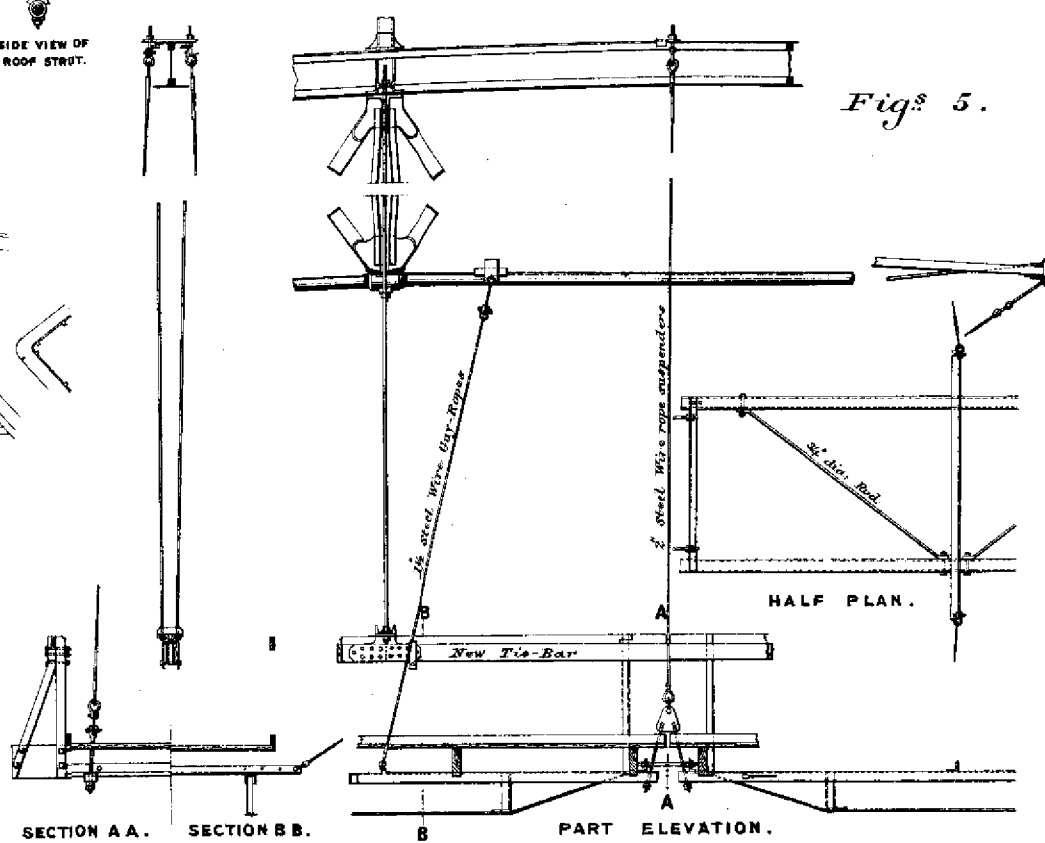


Fig. 5.

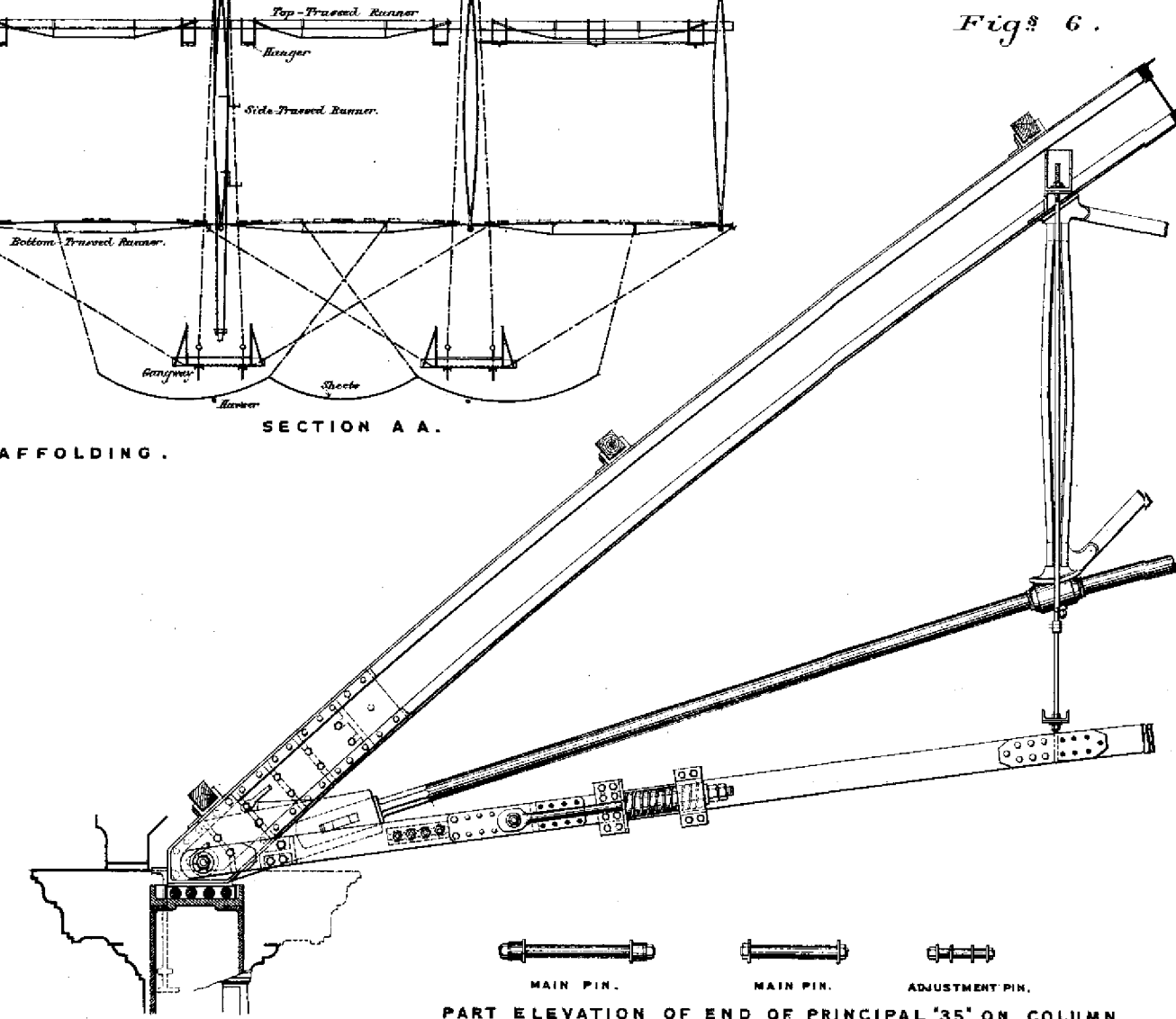


Fig. 6.

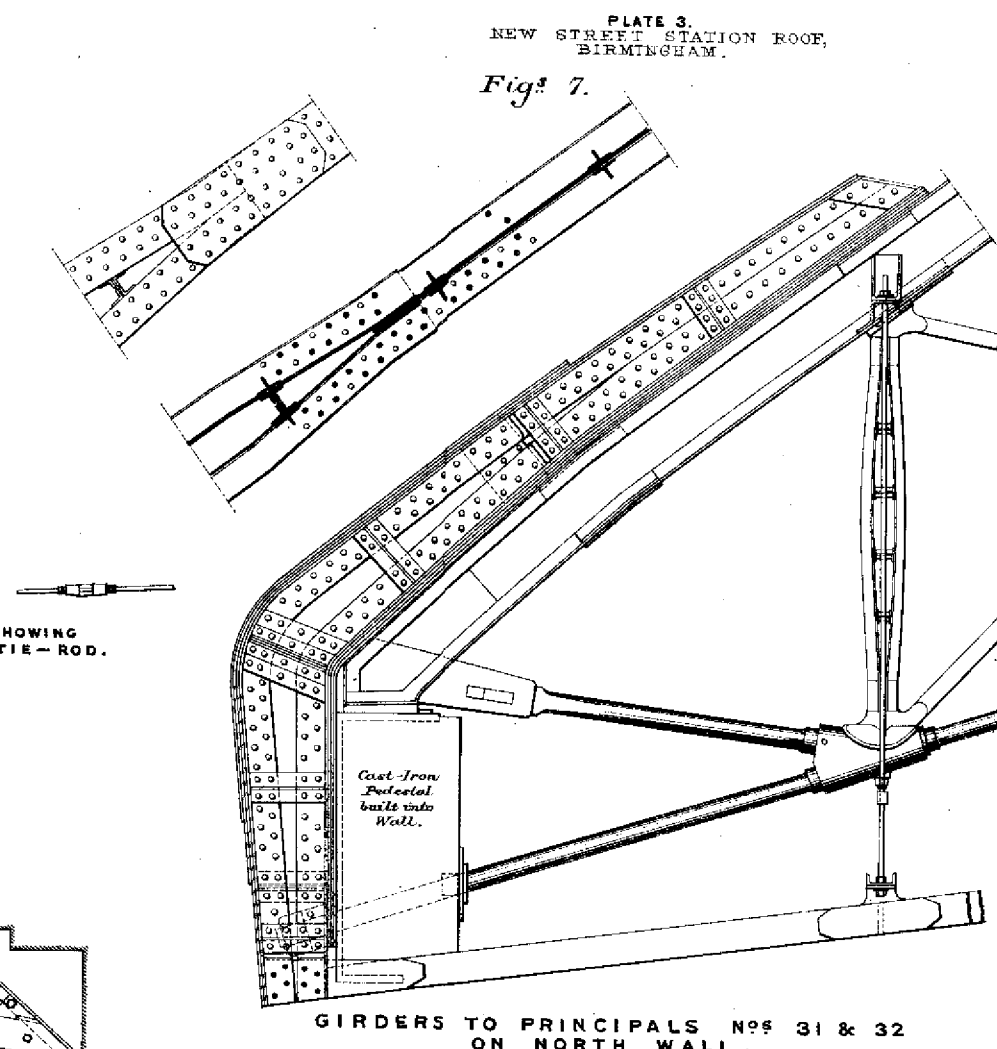
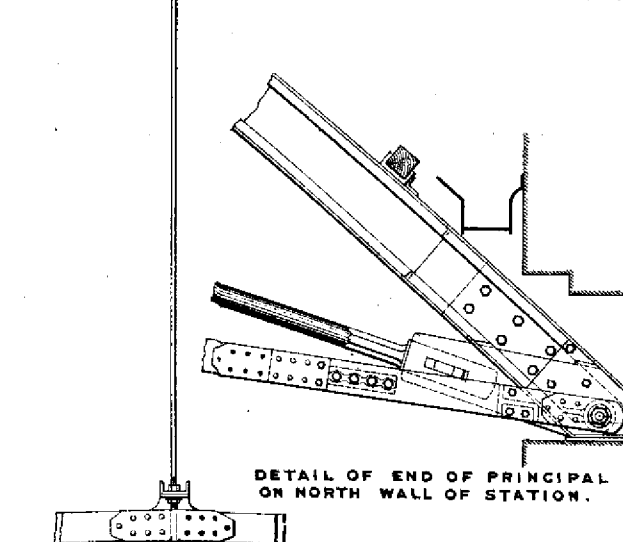
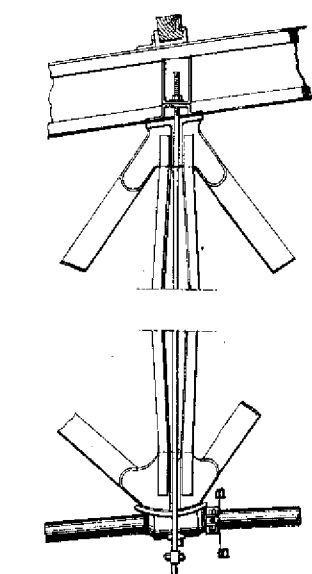
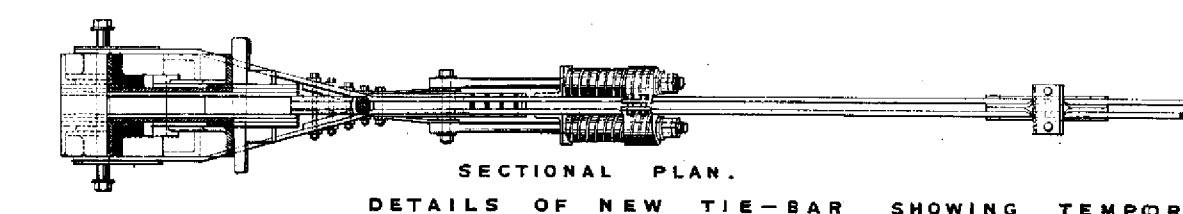
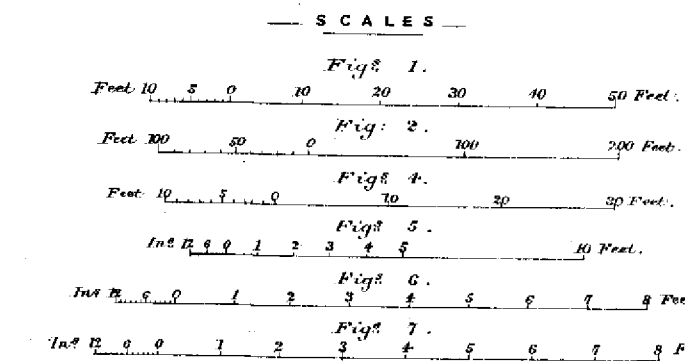


PLATE 3.
NEW STREET STATION ROOF,
BIRMINGHAM.

Fig. 7.



W. DAWSON.

of 18 feet. It was erected by means of travelling stages from the platform, Messrs. Fox, Henderson and Company, of Smethwick, being the contractors. The covering consisted partly of corrugated iron and partly of glass, but owing to corrosion the former was replaced by zinc on spars, as shown in Figs. 1, Plate 3, the alteration being completed in 1873.

NECESSITY FOR STRENGTHENING.

This roof is practically of the same design and construction as the one over Charing Cross Station, which was erected about the year 1863 and removed early in 1906; indeed, it may be taken as the prototype of that roof. The span of the Charing Cross roof was 48 feet less than the Birmingham roof, namely, 164 feet as against 212 feet, but the principals were farther apart, ranging from 35 feet to 37 feet between centres as against 24 feet at Birmingham.

In view of the failure of one of the tie-rods of the roof at Charing Cross it was deemed advisable to subject the welded joints in the tie-rods at Birmingham to a close scrutiny under a magnifying glass, after they had been thoroughly cleaned. No serious defects were discovered, but having regard to the results of the inquiry into the Charing Cross disaster, when it was proved that an original bad flaw in the weld of a bar could remain undetected in spite of the fact that every examination and test of the ironwork was made;¹ and in view of the length of time the roof had been subjected to the deleterious effect of steam and fumes from the locomotives, together with the fact that it was of rather lighter construction than accorded with modern practice, Mr. E. B. Thornhill, then Chief Engineer, advised the Railway Company to provide auxiliary tie-bars.

It was found by calculation that the tensile stress in the main tie-rod was 4·37 tons per square inch for the dead load alone, or 5·84 tons if the weight of the painters' scaffolding and 6 inches of snow were added; and that for each 10 lbs. per square foot of horizontal wind-pressure, the stress would be increased by an additional 0·42 ton per square inch. Therefore, for a wind-pressure of 40 lbs. and other loads as stated, there would be a stress in the tie-rod of 7·52 tons per square inch; but as it is unlikely that 6 inches of snow will remain on the roof with a wind blowing

¹ Major Pringle's Report to the Board of Trade, 25th March, 1906. See "Returns of Railway Accidents and Casualties . . . during the Three Months ending the 31st March, 1906," p. 122. London, 1906.

at a velocity corresponding with a horizontal pressure of 40 lbs. per square foot, 0·59 ton should be deducted, making a net unit stress of 6·93 tons.

It was also found that the compressive stress in the main arch-rib would be 2·75 tons per square inch for dead load plus painters' scaffolding, and 10 lbs. horizontal wind-pressure; or 3·17 tons per square inch with 40 lbs. wind-pressure. As the rib was well braced with purlins, wind-ties, and roof-covering, it was decided that the strengthening of the tie-rod alone need be dealt with.

SCAFFOLDING.

The problem that had to be solved was, how to fix an auxiliary tie-bar, calculated to weigh $5\frac{3}{4}$ tons, without interfering with the enormous traffic that day and night was to be found on the platforms below. To erect travelling stages, as had been done when the roof was originally constructed, was out of the question, and ultimately it was decided to suspend scaffolding from the existing principals and to strengthen them temporarily in such a way as to put no additional stress on the tie-rods.

The scaffolding adopted, shown in Figs. 4 and 5, Plate 3, involved placing a load of 24 tons on each principal, which would have produced an additional stress of $26\frac{1}{2}$ tons on the existing tie-rod. In order to avoid this additional stress, a steel-wire hawser with a breaking strength of $63\frac{1}{2}$ tons was attached to the feet of each principal during the whole of the time scaffolding was suspended from that principal. The hawsers were provided at each end with long screws, which at the column end passed through springs securely nested in a channel-bar bracket bolted to the shoes of the principals, as shown in *Figs. 3*. The length of the screws allowed for adjustments to suit the varying spans of the principals.

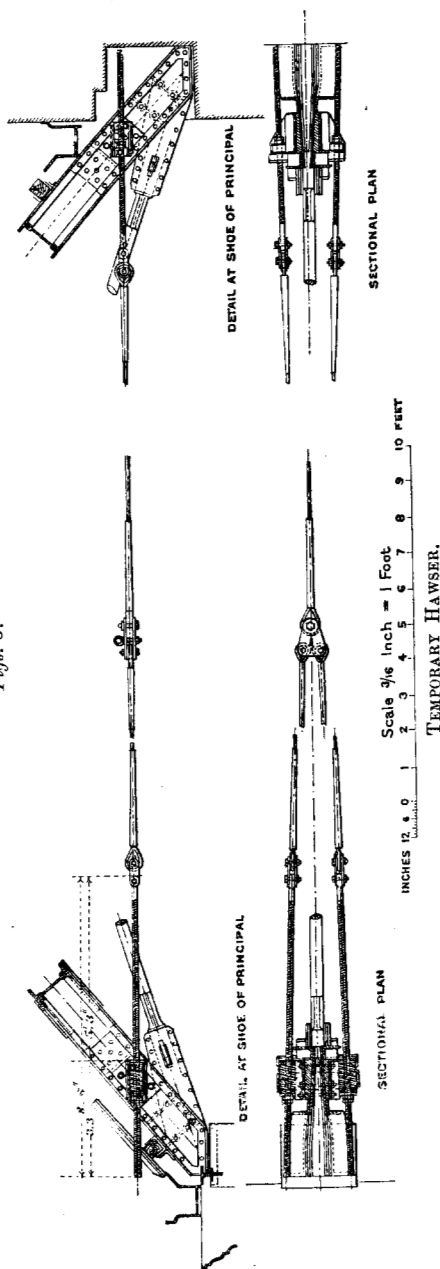
The springs had been previously loaded in a testing-machine, when their lengths between two gauge-points under the required load was accurately noted. The tension required in the hawser to counterbalance the effect of the scaffolding on the tie-rods was 16 tons, and to determine when this tension was on the hawser it was only necessary to screw down the nuts on the hawser-screws until the springs were compressed to the required extent. The adjacent end of the roof-principal was lifted clear of its bearing during the time the hawser was adjusted, and lowered after the adjustment had been completed. The erection of the scaffolding was then proceeded with, working from each end of the principal towards the centre, so as to avoid unequal loading. Three hawsers were in use with each

set of scaffolding, one hawser being lowered and refixed to the third principal in advance during an interval of an hour between trains on successive Sundays.

The general arrangement of the scaffolding is shown in Figs. 4, Plate 3. It will be seen from the Section A A that sufficient scaffolding was provided to enable the work of fixing the tie-bars to be continued without interruption. Thus, while the scaffolding required for fixing the tie-bar to one principal was in use, the other set of scaffolding was being moved forward from the principal in the rear to the one in advance of the work. Runners between adjacent principals and along a principal were fixed as shown, to facilitate this work; and altogether the scaffolding was found to answer its purpose very satisfactorily.

Each lower scaffold or gangway was made of twelve similar sections for each principal, suspended direct from the main rib and guyed from adjacent principals by wire ropes. All the

Figs. 3.



parts of one section were interchangeable with those of any other section. The lengths of the different units of the steelwork supporting the gangway were made to suit the shortest-span principal that it was intended to work under, and a series of connecting bolts was fixed between the different units. These connecting or adjusting bolts were provided with long screws and double nuts, so that the gangway could be adjusted to the necessary length and made perfectly rigid throughout its entire length by merely tightening up the adjusting bolts as required. When the tie-bar was completed, this gangway was readily stripped, hoisted to the stage at the level of the tie-rods, and refixed under the next principal but one in advance, no scaffolding being lowered to the platform until the whole of the work had been completed.

A gang of special scaffolders was continuously employed with each set, and, as they had previously been accustomed to this roof, they were able from the first to work with confidence at so great a height from the ground, and to remove and refix about 50 tons of material per week without mishap or inconvenience to any of the passengers using the platforms below.

Canvas sheets, commonly used in painting large roofs, were suspended below the whole area where work was in progress.

ERECTION OF NEW TIE-BARS.

The new tie-bars consist of two flat steel plates on edge, each measuring 6 inches by $\frac{3}{4}$ inch, with a space of $1\frac{1}{2}$ inch between them for painting purposes, and they are fixed about midway between the old tie-rod and the level of the springing-line. Steel rods, $1\frac{1}{8}$ inch in diameter, suspend the bar at each of the vertical struts of the old roof, one rod on each side of the principal hanging from brackets fixed on either side of the arched rib. Full particulars of this arrangement are shown in Figs. 6, Plate 3. Each suspension-rod was provided with screwed ends so that the nuts could be used to adjust either the rod itself or the flat bars of the new main tie in any required way.

Before any material for the tie-bars could be ordered, it was necessary to ascertain the length of each individual tie-bar, since, owing to the variation in the span and rise of the roof and to local irregularities in the principals themselves, no two lengths of bar were exactly the same. Two lines parallel to the north wall of the station and 130 feet apart were very accurately set out on the platforms on two successive Sundays, when the station was specially

closed for a few hours for that purpose. Heavy plummets were then suspended from each arch-rib in succession vertically over these lines, and other plummets were similarly suspended at each vertical of the principal, and the distance between them was accurately measured and checked with reference to the main plumb-lines.

The rise of the tie-bar having been determined, the length of the individual bars could then be calculated. This method proved efficient, and in no case was any alteration of a bar necessary. The vertical rods were delivered of the approximate length required, and were cut and screwed on the ground as the work proceeded. The tie-bars were delivered with cover-plates riveted on one end and with the holes drilled in the cover-plates for the end of the adjacent length in which no holes were provided. The whole tie-bar was then adjusted to the correct camber by means of the vertical suspension-rods, and the holes were drilled through, using the cover-plates as templates. The joints were riveted up by pneumatic riveting-hammers.

The method of attachment of the new tie-bars to the old principals required special consideration, owing to the complex nature of the ironwork where the old tie-rods intersected the foot of the arched rib. It was eventually decided to bore a hole through the entire thickness of the web-plates and the cast-iron packings forming the splayed portions of the main rib, and to insert tightly fitting steel pins $2\frac{3}{4}$ inches in diameter.

Castings were made and fitted to the shoes of each principal, to provide support for the pins between the web-plates of the roof-principal and the new tie-bar.

In view of the importance of the pins being absolutely level and at right angles to the new tie-bars, plant was devised to enable the holes to be bored with great accuracy. A hole $2\frac{1}{4}$ inches in diameter and 10 inches long was first drilled through the shoe of the principal by a Whitelaw pneumatic drilling-machine, and the castings, which were delivered with a $2\frac{1}{2}$ -inch diameter hole already drilled through them, were then fitted in place and secured to the principal by three 1-inch bolts. A boring-rig, consisting of a modification of an ordinary side-light cutter attached to a face-plate, was then secured to the principal. The boring-bar passing through the $2\frac{1}{4}$ -inch hole was housed in a bearing attached to a similar face-plate on the opposite side of the principal, and was driven by an ordinary pneumatic drill through worm-gear. After the boring-bar had been carefully adjusted, the hole—passing through 10 inches of old metal and 12 inches of new, i.e., 22 inches in all—was bored out to a plug-gauge $2\frac{3}{4}$ inches in diameter, with four to six passes

of the boring-bar. The pins, provided with pilot-nuts, were forced into the holes by means of jacks.

On opening out the shoes of the principals on the north wall of the station, it was found that the main wall of the hotel was carried by brick arches, and boring the holes for the main pins through the shoes of the roof-principals involved cutting away a considerable portion of the haunches of these arches. This was a difficult and costly operation, necessitating modifications of the boring-plant, and requiring the utmost care to prevent any disturbance in the main wall of the Queen's Hotel above.

The work in the hotel smoke-room and corridor also proved difficult. Owing to the confined space available and to the close proximity of the steel framing of the smoke-room to the roof-principals, the fixing of the new tie-bars involved making special arrangements for obtaining access to each of the seven principals; further, owing to the necessity of avoiding complaints from visitors at the hotel, the work could be carried on only between Friday night and Monday morning.

When detailed particulars of the principals resting on the garage were obtained, it was found necessary to provide angle-shaped girders to transfer the stress from the new tie-bars to the foot of the principal (Figs. 7, Plate 3). These girders weighed $2\frac{1}{2}$ tons each, and the feet of the old principals had to be underpinned and propped during the time the tie-bars were being fixed.

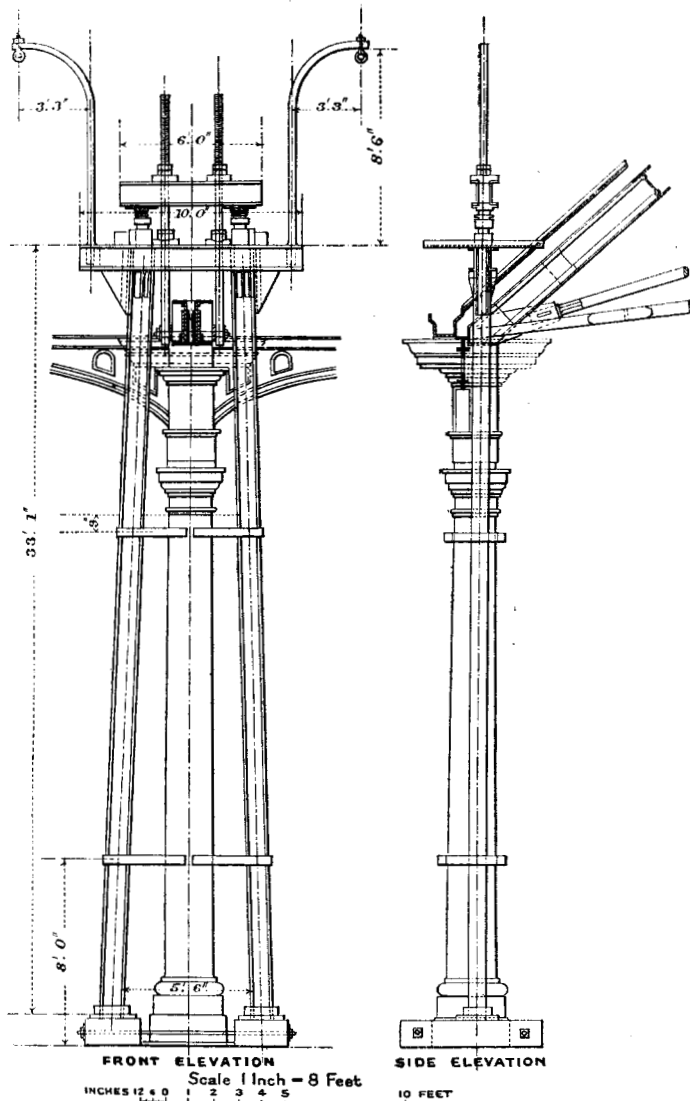
STRESSING THE NEW TIE-BARS.

In strengthening a roof or any engineering structure by the addition of auxiliary members, it is of course very important that initial stress shall be put upon the latter, or in other words, that the old members shall be relieved of a certain amount of the stress; and where the amount can be determined accurately this procedure makes the work much more satisfactory and efficient. The method adopted in the present case for ascertaining accurately the amount of stress put upon the new bars was the employment of calibrated springs, secured temporarily to the new tie-bars in the manner shown in Figs. 6, Plate 3.

A hole $2\frac{1}{4}$ inches in diameter was drilled through the column end of the tie-bar, 6 feet from the main pin, and a pin was inserted to take the ends of two adjusting-screws, which passed through the springs secured to the adjoining lengths of tie-bars. One inside plate and two outside cover-plates were attached to the column end of the new tie-bars, between which plates the two ends

of the adjoining length of tie-bar were inserted, the cover-plates

Figs. 8.



DERRICK FOR LIFTING END OF PRINCIPAL.

acting as guides when the bars were drawn together by compressing

the springs. The springs had been carefully calibrated under a load of 12 tons each. The adjusting-screws were eventually tightened until this load was put upon them and thereby transferred to the new tie-bar, the ends of which were drawn together to the extent of $\frac{7}{8}$ inch.

Simultaneously with the operation of putting stress upon the new tie-bars, the hawser-screws were tightened, so as to maintain the full tension of 16 tons in the hawser, and this stress, with the stress of 24 tons in the tie-bars, was continued while holes were drilled in the end of the tie-bar and the joint was riveted up and allowed to cool; the amount of stress in the new tie-bars was thus definitely assured.

The adjusting-gear on the tie-bar was then dismantled, but the hawser remained until the scaffolding on the principal had been removed. The springs were tested periodically during the progress of the work, but no alteration in their gauge-length under load was observed.

While the adjustment was in progress, the end of the principal was lifted entirely off its bearing by a specially-constructed derrick provided with long rods forming a sling (*Figs. 8*). The object was to ensure that none of the stress put upon the springs could be transferred to the columns in any way, otherwise the efficiency of the whole operation would have been seriously impaired. The adjustment, therefore, was made with the principal swinging clear of its bearing. Moreover, there would have been some risk to the column itself had not the weight of the principal been removed before the adjustment was made. The weight lifted by the derrick was 42 tons at a level of 35 feet above the ground.

Roller-bearings had been provided in the original design of the roof, but after lifting the principals it was discovered that the rollers were completely rusted up and were of no value for the purpose intended. They were therefore removed and replaced by solid packings.

EFFECT OF ALTERATION.

As was anticipated, the effect of shortening the old tie-rod by relieving it of some portion of its stress was to cause a slight buckling of the cross bracing in the principals.

The initial tension put upon the new tie-bars resulted in the stress on the tie-rods due to dead load only being reduced from 4.37 tons to 2.08 tons per square inch. It is uncertain to what extent stress due to snow, wind-pressure, painters' staging, etc., will be divided between the old tie-rod and the new bar.

Taking the extreme view of assuming that the old rod takes the whole of these extraneous loads and that the roof is subject to a wind-pressure of 40 lbs. per square foot, the maximum stress that could then be brought upon it will be 4·27 tons per square inch. In other words, the unit stress in the old tie-rod will vary between a minimum of 2·08 and a maximum of something less than 4·27 tons per square inch; whilst in the event of failure of one of the old rods, the new tie-bar would safely carry the whole of the load.

Owing to the greater depth of the principal, the effect of the new tie-bar is to reduce the compression in the arched rib; and as the stress was originally not excessive, the question does not call for further consideration.

The effect of the weight of the painters' scaffolding on these large-span roofs, especially when the distance between the principals is considerable, is perhaps of greater importance than is sometimes realized. The ordinary painters' scaffold for maintenance purposes in this case weighs 8·12 tons for each bay of roofing, and the additional stress in the tie-rod due to this scaffolding is 0·88 ton per square inch. Further, the tie-rods themselves may be strained locally by having to bear the bending-moments caused by the scaffolding, as well as the stress due to their position in the principal. In the Birmingham roof it is arranged that the scaffolding shall in all cases bear upon the parts of the tie-rod immediately adjoining the coupling-boxes or points of support of the rod, so as to avoid these additional bending-moments as far as possible.

The old tie-rods were subject to considerable vibration, especially when the painters' scaffolding was being moved along the roof from one bay to another. In order to obviate this, two longitudinal tie-rods have been fixed from end to end of the roof, as shown at B B in Figs. 6, Plate 3. Each of these tie-rods is provided with a collar bolted around the old tie-bar at one-third points in the span of the roof, and is also provided with a coupling-box in the centre of each bay, so that any required adjustment can be made to keep the old roof-principal strictly in line.

GENERAL.

Two sets of pneumatic plant, worked at 100 lbs. per square inch pressure, were used for drilling and riveting, compressed air being conveyed over the roof by pipes in the usual manner. The engines and boilers were fixed in a convenient place below.

Artificial light was provided by means of a number of 50 candle-power acetylene lamps. A 1,000 candle-power light was also tried

for a time, but the shadows cast by it rendered its use on the scaffolding dangerous.

The work throughout was so arranged that two new tie-bars could be fixed simultaneously. Principals Nos. 18 and 35, shown in Fig. 2, Plate 3, were commenced at the same time, and the work was carried on from these two points in an easterly direction, until the whole of the work was completed at the rate of two principals per week. The first two principals were finished on the 28th October, 1906.

The steel was made by the Siemens-Martin process, and was specified to have an ultimate tensile strength of 28 to 32 tons per square inch, an extension in 10 inches of not less than 20 per cent., and a contraction of area at the point of fracture of not less than 40 per cent.

In addition to the usual tensile tests, one of the main pins was tested in single shear at each end by a shear-blade with a cutting-angle of 90°. The 2½-inch pin was cut down to an area of 5·92 square inches, and the results of the tests were as follows:—

Maximum shearing load at end—

- (i) 129·31 tons = 21·84 tons per square inch.
 (ii) 118·1 „ = 19·8 „ „ „ „

The tensile test of the material from which the pin was made gave the following results:—

	Diameter of Test-Piece.	Breaking Weight.	Elongation in 10 Inches.	Contraction of Area.
	Inch.	Tons per Square Inch.	Per Cent.	Per Cent.
1st test	0·81	30·8	23	45·2
2nd „	0·94	31·3	22	46·1

The steelwork was made by Messrs. Cochrane and Company of Dudley, to dimensions and patterns supplied by the Railway Company. The whole of the work on site was carried out by the Railway Company under the direction of the Author, acting for the Chief Engineer, who at that time was Mr. E. B. Thornhill, M. Inst. C.E. The Author's assistant, Mr. W. J. Fuller, was the Resident Engineer.

The Paper is accompanied by eight drawings, from which Plate 3 and the Figures in the text have been prepared, and by a number of photographs.