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Prestressed Steel Girders

Poutres métalliques précontraintes

Vorgespannte Stahlträger

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It is gratifying to learn that interest in prestressing of steel structures has been aroused in countries as diverse as U.S.A., U.S.S.R., Japan and France. In Great Britain work on the subject, by Professor MAGNEL of Belgium, was published first in 1950. Subsequently, publications by R. A. SEFTON JENKINS in 1954 and FELIX J. SAMUELY in 1955, described the use of prestressed steel in lattice roof structures. These structures are at present in use.

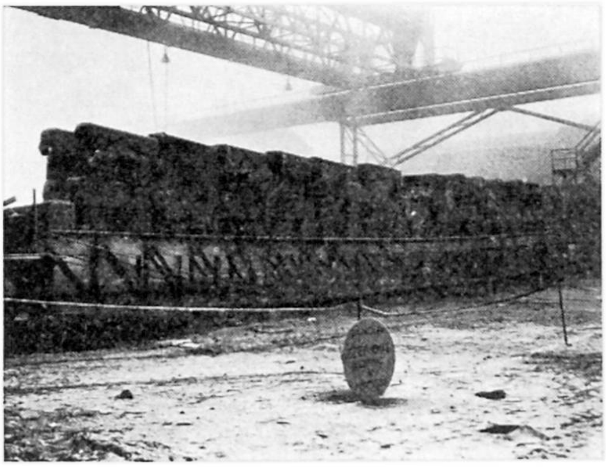

Fig. 1. 1st Full Scale Test. View of Girders at Failure.

Photograph by courtesy of Appleby-Frodingham Steel Co. (Branch of the United Steel Companies Ltd.)

However, until recently, little interest in this form of construction has been apparent. Apart from remedial work on existing structures, notably the strengthening of railway bridges by British Railways, the prestressing of steel by the pre-tensioning of high tensile steel tendons has not been used as a viable alternative to more conventional structural forms.

Two years ago, my sponsoring organisation, The British Iron and Steel Research Association, which is the central co-operative research body of the British Iron and Steelmaking Industry, decided to investigate again the economical potential of prestressed steelwork. The aim was to make steel construction more economic, and with other current investigations on high strength steels and corrosion techniques, to combat the competition from other structural materials such as prestressed concrete, which has made such great progress in Britain in recent years.

It was first confirmed analytically that savings in weight of steel girders were possible, and since the cost of the mild steel saved would exceed the cost of the prestressing system, savings of cost should result.

We were planning a series of model tests when it was learnt that a leading firm of consulting engineers, Messrs. FREDK. S. SNOW and Partners, were considering recommending prestressed steel construction for the girders of a major highway flyover. The reasons which prompted this proposal were firstly that the structure had to be erected over a heavily trafficked road, which could not be obstructed for long periods, secondly that the dead load

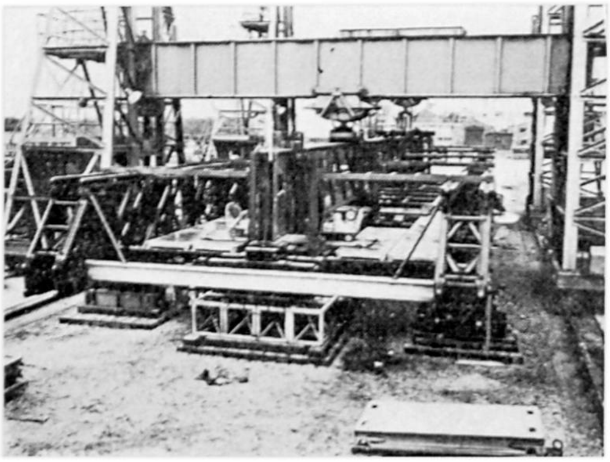

Fig. 2. 2nd Full Scale Test. View of test rig and test girder.

Photograph by courtesy of M.E.X.E.

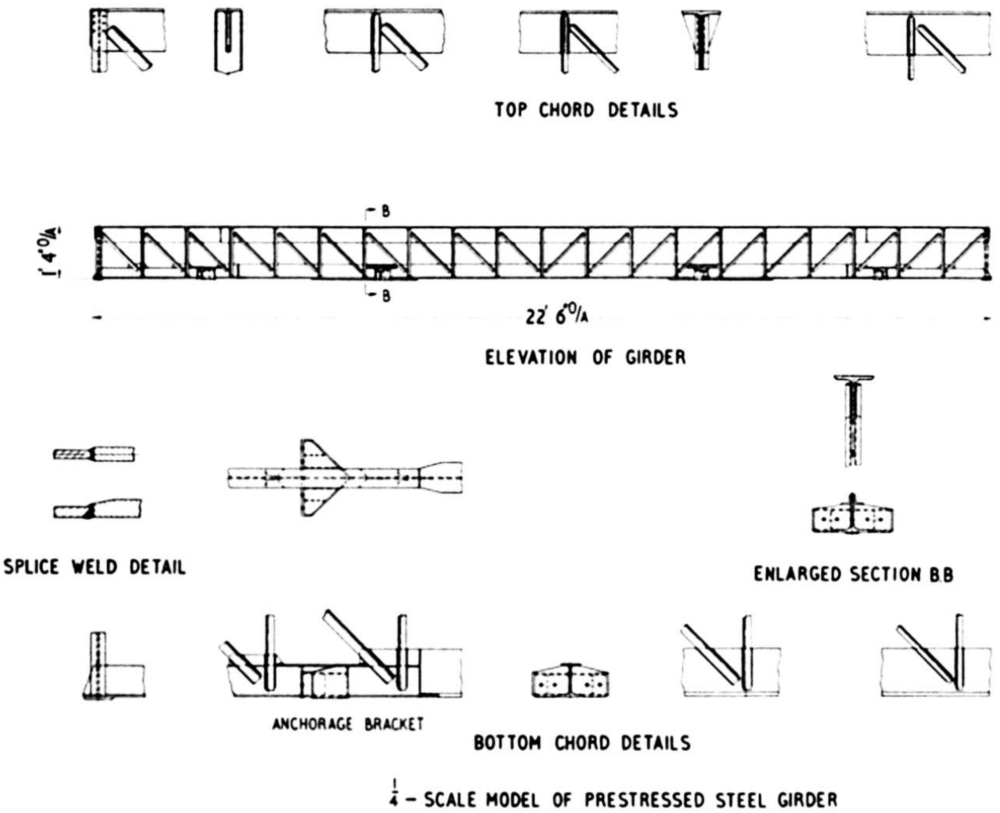

Fig. 3. $\frac{1}{4}$-scale model lattice girder.

of the steel structure would be only a quarter of that for a prestressed concrete structure, thus saving on foundation work, and thirdly the estimated cost of this form of construction was no greater than the equivalent prestressed concrete flyover. However, they were inhibited from recommending this design by the lack of any precedant in Great Britain, and desired to have

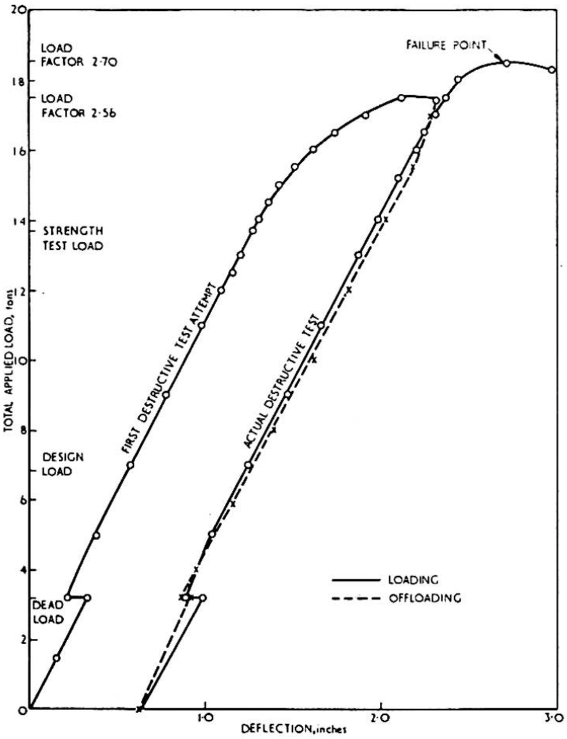

Fig. 4. Load/Deflection relationship of model lattice girder.

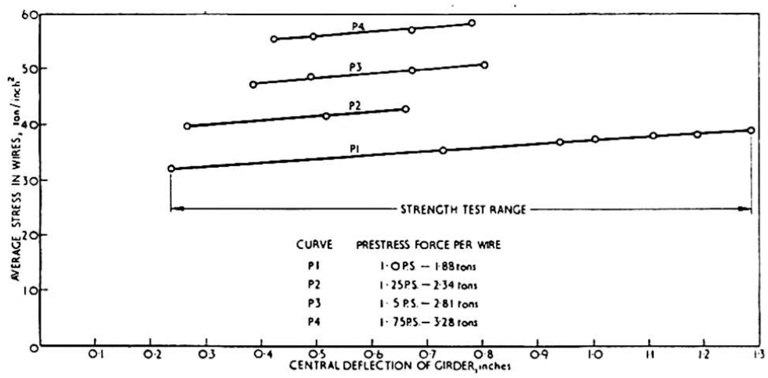

Fig. 5. Prestressing bar forces of model lattice girder.

proving tests carried out. After joint consultation with the client, B.I.S.R.A. undertook to finance and organise a full scale test of the girders in question. These were mild steel welded lattice girders of 90 feet span, 5′—3″ deep (27 m $\times$ 1.5 m) each prestressed at bottom chord level by 4 no. $1^{1}/_{8}$″ (2.86 cm) dia. Macalloy bars.

The test was conducted in accordance with the loading requirements, for structures of unusual design, specified in Appendix A of British Standard 449, the Use of Structural Steel in Buildings. The girders were required to pass two loading tests in which static loads were applied for a period of 24 hours. The first, a Stiffness Test, required that the structure should not deflect excessively under the application of dead load + 1.5 times live load and that the recovery on release of the sustained load should exceed 80% of the maximum deflection. The second, a Strength Test, required the structure to withstand twice dead load + twice live load with no part completely failing and with a recovery on release of not less than 20% of the maximum deflection. In simpler terms, an ultimate load factor of at least 2.0 was required.

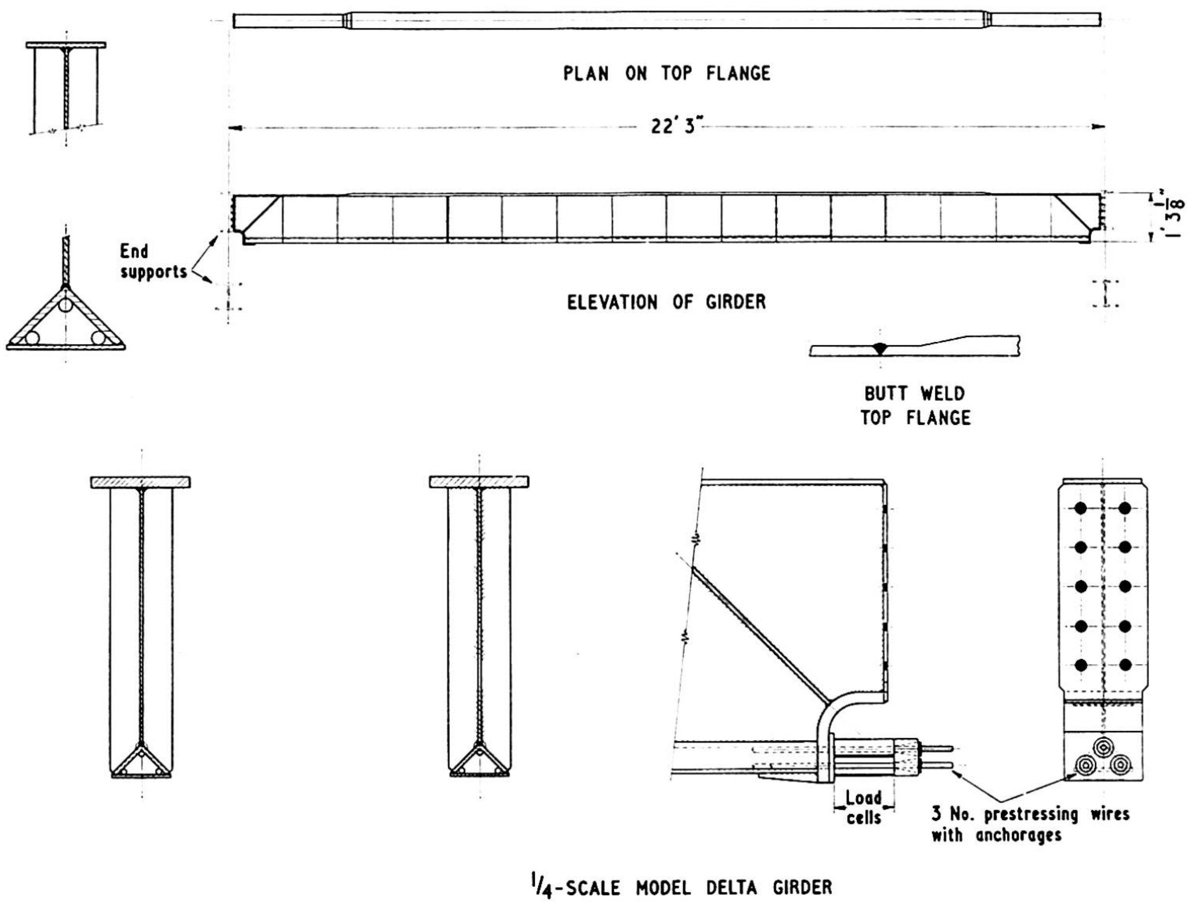

Fig. 6. ¼-Scale model delta girder.

The B. S. 449 test was adopted in default of any corresponding test being specified in the British Standard 153, Steel Girder Bridges. The girders had been designed in accordance with the provisions, where appropriate, of this latter standard which implies, by virtue of the magnitude of the working stresses laid down, a load factor of about 1.7. In the first instance the stiffness test of the Appendix A test was successfully passed but in the strength test failure took place at a load factor of 1.92, the bottom mild steel chord failing in tension. A second full scale test was authorised and was carried out at the Military Engineering Experimental Establishment. The girder tested was of a modified design, in the light of previous experience. In this case only one girder was tested, instead of a braced pair as previously, and lateral restraints, incorporating rollers to permit vertical deflection, were provided at 5 points along the 90 ft. span. In this second test, again the stiffness test was successfully passed but in the ultimate condition the spacing of the lateral restraints proved too large and failure took place by lateral buckling of the top chord at a load factor of 1.97. In practice, full lateral restraint to the top chord would be provided by the concrete deck slab, which the girders will support. After due consideration the governing authority accepted the results of these two tests as demonstrating the structural soundness of the design.

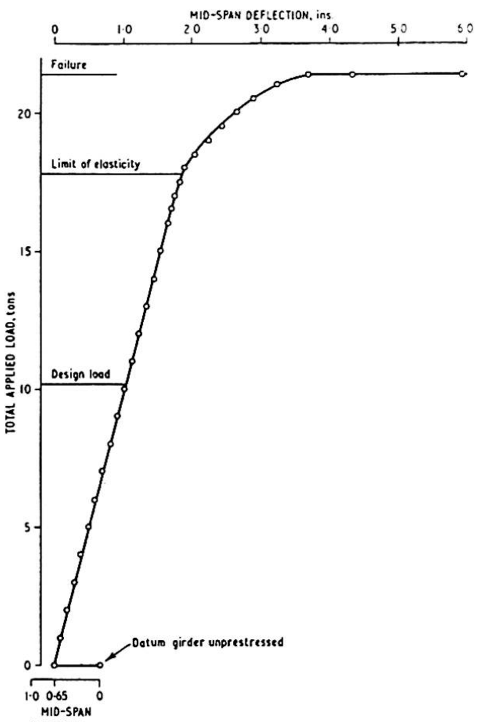

Fig. 7. Load/Deflection relationship of model delta girder.

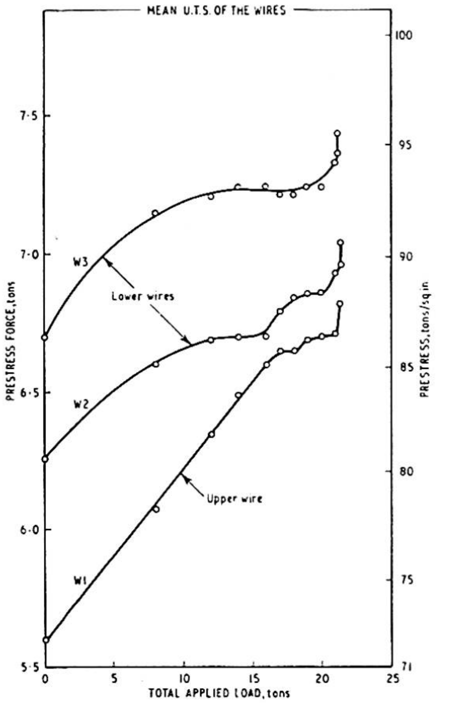

Fig. 8. Prestressing bar forces of model delta girder.

Whilst therefore having achieved the desired end, it was decided that this particular design had more information to yield and hence we constructed a quarter scale model which could be laboratory tested, under conditions of greater control than were possible in the open. The strain gauge system used on the second full scale test proved unreliable due to foul weather throughout the tests. The laboratory test however enabled a full range of strain gauge readings to be taken. In the event the model proved wholly satisfactory and ultimate failure did not take place until a load factor of 2.7 had been reached. This high figure is partly due, of course, to the higher yield stress usually shown by very thin sections, coupled with a degree of work hardening during testing, but the margin is sufficiently wide to assert that the girder complied fully with the design requirements.

We have subsequently fabricated and tested a novel design of prestressed welded plate girder. This is a B.I.S.R.A. design and incorporates a hollow bottom flange built up of a rolled steel angle and plate, which contains three prestressing wires. The wires are thus protected from accidental damage and corrosion. The top flange is a normal plate of cross sectional area some 2.3 times that of the compound bottom flange. Subsequently we propose to test a further girder, acting compositely with a concrete deck. The particular advantage of studying plate girders, vis a vis lattice girders, lies in the fact that when treating the girder as a whole, prestressing of the lower flange, in addition to creating initial compressive stress in that flange, also induces a tensile stress in the upper flange, albeit much smaller. This is not apparent in a lattice girder design if, as is usual, the average stresses across the individual chords are calculated, for design purposes.

Appended are photographs and figures depicting our early full scale tests and the model lattice and plate girders, together with comparisons of properties and behaviour.

Property	Model lattice girder	Model plate girder
Span	22′ 3″	22′ 3″
Depth	1′ 3¾″	1′ 3⅛″
Weight	398 lbs.	393 lbs.
Area of prestressing steel	0.239 sq. ins.	0.234 sq. ins.
Initial prestressing force	7.52 tons	17.6 tons
Moment of inertia	208 ins.^{4}	184 ins.^{4}
Design bending moment	234 tons-ins.	353 tons-ins.
Corresponding ¼ pt. loads	3.5 tons	5.1 tons
Ultimate moment of resistance	630 tons-ins. (load factor 2.7)	720 tons-ins. (load factor 2.04)
Corresponding ¼ pt. loads	9.45 tons	10.7 tons

Fig. 9. Comparisons of model girder properties.

In the course of the work described, much of the philosophy of steelwork design and prestressing has had to be re-examined. Fundamentally, what is being attempted is the replacement of a certain quantity of low strength/cost steel by a significantly smaller quantity of high strength/cost steel, thus affecting overall economy. It is clear that as and when really high strength steels, of weldable quality, manufactured in plate form, become available at a reasonable cost, prestressing may not have a great part to play. However, in the meantime, the technique is worth pursuing.

Regarding design, it has been argued that prestressing does not increase the ultimate moment of resistance of a girder. This is true when one compares girders embodying a certain proportion of high strength steel, in one case prestressed and in the other not so. If tested to destruction, the mild steel in the latter girder would reach yield stress at an early stage and the girder would deform, thereby increasing the stress in the high tensile element. Not until both elements depart from elasticity will ultimate failure take place. In the former case, by prestressing, the onset of yielding in the mild steel is postponed, and deformation up to full plasticity is reduced. In short, prestressing is necessary to limit stresses under working load and to enable the high strength steel to carry its due proportion of load under working conditions.

It is worth noting that loss of prestress in steel structures is much less than in prestressed concrete, the losses being wholly confined to creep in the high tensile steel, and perhaps slip in the anchorages.

It has also been argued that prestressing, by increasing the range of stress through which the mild steel tension flange will pass under the application of load, achieves increase in strength at the expense of the factor of safety. As has been pointed out in the paper of SHU-TIEN LI for a comparable factor of safety to be achieved, mild steel tensile stresses under working loads have to be limited to less than the permissible tensile stress laid down by codes of practice for the design of normal girders. This calls in question whether the principle of limiting working stresses is approriate for prestressed girders, and whether one ought not to design against ultimate conditions. It can be postulated that in elastic design maximum stresses are limited to some arbitrary figure and that the load at which failure takes place is of less importance. In ultimate load methods, a load factor is applied to the collapse load and stresses under working conditions are regarded as of academic interest only. The choice of design philosophy in prestressed steelwork is complicated by the fact that the prestressing force cannot be regarded as constant in the way that it is in prestressed concrete. For instance, in the case of our model lattice girder an increase in prestressing force of some 25% was recorded at ultimate load.

It would appear logical that in an elastic design both the top and bottom mild steel flanges, as well as the prestressing element, should reach working stresses at the same applied bending moment, and the relative increase in effective prestress should be allowed for, being calculated on the basis of anticipated deflections. Similarly, in ultimate load methods it would seem logical for both mild steel flanges and the prestressing tendons to reach ultimate load at the same applied bending moment. Two complications however arise; firstly the initial prestressing force required for the ultimate load case is higher than that required in elastic design. Secondly, in the ultimate load case, the stresses present in the prestressing tendons under *working load* would be higher than normally allowed in prestressed concrete. Is this acceptable? I do not know the answer, I merely put the question.

It follows therefore that upon the design philosophy adopted depends the choice of the ratio of the compression and tension flange areas and the initial prestressing force. In making comparisons with conventional girders, it is essential to compare similar girders designed on the same basis, and thus to compare like with like. If this is not done, for instance comparing a plastically designed conventional girder with an elastically designed prestressed girder, misleading conclusions will be drawn.

Referring again to the matter of the increase in effective prestressing force, depending upon the proportions of the particular girder considered, it should be noted that this increase tends further to strengthen a girder during the application of load. Hence one can anticipate higher ultimate load figures than an analysis assuming a constant prestressing force would indicate. However, this brings in its train a further difficulty. In structures carrying dynamic loading, particularly where dead load is low, the prestressing elements will be subjected to stress cycles, which will be larger than those experienced in tendons in prestressed concrete, but not as large as those experienced by the mild steel, since the tendons are not bonded to it. Consequently, fatigue in the tendons must be considered, and it does appear that a cable or rod anchorage which is not fatigue sensitive has yet to be developed.

These, therefore, are some of the difficulties that we face. None of these problems is insoluble but it will take time before satisfactory solutions can be found to all of them.

Summary

A series of two full scale tests on 90 ft span lattice girders is described and also two tests on $\frac{1}{4}$ scale lattice and plate girders. The girders were all prestressed at bottom chord level. The philosophy of prestressing steel is examined and differences between elastic and ultimate load methods on design properties highlighted. Future work and remaining problems are outlined.

Résumé

L'auteur décrit deux essais échelle grandeur effectués sur des poutres à treillis de 27 m (90 ft) de portée ainsi que deux essais sur des poutres à treillis et à âme pleine à l'échelle 1 : 4. Toutes les poutres étaient précontraintes au niveau de la membrure inférieure. On examine toutes les implications que comporte le principe de la précontrainte de l'acier et l'on fait ressortir les différences des méthodes élastiques et du calcul en plasticité en ce qui concerne les caractéristiques de l'étude. On esquisse enfin les travaux futurs et les problèmes qui subsistent.

Zusammenfassung

Es werden zwei Großversuche mit Fachwerkbalken von 27 m (90 ft) Spannweite in natürlicher Größe beschrieben sowie zwei Versuche mit Fachwerk- und Vollwandträgern im Maßstab 1 : 4. Alle Träger wurden am Untergurt mit dünnen Stahldrähten vorgespannt. Die Probleme im Zusammenhang mit der Vorspannung von Stahlkonstruktionen werden dabei untersucht und die Unterschiede zwischen der klassischen und der Traglastmethode in bezug auf Entwurf und Berechnung erwähnt. Zukünftige Arbeiten und noch nicht abgeklärte Probleme werden angedeutet.

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