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Einige Gesichtspunkte der Vorspannung von Stahlbrücken[1])

Some Aspects of Prestressing in Steel Bridges

Quelques aspects de la précontrainte des ports métalliques

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Die Vorspannung von Stahlbrücken hat vier verschiedene Aufgaben zu erfüllen:

1. Verstärkung bestehender Brücken.
2. Anordnung von hochfesten Vorspanngliedern bei Neubauten zur Erzielung einer günstigen Spannungsverteilung im Bauwerk.
3. Einleitung eines Eigenspannungszustandes in die statisch unbestimmte Konstruktion durch Stützenverschiebungen mit hydraulischen Pressen.
4. Vorspannung von Seilkonstruktionen (Hängesystemen) zur Stabilisierung und Verhinderung des Schlaffwerdens von Seilen.

Zu den einzelnen Punkten ist folgendes zu bemerken:

1. Verstärkung bestehender Brücken

Sie wird in der Regel mit einer Vorspannung so kombiniert, daß ein Teil oder das gesamte Eigengewicht der Brücke durch Verstärkungsglieder — die auch aus gewöhnlichem Baustahl sein können — übernommen wird. Für die Verkehrslast tritt sodann das neue Gesamtsystem in Aktion. In diesem Zusammenhang müssen sowohl der Spannungs- und Formänderungszustand unter Gebrauchslast als auch die Tragsicherheit untersucht werden, wobei bei häufigem Lastwechsel auch noch Fragen der Materialermüdung zu behandeln sind. Schließlich ist noch die Sicherheit gegen Instabilwerden des Gesamtsystems und einzelner Bauglieder zu ermitteln, wobei auch die baupraktisch unvermeidlichen Imperfektionen zu berücksichtigen sind.

2. Anordnung von hochfesten Spanngliedern bei Neubauten

Diese Methode gestattet die Aufbringung eines Eigenspannungszustandes im Bauwerk vom entgegengesetzten Vorzeichen des Lastspannungszustandes. Man wird vor allem gezogenen Konstruktionsgliedern eine Druckvorspannung erteilen (Fig. 1), deren Höhe in der Regel durch die Knick- beziehungsweise

[1]) Teil des am Kongreß vorgetragenen Generalberichtes.

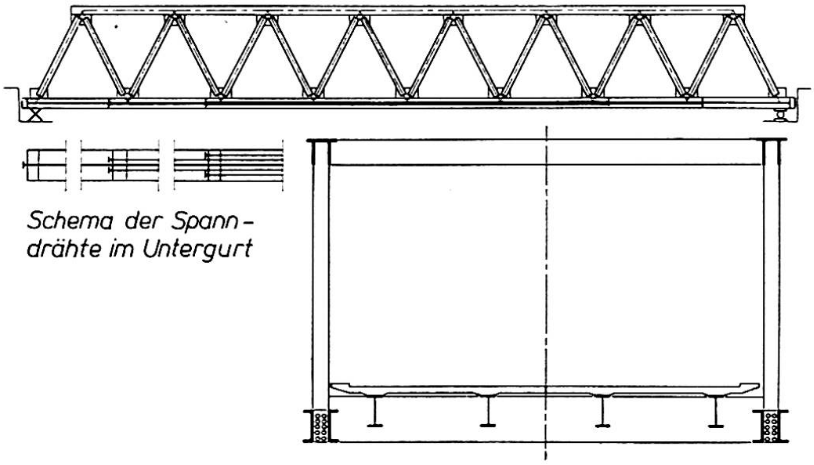

Fig. 1.

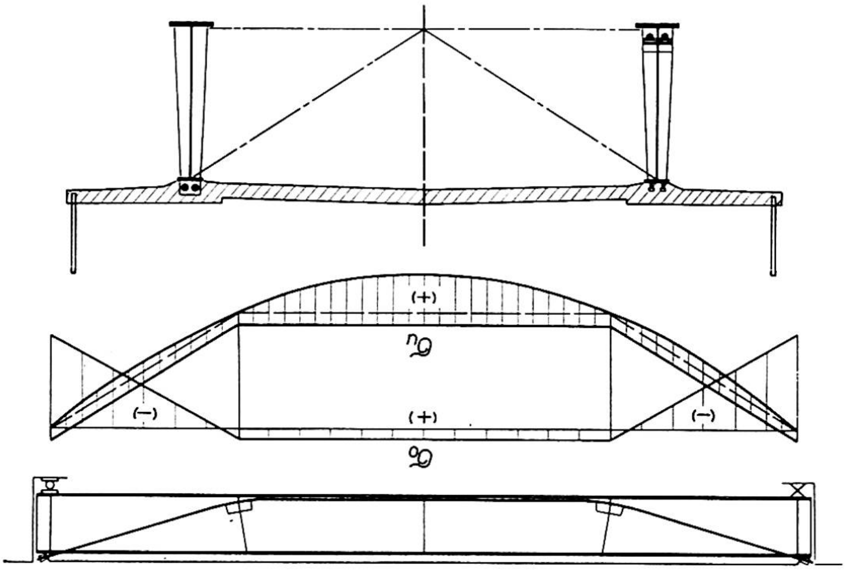

Fig. 2.

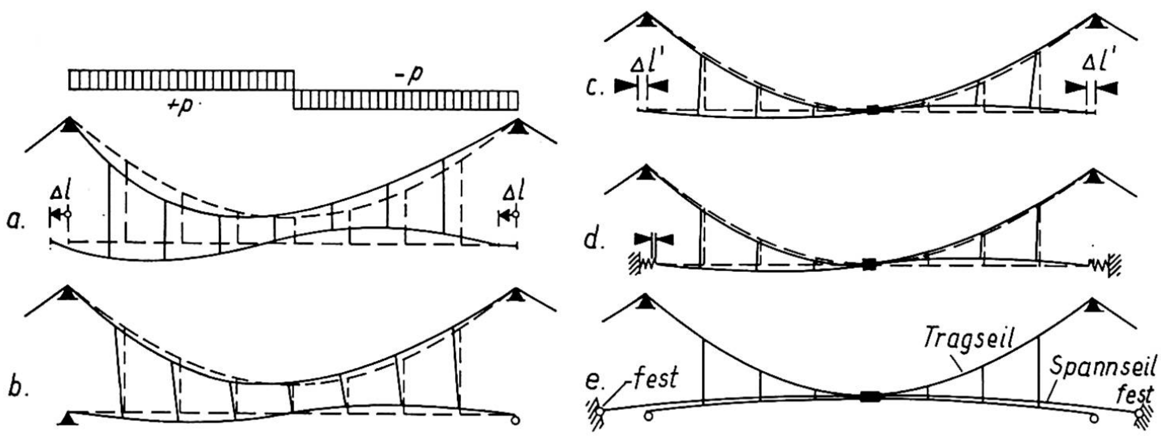

Fig. 3.

Kippsicherheit des Baugliedes begrenzt ist. Diese Sicherheit braucht unter Berücksichtigung aller Imperfektionen nur wenig größer als 1 zu sein, da dieser Spannungszustand schon durch das Eigengewicht abgebaut wird.

Während bei der Vorspannung von Fachwerkstäben in statisch bestimmten Systemen in erster Linie der betreffende Stabzug (zum Beispiel Untergurt) die Vorspannung erhält und im übrigen System nur Spannungen sekundärer Natur infolge der Biegesteifigkeit der Knotenverbindungen und der Stäbe auftreten, ist bei der in Fig. 2 gezeigten Vorspannung mit polygonal geknickter Kabelführung das ganze System in einen Eigenspannungszustand versetzt, der sich dem Lastspannungszustand überlagert. Die Größe der Vorspannung kann so gewählt werden, daß der größte Teil der ständigen Last vom Vorspannkabel allein aufgenommen und die Nutzlast sodann durch das kombinierte System getragen wird. Auf diese Weise können sowohl einfache Balken als auch Durchlaufträger und Rahmensysteme wirtschaftlich vorgespannt werden.

Neben dem Spannungsnachweis im Vorspannzustand und unter maximaler Gebrauchslast ist bei vorgespannten Systemen auch ein Tragsicherheitsnachweis zu führen. Eine Dimensionierung nur unter Berücksichtigung der Tragsicherheit halte ich nicht für ausreichend, da die Kenntnis des Spannungs- und Formänderungszustandes unter Gebrauchslast unbedingt notwendig ist, um zu einer abschließenden Beurteilung der Brauchbarkeit des Bauwerkes zu kommen. Es ist besonders bei vorgespannten Systemen fraglich, ob die plastische Reserve im Tragwerk ausgenützt werden kann. Auch dem Stabilitätsnachweis kommt hier große Bedeutung zu. Die Wirksamkeit einer Vorspannung steht daher, trotzdem diese auf den Grenzzustand keinen unmittelbaren Einfluß hat, weil sich die Eigenspannungszustände infolge Plastizierung vor dem Zusammenbruch im allgemeinen ausgleichen, einwandfrei fest. Außerdem kann sich fallweise bei Plastizierung der Stahlkonstruktion noch ein nur aus Vorspannkabeln bestehendes tragfähiges System (Hängesystem) ausbilden, welches in der Lage ist, den Zusammenbruch zu verzögern, eine Tatsache, die bei der Festsetzung des Sicherheitskoeffizienten zu berücksichtigen ist.

3. Einleitung eines Eigenspannungszustandes durch Auflagerverschiebungen

Dieser Art der Vorspannung kommt bei schiefen Brücken eine besondere Bedeutung zu, um entweder negative Auflagerdrücke ganz auszuschalten oder auf ein gewünschtes Maß zu reduzieren. Zur Verkleinerung von unerwünscht hohen Zugspannungen in der Stahlbetonfahrbahnplatte sowie zum Momentenausgleich im elastischen Bereich kann das Anheben beziehungsweise Absenken der Lager ebenfalls mit Vorteil angewandt werden. Ähnliche Betrachtungen lassen sich auch für Rahmen- und Bogentragwerke anstellen.

4. Vorspannung von Seilkonstruktionen

Für die Vorspannung von aus Seilen aufgebauten Systemen hat ILJASEVITSCH interessante Beispiele gebracht. Sie dient dazu, um die Wirksamkeit der Seilkonstruktionen unter maximaler Belastung zu gewährleisten (Verhin-

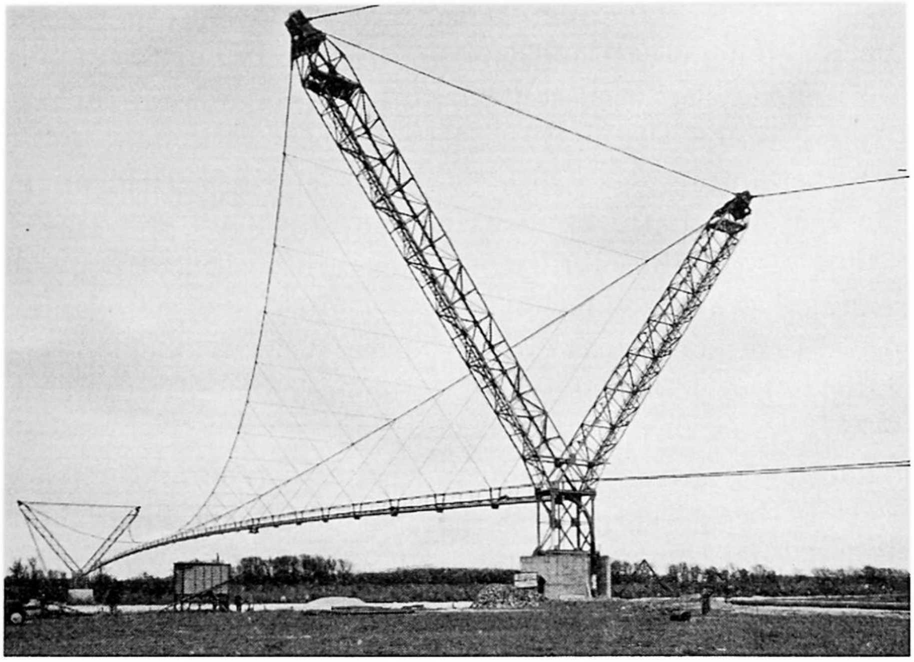

Fig. 4.

derung des Schlaffwerdens von Seilen) und um eine ausreichende Stabilität des Systems gegen aerodynamisch erregte Schwingungen zu erzielen. Fig. 3 zeigt die Stabilisierung eines Hängesystems durch Anordnung eines zusätzlichen Spannseiles, das mit dem Tragseil durch einen Mittelknoten fest verbunden ist. In Fig. 4 ist ein vom Verfasser gemeinsam mit der Firma Waagner-Biró AG entwickeltes System für eine Rohrbrücke dargestellt, das auch eine hohe aerodynamische Stabilität aufweist.

Besondere Beachtung verdient die *Tragsicherheit* von vorgespannten Systemen. Shu tien Li hat in seiner Arbeit gezeigt, daß die Ausnützung der zulässigen Spannungen für den Baustahl und den Vorspannstahl zu einer Herabsetzung der Sicherheit gegenüber den für nicht vorgespannte Konstruktionen gültigen Werten führen kann. Allerdings ist hier auch die Wahrscheinlichkeit der Abweichung der Spannungen von den Rechnungswerten maßgebend, die eng mit der Ursache ihres Auftretens in Zusammenhang steht. Man kommt dem Ziel einer kohärenten Bauwerkssicherheit näher, wenn man die spannungserzeugenden Faktoren mit Unsicherheitskoeffizienten versieht. Hierbei hat der Unsicherheitskoeffizient der Vorspannung sowohl die Ungenauigkeit ihrer Aufbringung als auch den Vorspannverlust infolge bleibender Verschiebungen der Anker- und Stützpunkte sowie Kriechen des Vorspannstahles zu berücksichtigen. Wie Iljasevitsch gezeigt hat, ist mit einem 5 bis 10%igen Vorspannverlust infolge Nachgiebigkeit der Verankerung und Kriechen des Vorspannstahls zu rechnen.

Die mit diesem Verfahren erhaltenen Spannungswerte werden dann jenen gegenübergestellt, die ein Unbrauchbarwerden des Bauwerkes bedingen, wobei auch die festigkeitsvermindernden Faktoren durch Unsicherheitskoeffizienten berücksichtigt werden müssen. Man wird zur erschöpfenden Beantwortung der Frage der Bauwerkssicherheit zwei Berechnungen durchzuführen haben. Die eine beschäftigt sich mit dem Spannungszustand an der Grenze des elastischen Bereiches, während die zweite jenen Grenzzustand untersucht, bei dem im Baustahl und gegebenenfalls auch im Spannstahl die Fließgrenze an so vielen Stellen erreicht wird, daß ein Mechanismus entsteht. Hat der Vorspannstahl keine ausgeprägte Fließgrenze, was meist der Fall ist, so wird man je nach der Form des Spannungs-Dehnungs-Diagrammes jene Spannung als maßgebend ansehen, die eine bestimmte bleibende Verformung hervorruft. Hierbei ist noch offen, ob man die 0,2-Dehngrenze nimmt oder eine andere Festlegung trifft. Je nach der Art der Vorspannung und der Wahl der Querschnittsverhältnisse sowie nach der Art der Belastung wird man die Spannung im Vorspannstahl mehr oder weniger ausnützen können. Eine allgemein gültige Regel läßt sich hier nicht geben.

Die Wirtschaftlichkeit einer vorgespannten Stahlkonstruktion kann nicht allein aus dem Vergleich der Gewichte, unter Berücksichtigung der Materialpreise, mit jenen einer nicht vorgespannten Konstruktion beurteilt werden. Man wird hierbei auch den konstruktiven Aufwand für die Einleitung der Vorspannung in die Stahlkonstruktion ebenso berücksichtigen wie die Kosten der Aufbringung der Vorspannkraft im Werk beziehungsweise an der Baustelle.

Zusammenfassung

Der Diskussionsbeitrag geht kurz auf die Aufgaben der Vorspannung und die Wege zu deren Lösung ein. Die Vorspannung einzelner Konstruktionsglieder und ganzer Systeme sowie von Seilkonstruktionen wird kurz besprochen. Schließlich behandelt der Verfasser noch Probleme der Tragsicherheit von vorgespannten Konstruktionen.

Summary

The paper presents a short review of the problems of prestressing and of possible means for their solution. The prestressing of single members and of structural systems, as well as of wire rope structures, are briefly discussed. Finally, the author deals with questions of safety and the limit design of prestressed structures.

Résumé

L'auteur décrit brièvement divers modes d'utilisation de la précontrainte. Il discute la mise en précontrainte d'éléments d'ouvrages, d'ouvrages entiers et de systèmes formés de câbles. Pour terminer, il examine certains aspects de la sécurité et de la résistance à la ruine des constructions précontraintes.

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IIIc2

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.

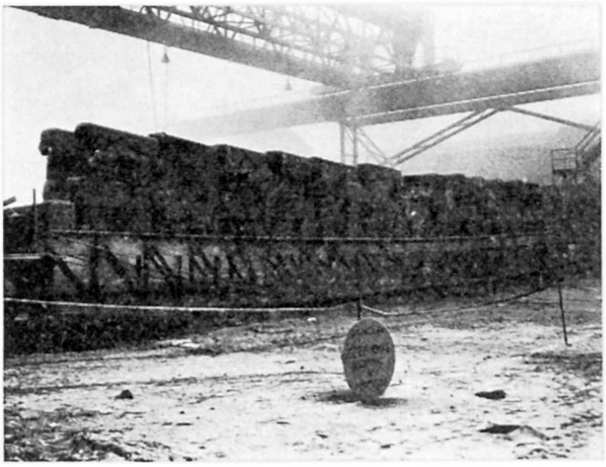

Fig. 1. 1st Full Scale Test. View of Girders at Failure.

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

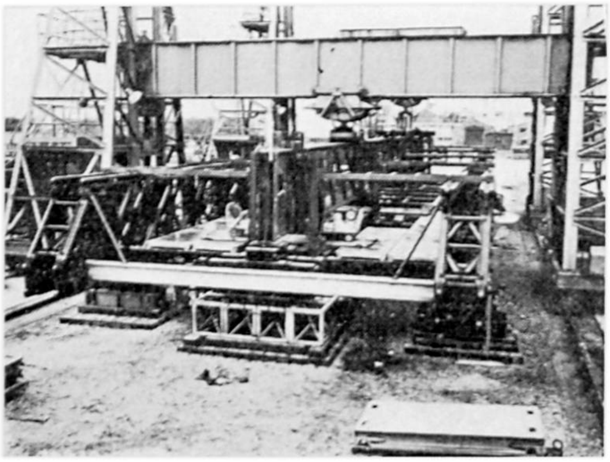

Fig. 2. 2nd Full Scale Test. View of test rig and test girder.

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

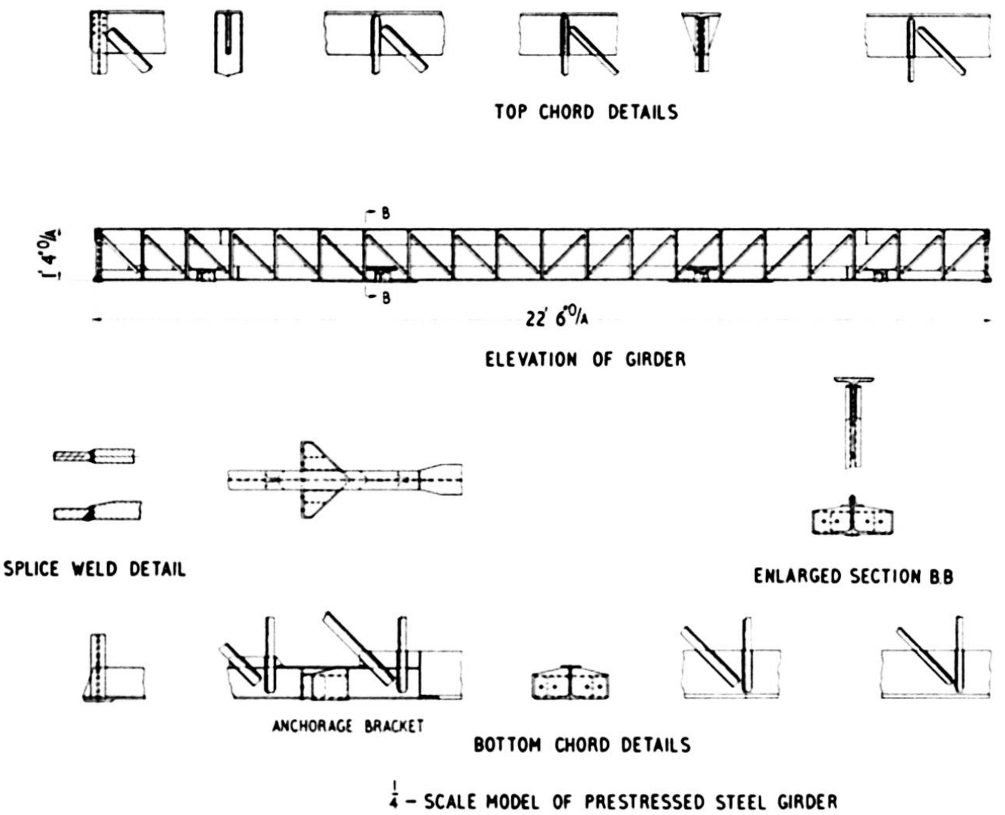

Fig. 3. $\frac{1}{4}$-scale model lattice girder.

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

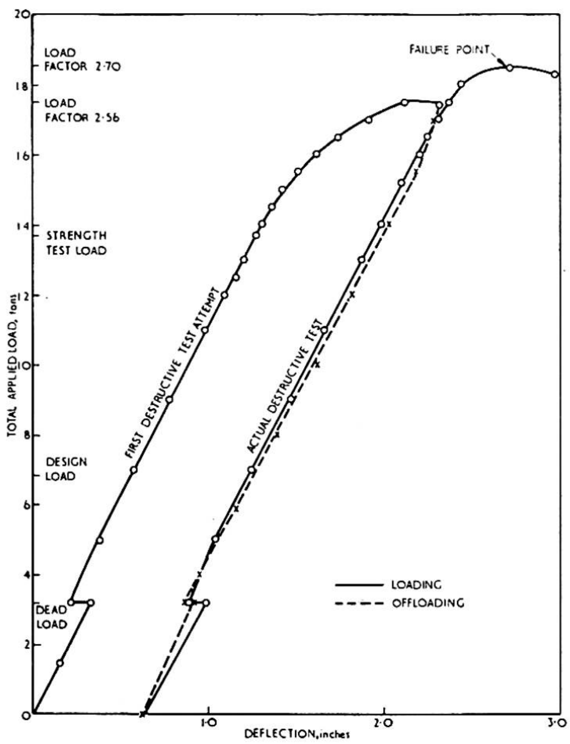

Fig. 4. Load/Deflection relationship of model lattice girder.

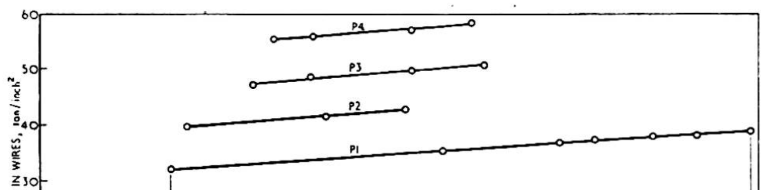

Fig. 5. Prestressing bar forces of model lattice girder.

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 × 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.

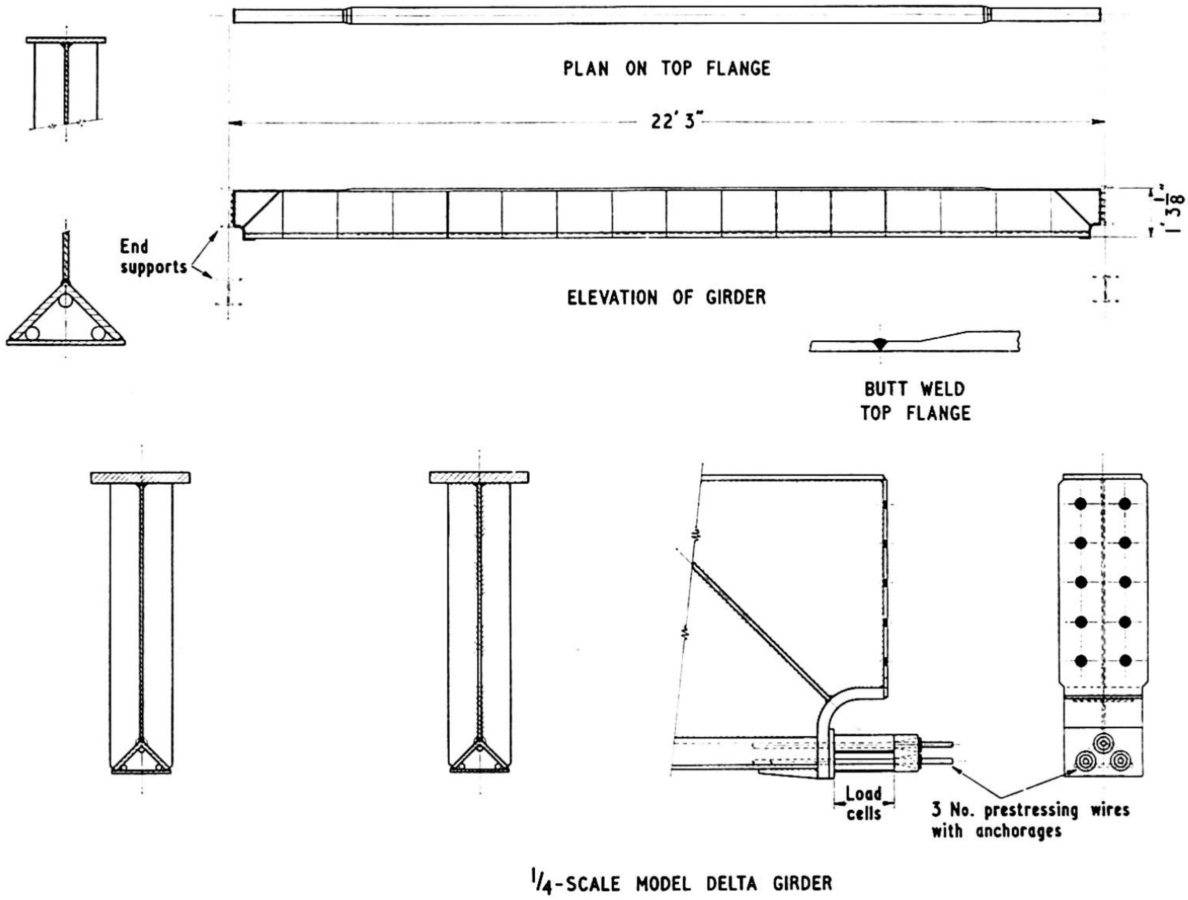

Fig. 6. 1/4-Scale model delta girder.

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.

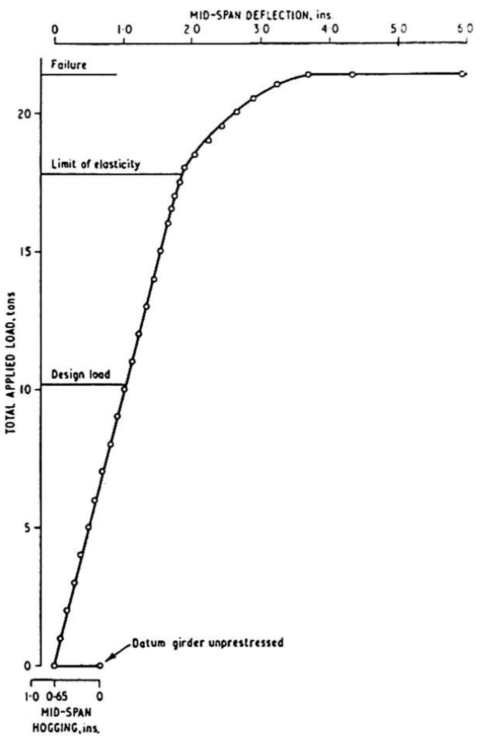

Fig. 7. Load/Deflection relationship of model delta girder.

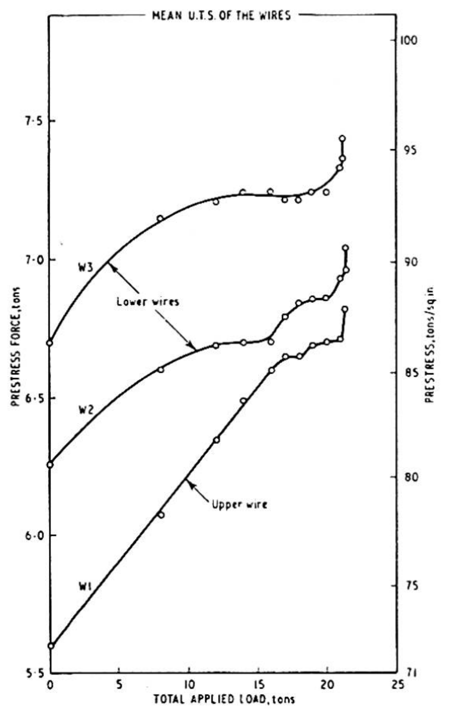

Fig. 8. Prestressing bar forces of model delta girder.

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′ $3\frac{3}{4}$″	1′ $3\frac{1}{8}$″
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 $\frac{1}{4}$ 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 $\frac{1}{4}$ 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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IIIc3

Prestressing Steel Girder Bridges

La précontrainte des ponts à poutres métalliques

Vorspannung von Stahlfachwerkbrücken

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The technique of prestressing steel girder bridges and structures is of recent origin and has attracted the attention of many structural engineers. There are two purposes for which this technique is utilised in steel girder bridges, viz., 1. for strengthening existing bridges to enable them to carry loads that are havier than the design loads, and 2. for the design of new bridges by prestressing the members and thereby effecting economy. Both these aspects have been covered in the papers presented to the Association. The strengthening of existing railway bridges is, in my opinion deserving of serious consideration in many countries on account of the increase in present-day loads and speeds as compared with the design loads. Unfortunately, not much experimental evidence of the results of prestressing, especially on the durability of prestressing in the case of riveted structures, is yet available. For instance, it would be interesting to know whether:

1. The actual stresses developed in the structure agree with the calculated stresses.
2. Any records have been kept which would make it possible to determine the effect of vibrations and the loss of prestress due to slip at rivets, creep etc., in course of time.
3. Any records have been kept of the increase in camber due to prestressing and the loss of camber in course of time, especially in railway bridges which are subjected to heavy vibrations.

Just as a loaded bridge girder develops secondary stresses due to restraining action at the joints, an important consequence of prestressing would be that the initial prestressing would develop secondary stresses in the chord and web members, having an opposite sign to that which would occur under normal loads. Therefore, an important advantage of prestressing would be the elimination, or reduction, of secondary stresses. According to the British and American codes of practice the secondary stresses need not be considered if the ratio of depth to length of members is kept within certain limits. In the B.S. Specification it is 1/12 for chord members and 1/24 for web members, and in the AREA

Specification it is 1/10. In the case of heavily loaded girders, for instance, the girders in bridges carrying double or multiple tracks, the members are sufficiently rigid and the limiting ratio of depth/length cannot always be maintained, with the result that the secondary stresses are bound to the appreciable. For instance, in the open-web girders of the deck type supported at the top chord level, as used in the double track, 150 ft. spans, for the new Yamuna Bridge at Delhi (India), the length/depth ratio is as much as about 7,5. If in such cases the limits mentioned above had to be maintained, the depth would have to be reduced. The value of l/r for compression members would increase even more, thereby reducing the permissible axial stress. Unter such conditions, according to the British Code, the girders have to be designed taking into consideration the secondary stresses. Here, prestressing will have a distinct advantage in reducing the secondary stresses and reducing the areas of members, required for direct stresses under service loads, so that the effect of secondary stresses can be ignored and higher axial stress permitted in compression. It would be interesting to know whether this effect of structural prestressing on the secondary stresses has been studied.

While dealing with secondary stresses, it may interest the members to know that on the Indian Railways, a method to eliminate secondary stresses by pre-deforming the girders was developed and is now a standard practice. Briefly, the method is as follows:

1. The change in length of each member under full service load is calculated, and to ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i. e., to avoid changes in lengths of floor and the bracing system between the chords which carry the floor system, a further change in length is applied in the length of all members equal to

$$\frac{\text{change in length of loaded chord}}{\text{length of loaded chord}} \times \text{length of member}$$

   In through spans, this change will be an increase while in the case of deck spans a decrease in the lengths of all members.

2. The actual manufactured lengths of members are the nominal lengths altered as above.

3. The positions and directions of gauge lines of all connection holes in the main gussets and also those in chord joints and the machining of ends are according to the nominal dimensions.

4. The bottom chord is first laid on camber jacks and the required camber is given.

5. The web members are then fitted to the bottom chord and the top chord is also placed in position. The holes at the top end of web members and the top chord joints will obviously not correspond.

6. The members are strained into position and riveted up with the permanent gussets.

This method of pre-deforming the girders was developed during the design of the Wellingdon Bridge near Calcutta (span 350 ft.) in 1930. The method although apparently elaborate, is not difficult to apply in practice and does not entail any appreciable cost. Subsequently, tests were conducted by Dr. NICOLS, on the girders of the Nerbudda Bridge (span 282 ft.) to determine the efficiency of the method. A full report of the tests appears in the Proceedings of the Institution of Civil Engineers, London (1937), in Paper No. 507 "Prestressing Bridge Girders". Tests were also conducted on the girders of a small span (150 ft.) Wunna Bridge for the same purpose. In the new girders for the combined rail-road bridges across the Ganga and the Brahmaputra, spans 400 ft., designed by Messrs. FREEMAN, FOX and Partners of the U.K., the effect of pre-deformation was taken into consideration. These girders were erected by the cantilever method and as the method of pre-deforming, given above, could not be followed, it was a matter of doubt as to whether the required pre-deformation could be achieved. Elaborate tests were recently conducted on the Brahmaputra Bridge also to determine the secondary stresses during erection. It was found that pre-deformation could be achieved only to the extent of about 30% in the case of the girders of the Brahmaputra bridge erected by the cantilever method and of 40% in the other two girders erected on camber jacks. The full theoretical pre-deformation obviously could not be achieved which could be attributed to tolerances in manufacture and fabrication. In the girders of the Brahmaputra bridge the members were of high-tensile steel and consequently were sufficiently slender, so that secondary stresses were not of much consequence. The Nerbudda and Wunna bridge girders were of mild steel and deformation stresses in some members were as high as 50% of the primary stresses, and at one or two points they were even higher.

Hence, the elimination of secondary stresses may be necessary and desirable under certain circumstances and this can be done either by pre-deforming the girder by the method given in the Indian Railway Code of Practice for Steel Bridges or by prestressing the girders by the methods recently developed.

Summary

The report points out an important advantage of the prestressing of open web girders in eliminating secondary stress and thereby, in some cases, permitting higher axial stresses in compression. An alternative method of reducing secondary stresses by pre-deforming the girders, as adopted in Indian Railways, is mentioned, and the respective tests are described.

Résumé

Cette communication met en évidence le considérable intérêt qui s'attache à la précontrainte des poutres à treillis du fait de l'élimination des contraintes secondaires et, par voie de conséquence, de l'accroissement parfois possible des contraintes axiales de compression. On se réfère également à un autre moyen permettant de réduire les contraintes secondaires et qui consiste à réaliser une déformation préalable des poutres, solution que les Chemins de fer de l'Inde ont adoptée. Les essais correspondants sont décrits.

Zusammenfassung

Der Bericht behandelt einen wichtigen Vorteil der Vorspannung von Stahlfachwerkträgern: Die Möglichkeit der Ausschaltung von Nebenspannungen, womit in einigen Fällen eine bessere Materialausnützung erreicht werden kann. Eine Methode zur Verringerung der Nebenspannungen durch Vorkrümmung der Träger, wie sie bei den Indischen Staatsbahnen im Gebrauch ist, wird zusammen mit zugehörigen Versuchen beschrieben.