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Mass-Produced Steel Structures of Open-Web Sections

Production en série de treillis

Massengefertigte Stahlbauten aus Fachwerkträgern

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Abstract

The light weight steel structures developed by the Central Mechanical Engineering Research Institute, Durgapur using openweb sections have effected considerable saving in structural steel. A brief outline of the design procedures, production technology developed and experiments done, have been reported in this paper.

Notation

P : Load (Kg)

E : Young's modulus (Kg/Cm^2)

G : Shearing modulus (Kg/Cm^2)

h : Depth of the joist section (Cm)

l : Length of the joist (Cm)

I_f : Moment of inertia of flange section (Cm^4)

J_f : Torsional property of section of flange member (Cm^4)

θ : Angle of inclination of web lacing with flanges (rad)

I_w : Moment of inertia of web member about its axis of bending (Cm^4)

J_w : Torsional property of section of web member (Cm^4)

J : Equivalent torsional property of section of openweb beam (Cm^4)

A_w : Cross sectional area of web lacing (Cm^2)

A_f : Cross sectional area of flange member (Cm^2)

I : $A_f.h^2/2$ (Cm^4)

$(I)_e$: Equivalent second moment of area of an openweb beam across the lacing assumed as a solid beam (Cm^4)

δ_s : Deflection due to shear (Cm)

δ_b : Deflection due to bending (Cm)

η_b : Equivalent length factor for the bending of the main members (dimensionless)

ϕ : Angle of twist of joist in radins

ϕ_1 : Angle of twist of joist flanges in radians.

1. Introduction

With a view to effect economy in structural steel by reducing the weight of the structure[1] and also to develop methods of easy and quick fabrication and erection, research on light weight steel structure are in progress at the Central Mechanical Engineering Research Institute in India. This is of immediate practical importance in the country like India where there is a continued chronic shortage of steel. The openweb construction meets this very advantageously. Furthermore, this type of construction fits in Indian conditions since it is labour intensive and the results of these fabricated sections economically are also favourable. The sections provided are ideally used both in shear as well as bending which is rarely the case with rolled steel sections. The pioneering work done by the American Steel Joist Institute[2] in developing this type of joists for floor beam construction revealed its economic values and merits over the rolled joists. These sections may be 'tailor made' to suit the exact design requirements unlike the rolled sections which are dependent on standard rolling dimensions. The advantages of this type of construction are (a) strength with lightness, (b) substantial savings in material and fabrication cost, (c) speedy erection, (d) ease in handling, (e) reduced foundation costs, (f) clean appearance, and (g) reduced surface of steel work, bringing savings in initial and periodic maintenance.

2. Design concept, properties and stability of openweb sections

a. Openweb sections

A typical view of open beam is shown in Fig 1.

In the openweb sections, angles are used to form the flanges whereas rods between the flanges forming an warren trusses are used to resist shear force For the design of the section, the maximum bending moment to be resisted is calculated and the product of area of each angle with depth is approximately obtained by dividing the bending moment by one and half time the allowable stresses. Number of combination of depth and the area can be chosen to get the required resistance. However, to utilise fully, it is better to use standard available angles and calculate the depth accordingly. For shear the vertical component of the angle lacings has to be equal to the shear force acting at a particular section and as such again two factors are available to control namely, the angle of inclination of lacing and area of lacing. Here again to utilise the material fully the area of standard rod is chosen and the angle is adjusted as required.

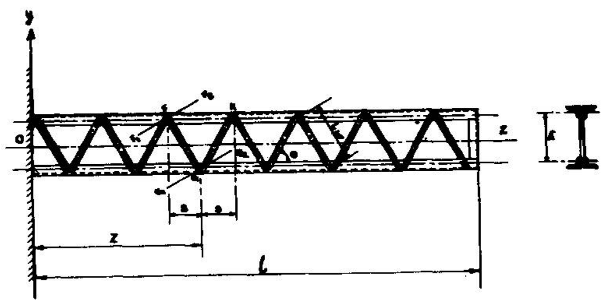

Fig 1
Geometry of an openweb section of cantilever beam

b. Properties

Knowing the different components the moment of inertia, can be calculated for this fabricated section by following elementary theories and hence the bending and shear resistance. However, experiments have shown that the moment of inertia, the torsion, deflection and stresses are function of depth, area of flange, area of the lacing and inclination of the lacing. Even under optimum condition it is rather difficult to get the values of rigidities as calculated by elementary theory. Hence a form factor has to be used which modifies the moment of inertia. Hence, stresses can thus be easily calculated. The openweb beam has been treated as a solid beam with modified value of bending and torsional rigidity. A simple method for establishing the bending rigidity has been given in earlier publications[3,4] where the web lacing has been treated as hinged with the flange and thus a correction factor is incorporated. The test set up for determination of bending rigidity is shown in Fig 2.

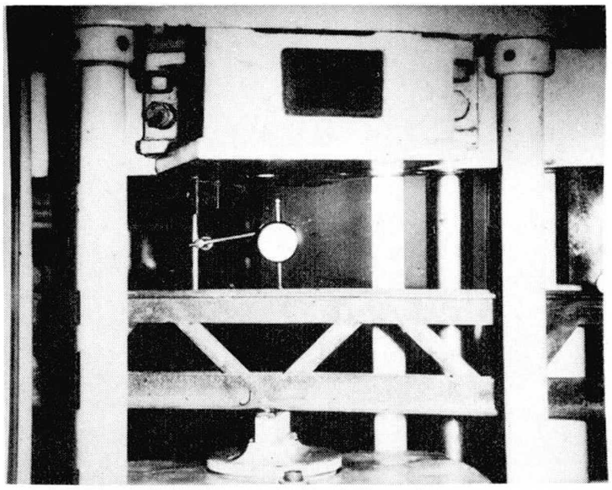

Fig 2
Test set-up for openweb joists subjected to mid-point loading

The modified value of moment of inertia of the openweb beam is given by -

$$(I)_e = \frac{1}{\frac{(\delta_b)^3}{I} + \frac{24\,\delta_b}{A_w l^2 \operatorname{Sin}^2\theta \operatorname{Cos}\theta}} = I.K$$

$$\text{where, } K = \frac{1}{(\delta_b)^3 + \frac{12\,\delta_b}{\operatorname{Sin}^2\theta \operatorname{Cos}\theta}\,\frac{A_f}{A_w}\,(h/l)^2} \quad ..1$$

The torsional rigidity investigation[4,5] has been done where the distortion of the cross section during twist was taken into consideration. The computed values of torsional rigidity are compared with the results of torsion tests on cantilever beams of openweb sections having different web stiffnesses and a good agreement was found between the experimental and calculated values. The torsion test set up of the joists is shown in Fig 3. The torsional rigidity obtained can be utilised in design formulae for calculating the critical compression stress in bending and the allowable stress for the openweb section.

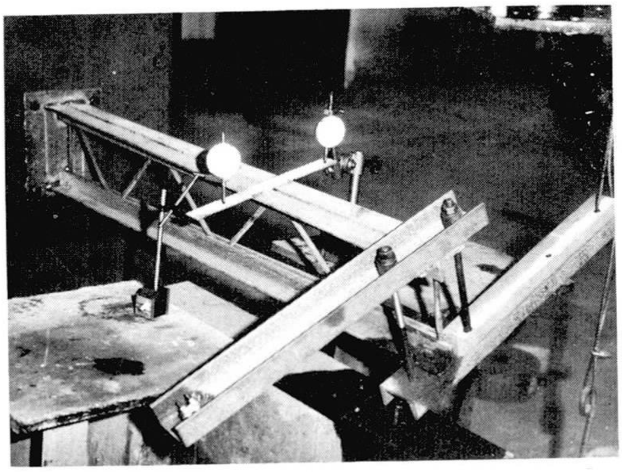

Fig 3
Torsion test set-up for the joists

The expression for ϕ and ϕ_1 are given by

$$\phi = \frac{2\kappa}{\lambda\kappa - \mu^2} \cdot \frac{Tl}{2\,GJ_f} = K\,\frac{Tl}{2GJ_f}$$

$$\phi_1 = \frac{2\mu}{\lambda\omega - \mu^2} \frac{Tl}{2\,GJ_f} = K_1 \frac{Tl}{2\,GJ_f}$$

where, $\lambda = \frac{EI_f}{64GJ_f}\pi^4(h/l)^2 + 2\frac{EI_w}{GJ_f}\tan\theta \sin\theta \Big[\frac{\pi^2}{16}\cos^2\theta\,(2 - \cos\frac{\pi s}{2l})$

$+ 3/2(l/h)^2 \sin^2\theta\,\{5 - 8/\pi - (8/\pi - 1)\cos \pi s/2l\} - 9/4(l/h)\sin\theta \cos\theta \sin\pi s/2l\Big]$

$+\frac{J_w}{J_f}\tan\theta \sin^3\theta\,\pi^2/16\,(1 + \cos\pi s/2l)$

$\mu = \frac{EI_w}{GJ_f}\tan\theta\sin\theta\,[\pi/2\,(l/h)\sin\theta\cos\theta\sin\pi s/2l + 3(l/h)^2\sin\theta$

$\{5 - 8/\pi - (8/\pi - 1)\cos\pi s/2l\} - 3(l/h)\sin\theta\cos\theta\sin\pi s/2l]$

$+\frac{J_w}{J_f}\tan\theta\sin^2\theta\,\pi/4\,(l/h)\cos\theta\sin\pi s/2l$

$\omega = \pi^2/4 + 2\frac{EI_w}{GJ_f}\tan\theta\sin^3\theta\,(l/h)^2\,\{8 - 12/\pi - (12/\pi - 1)\cos\pi s/2l\}$

$+\frac{J_w}{J_f}\tan\theta\sin\theta\,(l/h)^2\cos^2\theta\,(1 - \cos\pi s/2l)$

c. Stability

From the shape of the openweb beam it becomes obvious that the ratio of the moment of inertia in the stiff and slender plane is quite high and as such critical stresses in compression bending is quite low which imposes certain more restriction on the total use of the material. The section is also very much susceptible to lateral buckling and it has been found that permissible load goes on increasing as point of application of the load is shifted from top flange to the bottom flange. The buckling load for such sections have been found theoretically by Basole[6] to be

$$P_{cr} = 45.66\,\frac{I_{yy}Eh}{l^3}$$

when load is applied at bottom flange

and $$P_{cr} = 15.80\,\frac{I_{yy}Eh}{l^3}$$

when load is applied at top flange.

These results have also been verified experimentally, where

I_{yy} : Moment of inertia in the slender plane
E : Modulus of elasticity
h : Depth of the section
l : Span of the beam

In the extreme case the allowable load by applying to lower lacing flange may be as high as three times, from the point of view of lateral stability. In addition to these, even the small amount of prestressing is found to increase the stability of the girder.

3. Application of openweb section in different structres

a. Portal frames

The analysis of openweb sections has been extended to portal frames.[7] A simple rig was set up for the portal frame tests (EP 1 series) as shown in Fig 4. This consisted of a reaction frame which was identical in shape to the test frame but of enlarged dimensions to accommodate the test frame and loading tackles. The loads were applied by means of turn buckles and the load indications were obtained by spring dynamometers designed for this purpose. The dynamometers were calibrated in a universal testing machine. The loads were gradually applied at the purlin points as shown in Fig 4 and increased in steps of 20 Kg up to a maximum of 140 Kg at each point. The strain gauge measurements were made at some selected points along the portal frame.

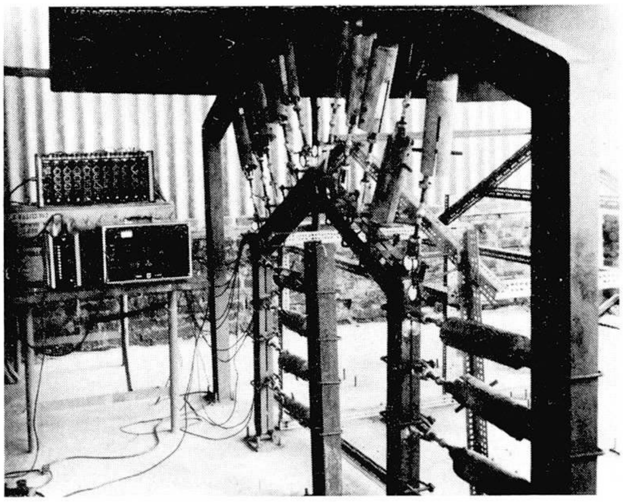

Fig 4
Test set up for the portal frame

The tests up to destruction of portal frames, however, could not be carried out with the turn backle loading system and the spring dynamometer loads indicating device. These tests were carried out with Lossenhaus-seunerk hydraulic jacks and 'individual testing cylinder plant'. These reaction frames were built up in parallel providing enough rigidity against deflection.

Four point loading with suitable loading links was applied on each member of the frame and the frames were tested up to destruction. The strain measurements at selected points along the flange of the portal frames, EP 2 series were taken. A further series of four portals varying the section parameters were tested up to destruction. The failure loads are given in Table 1.

It may be seen that the ultimate failure loads vary considerably depending on the nature of failure of the portals. In EP 2 series, where two portals were tested up to destruction, the eave joint failure of one of the portals, EP 2A, resulted in its premature collapse well before the material had reached its yield stress. Particular care was taken in welding and stiffing the cave joints of the portal, namely

TABLE 1

The ultimate failure loads of openweb portal frames subjected to joint loads

Portal series	Span Cm	Height Cm	Dimensions Section depth Cm	Flange area Cm^2	Web rods dia mm	Ultimate failure load (experimental) of portal frame Kg	Yield load (Theoretical Web Kg	Flange Kg
EP2A	160	95	10	5.66	10	275	350	412
EP2B	160	95	10	5.66	10	400	350	412
EP3	160	97	11	5.66	6	275	138	470
EP4	160	97	11	5.66	12	850	560	470
EP5	160	97	11	9.00	10	782	390	700
EP6	160	97	11	9.00	12	1150	560	700

EP2B and the next series of portals. However, when the stresses in web rod and a flange element just reached the yield point, the frame buckled laterally which precipitated its instantaneous collapse Fig 5(i).

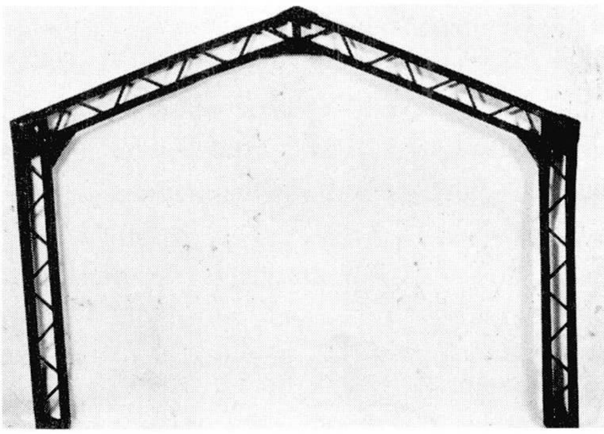

Fig 5(i)
Failure by lateral instability of portal

In the next series of portal tests, adequate lateral supports to the frame were provided. In EP3 series, the yield loads predicted by theory for the web rods at the section of maximum shear force and for the flange element at the section of maximum bending moment were respectively 138 Kg and 412 Kg. In the test, the ultimate loads of the portal reached was 275 Kg which was twice the yield load of the web, but much lower than the theoretical yield load predicted for the flange element. This shows that the theoretical yield load predicted for the flange element is not valid in this case, because, in the post-buckling range of the web rods, the depth and unsupported length of the flange elements have changed completely thus changing the yield loads of the flange element. A number of rods had buckled in the two highly stressed members of the portal before its ultimate failure as shown in Fig 5(ii). Similar trend was observed in two other test series EP5 and EP6. The ultimate failure loads of these portals were again twice the yield loads of the web rods.

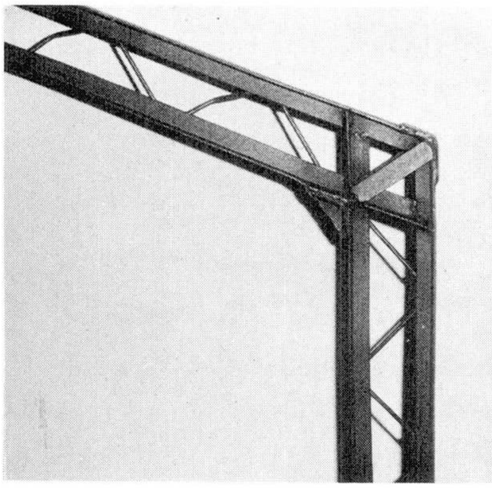

Fig 5(ii)
Buckling of the web rods of the portal

In EP4 series, the ultimate failure load of the portal is only 52 per cent higher than the yield load of the web rod, whereas it is 81 per cent higher than the flange yield load. This result fairly agrees with the ultimate loads obtained for the openweb joists. This portal tested up to destruction is shown in Fig 5(iii).

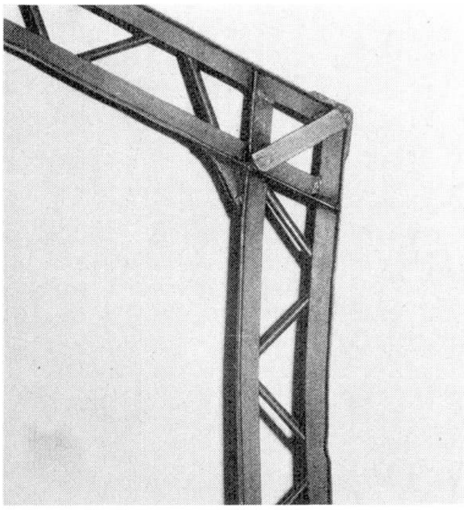

Fig 5(iii)
Bending failure of the flange angles of the portal

In addition to model test on such structures prototype testing were also carried out. The photograph of the portal testing is shown in Fig 6.

b. Grid frames

The analysis of openweb sections was next applied to grid frames. The grid consisted of four main beams and three cross beams which were uniformly spaced. The main cross beams were fabricated using a pair of 40 x 40 x 5 ms angles for the top and bottom flanges and 10 mm dia rods for the web loadings. The angle of web lacing was kept constant at 45° in each case. The dimensions and sectional properties of these built-up joists are as shown in Table 2.

The main beams were provided with rollers at 480 cm centre, the effective span of the simply supported beams. The rollers were made from 2.54 cm dia rod. The whole frame is placed over eight brick pillars. The set up for the openweb interconnected beam system is shown in Fig 7. The deflection and bending moment curves for

TABLE 2
Properties of sections of main and cross beams

	Span Cm	Section depth Cm	Flange area Cm^2	Web rod mm	Ic Cm^4	J Cm^4
Main beam	480	20	7.56	10	935	1.67
Cross beam	120	20	7.56	10	156	3.88

the interconnected beam system are shown in Fig 8 when the grid is centrally loaded.

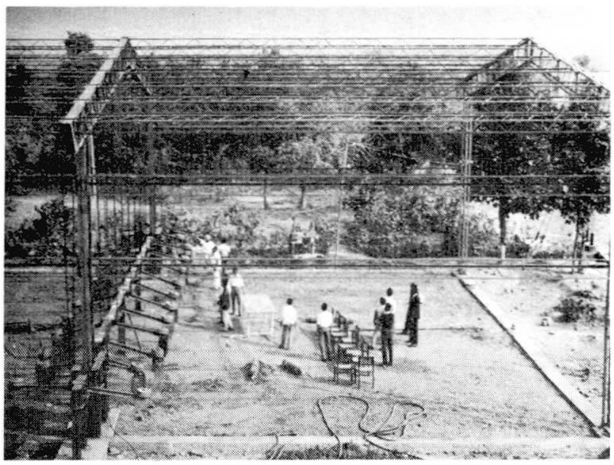

Fig 6
Prototype testing of portal frame

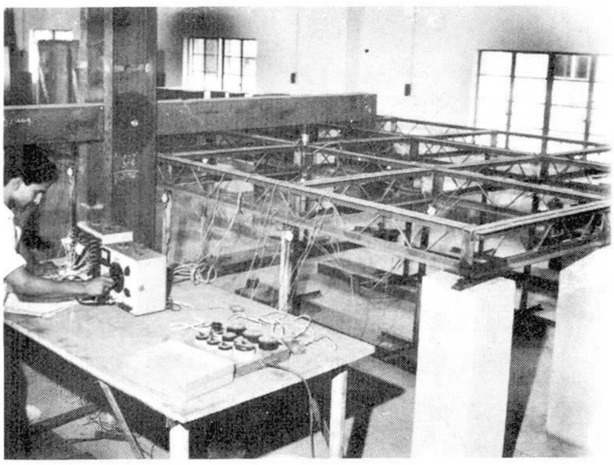

Fig 7
Test set-up for the interconnected beam system

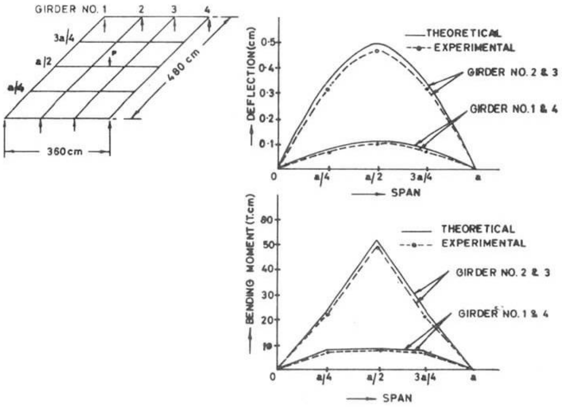

Fig 8

Deflection and bending moment curves for the interconnected beam

The same grid is tested with the Cast in situ slab which results in composite construction. This sort of construction results in appreciable economy as well as speedier construction. The thickness of the slab provided is 7.5 cm. Channel shear connections of 50 x 50 x 6 were used for the composite action of the slab, which are spaced at 90 cm c/c. The dimensions and sectional properties of one composite beam are shown in Table 3.

The test set up for the open web interconnected beam with cast in situ slab is shown in Fig 9. Deflections were measured, one at the central section and two at the quarter section. The strain measurements were also made at the bottom flange of the beam at some selected points.

TABLE 3

Properties of sections of main and cross beams of openweb interconnected beams with cast in situ slab

	Span Cm	Section depth Cm	Web rod mm	Ic Cm^4	J Cm^4
Main beam	480	27.5	10	3407	994
Cross beam	120	27.5	10	156	3.88

As before the deflection and bending moment curves for the interconnected beams with cast in situ slab is shown in Fig 10.

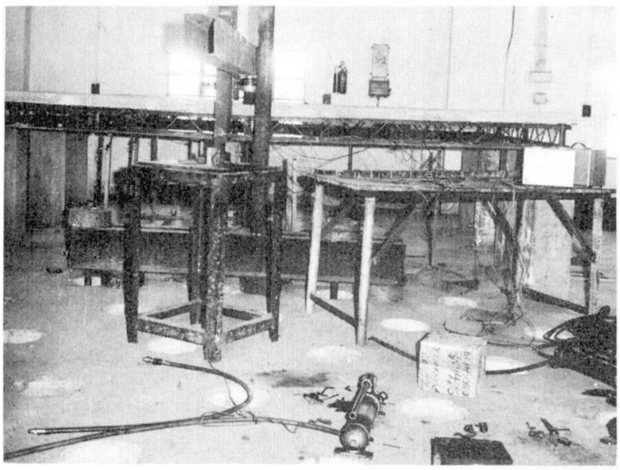

Fig 9

Test set-up for the interconnected beam system with cast-in-situ slab

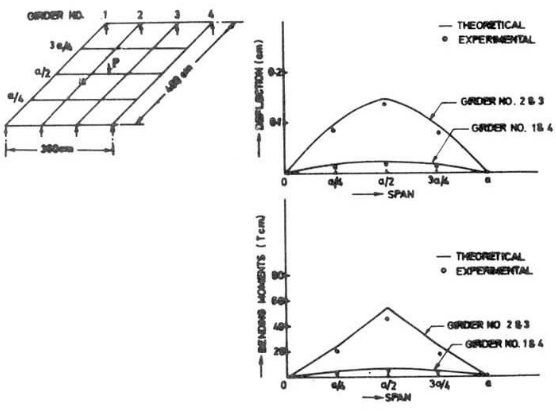

Fig 10

Deflection and bending moment curves for the interconnected beam with cast-in-situ slab

In all the above cases it was found that the theoretical values are in good agreement with the experimental results in elastic range which justifies that the openweb beams behaves like solid web beams with modified values of rigidities and hence the stresses.

4. Production Technology

Efforts have been made in the structure division of Central Mechanical Engineering Research Institute in standardising the Sections with its suitability for mass production. The cost of fabrication has been reduced to a minimum by using zigs and fixtures developed here. CMERI has become the best place for development of this type of structures in view of its mechanical engineering bias, in Workshop practice. Fixture have been developed for the bending of bars in different angles, and for welding arrangements to reduce the distortion of the sections to the minimum.

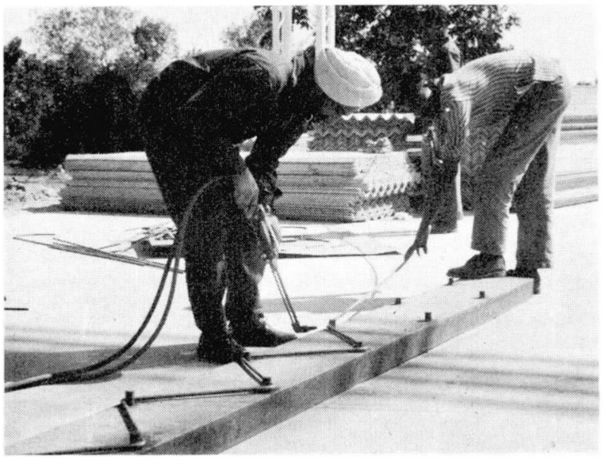

Fig 11
Bending of rod for formation of lacing

Fig 11 shows the bending of rod for formation of lacing, for the purlin. The arrangement was simple as can been seen from the figure. Rod posts were so arranged on the steel fixture as to bend the rod at right angle at each bend. Oxy-acetylene torch was used for quick heating of the rod at bends to facilitate manual bending to close dimensional tolerances.

The second fixture, shown in Fig 12(a), was made to hold the two side angles in position ready to take the bend lacing. Lacings were welded to angles in this manner to minimise distortion. Clamps holding the angles to the frame were released after the welds had cooled down. A close-up of the fixture is shown in Fig 12(b).

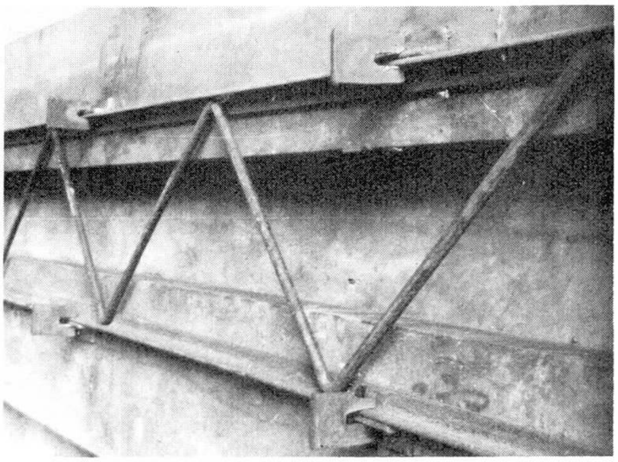

Fig 12(a)
Welding of lacing to the angles

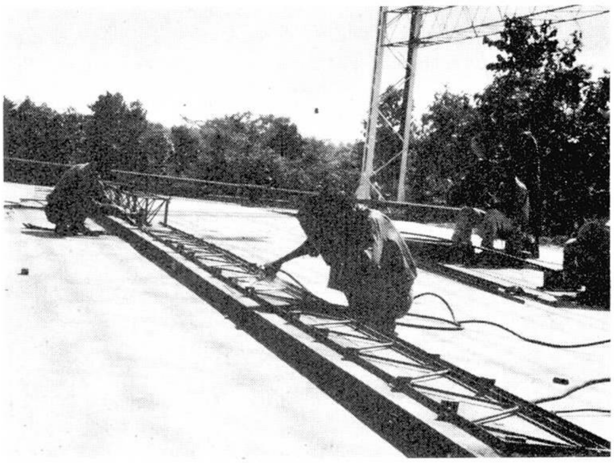

Fig 12(b)
Close-up photograph showing the clamping device

The third fixture was made to hold a third angle in position to facilitate welding of two similar lacings with this angle and the already formed section of the purlin without distortion. The complete purlin was thus formed. The fixture is shown in Fig 13.

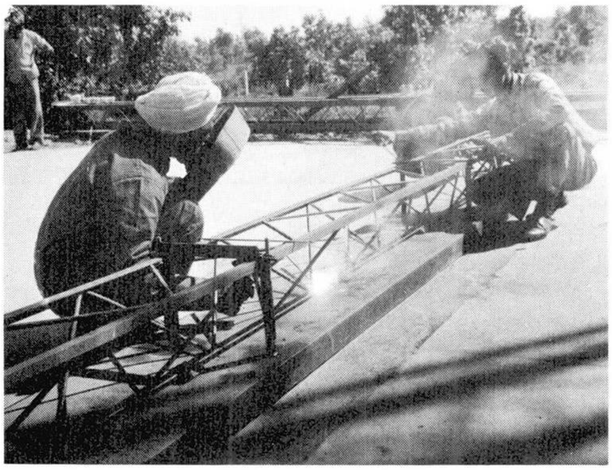

Fig 13
Fixture for assembly of the complete purlin

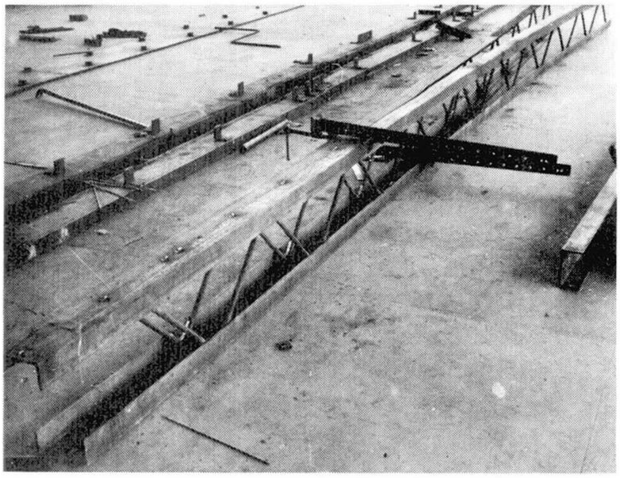

Fig 14
Assembly of a complete column

For simplicity of fabrication, the pitched portals were broken up in columns and rafters. The rafters or columns were first fabricated in halves using a fixture similar to that used for purlins (Fig 12a). The two halves were assembled by means of hand-vice clamps and then tack-welded as shown in Fig 14. The sequence followed in welding was 'Centre-Outwards' and this ensured consistent alignment. Most of the columns and rafters were straight excepting a few which were straightened by heating the convex sides to black heat and then allowing them to cool in air.

Fig 15
Assembly of a complete portal

Fig 15 is shown the assembly of columns and rafters into complete portals.

5. Acknowledgements

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SUMMARY

The research for an economical and light weight design for industrial structures by CMERI, Durgapur, has resulted in the development of openweb steel joints of different shapes for portal frames and grid frames. The openweb sections are now extensively used in different types of structures. After extensive investigation simple design methods were established for openweb beams and openweb portal frames and grid frames. It has provided test and research data for development of new types of openweb sections. The experiments have shown that the design methods established give reasonable estimates of stresses.

RESUME

La recherche d'une solution économique et légère pour la construction de halles industrielles en acier a conduit la CMERI de Durgapur à développer des treillis de différentes formes pour les cadres. Les treillis sont employés maintenant couramment pour différentes applications. Des recherches approfondis établissent des méthodes de calcul simples pour les poutres et les portiques. Des données expérimentales permettent de développer de nouveaux types de treillis. Des essais ont montré que la nouvelle méthode donne une bonne estimation de la réalité.

ZUSAMMENFASSUNG

Die Suche nach einer wirtschaftlichen und gewichtsmässig leichten Ausführung für Stahlbauten durch das CMERI in Durgapur führte zur Entwicklung von Fachwerk-Stahlträgern verschiedener Formen für Portalrahmen. Die Fachwerkträger werden jetzt bei verschiedenen Bauwerken angewandt. Nach eingehender Untersuchung wurden einfache Methoden für Balken und Rahmen festgelegt. Es ergaben sich daraus Versuchs- und Forschungsdaten für die Entwicklung neuer Typen von Fachwerkträgern. Experimentelle Versuche zeigten, dass die neue Methode eine gute Schätzung der Beanspruchung erlaubt.

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