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ISI HANDBOOK
FOR
STRUCTURAL ENGINEERS

7. SIMPLE WELDED GIRDERS

INDIAN STANDARDS INSTITUTION

ISI HANDBOOK
FOR
STRUCTURAL ENGINEERS

No. 7

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FOR
STRUCTURAL ENGINEERS

7. SIMPLE WELDED GIRDERS

INDIAN STANDARDS INSTITUTION
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FOREWORD

This handbook, which has been processed by the Structural Welding Sectional Committee, SMDC 15, the composition of which is given in Appendix C, had been approved for publication by the Structural and Metals Division Council of ISI.

The need for increase in steel production and conservation of steel through efficient use of available resources and technique received the attention of the Planning Commission from the earlier stages of the First Five-Year Plan. At the instance of the Planning Commission, ISI took up a steel economy project involving formulation and implementation of Indian Standards relating to production and use of steel. One of the objectives of this programme was to prepare Indian Standards on use of welding as a mode of structural fabrication.

In order to reduce the work involved in design production and to facilitate the use of various Indian Standard codes of practice, ISI undertook preparation of a number of design handbooks. This handbook, seventh in the series, relates to simple welded girders. Other handbooks in the series are:

SP : 6 ISI Handbook for structural engineers:

- SP : 6 (1)-1964 Structural steel sections
- SP : 6 (2)-1962 Steel beams and plate girders
- SP : 6 (3)-1962 Steel columns and struts
- SP : 6 (4)-1969 Use of high strength friction grip bolts
- SP : 6 (5)-1970 Cold formed, light gauge steel structures
- SP : 6 (6)-1971 Application of plastic theory in design of steel structures

Intelligent and economical use of a code by a designer may be made only by a thorough understanding of the physical behaviour of the structures to which the code applies, the basic information on which the code is based and the method of fabrication. Since such knowledge is not till now available to the structural engineer who has undergone the usual university training it is one of the aims of this handbook to supply this.

Clauses 1 and 2 describe in brief the various welding and cutting processes. Clause 3 covers the effects of shrinkage and distortion due to welding and methods of reducing them. The preparation of fusion faces for different types of welding are covered in **4**. The welded joints and their configuration are covered in **5** and **6**. Analysis of structural members and procedure of designing structural joints using welding are given in **7**.

In the preparation of this handbook the technical committee has derived valuable assistance from Dipl.-Ing. Claus Hofmann, Consulting Engineer, Germany. The preliminary draft of this handbook was prepared by Mr. Hofmann. This assistance was made available to ISI through the good offices of the Government of West Germany.

No handbook of this kind can be made complete for all times to come at the very first attempt. As designers and engineers begin to use it, they will be able to suggest modifications and additions for improving its utility. They are requested to send such valuable suggestions to ISI which will be received with appreciation.

1. SCOPE

1.1 This handbook deals with simple welded beams and girders subjected to static loads and covers the use of steel in general construction using welding as the method of fabrication.

1.2 This handbook makes reference to a number of Indian Standards on use of steel and welding, a list of these and other relevant standards is given in Appendix C.

2. METHODS OF WELDING

2.1 Metal Arc Welding — Manual metal arc welding is the most widely adopted process used in the fabrication of steel structures. The advantage of this process is that the arc concentrates a large amount of heat at high temperature (5 000 to 6 000°C) required to melt the parent metal and welding electrode at the fusion zone. There will also be no atmospheric contamination of weld metal that may cause any deterioration in the weld deposit and the joint.

2.1.1 In metal arc welding the filler material and the parent metal are fused into weld metal. An arc is struck with a voltage of 45 to 100 V and a current of 50 to 400 A depending on the size of electrode used. Either dc or ac can be used for metal arc welding.

2.1.2 The metal from the tip of the electrode is transferred to the joint. The metal transfer can be either as fine spray or as globules (short arc) depending on the method of welding. Melting rate is higher in the case of the former. The rate of deposition depends on the flux covering and the performance characteristic of the electrode (*see* IS : 815-1966*).

2.1.3 When dc is used for welding, the positive pole is the hotter one in the case of flux cored electrodes, rutile coated electrodes and the electrodes with substantial amount of iron oxide in the flux covering. It is cooler in the case of basic coated electrodes. In ac welding, the flux coating does not have any such influence.

2.1.4 Flux coated electrodes are normally employed in manual metal arc welding but continuous coated electrodes are used in automatic welding processes to have increased rates of welding.

2.2 Submerged Arc Welding — The deposition rate is high in submerged arc welding, where bare wire (or a number of wires) of diameter 1·6 to 12 mm are used. The arc is struck under a layer of flux powder. The process could be automatic or semi-automatic. In the automatic

*Classification and coding of covered electrodes for metal arc welding of mild steel and low alloy high-tensile steel (*revised*).

process the feed of flux and wire and the rate of welding are automatically regulated according to the size of weld desired. In the case of semi-automatic process the feed of wire and flux are automatically regulated while the operator has to control the rate of deposit. Both ac and dc can be used for submerged arc welding. Voltage between 25 and 40 V and currents 250 to 1 200 A are used. With wires of diameter above 4 mm currents up to 2 000 A may be used. A portion of flux powder melts and in the form of slag protects the weld metal from atmospheric influence and prevents rapid cooling of the joint (*see* Fig. 1). The unfused portion of the flux is collected and used again.

2.2.1 For general recommendations regarding settings, rate of deposition, size of the filler wire, etc, reference may be made to Appendix C of IS : 823-1964*.

2.2.2 The application of automatic submerged arc welding is generally limited to long, continuous and uninterrupted seams of welds in plates thicker than 5 mm. The welds are normally made in flat position. In the manufacture of boilers and pressure vessels even circumferential joints are made by automatic submerged arc welding process, the welding head remaining stationary and the vessel rotating about its axis. Submerged arc welding process is also used as a manual welding process when it is called semi-automatic process and it has certain inherent limitations because the arc is submerged.

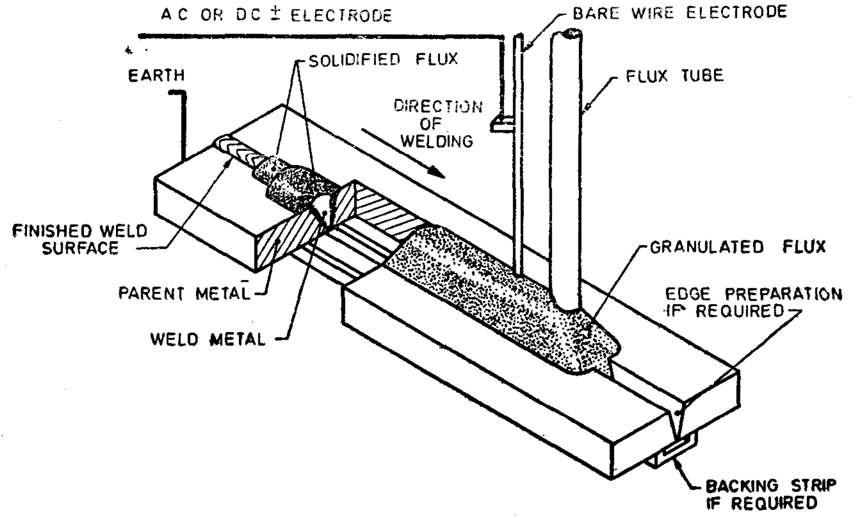

FIG. 1 SUBMERGED ARC WELDING

*Code of procedure for manual metal arc welding of mild steel.

2.3 Inert-Gas Metal Arc Welding — In recent years, inert-gas metal arc welding has gained importance in structural welding. In this process bare wires are used as electrodes. The arc is surrounded by shielding gases like air, carbon dioxide, argon, helium, etc. The process permits currents of very high density (50 to 170 A/mm^2) with temperature up to 20 000°C. High welding rates may, therefore, be achieved through thin wires. Shielding gases are used either in their pure form or with addition of oxygen in proper proportions. The heat is transmitted by dissociation of the gases in the arc and a recombination at the fusion zone. The transfer of metal in the argon protected arc is in very fine drops, while in the case of carbon dioxide with short-circuited arc it is in big droplets which results in high spatter loss.

2.3.1 Where argon is used for inert gas welding of steels, 1 to 3 percent oxygen mixed with argon which reduces the surface tension of the weld metal and of the droplet during their transfer.

2.3.2 The composition of filler wires for inert gas arc welding will normally be the same as that of the material to be welded. They are covered in IS : 6419-1971* and IS : 6560-1972†.

2.4 Gas Welding — Use of gas welding is very limited in structural steel fabrication since the wider heat affected zone leads to excessive deformation of the welded members. If the plate is thicker than 8 mm, initial heating of each run takes a long time resulting in very slow rate of welding. Nowadays gas welding is used for welding sheets and in pipe work.

2.4.1 The fusion faces of the components to be joined and the filler material are melted from an oxy-acetylene flame from the welding torch. Oxygen and acetylene are normally used in the proportion of 1 : 1. This mixture produces a temperature of about 3 000°C. Propane and hydrogen gases are also occasionally used.

2.4.2 For sheets of thickness up to 4 mm, welding is done with leftward technique, that is, the torch is moved from right to left the filler material melting in advance of the parent metal. For thicker material, the rightward technique is adopted since it is quicker and economical and there is better utilization of heat.

2.4.3 In Table 1 is given consumption of oxygen and acetylene gases and their pressures, rate of welding of plates using different sizes of welding torches are given. The information is based on well accepted practice being followed in the welding industry. The welding rate given are the average values including the time for flame adjustment and change of filler

*Specification for ferritic steel rods and bare electrodes for gas shielded arc welding.

†Specification for molybdenum and chromium-molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding.

rods but excluding the time required for the preparation of weld joints. The consumption of gases and rate of welding fluctuate according to the thickness and the condition of the metal to be welded and the experience and the skill of the welder. The consumption of acetylene gas could be on the average up to 10 percent below the consumption of oxygen depending upon whether the welding flame is oxidizing, neutral or reducing.

2.5 Gas Cutting — Gas cutting is the most commonly used thermal cutting process and for the preparation of the fusion faces in steel. Gas cutting is possible in the case of metals whose oxidation temperature is lower than the fusion temperature, for instance steel. The process of heating is the same as for the gas welding. Once the fusion temperature is reached at the place to be cut, the flow of oxygen concentrated through a nozzle impinges and cuts the steel by oxidizing it. The oxide is blown away as slag due to the high velocity of oxygen. Oxidation of steel is an exothermic reaction and, therefore, the heat of the flame is sufficient to maintain the cutting temperature. For practical reasons, however, the pre-heating flame remains on.

2.5.1 In Table 2 are given the consumption of oxygen and acetylene, speed of cutting and the nozzles to be used for plates of different thicknesses. Information in regard to gas cutting using hydrogen and liquid petroleum gas are also included in the table. Gouging, which is carried out on the same principle as gas cutting, is done using special nozzles. The nozzles are held at an angle of 15° to 30° to the work-piece. The gouging flame blows out the molten material out of the kerf.

2.5.2 For cutting high alloy steels like austenitic chromium nickel steels and mild steels of higher thicknesses, powder cutting torches are employed in which iron powder is fed along with oxygen. This facilitates quicker oxidation of the base material.

2.5.3 For application of the gas cutting in fabrication of steel structures reference may be made to Section V of IS : 800-1962*.

2.6 Arc Cutting—Arc cutting process is employed for cutting alloy steels, cast iron and non-ferrous metals. In this process the arc melts the base metal only locally. The molten metal is then blown off by a high velocity gas or gas mixture.

2.6.1 The arc cutting using inert gas can be effected using the same instrument as for inert gas welding with slightly higher current and voltage so that the metal not only melts but also evaporates. The gas with high velocity and at high temperature from the nozzle blows away the molten metal and simultaneously protects the cut edges from oxidation.

*Code of practice for use of structural steel in general building construction (*revised*).

TABLE 1 AVERAGE VALUES FOR GAS CONSUMPTION AND WELDING RATE IN GAS WELDING

(*Clause* 2.4.3)

Torch Size	Plate Thickness	Pressure	Oxygen Consumption		Acetylene Consumption		Welding Time Rate*		Welding Technique
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
mm	mm	kg/cm	l/h	l/m	l/h	l/m	min/m	m/h	
0·3 to 0·5	0·3	2·5	45	3	45	3	4	15	Leftward welding
	0·5	2·5	45	4·5	45	4·5	6	10	
0·5 to 1	0·5	2·5	80	6·7	80	6·7	5	12	
	1	2·5	80	13·3	80	13·3	10	6	
1 to 2	1	2·5	150	16·7	150	16	6·7	9	
	2	2·5	150	37·5	150	37·5	15	4	
2 to 4	2	2·5	330	44	330	44	8	7·5	
	4	2·5	330	132	330	132	24	2·5	
4 to 6	4	2·5	500	111	500	111	13·3	4·5	
	6	2·5	500	250	500	250	30	2	
6 to 10	6	2·5	800	267	800	267	20	3	
	10	2·5	800	615	800	615	46	1·3	
10 to 14	10	2·5	1 250	625	1 250	625	30	2	Rightward welding
	14	2·5	1 250	1 390	1 250	1 390	67	0·9	
14 to 20	14	2·5	1 800	1 385	1 800	1 385	46	1·3	
	20	2·5	1 800	3 000	1 800	3 000	100	0·6	
20 to 30	20	2·5	2 400	2 670	2 400	2 670	67	0·9	
	30	2·5	2 400	6 000	2 400	6 000	150	0·4	

*Average values including flame adjustment and change of filler rods, excluding preparation of weld. The gas consumption, welding time and welding rate fluctuate according to the condition of the metal, conscientiousness and experties of the welder. The acetylene consumption can be on an average up to 10 percent below the consumption of oxygen, depending on the adjustment of the flame.

For special details, such as permissible stresses, throat thickness, workmanship, flame conditions, etc, *see* IS : 1323-1966 'Code of practice for oxy-acetylene welding for structural work in mild steel'.

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TABLE 2 CONSUMPTION OF OXYGEN AND ACETYLENE GASES FOR CUTTING STEELS

(*Clause* 2.5.1)

Plate Thickness	Cutting Speed	Distance from Work-piece	Nozzle		Oxygen Consumption			Acetylene* Consumption		Hydrogen Consumption		City Gas Consumption	
			Cutting Nozzle	Heating Nozzle	Pressure	Litres per Hour of Cutting	Litres per Metre of Cutting	Litres per Hour of Cutting	Litres per Metre of Cutting	Litres per Hour of Cutting	Litres per Metre of Cutting	Litres per Hour of Cutting	Litres per Metre of Cutting
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
mm	mm/min	mm			atm								
3	300 to 335	2 to 3	3 to 10	3 to 30	1·5	1 500	87 to 75	250	14 to 12	1 000	55 to 50	650	36 to 33
10	260 to 300	2 to 3	10 to 30		2·5	2 100	134 to 117	350	22 to 20	1 400	90 to 78	900	58 to 50
30	195 to 240	3 to 5	30 to 60	30 to 100	3·5	3 600	308 to 250	600	51 to 42	2 300	197 to 160	1 450	122 to 101
60	150 to 185	3 to 5	60 to 100		5·0	7 000	780 to 625	750	83 to 68	3 000	333 to 270	1 900	211 to 171
100	120 to 150	4 to 6	100 to 150	100 to 250	6·5	11 300	1 577 to 1 255	900	125 to 100	3 800	528 to 422	2 400	333 to 267
150	100 to 130	4 to 6	150 to 200		7·5	15 000	2 500 to 1 666	1 000	183 to 141	4 400	733 to 564	2 800	467 to 359
200	80 to 115	6 to 9	200 to 250		9·0	19 300	4 020 to 2 797	1 200	250 to 174	4 800	1 000 to 696	3 000	625 to 435
250	70 to 100	7 to 10	200 to 250		10·0	23 600	5 619 to 3 933	1 300	310 to 217	5 200	1 083 to 867	3 300	786 to 550

NOTE — The above values are average, differences are possible due to the following causes:

a) Difference in the composition of the metal;

b) Difference in purity and accuracy of the cut surface; and

c) Difference in the conditions of the gas, especially temperature and purity of the oxygen.

*If taken from low-, medium- and high-pressure generators. If taken from cylinders, set the acetylene pressure to 0·5 atm.

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3. STRESSES AND DEFORMATIONS CAUSED BY WELDING

3.1 In a welded joint stresses and deformation caused by welding can hardly be calculated beforehand. This is due to the complicated nature of the problem. The magnitude by which the metals expand due to heat differs for different metals. Likewise the modulus of elasticity on which the strength properties of the metals are calculated also varies with the temperature. The yield point reduces with the increasing temperature approaching zero in the case of steel at about 600°C (*see* Fig. 2 and Table 3). These three factors, the coefficient of linear expansion α, the modulus of elasticity 'E' and the yield point, influence the deformations and stresses caused in the material due to welding. In addition we have the conditions existing in the areas surrounding the weld. The first is the high temperature difference between the weld metal and the adjacent metal. Its effect differs from metal arc welding to gas welding as can be seen from Fig. 3. The shape of the isotherms, and hence the pattern of a temperature drop, are also influenced by the rate of welding. The ellipses become more and more elongated with increasing speed of welding.

3.1.1 The thickness of the work-piece, the shape of the member and the preventive measures adopted to reduce shrinkage, the position and sequence of welding and the type of tacking constitute the second important factor influencing shrinkage distortion in the welded joint.

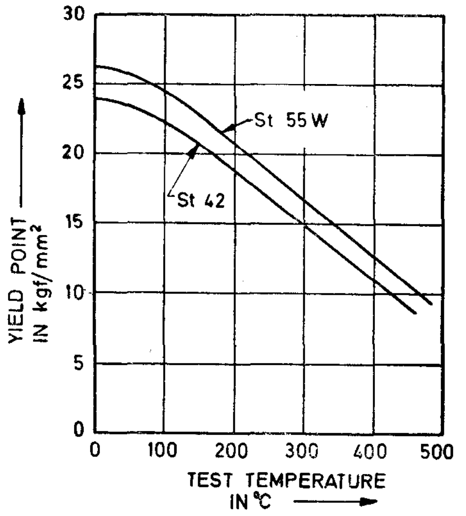


FIG. 2 THERMAL YIELD OF STEEL AT AN ELONGATION OF 0·2 PERCENT

TABLE 3 EFFECT OF HEAT ON THE LINEAR EXPANSION OF STEEL, ALUMINIUM AND COPPER

(*Clause* 3.1)

Heat Property	Material	Temperature Rise from 20°C to								
		100°C	200°C	300°C	400°C	500°C	600°C	700°C	800°C	900°C
Coefficient of linear expansion $\times 10^{-3}/°C$	Steel	11	12	13	13	14	14	—	—	—
	Copper	18	18	18	19	19	19	—	—	—
	Aluminium	22·4	24·9	25·8	26·8	27·9	28·5	—	—	—

3.1.2 If steel conforming to steel St-42S of IS : 226-1969* is subjected to elastic behaviour up to the yield point of about 24 kgf/mm² a strain according to Hook's law of 0·115 percent will result. Comparing this with the free expansion of 0·7 percent due to a temperature difference of 500°C it becomes clear that the majority of the stresses caused by resistance to shrinkage is reduced because of plastic expansion yielding.

Or in other words, we have the simplified relations as follows:

$$\sigma_w = E\,\alpha_w\,\Delta_t$$

where

σ_w = stress due to thermal expansion in kgf/mm²,

α_w = coefficient or expansion per °C, and

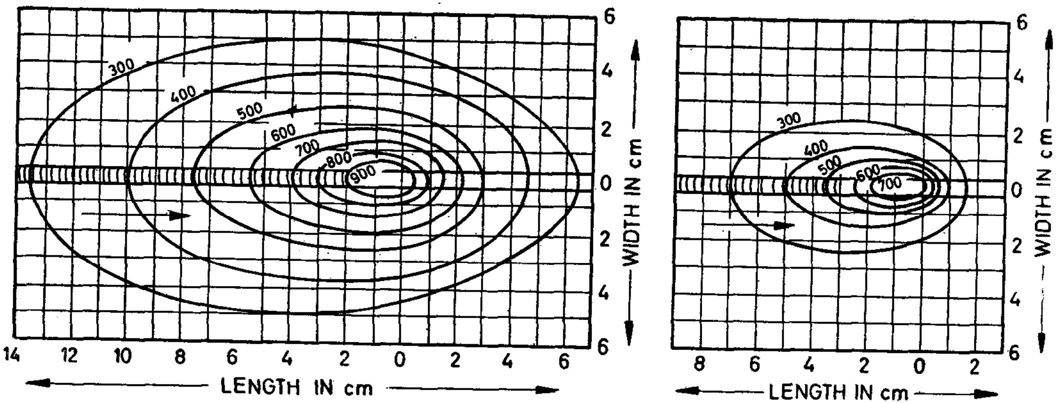
Δ_t = difference in temperature in °C

neglecting the temperature dependency of the modulus of elasticity E and the expansion coefficient α, and the stress due to the shrinkage prevention.

3.1.3 This gives a thermal stress of 25 kgf/mm² in steel when the difference in temperature is of the order of 100°C. This shows that even at this relatively low temperature difference when the shrinkage is fully prevented thermal stresses occur in the order of yield point in normal structural steel.

3.1.4 For example, if a plate is locally heated (*see* Fig. 3) the expansion due to heat is prevented by the surrounding cold metal. This causes compression stresses whose limit is represented by the thermal yield point corresponding to the respective temperature. The theoretical excessive stresses result in upsetting of the metal. The internal compressive forces are balanced by tensile forces in the unheated portion of the plate as there is no action of any external force.

*Specification for structural steel (standard quality) (*fourth revision*).



3A Gas Welding

3B Metal Arc Welding

FIG. 3 ISOTHERMS DURING WELDING ON STEEL PLATES 5 mm THICK

3.1.5 During cooling, the internal stresses caused due to heat are reduced, that is, both the compressive stresses in the previously highly heated zones as well as the corresponding stresses due to erection disappear. This phenomenon first takes place in the metal surrounding the weld because the weld metal itself is free from stresses on account of high temperature. With further cooling the maximum shrinkage stresses occur in the weld seam due to the maximum temperature difference, the amount of these stresses depending on the factors previously mentioned. The tensile stresses in the weld seam of a finished joint are compensated by the compressive stresses occurring in the weld metal, thus resulting in two zones free of stress on either side of the weld seam. In Fig. 4 is shown the distribution of the longitudinal stresses in a single-V butt welded joint.

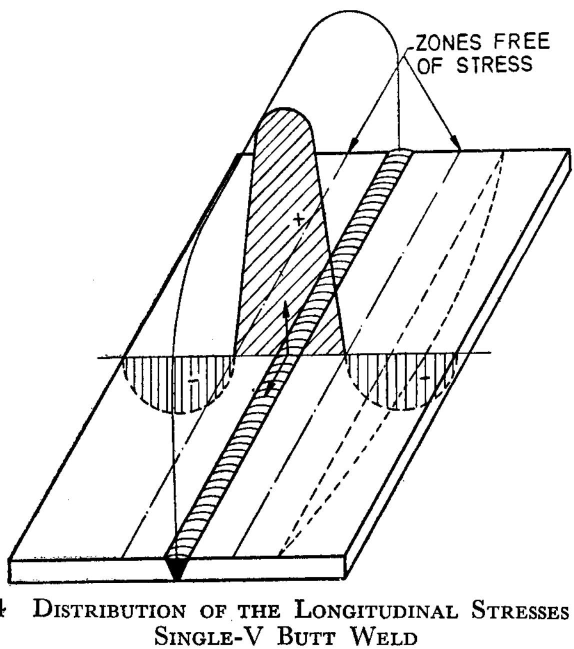


FIG. 4 DISTRIBUTION OF THE LONGITUDINAL STRESSES IN A SINGLE-V BUTT WELD

3.2 Effects of Shrinkage Stresses

3.2.1 The shrinkage stresses and the distortion are influenced by the method of welding, filler material and the number of passes. Thus, for example, gas welding results in lower shrinkage stresses but with greater distortion than in the case of metal arc welding. Manual welding as against automatic welding requires more number of runs and, therefore, introduces more heat, thus resulting in less shrinkage in the case of automatic welding than in the case of manual welding. Recommendations in regard to estimation of shrinkage in plates of different thicknesses are given in Tables 4 to 7. The information covers the type of edge preparation, type of joint and the welding processes adopted. The values are based on extensive research carried out in West Germany.

3.2.2 Besides transverse shrinkage covered in **3.2.1**, welding gives rise to longitudinal shrinkage also. However, no general formula or values could be given for longitudinal shrinkage because of the very complicated relations between the various factors involved. In the case of uninterrupted long weld seams a longitudinal shrinkage of 0·1 mm/m run of weld may be assumed. If the joint is made in very thick plates, the shrinkage in the direction of thickness also becomes significant.

When shrinkage is prevented, it results in stresses from zero at the surface to a maximum value around the neutral axis of the plate creating conditions for tri-axial stresses to develop in addition to secondary stresses in both transverse and longitudinal directions. Under certain unfavourable conditions these shrinkage stresses lead to the total disappearance of the yield and hence to the sudden failure of the member without deformation (brittle failure).

Furthermore, hardening, strain ageing and low ambient temperatures can also lead to the disappearance of the yield point. Normally, however, the shrinkage in the direction of stress only is considered.

3.2.3 In addition to longitudinal and transverse shrinkages mentioned above, asymmetric joints like fillet welds or T-joints give rise to angular shrinkage. For the same reason eccentric weld joints result in bends (I sections in crane girders). In plate construction we have displacements, buckling and bulging due to peripheral or central heating of restrained plates. The amount of shrinkage and distortion in such cases can be estimated and preventive measures taken based only on the experience over a long period of time.

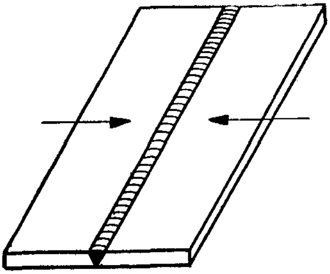
3.3 Reduction or Prevention of Shrinkage Stresses and Distortion

3.3.1 The statement ‘ the best welded construction is one in which the welding is minimum ’ is a useful guide to the designer of welded construction. A structural joint should have the barest minimum number of parts to be welded. This reduces the amount of weld metal to be deposited thereby reducing the heat input. The individual structural elements should be as few as the design provisions could permit. It is also advantageous to split up structural members of bigger dimensions into assemblies and sub-assemblies. They should be assembled together and welded only after they are straightened and stress-relieved, if necessary, to make them free from distortions and shrinkage stresses.

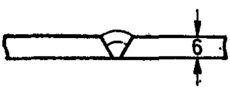
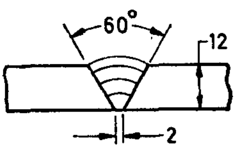
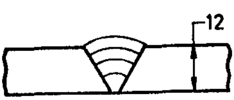
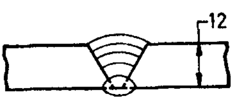
It is generally the practice to allow for longitudinal and transverse shrinkage rather than prevent this in order to minimize residual stresses and deformations.

TABLE 4 TRANSVERSE SHRINKAGE OF BUTT WELDS IN STEEL

(*Clause* 3.2.1)



The amount of shrinkage is for the plate of average measurement of large, properly pretacked plates, of steel conforming to IS : 226-1969* and IS : 2062-1969†.

Joint Detail	Method of Welding	Number of Weld Runs	Shrinkage Excluding Gap, mm
	Metal arc welding from one side only with covered electrode	2	1·0
	Metal arc welding with bare electrode from one side only	4	1·4
	Metal arc welding from one side only with covered electrode	5	1·6
	Metal arc welding from both sides with covered electrode, root machined before welding	5 Sealing runs 2	1·8

*Specification for structural steel (standard quality) (*fourth revision*).
†Specification for structural steel (fusion welding quality) (*first revision*).

(*Continued*)

TABLE 4 TRANSVERSE SHRINKAGE OF BUTT WELDS IN STEEL — *Contd*

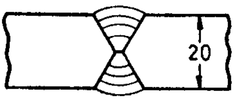
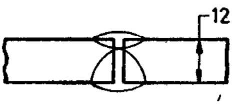
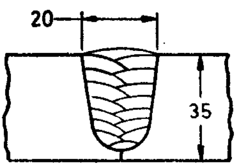
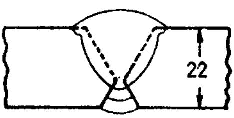
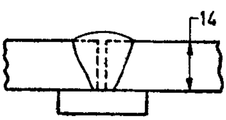
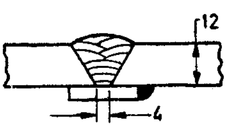
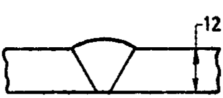
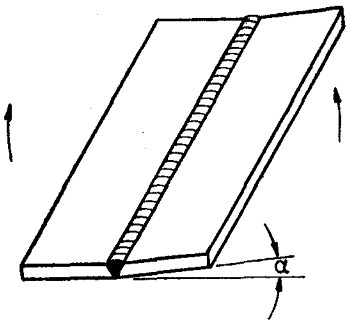
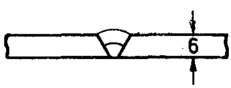
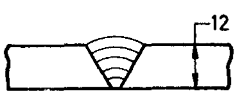
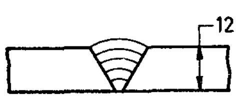
Joint Detail	Method of Welding	Number of Weld Runs	Shrinkage Excluding Gap, mm
	Metal arc welding from both sides with covered electrode	4 on either side	1·8
	Metal arc welding using deep penetration electrode	2	1·6
	Metal arc welding from one side only with covered electrode	20	3·2
	Submerged arc welding sealing runs by metal arc welding using covered electrode	—	2·4
	Submerged arc welding with copper backing strip	1	0·6
	Metal arc welding with backing strip using covered electrode	—	1·5
	Gas welding	—	2·3

TABLE 5 ANGLE SHRINKAGE OF BUTT WELDS

(*Clause* 3.2.1)



Average of measurements on plate not clamped rigidly.

Joint Detail	Method of Welding	Number of Runs	Angular Shrinkage ∝ Degree
	Metal arc welding using covered electrodes	2	1
	Metal arc weldin usingg bare electrodes	3	1
	Metal arc welding using covered electrodes	5	3½
	Metal arc welding using covered electrodes, root machined	5 3 sailing runs	0

(*Continued*)

TABLE 5 ANGLE SHRINKAGE OF BUTT WELDS — *Contd*

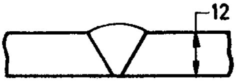
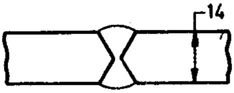
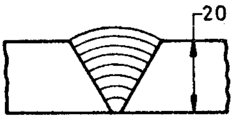
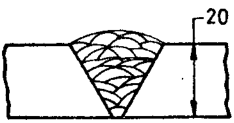
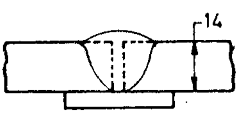
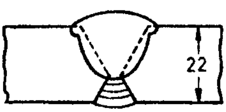
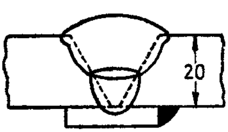
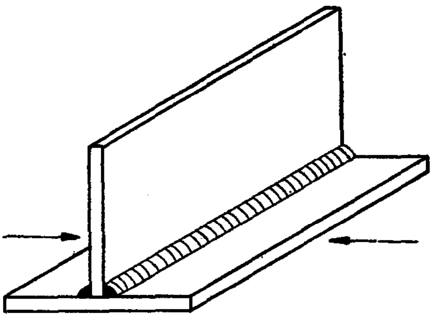
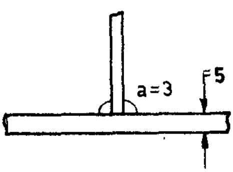
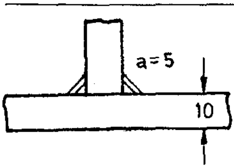
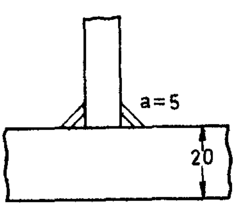
Joint Detail	Method of Welding	Number of Runs	Angular Shrinkage α Degree
	Rightward gas welding	—	1
	Gas welding in vertical position from both sides simultaneously	—	0
	Metal arc welding using covered electrodes	8 broad runs	7
	Metal arc welding using covered electrodes	22 narrow runs	13
	Submerged welding with copper backing strip	1	0
	Submerged welding, sealing runs by metal arc welding using covered electrode	1	2
	Submerged welding with steel backing strip	2	5

TABLE 6 TRANSVERSE SHRINKAGE IN JOINTS FILLET WELDED BY METAL ARC WELDING WITH COVERED ELECTRODES

(*Clause* 3.2.1)



The shrinkage values are average measurements in properly pretacked large size members of steels conforming to IS : 226-1969* and IS : 2062-1969†.

JOINT DETAIL	WELDING POSITION	LEG LENGTH OF FILLET mm	SHRINKAGE mm
	—	3	0·5
	Horizontal	5	0·3
	Horizontal	5	0

*Specification for structural steel (standard quality) (*fourth revision*).

†Specification for structural steel (fusion welding quality) (*first revision*).

(*Continued*)

TABLE 6 TRANSVERSE SHRINKAGE IN JOINTS FILLET WELDED BY METAL ARC WELDING WITH COVERED ELECTRODES — *Contd*

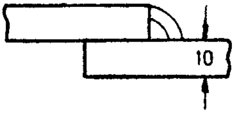
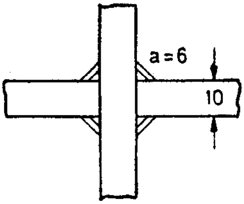
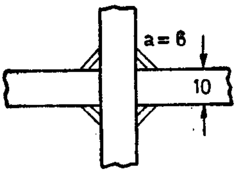
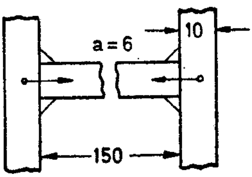
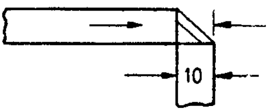
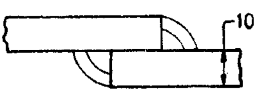
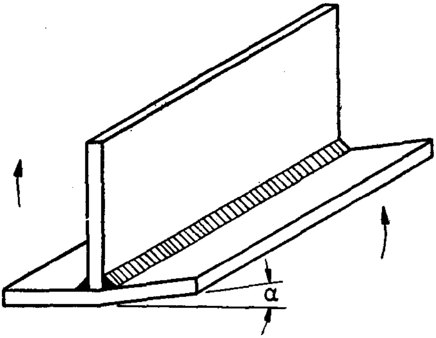
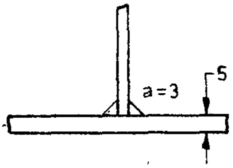
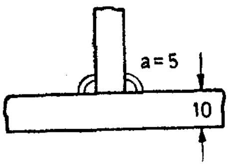
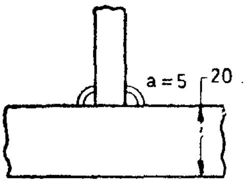
Joint Detail	Welding Position	Leg Length of Fillet mm	Shrinkage mm
	Horizontal	—	0·5
	Flat	6	1·0
	Vertical	6	1·3
	Horizontal	6	0
	Horizontal	—	0
	Horizontal	—	0·8

TABLE 7 ANGULAR SHRINKAGE IN FILLET WELDS AND T-JOINTS

(*Clause* 3.2.1)



Average measurements in members not rigidly clamped. The values can be used for steel conforming to IS : 226-1969* and IS : 2062-1969†.

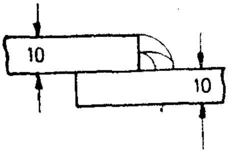
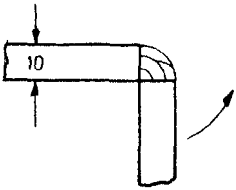
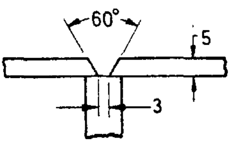
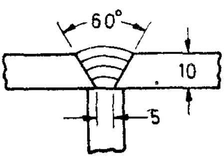
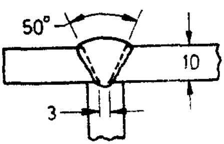
JOINT DETAIL	METHOD OF WELD	ANGULAR SHRINKAGE, α DEGREE
	Metal arc welding covered electrodes	3
	Metal arc welding covered electrodes, 2 horizontal runs	3
	Metal arc welding with covered electrodes, 2 horizontal fillet runs on either side	1

*Specification for structural steel (standard quality) (*fourth revision*).

†Specification for structural steel (fusion welding quality) (*first revision*).

(*Continued*)

TABLE 7 ANGULAR SHRINKAGE IN FILLET WELDS AND T-JOINTS — *Contd*

Joint Detail	Method of Weld	Angular Shrinkage, α Degree
	Metal arc welding with covered electrodes, 3 horizontal fillet runs	2
	Metal arc welding with covered electrodes, 4 horizontal runs	1½
	Metal arc welding with covered electrodes, 1 run	0
	Metal arc welding with covered electrodes, 3 runs	1
	Submerged arc welding, 1 run	0

3.3.2 The correct choice of the method of welding, joint preparation and the amount of deposited metal involved also contribute to the reduction in shrinkages stresses and distortion. If long welds are inevitable, automatic or semi-automatic welding should be preferred to manual welding. Angle between the fusion faces should be as small as possible and the number of runs as few. An angle between the fusion faces of 60° prescribed for manual welding can be reduced by about 10° for automatic welding. Crossing of welds should be avoided. For plates thicker than 12 mm double-V edge preparations should be preferred to single-V as these result in minimum of welding. The size of weld should be minimum consistent with the design requirements which ensures minimum of deposited weld metal. This applies particularly for fillet welds which are generally of a higher size than necessary.

Unless otherwise calculated the following fillet weld sizes are recommended:

Plate Thickness	*Fillet Size*
mm	mm
6	3
12	4
18	5
Over 18	6

The sizes given above are based on the formula:

$$t = 0{\cdot}3s$$

where

t = fillet size, and

s = plate thickness.

3.4 Welding Procedure — For optimum results, a welding procedure plan should be prepared for each weld joint. This should be followed particularly in the case of complicated structural joints.

The welding procedure plan should contain the following information:

a) Specification of steel being welded;

b) The filler material like electrodes and bare wires to be used for welding, their classification and specification, and special precautions to be taken where necessary;

c) Welding process (manual arc welding, gas welding, submerged arc welding, etc);

d) Preparation of fusion faces;

e) Whether the welds are to be made at shop or at site;

f) Sequence of welding;

g) Use of any auxiliary equipment like positioner, jigs and fixtures, clamping devices and stiffeners etc;

h) Details regarding pre- and post-weld heat treatment, for example pre-heating, post-heating, stress-relieving, etc; and

j) Inspection procedure before, during and after welding.

It is the usual practice for fabrication shops to develop by experience a welding procedure and welding sequence for different types of joints.

3.5 Tack Welds — Good welding begins with proper tacking. Where the tack weld is intended to be a part of the final weld its size should be that of the first run of the production weld and should be made with the same care applicable during the final welding. The length of the tack weld should be 2 to 3 times the plate thickness.

If the tack weld is not intended to form a part of the final weld it should be chipped and ground before the final weld is made. A tack weld, which is not intended to be included in the final weld, should never be left on the job.

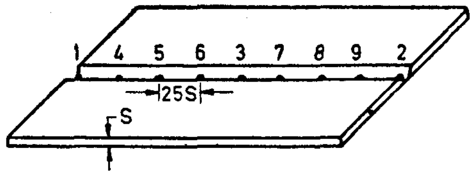
Recommended tack welding speeds and their spacing are given in Table 8.

TABLE 8 RECOMMENDED TACKING DISTANCES FOR BUTT WELDS

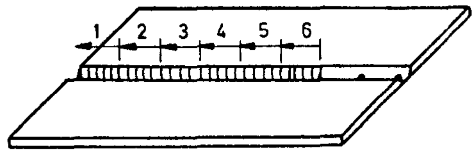
Welding Procedure	Welding Speed cm/min	Tacking Spacings mm
Arc welding with bare electrodes	8	20 minimum plate thickness
Arc welding with covered electrodes	10 to 15	25 to 32 minimum plate thickness
High duty welding machines (submerged welding, protected arc welding, electroslag welding, etc)	30 to 100	60 to 120 minimum plate thickness

3.6 The base metal should be heated as little as possible while welding. The number of runs should, therefore, be kept to the minimum considering always the possibility of free thermal expansion and free shrinkage. In welding plates of larger size the transverse welds splicing the plates should be made first and then the longitudinal welds. For example, in a plate girder the flange and the web plates should be spliced first before they are welded to form a T-joint. If butt and fillet welds occur in the same joint, butt weld should be made first. To achieve minimum shrinkage and distortion, back stop method is very useful in manual arc welding of plates (*see* Fig. 5).

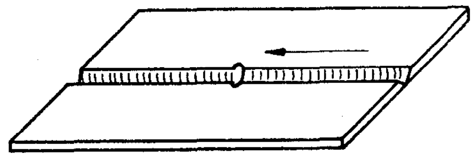
When two I sections are welded together the transverse weld should preferably be staggered at distance equal to 20 times the plate thickness. In a symmetrical weld the angular shrinkage can be avoided by first welding half on one side and welding full on the second side and finally completing



5A Tacking (tacking spacings: 25 times plate thickness)



5B First run by back stey welding from tacking point to tacking point



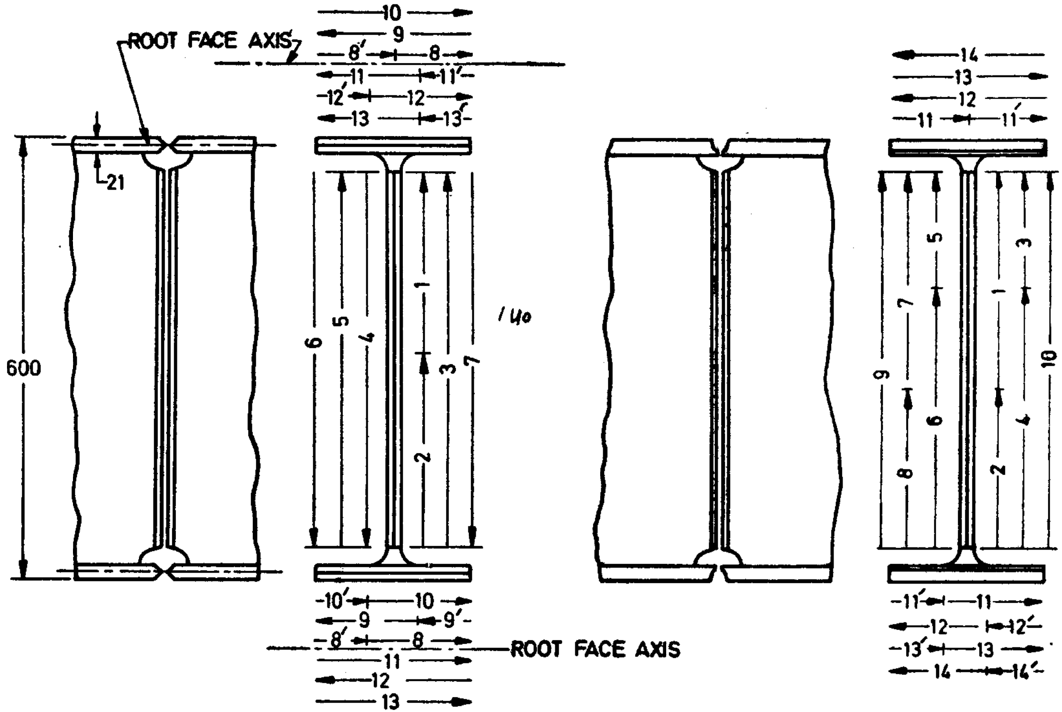
5C Further run continuous, but direction to be reversed for successive run

FIG. 5 WELDING SEQUENCE SCHEME

the weld on the first side (*see* Fig. 6). In the case of asymmetric double-V or double-U butt welds, the distortion is best avoided by chipping out and gouging the root run before completely welding the bigger half of the weld (*see* Fig. 7).

In the case of double fillet welds forming a T-joint between two plates a preliminary upward tilting of the flange plate is recommended to balance the final distortion due to shrinkage. When one of the plates is very thick, certain amount of angular shrinkage cannot be avoided.

3.7 Besides the methods indicated above for reducing the shrinkage stresses and distortion there are a number of other methods which can be profitably used. Pre-heating and post-heating when judiciously adopted minimizes buckling and distortion. Anticipated distortions in the case of plate girders, T-joints and splice in plates may be eliminated or reduced by forced elimination of their occurrence using proper jigs and fixtures. Annealing and cold-straightening are often used for the removal of locked up stresses. Hot- and cold-straightening are employed to remove distortion.



6A The beam is rotated about its longitudinal axis so that welding is done in the flat position

6B Position welding, when rotation of the beam to other position is not possible

FIG. 6 WELDING SEQUENCE SCHEME

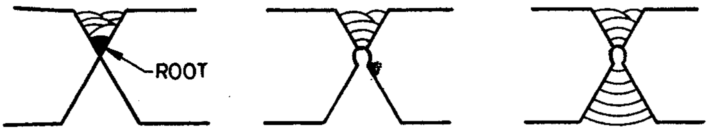


FIG. 7 WELDING ASYMMETRIC DOUBLE-V BUTT WELDS

3.8 As a summary a few recommendations are outlined below for prevention and reduction of shrinkage and distortion:

a) Do not over weld. Over welding results in accumulation of a large quantity of heat, thus increasing the shrinkage stresses and thereby the distortion. An effort should be made to use the optimum sizes of weld required and to reduce the length of the weld by using intermittent welds.

b) Use faster rates of welding. Slow welding results in local heating of the base metal near the weld.

c) The welds should be on the neutral axis of the joint or as close to it as possible. The welds around the neutral axis of the joint should be balanced to reduce the secondary stresses caused due to eccentricity of forces and also the distortion (*see* Fig. 5 and 6).

d) It is possible to orient parts in such a way that the pre-set or pre-camber is balanced by the shrinkage forces due to welding, thus bringing the components to the required alignment.

e) Use jigs and fixtures to clamp the members in position before welding. Clamping minimizes distortion. A good method is to clamp similar members back to back with one another and weld both at the same time.

f) Pre-heat, if necessary.

g) Welding should start from the strained end of the component towards the unstrained end.

h) Joints should be prepared accurately without leaving excessive gaps or large included angles.

j) When the component to be welded is large, use sub-assemblies. This will localize shrinkage to take place before the sub-assemblies are welded on into a completed member.

3.9 The following corrective measures may be used to remove distortions:

a) The distorted members may be straightened by presses, clamps or other means. This is a cold process which requires forces of large magnitudes to be applied.

b) If the distortion is due to shrinkage of the weld and its adjacent base metal, the member may be straightened by flame heating of the opposite side of the member which is resisting the shrinkage.

This is accomplished by heating the longer side of the member with a torch in a local spot or as shown in Fig. 8, and then allowing it to cool. This will cause certain amount of shrinkage. This is repeated as many times as required to bring the member back to the straight position.

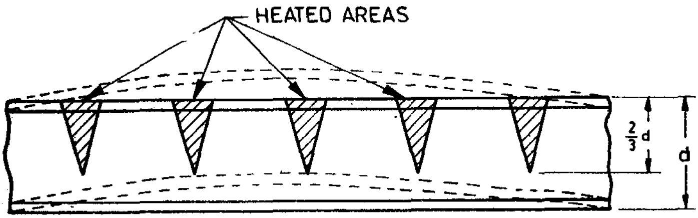


FIG. 8 POST WELD HEATING OF WELDED PLATE GIRDER

4. WELD PROFILES AND WELDING PROCEDURE

4.1 General

4.1.1 *Welding Symbols* — The basic symbols and the method of using them on drawings for various types of welds are given in IS : 813-1961*. In the examples of welding design given in the handbook these symbols have been used.

4.1.2 *Welding Procedure* — The general provisions in regard to welding procedure using metal arc welding are given in IS : 823-1964†. Recommendations in regard to submerged arc welding are covered in IS : 4353-1967‡. Tables regarding preparation of fusion faces in terms of thickness of plate and type of weld are, however, given in the following pages.

4.2 Gas Welding of Steels

4.2.1 Although gas welding is not much used for the fabrication of structural steel work certain recommended joint preparations are given in Table 9. These edge preparations are suitable for weld steels conforming to IS : 226-1969§, and IS : 2062-1969||. The design provisions should conform to IS : 1323-1956¶.

*Scheme of symbols for welding (*amended*).

†Code of procedure for manual metal arc welding of mild steel.

‡Recommendations for submerged arc welding of mild steel and low alloy steels.

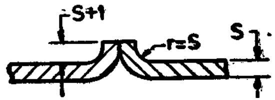
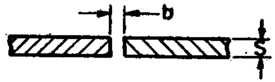
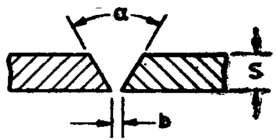
§Specification for structural steel (standard quality) (*fourth revision*).

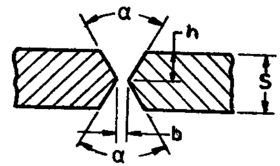
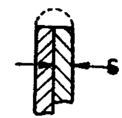
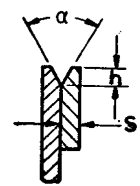
||Specification for structural steel (fusion welding quality) (*first revision*).

¶Code of practice for oxy-acetylene welding for structural work in mild steel (*revised*)

TABLE 9 DETAILS OF JOINTS FOR OXYACETYLENE GAS WELDING OF STEEL

(*Clause* 4.2.1)

Plate Thickness *s* mm	Execution of Weld	Type of Weld	Symbol	Joint Preparation	α Degree	*b* mm	*k* mm
Up to 1·5	One side	Stitch weld	JL		—	—	—
Up to 1	One side	Square butt	⊓		—	—	—
1 to 2 3 to 8	One side				— —	≈ 1 ≈ 2	— —
5 to 10	Both sides				—	≈ 4	—
3 to 12	One side	Single-V butt	▽		≈ 60	2-4	—

Over 12	Both sides	Double-V butt			≈ 50	≈ 4	$\frac{s}{2}$
Up to 5	One side	Bead weld			—	—	—
Over 3	One side	Bead V weld			≈ 60	—	≈ s

4.2.2 The filler material for gas welding should conform to IS : 1278-1967*. Welding may be made from either side of the joint or from both sides depending upon the thickness of material involved in all positions.

4.3 Metal Arc Welding with Covered Electrodes

4.3.1 Joint preparations for steels conforming to IS : 226-1969†, IS : 2062-1969‡ and IS : 961-1962§ for manual metal arc welding are given in Table 10.

This table is based on the recommendations contained in IS : 823-1964||. Reference may be made to this standard for further details regarding welding procedure, precautions like pre- and post-welded heat treatment to be taken, etc. The covered electrodes should conform to IS : 814-1970¶ and classified according to IS : 815-1966**. As in the case of gas welding, welds may be made from one or from both the sides of the joint. It is very important that the electrode selected for welds is suitable for the position of welding adopted. For selection of electrodes and definition of welding positions reference may be made to ISI Handbook for manual metal arc welding for welders.

4.4 Metal Arc Welding with Deep Penetration Electrodes — Due to the special characteristics of the flux covering penetrations deeper than those using normal electrodes can be obtained by the use of deep penetration electrodes (*see* IS : 814-1970¶). These electrodes are generally used for welding thicker material and where joint is not accessible from both the sides. The deep penetration electrodes should be stored with care because the flux covering is hygroscopic and gets damaged when exposed to atmosphere.

In Table 11 are given recommendations in regard to preparation of fusion faces while welding with deep penetration electrodes.

4.5 Submerged Arc Welding of Steel — Recommendations for joint preparations are given in Table 12 which is based on IS : 823-1964|| and IS : 4353-1967††.

The filler material for submerged arc welding which is available as a combination of bare wire and fluxes are covered in IS : 3613-1966‡‡.

---

*Specification for filler rods and wires for gas welding (*first revision*).

†Specification for structural steel (standard quality) (*fourth revision*).

‡Specification for structural steel (fusion welding quality) (*first revision*).

§Specification for structural steel (high tensile) (*revised*).

||Code of procedure for manual metal arc welding of mild steel.

¶Specification for covered electrodes for metal arc welding of structural steel (*third revision*).

**Classification and coding of covered electrodes for metal arc welding of mild steel and low alloy high-tensile steel (*revised*).

††Recommendations for submerged arc welding of mild steel and low alloy steels.

‡‡Acceptance tests for wire flux combinations for submerged-arc welding.

Both ac and dc may be used for submerged-arc welding. Welding may be done from either side or from both sides. Plates under 20 mm thicknesses are preferably welded using dc. Since submerged-arc welding is usually an automatic process using mechanical equipment, welds are made only in flat position.

5. WELD JOINTS

5.0 A few possible types of structural connections are illustrated in the following clauses. The applicability of these connections have to be decided by the design engineer, keeping in view the effect of the joints in relation to stress in the material, choice of material, welding process, welding equipment, etc. Estimated deformations and shrinkage stress caused by welding and the limitations of transportation and erection also guide the selection of welded joints.

5.1 Joints in Beams and Girders

5.1.1 I-sections may be formed by welding two flange plates and a web plate. The width of the flange plate is usually limited to 20 times its thickness (*see* **21.5.1.3** of IS : 800-1962*). The thickness of web is dependent on the total depth of the section and IS : 800-1962* limits it to 1/85 of the depth (*see* Fig. 9).

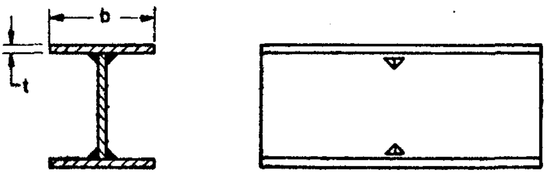


FIG. 9 WELDED I SECTION

5.1.2 I-sections may also be fabricated by welding two T-sections to a web plate. The thickness of web should, however, conform to the minimum values specified in IS : 800-1962* (*see* Fig. 10). Occasionally a longitudinal skew cut is made in a rolled section and welded together in opposite direction. The depth of the beam is thereby increased resulting in higher load carrying capacity of the girder (*see* Fig. 11). Using similar procedure tapered beams may also be fabricated (*see* Fig. 12). These special sections fabricated from rolled I-sections are often used as cantilever beams, purlins and roof girders to resist light loads.

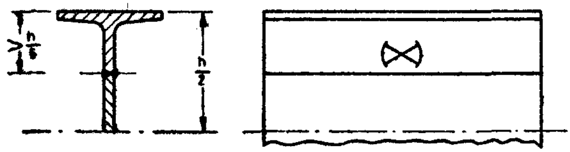
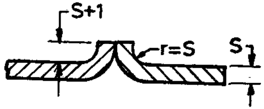
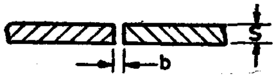
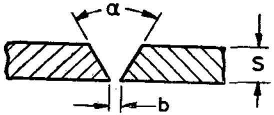
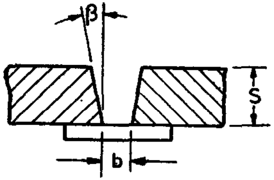


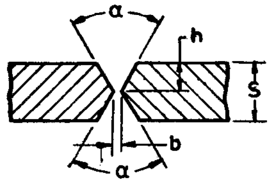
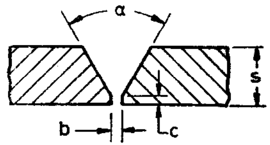
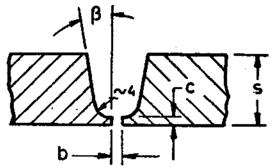
FIG. 10 TEE WELDED TO A WEB PLATE

*Code of practice for use of structural steel in general building construction (*revised*).

TABLE 10 DETAILS OF JOINTS FOR MANUAL METAL ARC WELDING OF STEEL

(*Clause* 4.3.1)

Plate Thickness s mm	Execution of Weld	Type of Weld	Symbol	Sectional Representation of Weld Preparation	α Degree	b* mm	e mm	h mm
Up to 2	One side	Stich weld			—	—	—	—
Up to 3	One side	Square butt			—	≈ s	—	—
Up to 6	Both sides				—	$\frac{s}{2}$	—	—
3 to 20	One side†	Single-V butt			≈ 60	≈ 2	—	—
5 to 20	Both sides							
Over 10	One side	Single-V butt with backing strip			≈ 10	6 to 10	—	—

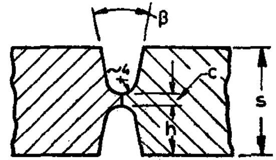
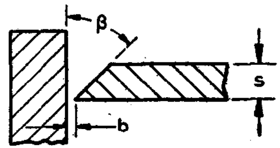
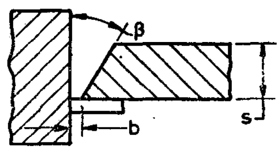
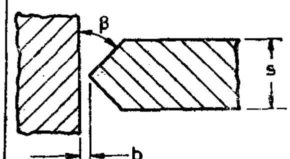
16 to 40	Both sides	Double-V butt			≈ 60	≈ 2	—	$\frac{s}{2}$
8 to 20	Both sides	Single-V butt			≈ 60	0-2	2-4	—
Over 16	One side	Single-U butt			≈ 10	≈ 2	≈ 2	—
	Both sides					0-2	≈ 3	—

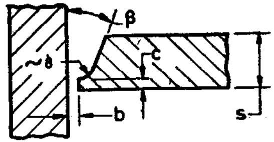
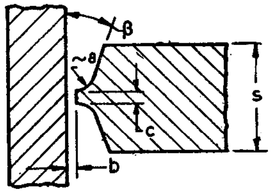
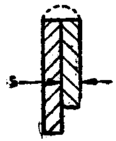
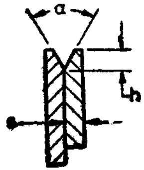
*Values for tacked plates.

†For one side execution, it is possible to have a backing strip.

(*Continued*)

TABLE 10 DETAILS OF JOINTS FOR MANUAL METAL ARC WELDING OF STEEL — *Contd*

Plate Thickness *s* mm	Execution of Weld	Type of Weld	Symbol	Sectional Representation of Weld Preparation	α Degree	*b** mm	*c* mm	*h* mm
Over 30	Both sides	Double-U butt			≈ 10	0-3	≈ 3	$s\frac{1}{2}$
3 to 16	One side	Single bevel butt			45-60	0-3	—	—
6 to 16	Both sides							
Over 16	One side	Single bevel butt with backing strip			15-30	6-10	—	—
16 to 40	Both sides	Double bevel butt			45-60	0-2	—	—

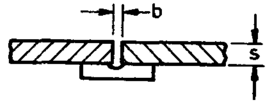
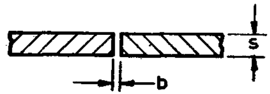
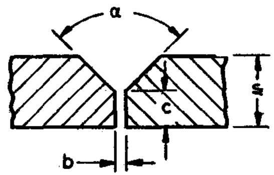
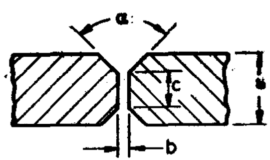
Over 16	One side Both sides	Single J butt			≈ 20	≈ 2	≈ 2	—
Over 30	Both sides	Double J butt			≈ 20	≈ 2	≈ 2	—
Over 3	One side	Bead weld			—	—	—	—
Over 4	One side	Bead V weld			≈ 60	—	—	5-1·2 *s*

Note — For one side execution, it is possible to have a backing strip.

*Values for tacked plates.

TABLE 11 DETAILS OF JOINTS FOR MANUAL METAL ARC WELDING USING DEEP PENETRATION ELECTRODES

(*Clause* 4.4)

Plate Thickness s mm	Execution of Weld	Type of Weld	Symbol	Joint Preparation	α Degree	b mm	c mm
5	One side				—	0	
6						1 to 2	
6 to 10	Both sides	Square butt			—	0	—
10 to 13					—	1 to 2	—
Over 10	Both sides	y weld			90	0	7
						1 to 2	10
Over 13						2 to 3	10
						3	10
Over 12	Both sides	Double y weld			90	0	4
Over 14						1 to 2	7

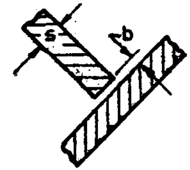
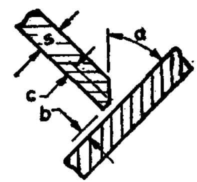
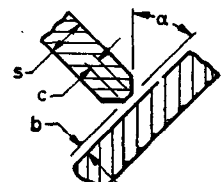
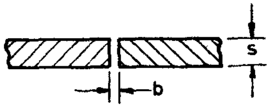
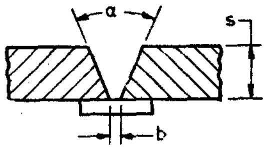
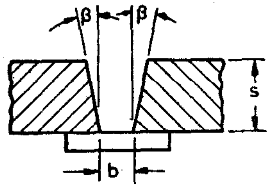
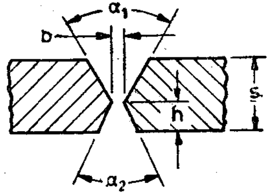
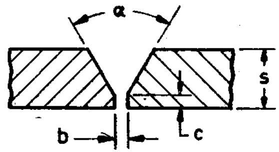
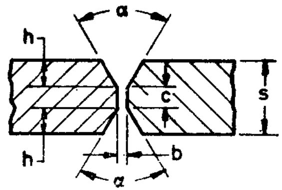
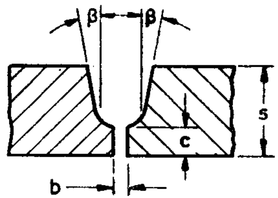
5	One side	Fillet weld			—	0	—
6						1 to 2	—
5 to 7	Both sides				—	0	—
7 to 9						1 to 2	—
9 to 12						2 to 3	—
12 to 15						3	
Over 5	One side	Half Y weld (HY weld)			45	0	2
Over 7						1 to 2	3
Over 9	Both sides				45	0	5
Over 11						1 to 2	6
Over 10	Both sides	Double bevel butt			45	0	2
Over 12						1 to 2	3

TABLE 12 DETAILS OF JOINTS FOR SUBMERGED-ARC WELDING OF STEEL

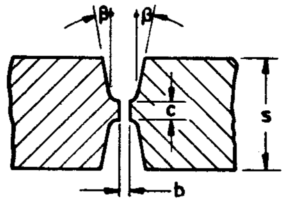
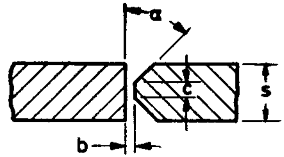
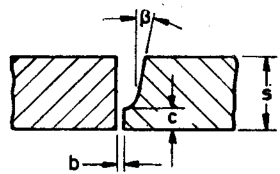
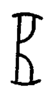
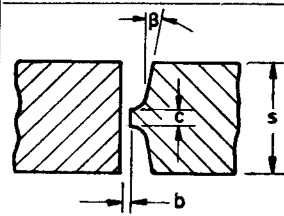
(*Clause* 4.5)

Plate Thickness s mm	Execution of Weld	Type of Weld	Symbol	Joint Preparation	α Degree	b mm	c mm	h mm
1·5 to 8	One side	Square butt			—	Up to 1·5	—	—
3 to 20	Both sides							
4 to 10	One side				—	2 to 4	—	—
12 to 30	Both sides					4 to 8	—	—
40 to 70	One side	Single V butt			30 to 50	Up to 3	—	—
Over 20	One side	Single V butt with backing strip			10 to 50	10 to 30	—	—

10 to 50	From both sides root run by manual welding	Double-V butt			a_1=50-90 a_2=50-60	1·5 to 3	—	1 to 15
15 to 30	From one side	Single-Y weld			40-90	Up to 1·5	3-12	—
Over 15	Both sides	Double-Y weld			40-90	Up to 1·5	5-9	—
Over 30	From one side	Single-U weld			5-10	Up to 1·5	5-10	6

(*Continued*)

TABLE 12 DETAILS OF JOINTS FOR SUBMERGED-ARC WELDING OF STEEL — *Contd*

Plate Thickness s mm	Execution of Weld	Type of Weld	Symbol	Joint Preparation	α Degree	b mm	c mm	h mm
Over 50	Both sides	Double-U weld			5 to 10	Up to 1·5	6	6
Over 15	Both sides	Double bevel butt			45 to 60	Up to 1·5	5 to 10	—
Over 30	From one side	Single-J butt			5 to 10	Up to 1·5	5 to 10	12
Over 50	Both sides	Double-J butt			5 to 10	Up to 1·5	6	12

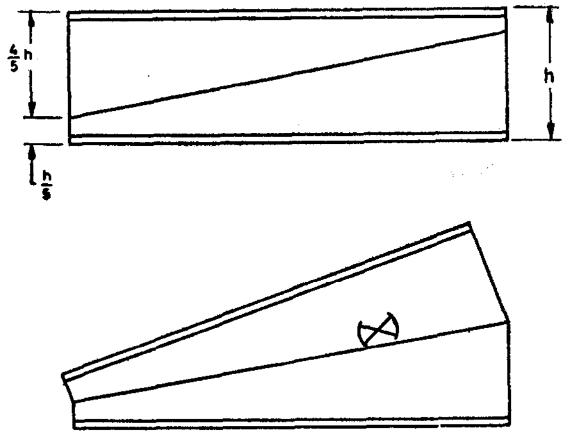


FIG. 11 TAPERED BEAM BY SLITTING A BEAM AND REWELDING

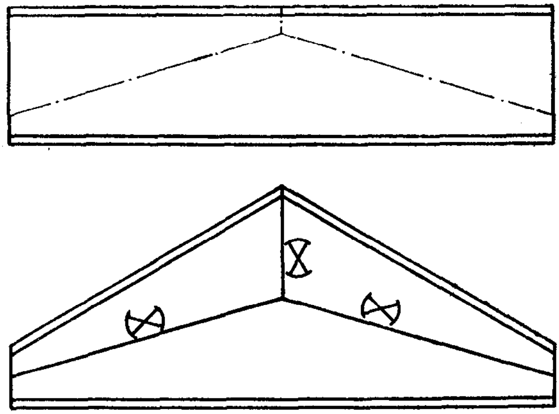
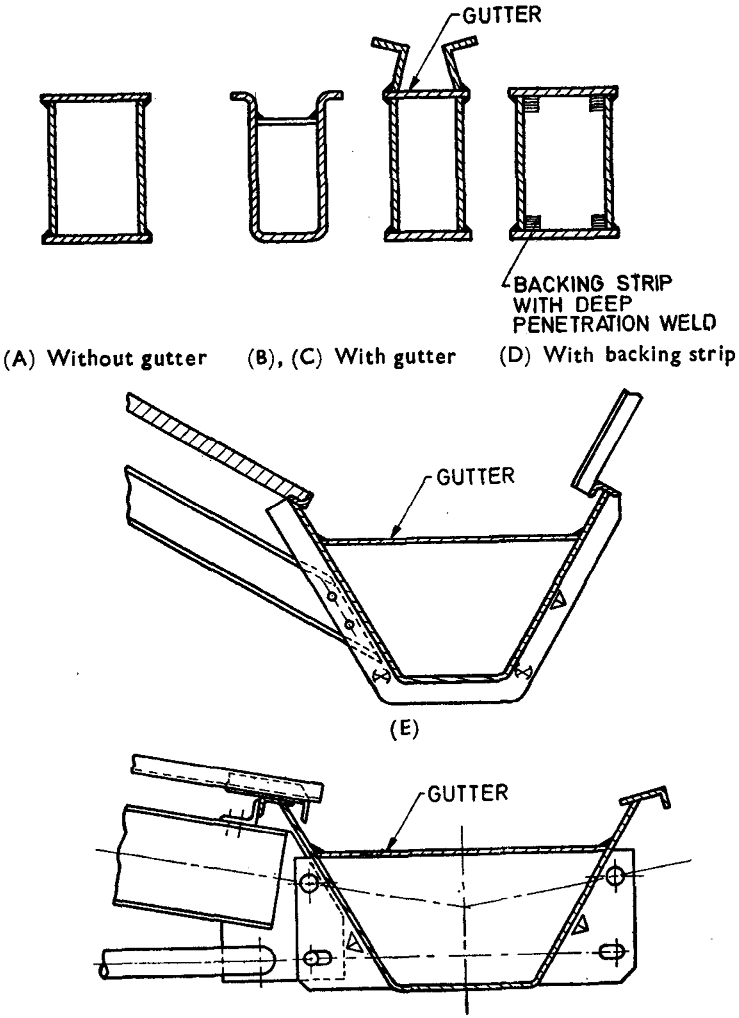


FIG. 12 A BEAM SLIT AND REWELDED TO FORM A TAPERED BEAM

5.1.3 *Box Girders* — Box girders may be fabricated by welding plates, sheets and strips. By virtue of their shape box girders possess high torsional resistance and are commonly used in the manufacture of cranes. In structural steel work they are often used as roof girders. The top flange if in the shape of a trough may be used as gutters. In Fig. 13 are shown some typical box sections and their use as beams cum gutters.



(A) Without gutter (B), (C) With gutter (D) With backing strip

(E)

(E), (F) Roof girders functioning as gutters

FIG. 13 BOX GIRDERS AND THEIR TYPICAL APPLICATIONS

5.2 Joints

5.2.1 *Butt Joints for Rolled I-Sections* — Rolled I-sections are often sliced by butt welding. Wherever possible, butt welds joining the flange and the webs should be staggered. If this is not possible and wherever the sections are subjected to very high stresses the joints should be radiographed. Typical connections for splicing rolled I-sections are shown in Fig. 14 and 15.

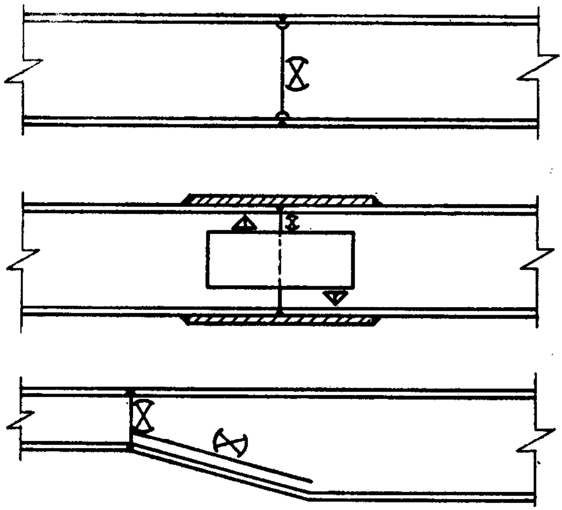


FIG. 14 METHODS OF SPLICING ROLLED I-SECTIONS

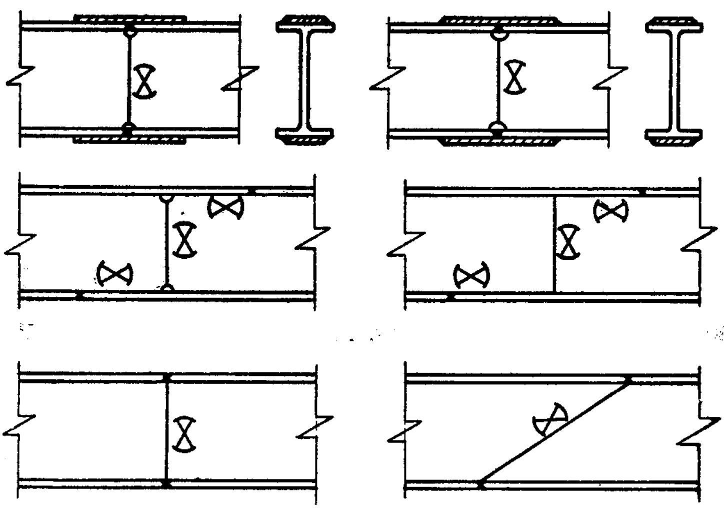


FIG. 15 METHODS OF SPLICING ROLLED I-SECTIONS

5.2.2 *Splicing of Welded Beams and Girders* — As far as possible the flange and web splices should be staggered. When joined by butt welds a splice joint in a welded girder is as strong as the continuous section itself (*see* Fig. 16).

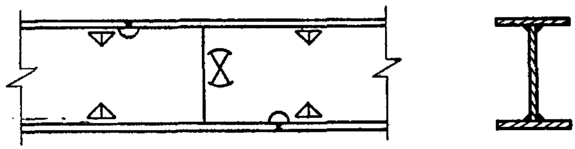
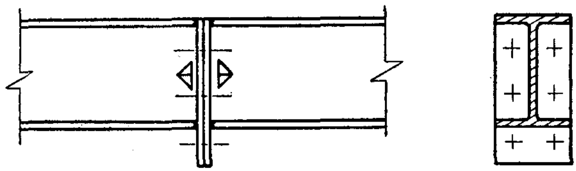
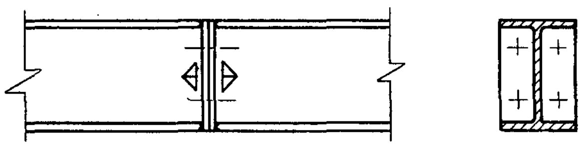


FIG. 16 SPLICING A WELDED GIRDER

5.2.2.1 For connections at site a simpler method would be to weld end plates to the beams to be joined. A connection will ultimately be made using high tensile friction grip fasteners. In Fig. 17 is shown a connection using ordinary bolts. Use of high tensile friction grip fasteners in joints predominantly subjected to moments and shear are also shown.



17A Joint where bending moment is predominant



17B Joint where shear is predominant

FIG. 17 SPLICES USING HIGH TENSILE

It is recommended that killed steel conforming to IS : 961-1962* or IS : 2062-1969† should be used for end plates. The plates should also be free from laminations. For design of joints using high tensile friction grip fasteners reference be made to IS : 4000-1967‡ [*see also* SP : 6(4)-1969 ISI Handbook for Structural Engineers. Use of High Strength Friction Grip Bolts.].

5.2.3 *Joints in Box Girders* — Joints for connecting box sections may be made using single-V butt welds with backing strip from inside when

*Specification for structural steel (high tensile) (*revised*).

†Specification for structural steel (fusion welding quality) (*first revision*).

‡Code of practice for assembly of structural joints using high tensile friction grip fasteners.

welding is done from outside only (*see* Fig. 18). It is also possible to weld two of the four plates of the box girders using double-V butt welds from both sides (*see* Fig. 19). The choice between the two methods of joining depends on the dimensions of the section and also the stresses to which they are subjected to. The first method has a disadvantage that the slag can easily flow out and lead to defects in the root. In the case of deep penetration welds there will be an undesirable transmission of forces through the backing strip (*see* Fig. 20). If welding has to be made from both sides the illustration given in Fig. 19 may be adopted, the final joint being in the web or the flange. If a hand hole is made in the web it should be big enough so that the root of the weld on the other side may be inspected without difficulty.

5.2.4 *Rigid Connection Between Beams* — Girders of equal depth may be joined with or without cover plates depending on the load to be transmitted (*see* Fig. 21). In the case of girders of different heights cleats may

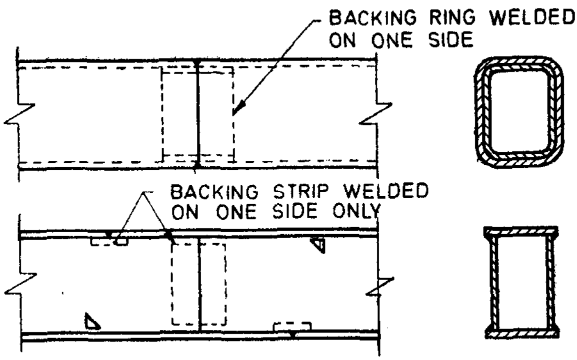


FIG. 18 BOX GIRDER, BUTT JOINTS WITH BACKING RING OR BACKING STRIPS

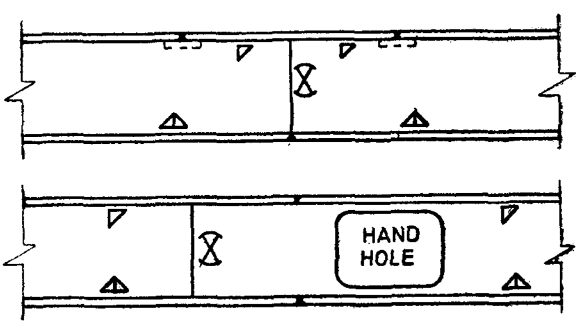
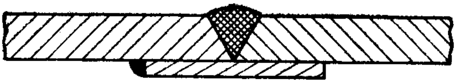
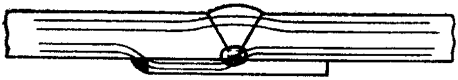


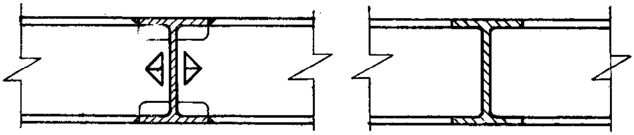
FIG. 19 BOX GIRDER WITH BUTT JOINTS WELDED FROM BOTH SIDES



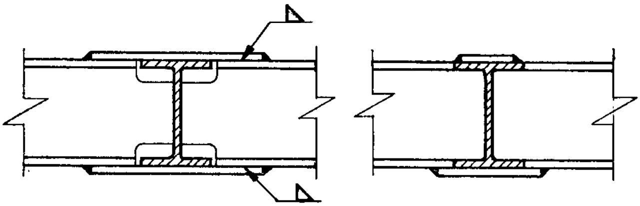
20A Single-V deep penetration butt weld with backing strip



20B Flow of stress in a single-V deep penetration butt weld with backing strip



21A Without cover plates

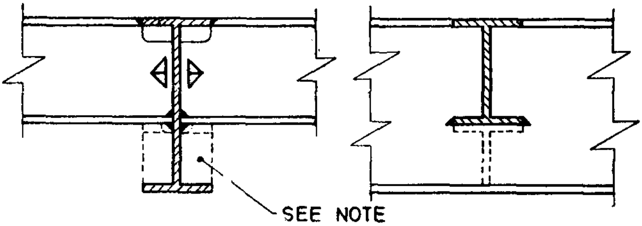


21B With cover plates

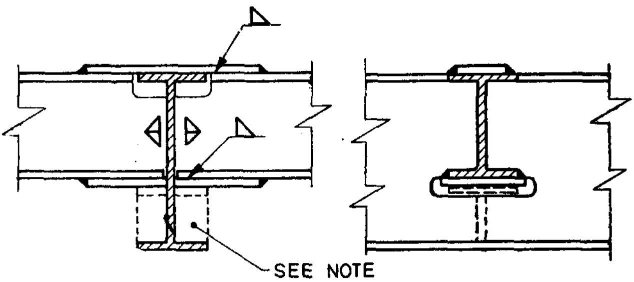
FIG. 21 JUNCTION OF GIRDERS OF EQUAL DEPTHS

be welded to the main girder to facilitate easy erection depending upon the importance of the secondary member (*see* Fig. 22). The girder connection may be made further rigid by extending the cover plates to serve as gusset plate for connecting the additional lateral bracings (*see* Fig. 23). For normal girder connection without involving any restraint due to the rigidity of the joint, simple connection involving end plates and bolts is recommended as shown in Fig. 24. The joint should, however, be checked for shear. If high tensile friction grip fasteners are employed, the end plates should be free from laminations.

5.2.5 *Welded Truss Girders* — Rolled sections are normally used for the fabrication of trusses. Welded trusses are of two types. The first type is derived from rivet construction where gusset plates are employed to connect the members (*see* Fig. 25). In the other type the members are directly welded to each other without using any gusset plate (*see* Fig. 26). While use of gusset plate results in certain amount of eccentricity it can be eliminated by properly making the connections (*see* Fig. 27).



22A Without cover plates



22B With cover plates

NOTE — Cleats may be welded to the main girder to facilitate easy erection depending upon the importance of the secondary member.

FIG. 22 JUNCTION OF GIRDERS OF UNEQUAL DEPTHS

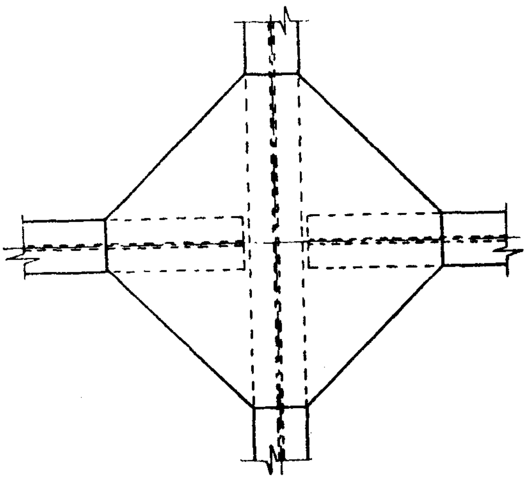


FIG. 23 JUNCTION OF RIGID GIRDERS

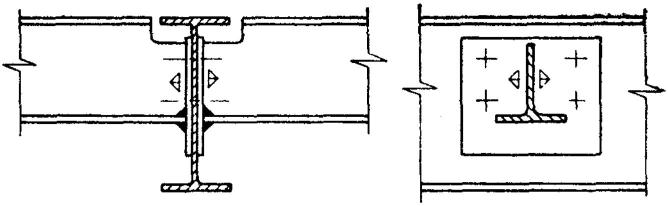


FIG. 24 SIMPLE GIRDER CONNECTION USING BOLTS AND END PLATES

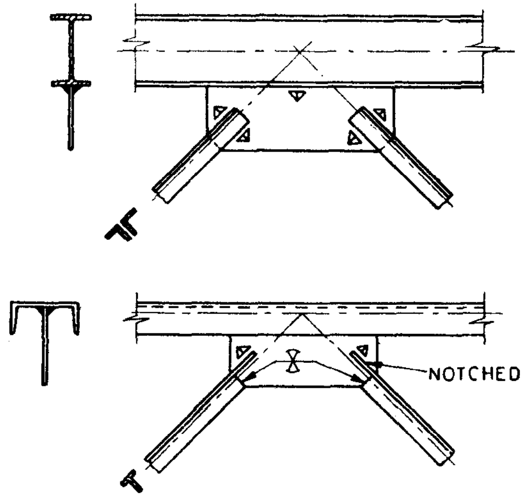


FIG. 25 WELDED TRUSS JOINTS WITH GUSSET PLATE

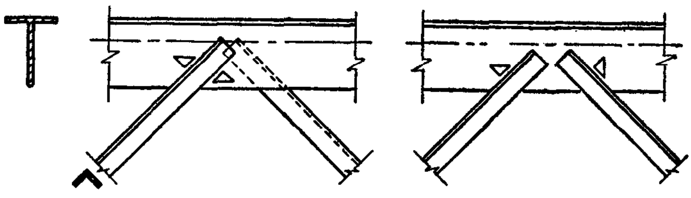


FIG. 26 TRUSS JOINTS WITHOUT GUSSET PLATES

5.2.5.1 Longitudinal fillet welds running along the axis of the member should be preferred to fillet welds running in transverse direction. Therefore, any constructions which are not directly exposed to the weather the transverse fillet welds can be altogether omitted. Combination of butt and fillet welds in forming T-joint of truss members requires the greater accuracy of fit and, therefore, makes the construction more expensive (*see* Fig. 25). This also may result in considerable distortion of the member.

5.2.5.2 Truss girders in which the main members are formed by T-sections or by slitting I-sections belong to the second group, that is, without gusset plate. The diagonals are usually connected directly to the web of the main chord (*see* Fig. 26). Generally only the end diagonal of the girder is welded with a gusset plate.

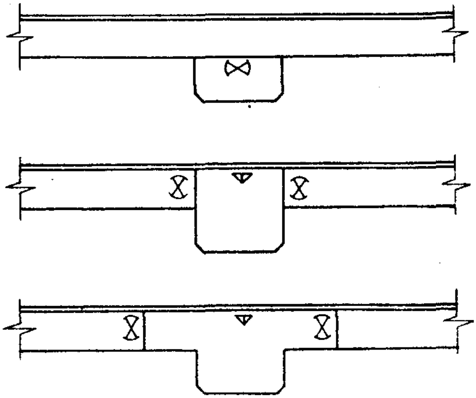


FIG. 27 USE OF GUSSET PLATES WITHOUT ECCENTRICITY

The diagonals and the vertical members of trusses are normally connected to either sides of the gusset plates or web of the main member to reduce the eccentricity and for uniformly distributing the moment at the joints (*see* Fig. 28). In the case of angles with slit corners the segregation zone is exposed which is a disadvantage. Furthermore, the welding can be made from only one side.

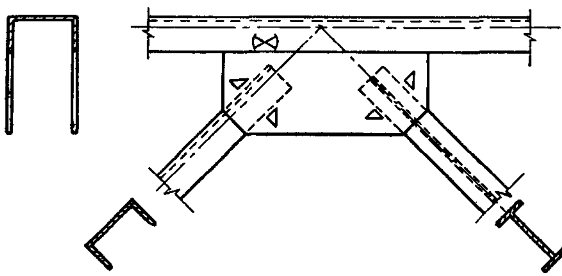


FIG. 28 JOINT IN A TRUSS GIRDER INVOLVING ROLLED CHANNELS AND BEAMS

5.2.5.3 In Fig. 28 and 29 are shown two ways of connecting webs of rolled sections to a gusset plate. In Fig. 30 is shown the method of joining round and square bars to gusset plates. For round bars a chamfer of 25° and for square bars a chamfer of 15° are recommended.

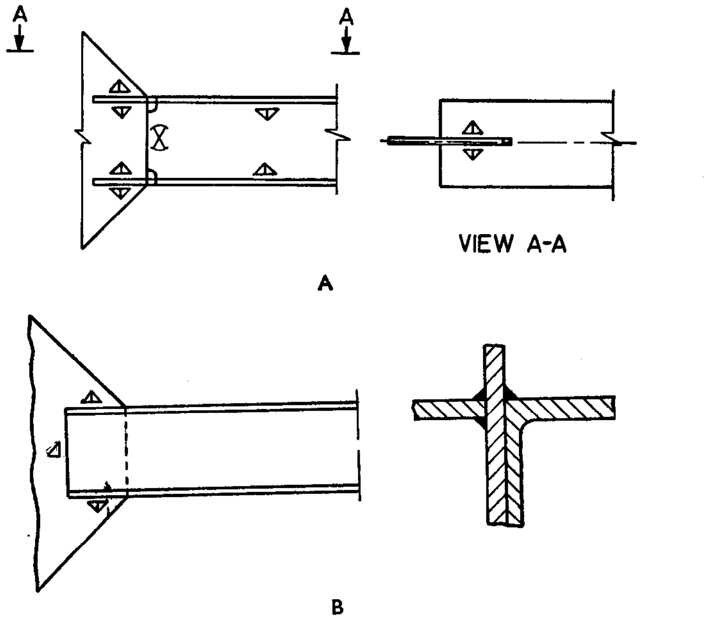


FIG. 29 METHODS OF CONNECTING WEB OF BEAM AND CHANNEL SECTIONS TO GUSSET PLATES

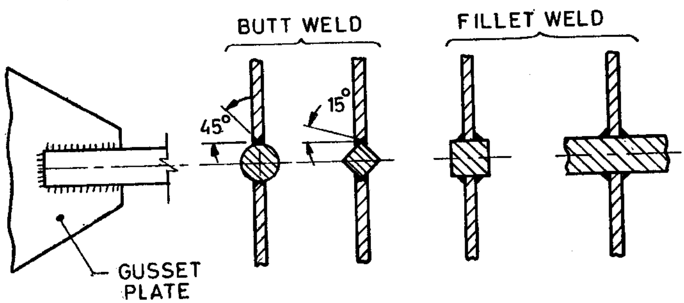


FIG. 30 METHODS OF CONNECTING ROUND, AND SQUARE BARS TO GUSSET PLATES

At crossing between the girders and secondary members the direct welding of two members should be avoided in order to eliminate the undesirable junction of transverse welds.

5.2.5.4 Methods of connecting bracings through gusset plates to the flange and webs of I-sections are shown in Fig. 31 and 32. For the type of construction shown in Fig. 31A the gussets should be welded to the flanges with single or double-V butt welds depending upon the thickness of the flange. The bracings may be either welded at site or connected by bolts. For the type of construction shown in Fig. 31B care should be taken to see that the gussets are satisfactorily welded to the web. These two types of constructions are normally adopted for connecting horizontal bracings. In the case of vertical bracing the method shown in Fig. 32 may be adopted.

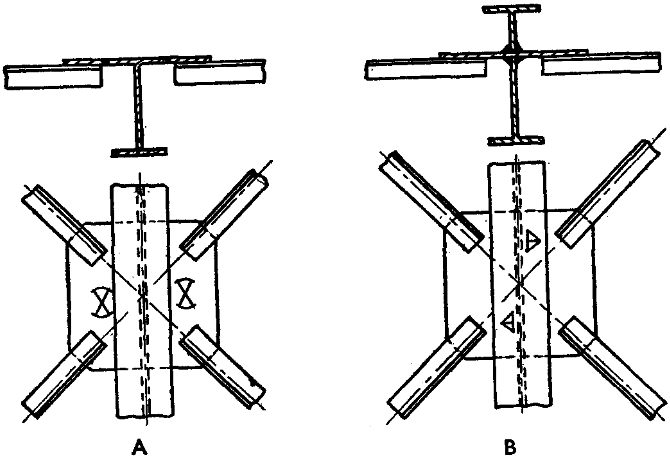


FIG. 31 JOINTS CONNECTING HORIZONTAL BRACINGS TO I-GIRDERS — ASSEMBLY BY SITE WELDING

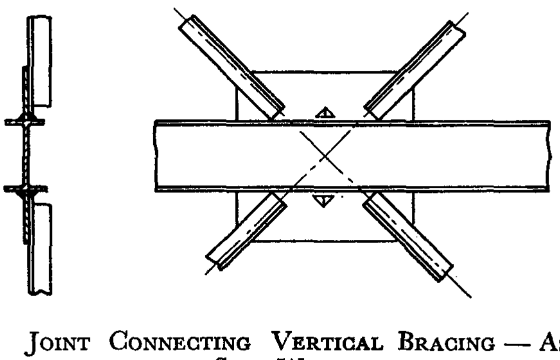
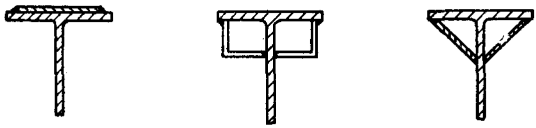


FIG. 32 JOINT CONNECTING VERTICAL BRACING — ASSEMBLY BY SITE WELDING

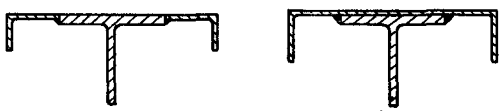
5.2.6 *Flanges of I-Sections* — While in built up girders the flanges are proportioned to the requirements of the designer the flanges of rolled sections may have to be strengthened or reinforced by the use of additional flange material in the form of flange plates, rolled sections like angles, channels and beams. Such reinforcement will be required not only to resist the bending moments but also to prevent the buckling of the flanges when the beams are too long and when they are not properly supported laterally (*see* **10** in IS : 800-1962*). In the case of crane way girders the flanges are strengthened to resist the horizontal forces due to the surge action of the cranes and the horizontal component of wind loads. Some typical methods of reinforcing the flanges are shown in Fig. 33 to 35. When channels are used, there should be enough space for the welder to deposit the fillets. Care should be taken that the welds are not deposited on the fillet portion of the channels which is a segregated zone (*see* Fig. 34B).

The flange reinforcement should be tapered to a suitable slope at the beam ends (*see* Fig. 35).



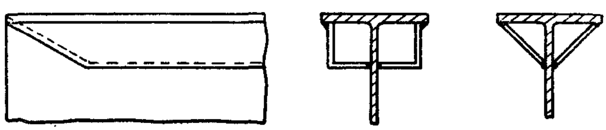
A B C

FIG. 33 FLANGE REINFORCEMENTS WHERE VERTICAL LOADS ARE PREDOMINANT



A B

FIG. 34 FLANGE REINFORCEMENTS TO RESIST HORIZONTAL FORCES



A B

FIG. 35 FLANGE REINFORCEMENTS ENDING WITH TAPER

5.2.7 *Strengthening of Webs* — Webs need reinforcement especially when subjected to very high shear stresses and in places where heavy vertical point loads are acting. By providing web reinforcements it is possible to permit higher shear stresses than specified in IS : 800-1962*. Two methods of strengthening webs normally employed in repair work are shown in Fig. 36. When the reinforcement has to extend to the full depth of the girder the ends of the reinforcing plates may be welded to the flanges as shown in Fig. 37.

*Code of practice for use of structural steel in general building construction (*revised*).

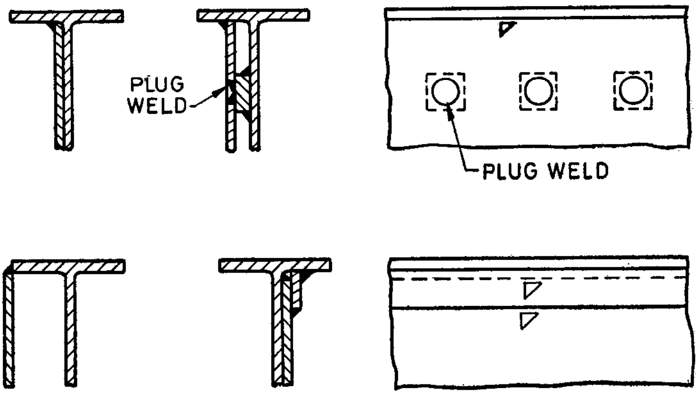


FIG. 36 REINFORCEMENTS FOR WEBS

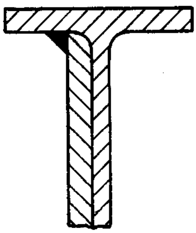


FIG. 37 WEB REINFORCEMENT WELDED TO FLANGE

5.2.8 *Web Stiffeners* — Webs need stiffening to prevent their buckling and to transfer effectively the loads from the flanges to the web especially when point loads are involved. Stiffeners are necessary when ratio of depth to thickness of webs exceeds 85 in steel conforming to IS : 226-1969*, IS : 2062-1969† and St 42-O of IS : 1977-1969‡ or 75 for steels conforming to IS : 961-1962§. With the use of stiffeners as specified in **10.3.3** of IS : 800-1962|| it is possible to use webs with clear dimensions of the web in the panel up to 270 times its thickness.

Stiffeners are of three types. The horizontal stiffeners are used to reduce the effective depth of the web so that the stresses permitted in

*Specification for structural steel (standard quality) (*fourth revision*).

†Specification for structural steel (fusion welding quality) (*first revision*).

‡Specification for structural steel (ordinary quality) (*first revision*).

§Specification for structural steel (high tensile) (*revised*).

||Code of practice for use of structural steel in general building construction (*revised*).

Tables 10 and 11 of IS : 800-1962* may be used. The design of horizontal stiffeners are covered in **21.7** of IS : 800-1962*.

The intermediate stiffeners which are vertical are employed to reduce the effective length of the panel of webs. Clause **21.7** of IS : 800-1962* specifies that intermediate stiffeners should be used throughout the length of the girder when the thickness of web is less than $d/85$ for steel conforming to IS : 226-1962†, IS : 2062-1969‡ and St 42-O of IS : 1977-1969§ and $d/75$ for steel conforming to IS : 961-1962|| where d is the distance between the flanges ignoring the fillets. The vertical stiffeners should be spaced at a distance apart not greater than 1·5 d and not less than 0·33 d.

The bearing stiffeners are to be used at points of concentrated loads and at points of support. Intermediate stiffeners should not be welded to the tensile flanges wherever possible especially when the girders are subjected to dynamic loads. Bearing stiffeners in accordance with **21.7.2.2** of IS : 800-1962* should be designed as columns and should be symmetrical about the web as far as possible. The ends of the bearing stiffeners should be machined or ground to fit tightly at both top and bottom and the stiffeners should be provided with sufficient welds to transmit to the web the whole of the concentrated load. In certain cases and on supports where the bearing on the web is not substantial the bearing stiffeners may be connected only to part height of the girder. In Fig. 38 are shown a few examples of intermediate stiffeners. Bearing stiffeners are shown in Fig. 39.

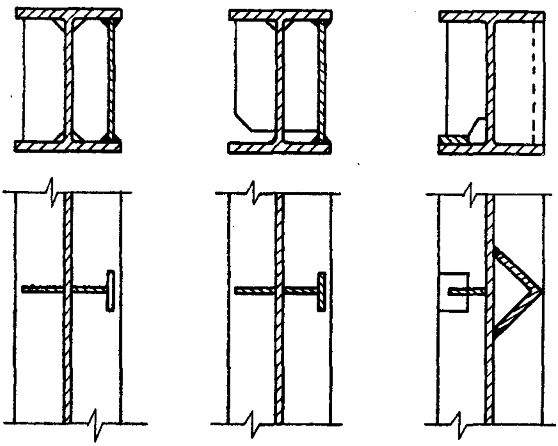


FIG. 38 EXAMPLES OF INTERMEDIATE STIFFENERS

*Code of practice for use of structural steel in general building construction (*revised*).

†Specification for structural steel (standard quality) (*fourth revision*).

‡Specification for structural steel (fusion welding quality) (*first revision*).

§Specification for structural steel (ordinary quality) (*first revision*).

||Specification for structural steel (high tensile) (*revised*).

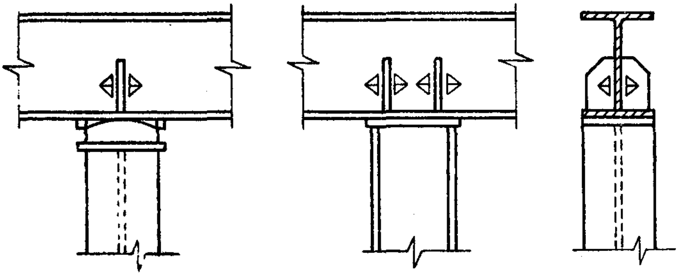


FIG. 39 BEARING STIFFENERS

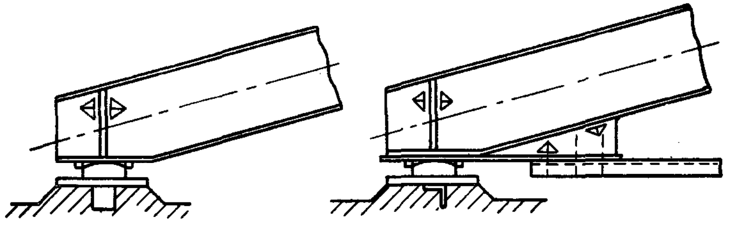
6. SUPPORTS FOR BEAMS AND GIRDERS

6.1 Of the many possibilities some typical examples of supports are shown in the following figures. A simple support for a sloping roof girder is shown in Fig. 40A. If an additional plate is joined to the lower flange, we get a support for a two-hinged or three-hinged girder where the position of the tension bar is mostly horizontal (*see* Fig. 40B). In this type of construction, the point of intersection of the axes of the sloping girder and the tension rod lies outside the plane of support. This situation is favourable statically because it results in a reduction in the moment in the sloping girder. If such eccentricities are to be avoided arrangements given in Fig. 41 may be followed.

Figure 42 shows the connection of girder and tension bar, which is more suitable from welding point of view. If the support of the sloping girder comes over a lateral plate connection, Fig. 43 shows a possibility. Fig. 44 shows a connection to a column flange.

7. DESIGN

7.1 Analysis of Beams for Bending — Analysis of statically determinate and statically indeterminate beams are given in Appendix A. In these



A B

FIG. 40 SUPPORTS FOR GIRDERS

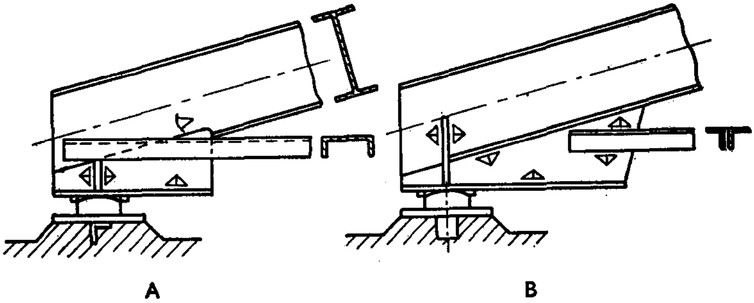


FIG. 41 SUPPORTS FOR GIRDERS

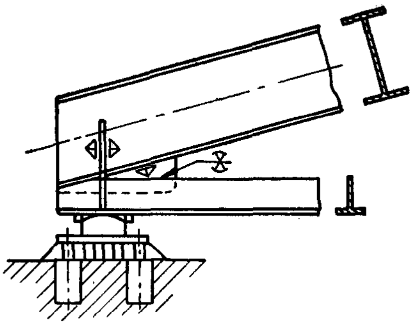


FIG. 42 GIRDER SUPPORT

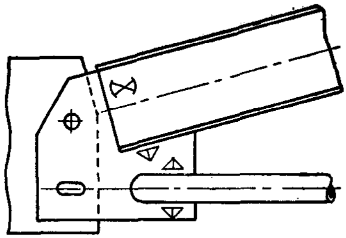


FIG. 43 CONNECTION OF GIRDER TO A PLATE

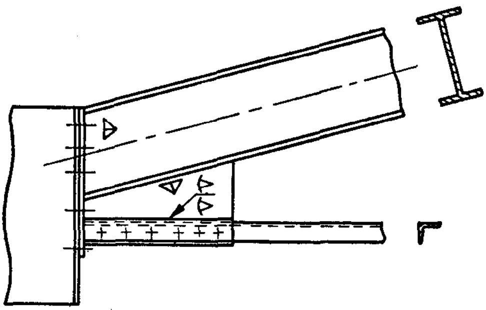


FIG. 44 CONNECTION OF GIRDER TO A COLUMN

formulae the deflection due to shear has been neglected. The deflection curves have been drawn assuming constant moment of inertia I over the entire length l of the beam.

7.2 Details for Dimensioning of Welds

7.2.0 The details given in the following pages are based on IS : 816-1969*.

7.2.1 *Butt Weld* — The size of a butt weld shall be specified by the effective throat thickness. The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part of the joint.

7.2.2 *Fillet Welds* — Fillet welds are of the following two types:

a) Normal fillet weld, and
b) Deep penetration fillet weld in which the depth of penetration beyond the root is 2·4 mm or more.

The size of this *normal fillet weld* should be taken as minimum leg length (*see* Fig. 45).

The size of a *deep penetration fillet weld* should be taken as the minimum leg length plus 2·4 mm (*see* Fig. 45).

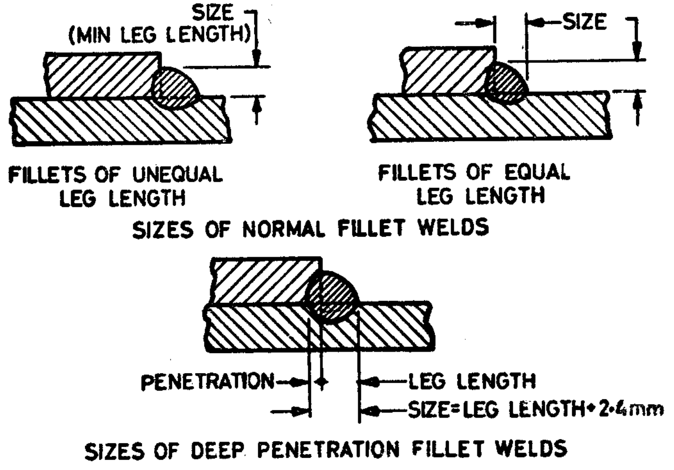


Fig. 45 Sizes of Fillet Welds

7.2.2.1 *Effective throat thickness* — The effective throat thickness of a fillet weld should be taken as $k \times$ fillet size, that is, $k \times t$, where k is a

*Code of practice for use of metal arc welding for general construction in mild steel (*first revision*).

constant. The value of k for different angles between fusion faces (*see also* **7.2.2.2**) shall be as given in Table 13.

7.2.2.2 *Angle between fusion faces* — Fillet welds should not be used for connecting parts whose fusion faces form an angle of more than 120 degrees or less than 60 degrees, unless such welds are demonstrated by practical tests to develop the required strength.

7.2.2.3 *Effective length* — The effective length of a fillet weld shall be taken only as that length which corresponds to the specified size and required throat thickness. For practical purposes, the effective length may be taken as the actual length minus twice the weld size.

7.2.2.4 *Minimum length* — The effective length of a fillet weld designed to transmit a load shall not be less than four times the size of the weld.

7.3 Stresses

7.3.1 *Symbols* — Unless otherwise specified the symbols used shall have the following meaning:

P_a = permissible stress due to axial force (kgf/cm²)

P_b = permissible bending stress (kgf/cm²)

f_a = calculated stress due to axial force (kgf/cm²)

f_b = calculated stress due to bending (kgf/cm²)

a, b, t, c, s, e, to = suffixes to indicate axial force, bending, tension, compression, shear, equivalent and torsion respectively

q = shear stress (kgf/cm²)

M = bending moment (kgf/cm)

Q = shear force (kgf)

S = static moment of area of parts to be joined about the centre of gravity of the whole section (cm³)

I = moment of inertia of the weld section about its centre of gravity (cm⁴)

Y = distance of the weld from the centre of gravity of the section (cm)

a = effective throat thickness (cm)

t = size of normal fillet weld leg (cm) = fillet size

k = constant for different angles between fusion faces

l = effective length of a fillet weld $= l' - 2\,t$

l' = total length of a fillet weld $= l + 2\,t$

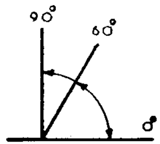
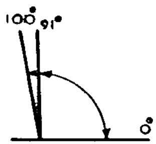
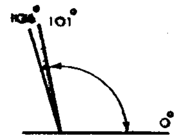
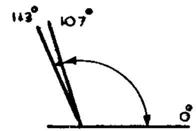
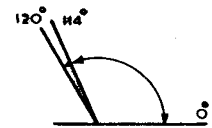
7.3.2 *Calculation of Stresses*

7.3.2.1 *Stresses due to compression, tension and shear* — When subjected to compressive, tensile or shear force the stress in the weld is given by:

$$f \text{ or } q = \frac{P}{\Sigma\,(a.l)}$$

TABLE 13 VALUE OF *k* FOR DIFFERENT ANGLES BETWEEN FUSION FACES

(*Clause* 7.2.2.1)

ANGLES BETWEEN FUSION FACES :	60° TO 90°	91° TO 100°	101° TO 106°	107° TO 113°	114° TO 120°
					
CONSTANT *k* :	0·70	0·65	0·60	0·55	0·50

where P is the type of force transmitted (axial load N or the shear force Q).

7.3.2.2 *Stresses due to bending moment* — When the weld is subjected to bending moment only, the normal stress is

$$f_b = \frac{MY}{I}$$

The horizontal shear (V) resulting from the bending forces is calculated from the formula:

$$V = \frac{Q \cdot S}{I}$$

7.3.3 *Combination of Stresses*

7.3.3.1 *Fillet welds* — The stresses shall be combined using the following formula:

$$f_e = \sqrt{f^2 + 1{\cdot}8\, q^2} \leqslant 1\,100 \text{ kgf/cm}^2$$

where

f = normal stress, compression or tension due to axial or bending forces.

7.3.3.2 *Butt welds* — Butt welds need not be checked for the combination of stresses if they are axially loaded.

7.3.3.3 *Combined bending and shear* — The equivalent stress f_e due to coexistent bending stress (tension or compression) and shear stress is obtained from the following formulae:

$$f_e = \sqrt{f^2_{bc} + 3\, q^2}$$

or

$$f_e = \sqrt{f^2_{bt} + 3\, q^2}$$

f_e should not exceed the value allowed for the parent metal.

7.3.3.4 *Combined bearing, bending and shear stresses* — Where a bearing stress f_{br} is combined with bending (tensile or compressive) and shear stresses under the most unfavourable conditions of loading, the equivalent stress f_e is obtained from the following formulae:

$$f_e = \sqrt{f^2_{bt} + f^2_{br} + f_{bt} \cdot f_{br} + 3\, q^2}$$

or

$$f_e = \sqrt{f^2_{bc} + f^2_{br} - f_{bc} \cdot f_{br} + 3\, q^2}$$

f_e should not exceed the value allowed for the parent metal.

7.3.4 *Permissible Stresses* — Permissible stresses as specified in IS : 816-1969* are given in Table 14.

7.3.5 Permissible stresses in site welds shall be 80 percent of the values specified in Table 14.

7.3.6 In Table 15 is given the strength of fillet welds of different sizes with fusion faces between 60° and 90°.

7.3.7 *Minimum Size of Fillet Welds* — In order to retard the rate of cooling and to prevent brittle failure of the joint certain minimum sizes of fillet welds are specified in Table 16. These are based on the provisions of IS : 816-1969*.

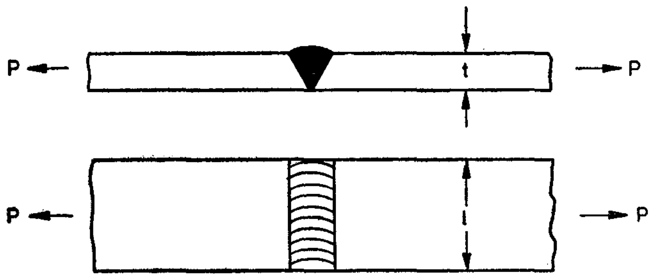
TABLE 14 PERMISSIBLE STRESSES IN WELDS SUBJECTED TO STATIC LOADING

(*Clause* 7.3.4)

Kind of Stress (Shop Welds)	Permissible Value, *Max* kgf/cm^2
a) *Butt welds*	
Tension and compression through throat of butt weld (for rolled I-beam and channels plates and bars)	1 500
Fibre stresses in bending through throat of butt weld (for rolled I-beam and channels), tension fibre and compression fibre	1 650
Fibre stresses in bending through throat of butt weld (for plate girders), tension fibre and compression fibre	1 575
Average shear stress through throat of butt weld	945
b) *Fillet welds*	
Compression, tension shear, equivalent stresses on throat of fillet weld	1 100
c) *Combined bearing, bending and shear stresses of welds*	2 285
d) *Plug welds*	
Shear	1 100

7.4 Formulae Illustrations

7.4.1 *Tension, Compression*



$$f_{\text{butt weld}} = \frac{P}{t\,l} \leqslant P_t = 1\,500 \text{ kgf/cm}^2$$

NOTE — Axially loaded butt welds need no check of stresses [*see* **7.5.2.1** in IS : 816-1969 Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)].

*Code of practice for use of metal arc welding for general construction in mild steel (*first revision*).

TABLE 15 STRENGTH OF FILLETS BASED ON A PERMISSIBLE STRESS OF 1 100 kgf/cm² ($k = 0·7$)

(*Clause* 7.3.6)

Leg Length of Fillet	Throat Thickness	*Strength of Fillet Weld
(1)	(2)	(3)
mm	mm	kgf
3	2·1	231
4	2·8	308
5	3·5	385
6	4·2	462
7	4·9	539
8	5·6	616
10	7·0	770
12	8·4	924
14	9·8	1 078
16	11·2	1 232
18	12·6	1 386
20	14·0	1 540
22	15·4	1 694
24	16·8	1 848
25	17·5	1 925

*80 percent of these values should be assumed for welds made at site.

TABLE 16 MINIMUM SIZE OF FIRST RUN OR OF A SINGLE RUN FILLET WELD

(*Clause* 7.3.7)

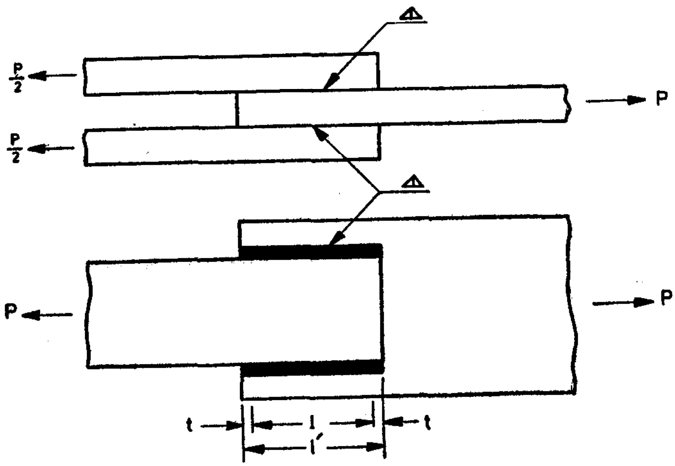
Thickness of Thicker Part		Minimum Size
Over	Up to and Including	
mm	mm	mm
(1)	(2)	(3)
—	10	3
10	20	5
20	32	6
32	50 (*see* Notes)	8 first run 10 minimum size of fillet

Note 1 — When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thickner part shall be adequately preheated to prevent cracking of the weld.

Note 2 — Where the thicker part is more than 50 mm thick, special precautions like preheating will have to be taken.

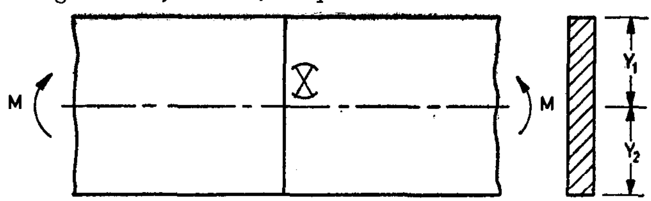
7.4.1.1 When subjected to compression the permissible stresses on butt welds will be the same as permitted for the parent metal.

7.4.2 *Fillet Weld*



$$q = \frac{P}{\Sigma\,(a.l)} \leqslant P_s = 1100 \text{ kgf/cm}^2$$

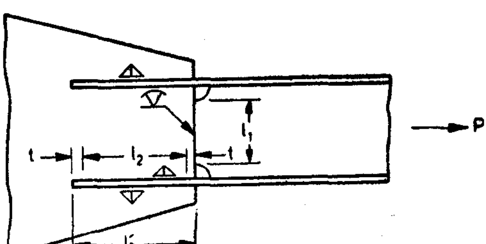
7.4.3 *Bending Moment, Tension, Compression*



$$f_b = \frac{M}{I}.Y = \frac{M}{Z} \leqslant P_b = 1\,575 \text{ and } 1\,650 \text{ kgf/cm}^2$$

for rolled I-beams and channels respectively.

7.4.4 *Combined Butt and Fillet Weld*



P = Strength of butt weld + strength of fillet welds

In the sketch shown above,

Strength of butt weld $= l_1 \times$ effective throat thickness $\times$ permissible stress in tension

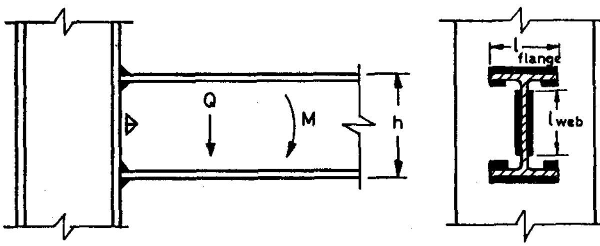
Strength of fillet welds $= \Sigma\, a l_2 f$

where

$a = K \times t = 0{\cdot}70\, t$, and

f = permissible shear stress in fillet welds.

7.4.5 *Fillet Weld Subjected to Moment and Shear* — The flange welds are proportioned to resist the bending moment. The welds joining the web to column are assumed to carry the shear.



Web:

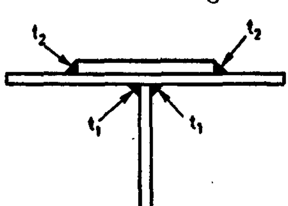
$$q = \frac{Q}{\Sigma(a.l_{\text{web}})} \leqslant P_s = 1\,100 \text{ kgf/cm}^2$$

Flange:

$$q = \frac{M}{h} \cdot \frac{1}{\Sigma\,(a.l_{\text{flange}})} \leqslant P_s = 1\,100 \text{ kgf/cm}^2$$

Tension is taken by upper fillet weld (top flange),
Compression by the lower fillet weld (lower flange), and
Shear stress by the web fillet weld.

7.4.6 *Horizontal Shear Stress Due to Bending Moments*



$$q = \frac{Q.S}{I \cdot \Sigma a} \leqslant P_s = 1\,100 \text{ kgf/cm}^2$$

where

Q = transverse force,

S = static moment of the part to be joined,

I = moment of inertia, and

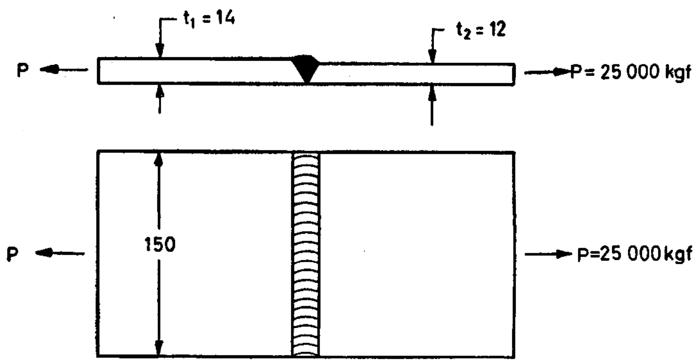
$\Sigma a = \Sigma\, k.t = \Sigma\, 0{\cdot}7.t.$

Substitute t by t_1 while designing the welds between the web and the first flange plate and by t_2 for designing the welds between the two flange plates.

7.5 Examples — The methods of using the formulae given in **7.3** and **7.4** in the design of welded joints are illustrated in the following examples. Steel conforming to IS : 226-1969* is used and the weld sizes and stresses are within the limits specified in Tables 14, 15 and 16.

7.5.1 *Example 1*

Two plates of thicknesses 14 and 12 mm are joined by a single-V butt weld as shown in the sketch. The joint is subjected to a tensile force of 25 000 kgf. It is required to find the tensile stress developed in the butt weld.



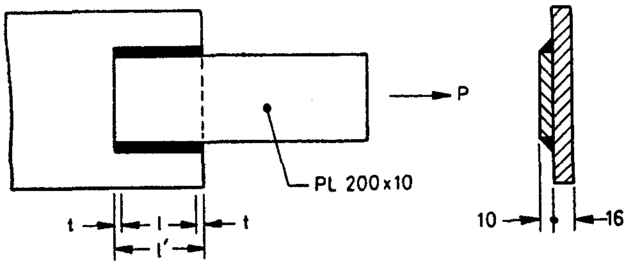
$t = t_2 = 12$ mm

$$f_{\text{butt weld}} = \frac{P}{t\,l} = \frac{25\,000}{1{\cdot}2 \times 15} = 1\,390 \text{ kgf/cm}^2$$

*Specification for structural steel (standard quality) (*fourth revision*).

7.5.2 *Example* 2

A fillet welded lap joint is subjected to a tensile force of 20 400 kgf. It is required to calculate the size and length of fillet weld required.



$$P = 20\,400 \text{ kgf}$$

Permissible shear stress in the fillet $P_s = 1\,100$ kgf/cm²

Weld area required

$$A = \frac{P}{P_s} = \frac{20\,400}{1\,100} = 18{\cdot}6 \text{ cm}^2$$

Adopted weld thickness

$$t = 0{\cdot}6 \text{ cm}$$

$$a = 0{\cdot}7\, t = 0{\cdot}7 \times 0{\cdot}6 = 0{\cdot}42 \text{ cm}$$

Required effective length of the weld

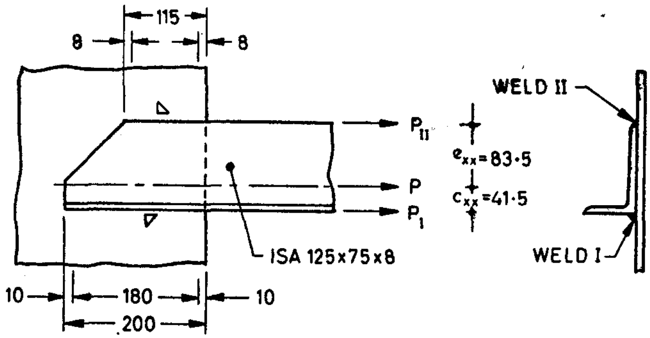
$$l = l' - 2t = \frac{18{\cdot}6}{2 \times 0{\cdot}42} = 22{\cdot}1 \text{ cm}$$

Total length

$$l' = 22{\cdot}1 + 1{\cdot}2 = 23{\cdot}3 \text{ cm, say 24 cm}$$

7.5.3 *Example 3*

An unequal leg angle ISA 125 × 75 × 8 lap welded to a 10-mm plate carries a tension of 20 400 kgf. In order to balance the forces Weld *I* should be designed to carry a load of 13 600 kgf and the Weld *II* to carry 6 800 kgf (*see* sketch below).



$$P = 20\,400 \text{ kgf}$$

$$P_s = 1\,100 \text{ kgf/cm}^2$$

$$P_I = 20\,400 \times \frac{8{\cdot}35}{12{\cdot}50} = 13\,600 \text{ kgf}$$

$$P_{II} = 20\,400 \times \frac{4{\cdot}15}{12{\cdot}50} = 6\,800 \text{ kgf}$$

Weld I

Throat area of weld required

$$A = \frac{P_I}{P_s} = \frac{13\,600}{1\,100} = 12{\cdot}4 \text{ cm}^2$$

Adopted weld thickness

$$t = 1 \text{ cm} > 0{\cdot}3 \text{ cm (\textit{see} Table 16)}$$

Effective throat thickness

$$a = 0{\cdot}7 \times 1{\cdot}0 = 0{\cdot}7 \text{ cm}$$

Required effective length of the weld

$$l = l' - 2.t = \frac{12{\cdot}4}{0{\cdot}7} = 18 \text{ cm}$$

Total length provided

$$l' = l + 2.t = 18 + 2{\cdot}0 = 20 \text{ cm}$$

Check shear stress in the fillet weld

$$q = \frac{P_I}{a.l} = \frac{13\,600}{0{\cdot}7 \times 18} = 1\,080 \text{ kgf/cm}^2$$

$$< 1\,100 = P_s \ldots\ldots\ldots\ldots\ldots\ldots \text{OK}$$

Weld II

Weld area required

$$= \frac{P_{II}}{P_s} = \frac{6\,800}{1\,100} = 6{\cdot}2 \text{ cm}^2$$

Adopted weld thickness

$$t = 0{\cdot}8 \text{ cm}$$
$$a = 0{\cdot}7 \times 0{\cdot}8 = 0{\cdot}56 \text{ cm}$$

Required effective length of the weld

$$l = l' - 2.t = \frac{6{\cdot}2}{0{\cdot}56} = 11{\cdot}0 \text{ cm}$$

Total length provided

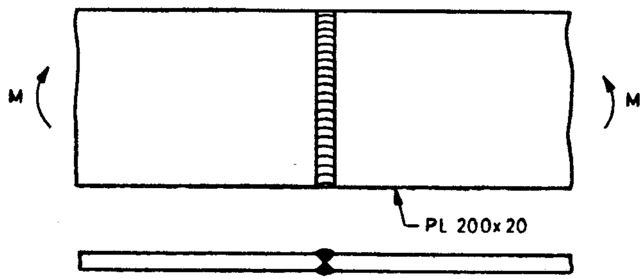
$$l' = l + 2.t = 11{\cdot}0 + 2 \times 0{\cdot}8 = 12{\cdot}5 \text{ cm}$$

Check shear stress in the fillet weld

$$q = \frac{P_{II}}{a.l} = \frac{6\,800}{11{\cdot}0 \times 0{\cdot}56} = 1\,100 \text{ kgf/cm}^2$$
$$= P_s \text{ .... OK}$$

7.5.4 *Example 4*

A butt weld connecting two plates as shown in the sketch is subjected to a bending moment. Calculate the stress developed in the weld and check if it is within permissible limits.



Bending moment $M = 140\,000$ kgf/cm

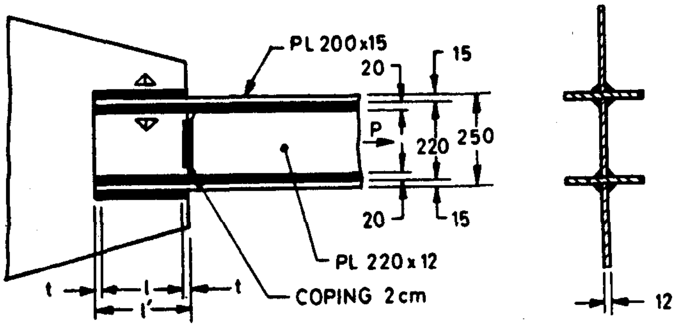
Permissible bending stress $P_b = 1\,575$ kgf/cm²

$$\text{Section modulus } Z = \frac{2{\cdot}0 \times 20{\cdot}0^2}{6} = 133 \text{ cm}^3$$

$$f_b = \frac{M}{Z} = \frac{140\,000}{133} = 1\,050 \text{ kgf/cm}^2$$
$$< P_b \text{ .... OK}$$

7.5.5 *Example 5*

The sketch shows a joint where a combination of butt and fillet welds has been used. It is required to design the fillet welds.



$P = 100\,000$ kgf tension

Permissible stresses in welds

Butt weld $P_t = 1\,500$ kgf/cm²

Fillet weld $P_s = 1\,100$ kgf/cm²

Area of cross section of the member $= 22 \times 1{\cdot}2 + 2 \times 20 \times 1{\cdot}5$

$= 26{\cdot}40 + 60{\cdot}00 = 86{\cdot}40$ cm²

$$P_{web} = 100\,000 \times \frac{26{\cdot}40}{86{\cdot}40} = 30\,600 \text{ kgf}$$

$$P_{flanges} = 100\,000 \times \frac{60{\cdot}00}{86{\cdot}40} = 69\,400 \text{ kgf}$$

Check total $P = 100\,000$ kgf

a) *Butt Weld of the Web*

Weld area required $= \dfrac{P_{web}}{P_t} = \dfrac{30\,600}{1\,500} = 20{\cdot}40$ cm²

Deducting for 2 copings, area of butt weld provided

$= 1{\cdot}2\,(22{\cdot}00 - 22{\cdot}00) = 21{\cdot}60$ cm²

Check tension stress in the butt weld

$$f_t = \frac{30\,600}{21{\cdot}60} = 1\,420 \text{ kgf/cm}^2$$

$< P_t$ .... OK

b) *Fillet Weld of the Notched Flanges*

Fillet weld area required $= \dfrac{P_{\text{flanges}}}{P_s} = \dfrac{69\,400}{1\,100} = 63{\cdot}0 \text{ cm}^2$

Adopt a weld thickness of 1·2 cm

Effective throat thickness

$$a = 0{\cdot}7 \times 1{\cdot}2 = 0{\cdot}85 \text{ cm}$$

Effective length required of one fillet weld (there are 8 welds of equal length) $l = \dfrac{63{\cdot}00}{8 \times 0{\cdot}85} = 9{\cdot}3$ cm

Total length l' of each weld $= l + 2\,t = 9{\cdot}30 + 2{\cdot}0 \times 1{\cdot}2 = 12{\cdot}0$ cm

Check shear stress of the 8 fillet welds

$$q = \frac{P_{\text{flanges}}}{a.l} = \frac{69\,400}{8 \times 0{\cdot}85 \times 9{\cdot}30} = 1\,100 \text{ kgf/cm}^2$$

$$= P_s \text{ ....OK}$$

Check effective length of one fillet weld

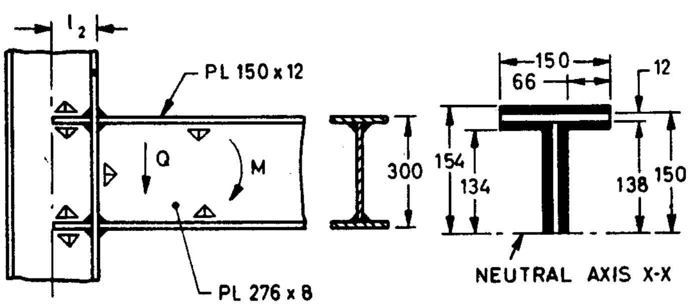
Minimum effective length $= 4\,t = 4 \times 1{\cdot}2 = 4{\cdot}8$ cm

Effective length of weld provided $= 9{\cdot}3$ cm

Actual length of weld provided $= 12{\cdot}0$ cm

7.5.6 *Example 6*

A rigid beam to column connection as shown in figure is subjected to a moment and a shear. It is required to determine the size of fillet welds.



Given:

Bending moment $M = 700\,000$ kgf/cm

Shear force $Q = 10\,000$ kgf

Assume 8 mm fillets for flange and 5 mm fillets for web

Moment of inertia of the weld

$$I_{x,\ \text{weld}} = 2 \times \frac{0{\cdot}5 \times 27{\cdot}6^3}{12} + 4 \times 6{\cdot}6 \times 0{\cdot}8 \times 13{\cdot}4^2 + 2 \times 15{\cdot}0 \times 0{\cdot}8 \times 15{\cdot}4^2$$

$$= 1\,750 + 3\,790 + 5\,700$$

$$= 11\,240 \text{ cm}^4$$

Modulus of section of the weld

$$Z_{x,\ \text{weld}} = \frac{11\,240}{15{\cdot}8} = 712 \text{ cm}^3$$

Fibre stress in the fillet weld

$$f_b = \frac{M}{Z} = \frac{700\,000}{712} = 984 \text{ kgf/cm}^2$$

$$< P_s \text{ ....OK}$$

Shear stress in the web weld

$$q = \frac{Q}{\Sigma\,(a.l)_{\text{web}}} = \frac{10\,000}{2 \times 0{\cdot}5 \times 27{\cdot}6}$$

$$= 362 \text{ kgf/cm}^2 \leqslant 1\,100 \text{ kgf/cm}^2 = P_s$$

Weld size $t = \frac{a}{k} = \frac{0{\cdot}5}{0{\cdot}7} = 0{\cdot}7$ cm

Stiffeners (upper two stiffeners tension, lower two stiffeners compression)

Force in two stiffeners $P = \frac{M}{h} = \frac{700\,000}{28{\cdot}8} = 24\,300$ kgf

$$q = \frac{P.}{\Sigma\,(a.l)} \leqslant P_s = 1\,100 \text{ kgf/cm}^2$$

$$\Sigma\,(a.l)\ \frac{P}{P_s} = \frac{24\,300}{1\,100} = 22{\cdot}0 \text{ cm}$$

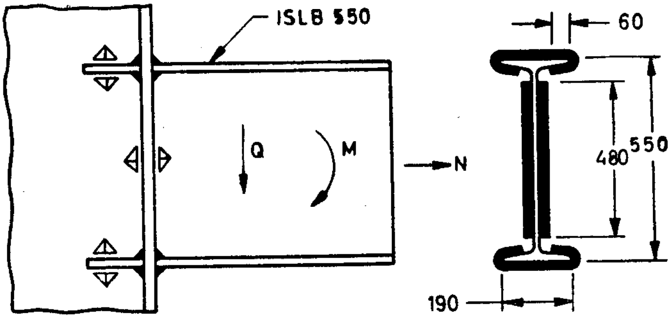
Adopt 4 fillet welds of size 7 mm.

Total length l_2 required

$$= \frac{23{\cdot}7}{4{\cdot}0 \times 7 \times 0{\cdot}7} + 2 \times 0{\cdot}7 = 12{\cdot}4 + 1{\cdot}4 = 14{\cdot}0 \text{ cm}$$

7.5.7 *Example 7*

A beam to column connection is subjected to a moment, shear and axial tension as shown in the figure. The flanges are connected with 14 mm fillet welds and the web with 8·5 mm fillets.



t_{flange} = 14 mm

$a_{flange} = k \times t_{fl} = 0{\cdot}7 \times 14 = 10$ mm

t_{web} = 8 mm

$a_{web} = k \times t_{web} = 0{\cdot}7 \times 8{\cdot}0 = 5{\cdot}6$ mm

Given:

Profile ISLB 550

Bending moment M = 1 800 000 kgf/cm

Shear force Q = 20 000 kgf

Tension force N = 12 000 kgf

Moment of inertia of the weld

$$I_{x,\,weld} = 2 \times 1{\cdot}0 \times 19{\cdot}0 \times \left(\frac{55{\cdot}0}{2}\right)^2 + 4 \times 6{\cdot}0 \times 1{\cdot}0 \times \left(\frac{53{\cdot}5}{2}\right)^2 + 2 \times \frac{5{\cdot}6 \times 48^3}{12}$$

$$= 28\,800 + 17\,200 + 10\,340 = 56\,340 \text{ cm}^4$$

Section modulus of the weld

$$Z_{x,\,weld} = \frac{56\,340}{55/2} = 2\,050 \text{ cm}^3$$

Fibre stress in bending

$$f_b = \frac{M}{w} = \frac{1\,800\,000}{2\,050} = 878 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2$$

$$A_{weld} = \Sigma a.l = 2 \times 0{\cdot}56\ (48{\cdot}0 - 2 \times 0{\cdot}56) + 4 \times 1{\cdot}0\ (6{\cdot}0 - 1{\cdot}0) + 2 \times 1,0 \times 19$$

$$= 52{\cdot}5 + 20 + 38$$

$$= 110{\cdot}5 \text{ cm}^2$$

$$\text{Tensile stress } f_a = \frac{N}{\Sigma (a.l)} = \frac{N}{\text{weld}} = \frac{12\,000}{110{\cdot}5} = 110 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2$$

Total normal stress = 848 + 110 = 958 kgf/cm²

Assume that the shear is resisted by the welds on the web only

$$\text{Shear stress } q = \frac{Q}{\Sigma (a.l)} = \frac{20\,000}{2 \times 0{\cdot}6\ (48{\cdot}0 - 2 \times 0{\cdot}6)} = \frac{20\,000}{56}$$

$$= 360 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2 = P_s$$

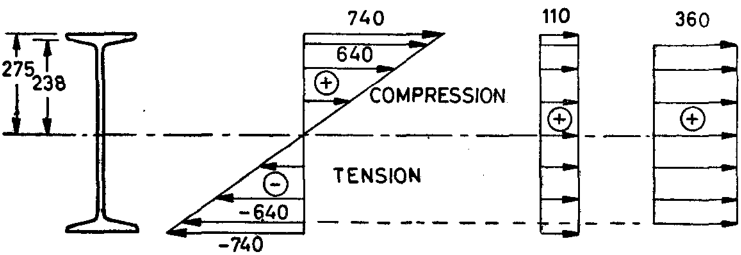
Equivalent stress

$$f_e = \sqrt{f^2 + 1{\cdot}8\, q^2} = \sqrt{958^2 + 1{\cdot}8 \times 360^2} = \sqrt{918\,000 + 234\,000}$$

$$= \sqrt{1\,152\,000} = 1\,070 < 1\,100 \text{ kgf/cm}^2$$

$$< P_s \ldots\ldots\ldots\ldots\ldots\ldots \text{OK}$$

Distribution of the stresses of the welds is shown in the sketch below:



7.5.8 *Example 8*

A leg welded on to the flange of a I-beam section as shown in the figure is subjected to an eccentric force of 12 tonnes. It is required to design the welded joint.

Given:

Tension force $N = 12\,000$ kgf

Bending moment $M = 12\,000 \times 3 = 36\,000$ kgf.cm

Adopted fillet size $t = 6$ mm

Effective throat thickness $a = 0{\cdot}7 \times 6 = 4{\cdot}2$ mm

$$A_{weld} = \Sigma (a.l) = 2 \times 0{\cdot}42\ (8{\cdot}0 + 15{\cdot}0) = 19{\cdot}3 \text{ cm}^2$$

Moment of inertia of the weld

$$I_{\text{weld}} = 2 \times \frac{0{\cdot}42 \times 8{\cdot}0^3}{12} + 2 \times 0{\cdot}42 \times 15{\cdot}0 \times (4{\cdot}2)^2$$

$$= 36 + 222 = 258 \text{ cm}^4$$

Section modulus of the weld

$$Z_{\text{weld}} = \frac{258}{4{\cdot}4} = 59 \text{ cm}^3$$

Shear stress

$$q = \frac{N}{A_{\text{weld}}} = \frac{12\,000}{19{\cdot}3} = 622 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2$$

$$< P_s \text{..................OK}$$

Fibre stress in bending

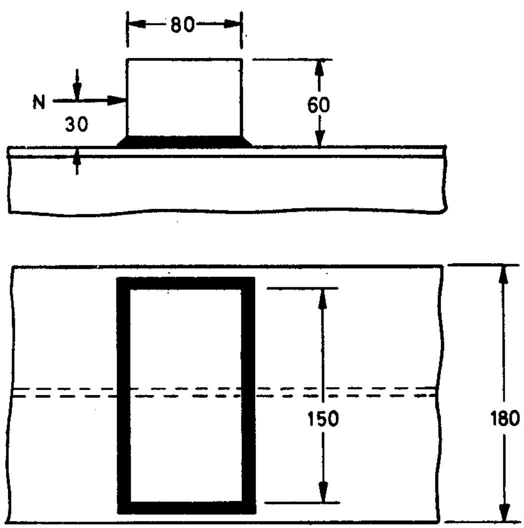
$$f_b = \frac{M}{Z_{\text{weld}}} = \frac{36\,000}{60{\cdot}0} = 600 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2$$

Equivalent stress

$$f_e = \sqrt{f^2 + 1{\cdot}8\,q^2} = \sqrt{593^2 + 1{\cdot}8 \times 622^2} = \sqrt{35\,649 + 613\,000}$$

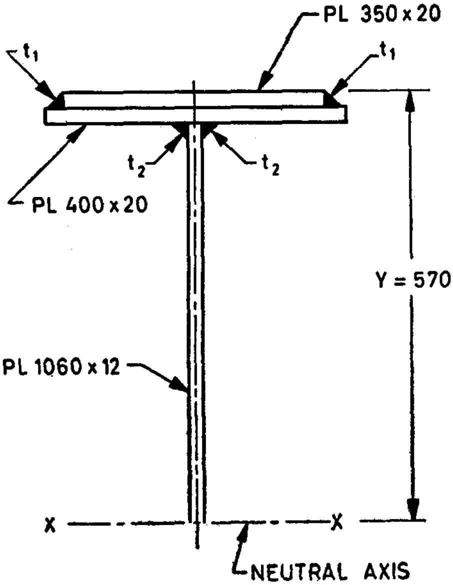
$$= \sqrt{964\,649} = 983 < 1\,100 \text{ kgf/cm}^2$$

$$< P_s \text{.........................OK}$$



7.5.9 *Example 9*

The profile of a plate girder fabricated from steel conforming to IS : 226-1969* is shown in the sketch. It is required to check the adequacy of the section for the loads given, and to design the welds connecting flanges and the web.



Given:

Bending moment $M = 9\,000\,000$ kgf/cm

Shear force $Q = 109\,000$ kgf

Properties of profile

$$I_{xx} = 1\,025\,000 \text{ cm}^4$$
$$Z_x = 18\,000 \text{ cm}^3$$
$$S_x = 9\,925 \text{ cm}^3$$

a) *Stresses in Steel Members (Flanges and Web)* :

Fibre stress in bending

$$f_{bt} = f_{bc} = \frac{M}{Z} = \frac{9\,000\,000}{18\,000} = 500 \text{ kgf/cm}^2 < 1\,575$$
$$= P_{bt} = P_{bc} \text{ (\textit{see} IS : 800-1962†, Table III)}$$

*Specification for structural steel (standard quality) (*fourth revision*).

†Code of practice for use of structural steel in general building construction (*revised*).

Maximum shear stress in the middle of the web

$$f_s = \frac{Q \cdot S}{I^b \cdot_{web}} = \frac{109\,000 \times 9\,925}{1\,025\,000 \times 1 \cdot 2} = 880 \text{ kgf/cm}^2$$

$$< 1\,100 \text{ kgf/cm}^2 = P_q \text{ (see IS : 800-1962*, Table VIII)}$$

Equivalent stress as per **10.5.4** of IS : 800-1962*

$$f_e = \sqrt{(f_{bt}^2 + f_b^2 + f_{bt} \cdot f_b + 3 f_q^2)}$$

In this case,

$$f_{bt} = \frac{M}{I_x} \cdot Y_2 = \frac{9\,000\,000}{1\,025\,000} \times 53 = 465 \text{ kgf/cm}^2$$

$$f_q = \frac{Q}{A_{web}} = \frac{109\,000}{106 \times 1 \cdot 2} = 860 \text{ kgf/cm}^2 < 945 \text{ kgf/cm}^2 = P_q$$

(*see* IS : 800-1962*, Table IX)

$$f_e = \sqrt{465^2 + 3 \times 86^2} = \sqrt{216\,000 + 222\,000}$$

$$= \sqrt{238\,200} = 488 \text{ kgf/cm}^2 < 2\,285 \text{ kgf/cm}^2 \text{ ....OK}$$

(*see* IS : 800-1962*, Table XIII)

b) *Stresses in the Welds*:

Welds connecting webs and flange plate:

Adopted fillet weld size $t_2 = 7$ mm

Effective throat thickness $a_2 = k \times t_2 = 0 \cdot 7 \times 7 = 5$ mm

$$q = \frac{Q \cdot S}{I_x . \Sigma a} = \frac{Q \cdot S}{I_x \cdot 2a}$$

$$S = 40 \times 2 \times 54 + 35 \times 2 \times 56 = 8\,240 \text{ cm}^3$$

$$2a = 2 \times 0 \cdot 5 = 1 \cdot 0 \text{ cm}$$

$$q = \frac{109\,000 \times 8\,240}{1\,025\,000 \times 1 \cdot 0} = 875 \text{ kgf/cm}^2 < 1\,100 \text{ kgf/cm}^2$$

$$< P_s \text{.........OK}$$

Welds connecting the flange and cover plate:

Adopted fillet weld size $t_1 = 0 \cdot 6$ cm

*Code of practice for use of structural steel in general building construction (*revised*).

Effective throat thickness $a_1 = k.t_1 = 0{\cdot}7 \times 0{\cdot}6 = 0{\cdot}42$ cm

$$q = \frac{Q{\cdot}S}{I_x{\cdot}\ \Sigma a} = \frac{Q{\cdot}S}{I_x.2a}$$

$$S = 35 \times 2 \times 56 = 3\,920 \text{ cm}^3$$

$$2a = 2 \times 0{\cdot}42 = 0{\cdot}85 \text{ cm}$$

$$q = \frac{109\,000 \times 3\,920}{1\,025\,000 \times 0{\cdot}85} = 490 \text{ kgf/cm}^2$$

$< 1\,100$ kgf/cm²
$< P_s$.........OK

APPENDIX A

(*Clause* 7.1)

ANALYSIS OF SIMPLE BEAMS

A-0. SYMBOLS

A-0.1 The following symbols are used in this appendix:

A, B = Reactions at supports

P = Single point load

q = Uniformly distributed load per unit length of beam

M = Bending moment

I = Moment of inertia

E = Modulus of elasticity

l = Length

x = Distance from the support of the point under consideration

Y = Deflection curve

Δ = Deflection

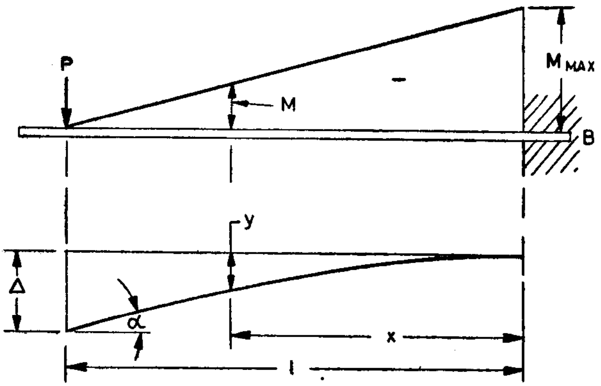
α = Slope at the support

W = Nodal point of the deflection curve

α_{th} = Thermal coefficient of expansion

A-1. STATICALLY DETERMINATE CASES

A-1.1 Cantilever Beam — Concentrated Load at Free End



$B = P$

$M = P\,(l - x)$

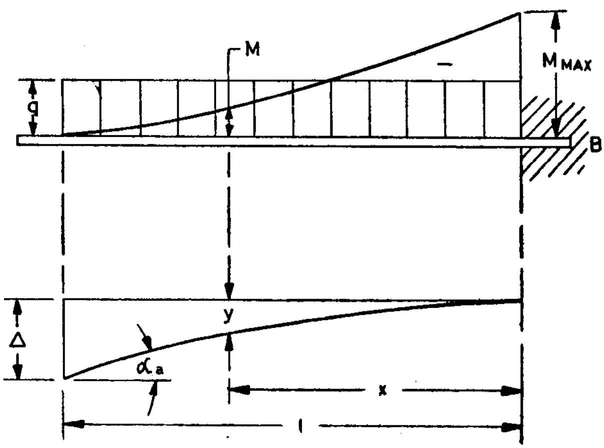
$M_{Max} = -\,Pl$

$y = \dfrac{Pl^3}{EI}\left(\dfrac{x^2}{2l^2} - \dfrac{x^3}{6l^3}\right)$

$\Delta = \dfrac{Pl^3}{3\,EI}$

$\alpha = \dfrac{Pl^2}{2\,EI} = \dfrac{3}{2}\dfrac{\Delta}{l}$

A-1.2 Cantilever Beam — Uniformly Distributed Load



Total load $P = ql$

$B = P$

$M = -\,\dfrac{Pl}{2}\left(1 - \dfrac{x}{l}\right)^2$

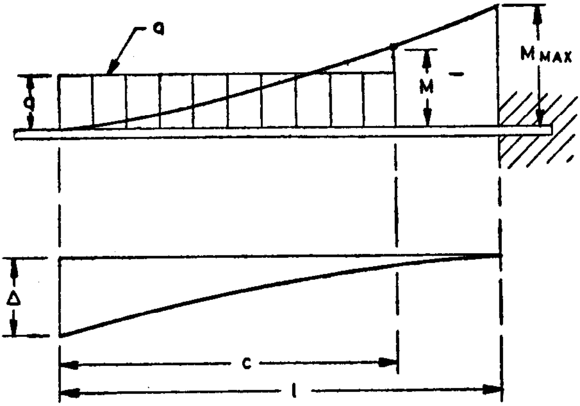
$M_{Max} = -\,\dfrac{Pl}{2}$

$= \dfrac{Pl^3}{EI}\left(\dfrac{x^2}{4l^2} - \dfrac{x^3}{6l^3} + \dfrac{x^4}{24l^4}\right)$

$\Delta = \dfrac{Pl^3}{8\,EI}$

$\alpha_a = \dfrac{Pl^2}{6\,EI} = \dfrac{4\,\Delta}{3l}$

A-1.3 Cantilever Beam — Uniformly Distributed for a Distance C From Free End



Total load $P = qc = B$

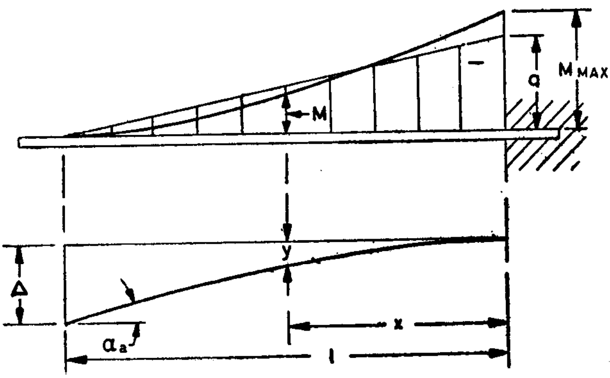
$$M = -\frac{qc^2}{2}$$

$$M_{Max} = -qc\left(l - \frac{c}{2}\right)$$

$$\gamma = \frac{c}{l}$$

$$\Delta = \frac{ql^3 c}{24\,EI}\,(8 - 6\gamma + \gamma^3)$$

A-1.4 Cantilever Beam — Load Uniformly Increasing to Fixed End



Total load $P = \frac{ql}{2}$

$B = P$

$$M = -\frac{Pl}{3}\left(1 - \frac{x}{l}\right)^3$$

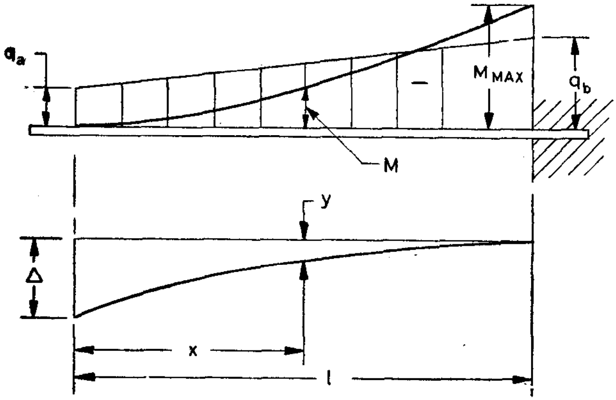
$$M_{Max} = -\frac{Pl}{3}$$

$$y = \frac{Pl^3}{EI}\left(\frac{x^2}{6l^2} - \frac{x^3}{6l^3} + \frac{x^4}{12l^4} - \frac{x^5}{60l^5}\right)$$

$$\Delta = \frac{Pl^3}{15EI}$$

$$\alpha_a = \frac{Pl^2}{12EI} = \frac{5\Delta}{4l}$$

A-1.5 Cantilever Beam — Trapezoidal Load



Total load $P = \frac{q_a + q_b}{2} \cdot l = B$

$M = -\frac{x^2}{6} q_a \left[(3 - {}'x\,\xi') + q_b \cdot \xi \right]$

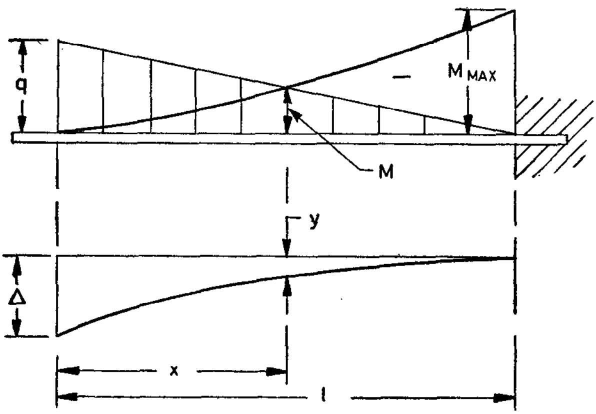
$M_{\text{Max}} = -\frac{l^2}{6} (2 q_a + q_b)$

$y = \frac{l^4}{120\,EI} \left[q_a (11 - 15\xi + 5\,\xi^4 - \xi^5) + q_b (4 - 5\,\xi + \xi^5) \right]$

$\xi = \frac{x}{l}$

$\Delta = \frac{l^4}{120\,EI} (11\,q_a + 4\,q_b)$

A-1.6 Cantilever Beam — Load Increasing Uniformly to Free End



Total load $P = \frac{ql}{2} = B$

$M = -\frac{qx^2}{6} (3 - \xi)$

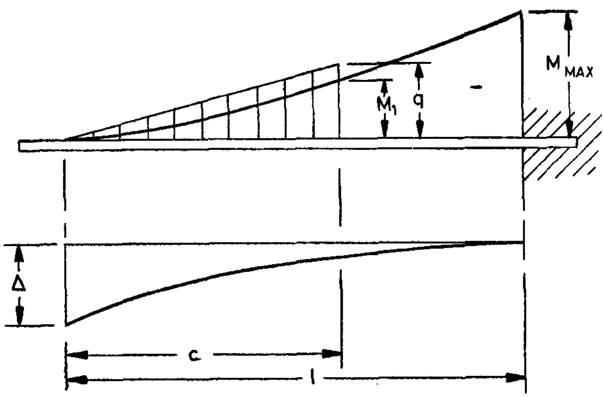
$M_{\text{Max}} = -\frac{ql^2}{3}$

$y = \frac{ql^4}{120\,EI} (11 - 15\,\xi + 5\,\xi^4 - \xi^5)$

$\xi = \frac{x}{l}$

$\Delta = \frac{11\,ql^4}{120\,EI}$

A-1.7 Cantilever Beam — Load Increasing Uniformly to Fixed End for a Distance *C*



$B = \frac{qc}{2}$

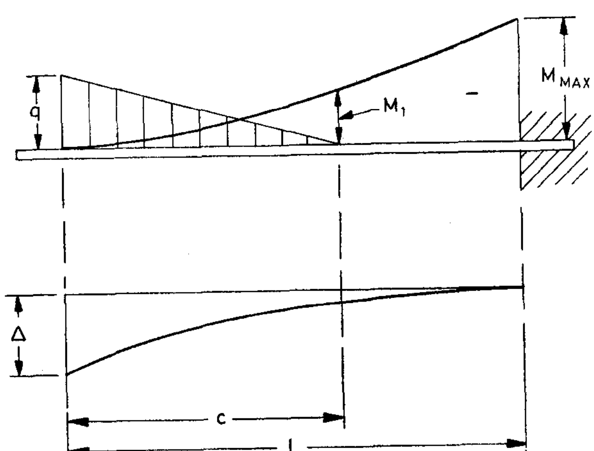
$M_1 = -\frac{qc^2}{6}$

$M_{Max} = -\frac{qc}{2}\left(l - \frac{2}{3}c\right)$

$\triangle = \frac{ql^3 c}{30\,EI}(5 - 5\gamma + \gamma^4)$

$\gamma = \frac{c}{l}$

A-1.8 Cantilever Beam — Load Increasing Uniformly for a Distance *C* to Free End



$B = \frac{qc}{2}$

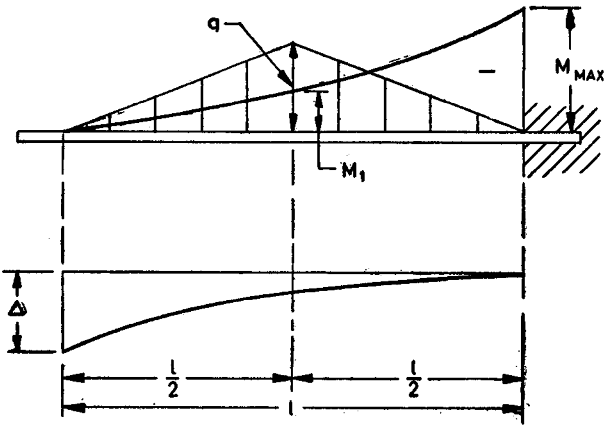
$M_1 = -\frac{qc^2}{3}$

$M_{Max} = -\frac{qc}{2}\left(l - \frac{c}{3}\right)$

$\triangle = \frac{ql^3 c}{120\,EI}(20 - 10\gamma + \gamma^4)$

$\gamma = \frac{c}{l}$

A-1.9 Cantilever Beam — Triangular Load with Apex at the Centre of Beam



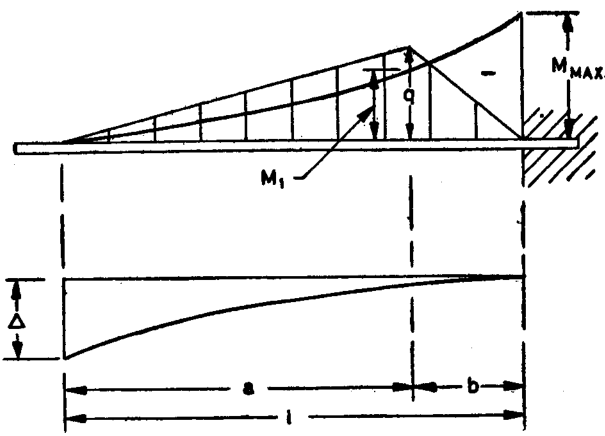
Total load $P = \frac{ql}{2} = B$

$M_1 = -\frac{ql^2}{24}$

$M_{\text{Max}} = -\frac{ql^2}{4}$

$\Delta = \frac{11\, ql^4}{192\, EI}$

A-1.10 Cantilever Beam — Triangular Load with Apex at *b* from Fixed End



Total load $P = \frac{ql}{2} = B$

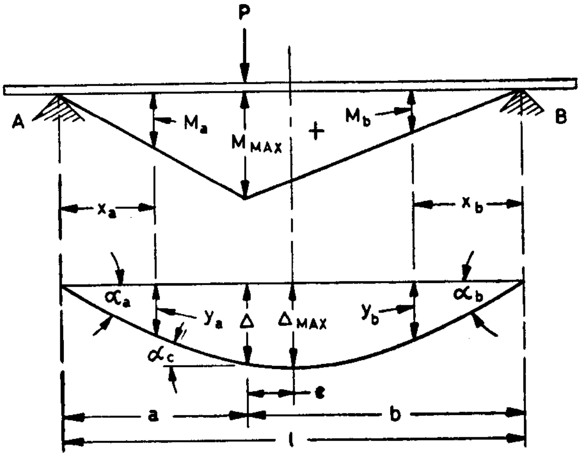
$M_1 = -\frac{qa^2}{6}$

$M_{\text{Max}} = -\frac{ql}{6}(l + b)$

$\alpha = \frac{a}{l}$

$\Delta = \frac{ql^5}{120\, bEI}(11 - 20\alpha + 10\alpha^2 - \alpha^4)$

A-1.11 Simple Beam — Concentrated Load at Any Point



$$A = P\frac{b}{l},\ B = P\frac{a}{l}$$

$$M_a = P\frac{b}{l}x_a,$$

$$M_b = P\frac{a}{l}x_b,$$

$$M_{Max} = P\frac{ab}{l}$$

$$y_a = \frac{Pbx_a\,(l^2 - b^2 - x_a{}^2)}{6\,EI\,l}$$

$$y_b = \frac{Pax_b\,(l^2 - a^2 - x_b{}^2)}{6\,EI\,l}$$

$$\Delta = \frac{Pa^2b^2}{3\,EI\,l}$$

Δ_{Max} for $e =$

$$b\sqrt{\frac{1}{3} + \frac{2a}{3b}} - \frac{1}{2},$$

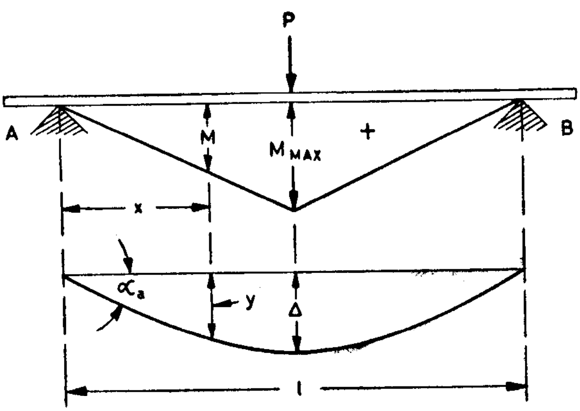
if $b > a$

$$\alpha_a = \frac{Pab\,(l + b)}{6\,EI\,l} = \Delta\left(\frac{1}{a} + \frac{1}{2b}\right)$$

$$\alpha_b = \frac{Pab\,(l + a)}{6\,EI\,l} = \Delta\left(\frac{1}{b} + \frac{1}{2a}\right)$$

$$\alpha_c = \frac{Pab\,(b - a)}{3\,EI\,l}$$

A-1.12 Simple Beam — Concentrated Load at Centre



$$A = B = \frac{P}{2}$$

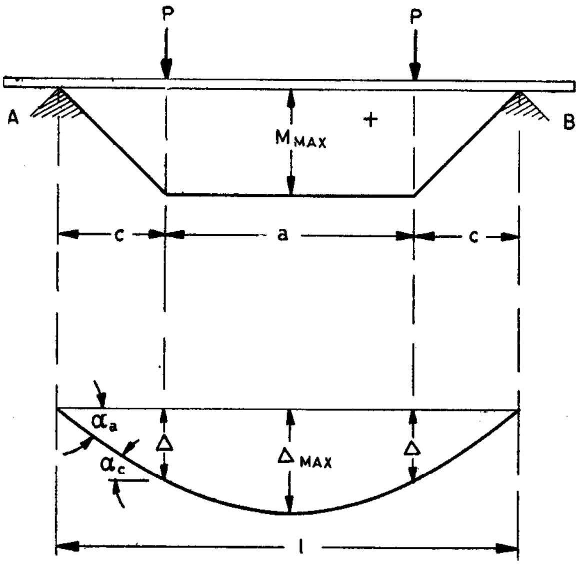
$$M = \frac{Px}{2},\ M_{Max} = \frac{Pl}{4}$$

$$y = \frac{Pl^3}{EI}\left(\frac{x}{16\,l} - \frac{x^3}{12\,l^3}\right)$$

$$\Delta = \frac{Pl^3}{48\,EI}$$

$$\alpha_a = \frac{Pl^2}{16\,EI} = \frac{3\Delta}{l}$$

A-1.13 Simple Beam — Two Equal Concentrated Loads Symmetrically Placed



$A = B = P$

$M_{Max} = Pc$

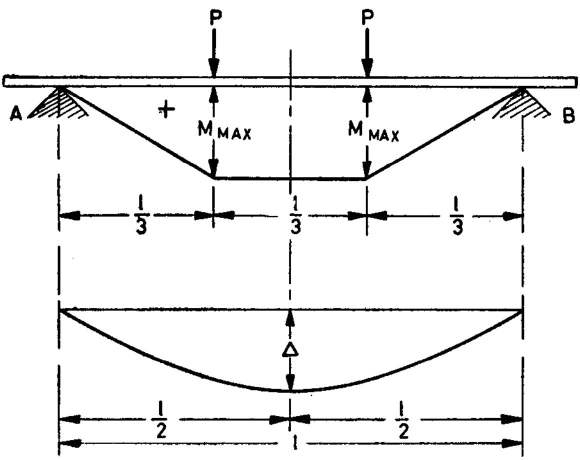
$$\triangle = \frac{Pc^2\,(3l - 4c)}{6\,EI}$$

$$\triangle_{Max} = \frac{Pc\,(3l^2 - 4c^2)}{24\,EI}$$

$$\alpha_a = \frac{Pc\,(a + c)}{2\,EI}$$

$$\alpha_c = \frac{Pac}{2\,EI}$$

A-1.14 Simple Beam — Two Equal Concentrated Loads Symmetrically Placed

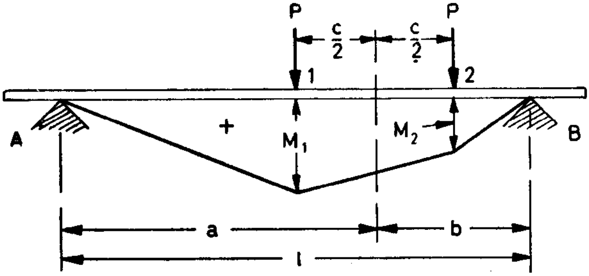


$A = B = P$

$$M_{Max} = \frac{Pl}{3}$$

$$\triangle = \frac{23\,Pl^3}{648\,EI} = \frac{M_{Max}\,l^2}{9{\cdot}39\,I}$$

A-1.15 Simple Beam — Two Equal Concentrated Loads Unsymmetrically Placed



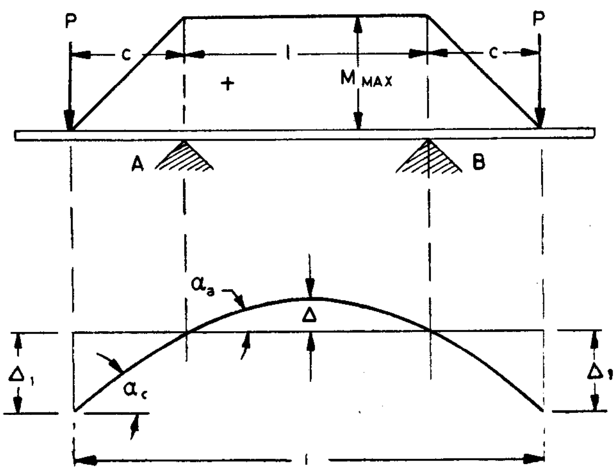
$$A = \frac{2\,Pb}{l},\ B = \frac{2\,Pa}{l}$$

$$M_1 = \frac{2\,Pb}{l}\left(a - \frac{c}{2}\right)$$

$$M_2 = \frac{2\,Pa}{l}\left(b - \frac{c}{2}\right)$$

$\gamma = \frac{c}{l}$	M_1 Max	A	B
0	0·500 Pl	1·00 P	1·00 P
0·1	0·451 Pl	0·95 P	1·05 P
0·2	0·405 Pl	0·90 P	1·10 P
0·3	0·361 Pl	0·85 P	1·15 P
0·4	0·320 Pl	0·80 P	1·20 P
0·5	0·281 Pl	0·75 P	1·25 P
0·586	0·250 Pl	0·707 P	1·293 P
> 0·586	Take single load in the middle of span		

A-1.16 Beam Overhanging the Supports — Two Concentrated Loads Symmetrically Placed on Overhangs



$$A = B = P$$

$$M_{Max} = Pc$$

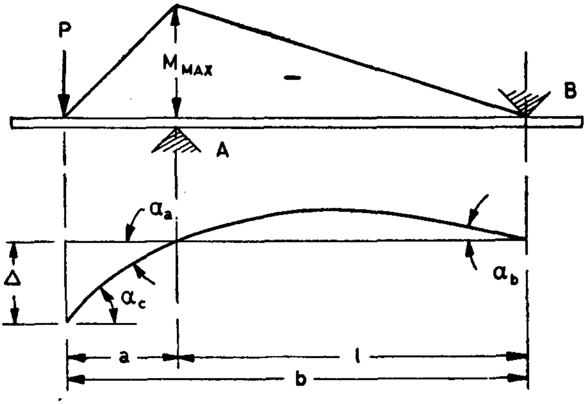
$$\Delta = \frac{Pcl^2}{8\,EI}$$

$$\Delta_1 = \frac{Pc^2\,(\,2c + 3l\,)}{6\,EI}$$

$$\alpha_a = \frac{Pcl}{2\,EI}$$

$$\alpha_c = \frac{Pc\,(\,l + c\,)}{2\,EI}$$

A-1.17 Beam Overhanging One Support — Concentrated Load at End of Overhanging



$$A = P\frac{b}{l},\ B = -P\frac{a}{l}$$

$$M_{Max} = -Pa$$

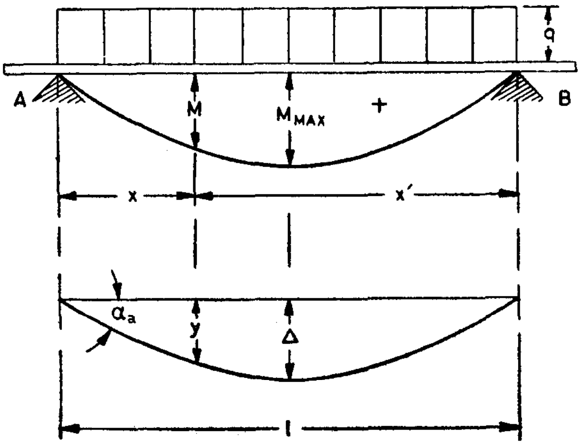
$$\Delta = \frac{Pa^2\,b}{3\,EI}$$

$$\alpha_a = \frac{Pal}{3\,EI}$$

$$\alpha_b = \frac{Pal}{6\,EI}$$

$$\alpha_c = \frac{Pa\,(2l + 3a)}{6\,EI}$$

A-1.18 Simple Beam — Uniformly Distributed Load



Total load $P = pl$,

$$A = B = \frac{P}{2}$$

$$M = \frac{P \times (l - X)}{2l}$$

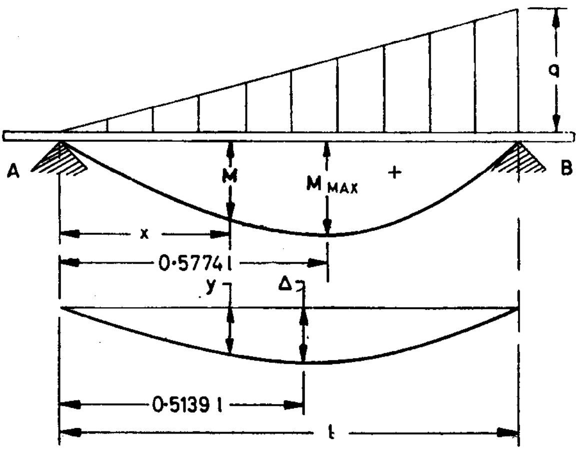
$$M_{Max} = \frac{Pl}{8} = 0{\cdot}125\,Pl$$

$$y = \frac{Pl^3}{EI}\left(\frac{X}{24l} - \frac{X^3}{12l^3} + \frac{X^4}{24l^4}\right)$$

$$\Delta = \frac{5Pl^3}{384\,EI}$$

$$\alpha_a = \frac{Pl^2}{24\,EI} = \frac{16\,\Delta}{5l}$$

A-1.19 Simple Beam — Load Increasing Uniformly to One End



Total load $P = \frac{ql}{2}$

$A = \frac{1}{3}P,\ B = \frac{2}{3}P$

$M = \frac{P\,X\,(l^2\,X^2)}{3l^2}$

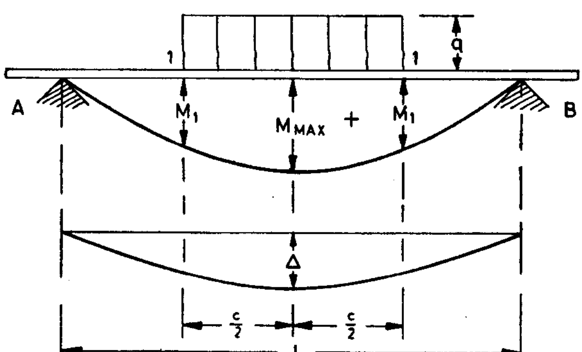
$M_{\text{Max}} = \frac{2Pl}{9\sqrt{3}} \sim 0{\cdot}128\,Pl$

at $x = \frac{l}{\sqrt{3}} = 0{\cdot}577\,4l$

$y = \frac{Pl^3}{180\,EI}\left(\frac{7X}{l} - \frac{10X^3}{l^3} + \frac{3X^5}{l^5}\right)$

$\Delta\ \text{Max} = \frac{Pl^3}{EI}\,0{\cdot}013\,04$ at $X = 0{\cdot}519\,3l$

A-1.20 Simple Beam — Partially Distributed Uniform Load Symmetrical to Centre



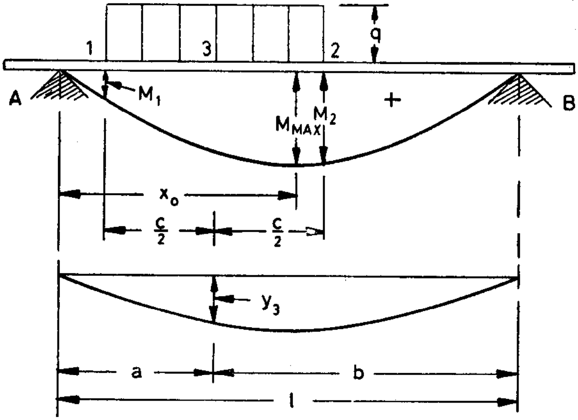
$A = B = \frac{qc}{2}$

$M_1 = \frac{qcl}{4}(1 - \gamma)$, with $\gamma = \frac{c}{l}$

$M_{\text{Max}} = \frac{qcl}{8}(2 - \gamma)$

$\Delta = \frac{qc}{96\,EI}(2l^3 - lc^2 + 0{\cdot}25c^3)$

A-1.21 Simple Beam — Uniform Load Partially Distributed



$\alpha = \frac{a}{l}$, $\gamma = \frac{c}{l}$

$A = qb\gamma$, $B = qa\gamma$

$M_1 = \frac{qbc}{2} (2\alpha - \gamma)$

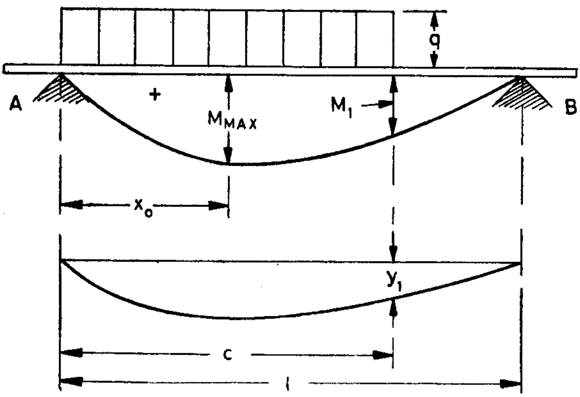
$M_2 = qac (1 - \alpha - \gamma)$

$M_{Max} = \frac{qab}{2} \gamma (2 - \gamma)$

at $x_o = a + c (\frac{1}{2} - \alpha)$

$y_3 = \frac{q\gamma}{384 EI} (lc^3 - 16 abc^2 + 128 a^2 b^2)$

A-1.22 Simple Beam — Uniform Load Partially Distributed at One End



$A = \frac{qc}{2} (2 - \gamma)$, with $\gamma = \frac{c}{l}$

$B = \frac{qc^2}{2l}$

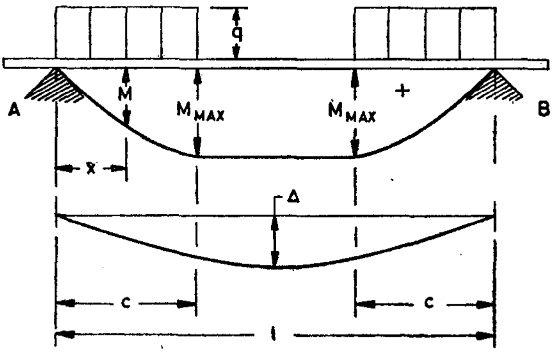
$M_1 = \frac{qc^2}{2} (1 - \gamma)$

$M_{Max} = \frac{qc^2}{2} (1 - \gamma - 0{\cdot}25 \gamma^2)$

at $x_o = c (1 - 0{\cdot}5 \gamma)$

$y_1 = \frac{qc^3 (1 - \gamma)}{6 EI} (l - 0{\cdot}75 c)$

A-1.23 Simple Beam — Uniform Load Partially but Symmetrically Distributed from Both Ends



$$A = B = qc$$

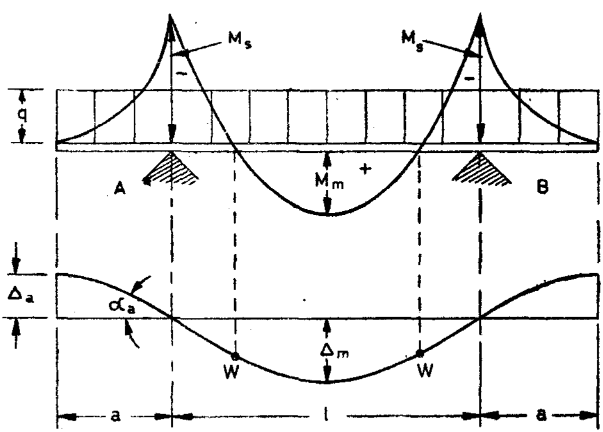
$$M = qx\,(c - 0{\cdot}5\,x) \quad \text{for } x \leqslant c$$

$$M_{\text{Max}} = \frac{qc^2}{2}$$

$$\Delta = \frac{ql^2c^2}{48EI}\,(3 - 2\,\gamma^2)$$

$$\gamma = \frac{c}{l}$$

A-1.24 Beam with Overhangs — Uniformly Distributed Load



$$A = B = q\left(\frac{l}{2} + a\right)$$

$$M_s = -\frac{qa^2}{2}$$

$$M_m = \frac{q\,(l^2 - 4a^2)}{8}$$

$$M_m = 0 \text{ for } a = \frac{l}{2}$$

$$M = -M_s = \frac{ql^2}{16} \text{ for}$$

$$a = \frac{l}{\sqrt{8}} = 0{\cdot}353\,6\,l$$

$$\Delta_m = \frac{ql^2\,(5l^2 - 24a^2)}{384\,EI}$$

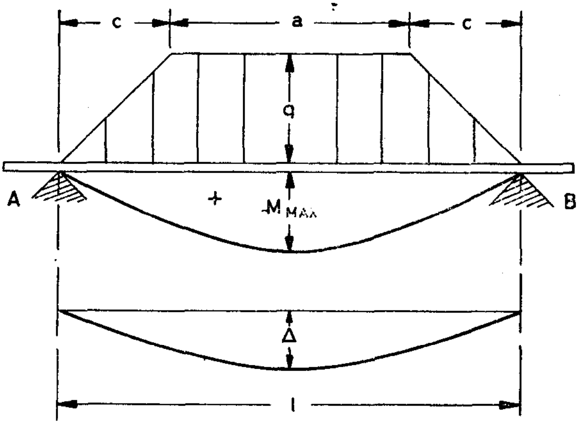
$$\Delta_m = 0 \text{ for } a = \sqrt{\frac{5}{24}}\,l = 0{\cdot}456\,4\,l$$

$$\Delta_a = \frac{qa\,(l^3 - 6a^2l - 3a^3)}{24\,EI}$$

$$\alpha_a = \frac{ql\,(l^2 - 6a^2)}{24\,EI}$$

$$\alpha_a = 0 \text{ for } a = \frac{l}{\sqrt{6}} = 0{\cdot}408\,2\,l$$

A-1.25 Simple Beam — Trapezoidal Load



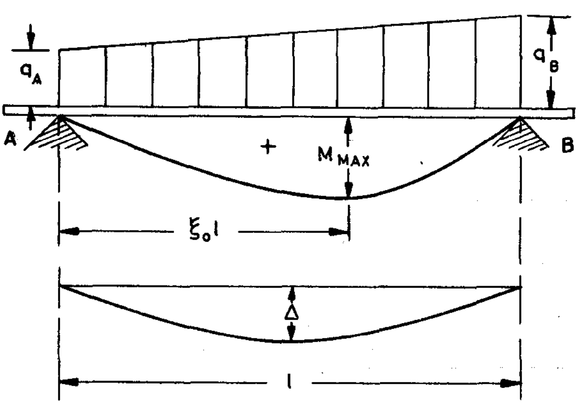
Total load $P = q\,(l - c)$

$$A = B = \frac{q\,(l-c)}{2}$$

$$M_{Max} = \frac{ql^2}{8} - \frac{qc^2}{6} \text{ at } x = \frac{l}{2}$$

$$\Delta = \frac{ql^4}{1\,920\,EI}\left[25 - 40\left(\frac{c}{l}\right)^2 + 16\left(\frac{c}{l}\right)^4\right]$$

A-1.26 Simple Beam — Trapezoidal Load



Total load

$$P = \frac{q_A + q_B}{2} l$$

$$A = \frac{2\,q_A + q_B}{6} l,$$

$$B = \frac{q_A + 2_B}{6} l$$

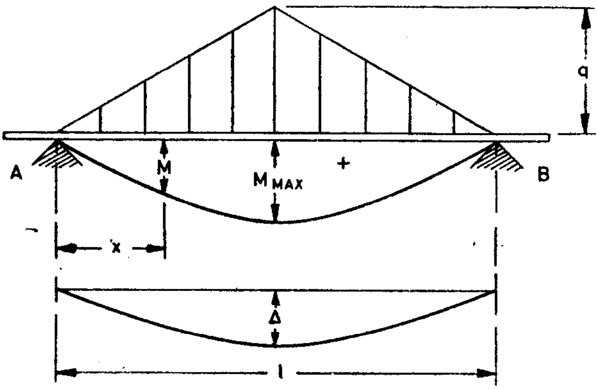
For $q_B > q_A$, the table directly gives the values for

$$M_{Max} = \frac{Pl}{n} \text{ at the point } x = \xi_0\, l$$

$$\Delta_{Max} \approx \frac{Pl^3}{EI} 0{\cdot}013\,03$$

$\frac{q_A}{q_B} =$	0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
$\xi_0 =$	0·577	0·566	0·555	0·545	0·536	0·528	0·520	0·514	0·508	0·504	0·500
$n =$	7·79	7·82	7·85	7·89	7·91	7·93	7·94	7·96	7·98	7·99	8·00

A-1.27 Simple Beam — Load Increasing Uniformly to Centre



Total load $P = \frac{ql}{2}$

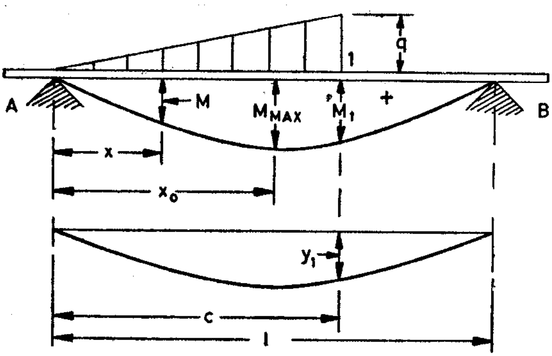
$A = B = \frac{P}{2}$

$M = Px \left(\frac{1}{2} - \frac{2x^2}{3l^2} \right)$

$M_{\text{Max}} = \frac{Pl}{6}$

$\Delta = \frac{Pl^3}{60EI}$

A-1.28 Simple Beam — Load Increasing from One Towards Any Point



$A = \frac{qc}{6}(3 - 2\gamma)$,

$\gamma = \frac{c}{l}$

$B = \frac{qc^2}{3l}$

$M_{\text{Max}} = \frac{qc^2}{3} \sqrt{\left(1 - \frac{2}{3}\gamma\right)^3}$

at $x_0 = c\sqrt{1 - \frac{2}{3}\gamma}$

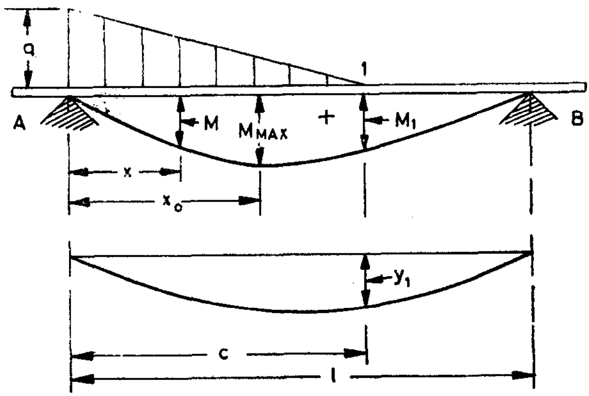
$M_1 = \frac{qc^2}{3}(1 - \gamma)$

$M = \frac{qcx}{6} \left(3 - 2\gamma - \frac{x^2}{c^2}\right)$

for $x \leqslant c$

$v_1 = \frac{qc^3}{45EI} (1 - \gamma)(5l - 4c)$

A-1.29 Simple Beam — Load Increasing from Any Point Towards an End



$A = \frac{qc}{6}(3 - \gamma)$,

$\gamma = \frac{c}{l}$

$B = \frac{qc^2}{6l}$

$M_{Max} = \frac{qc^2}{6l}\left(l - c + \frac{2}{3}c\sqrt{\frac{\gamma}{3}}\right)$

at $x_0 = c - c\sqrt{\frac{\gamma}{3}}$

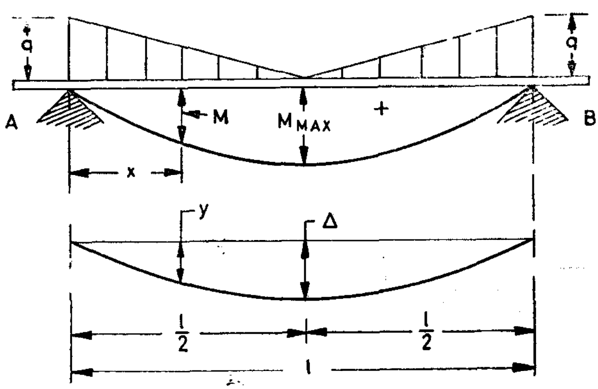
$M_1 = \frac{qc^2}{6}(1 - \gamma)$

$M = \frac{ql^2}{6}\frac{\xi}{\gamma}[\gamma^2(3 - \gamma) - \xi(3\gamma - \xi)]$

for $x \leqslant c$, and $\xi = \frac{x}{l}$

$y_1 = \frac{qc^3}{360\,EI}(1 - \gamma)(20l - 13c)$

A-1.30 Simple Beam — Load Increasing Uniformly from Centres to Ends



$A = B\frac{ql}{4}$, $\xi = \frac{x}{l}$

$M = \frac{qlx}{12}(3 - 6\xi + 4\xi^2)$

for $x \leqslant \frac{l}{2}$

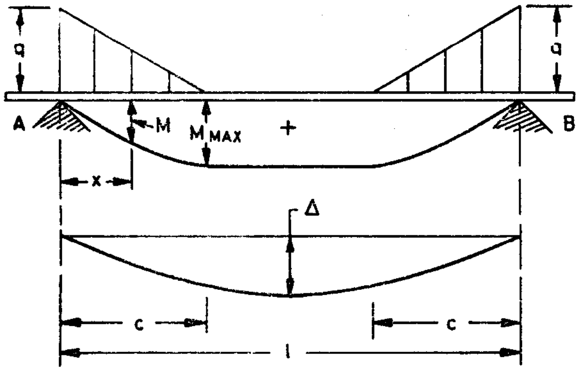
$M_{Max} = \frac{ql^2}{24}$

$\Delta = \frac{3ql^4}{640\,EI}$

$y = \frac{ql^3x}{120\,EI}\left(\frac{15}{8} - 5\xi^2 + 5\xi^3 - 2\xi^4\right)$

for $x \leqslant \frac{l}{2}$

A-1.31 Simple Beam — Load Increasing Uniformly to Ends



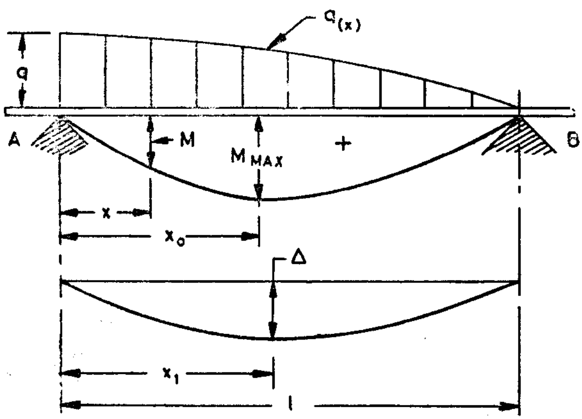
$$A = B = \frac{qc}{2}$$

$$M = \frac{q}{6} \times \left(3c - 3x + \frac{x^2}{c} \right) \text{ for } x < c$$

$$M_{\text{Max}} = \frac{qc^2}{6}$$

$$\Delta = \frac{qc^2}{240}(5l^2 - 2c^2)$$

A-1.32 Simple Beam — Parabolic Loading (Parabola of Second Order)



$$q(x) = q(1 - \xi^2),$$

$$\xi = \frac{x}{l}$$

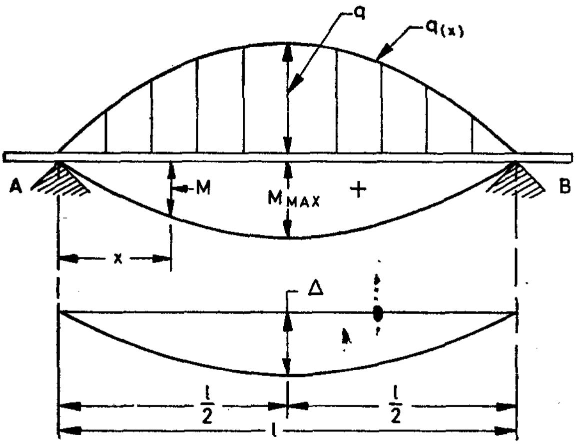
$$A = \frac{5}{12}\, ql, \qquad B = \frac{ql}{4}$$

$$M_{\text{Max}} = \frac{ql^2}{8{\cdot}97}$$
at $x_o = 0{\cdot}446\, l$

$$M(x) = \frac{ql^2}{12}(5\xi - 6\xi^2 + \xi^4)$$

$$\Delta = \frac{11\, ql^4}{120\, EI}$$
at $x_1 = 0{\cdot}486\, l$

A-1.33 Simple Beam — Parabolic Load (Parabola of Second Order)



$q(x) = 4q(\xi - \xi^2)$, $\xi = \frac{x}{l}$

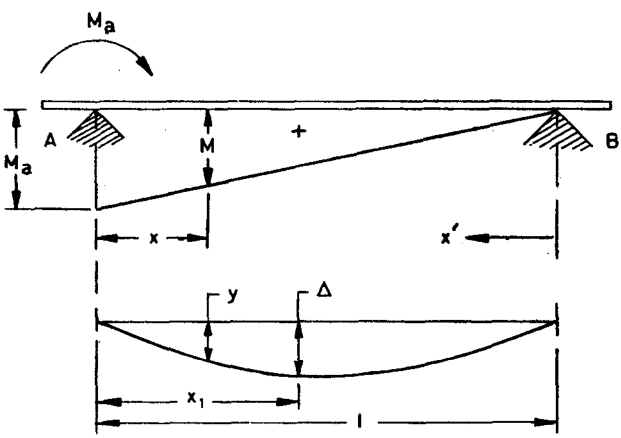
$A = B = \frac{ql}{3}$

$M_{Max} = \frac{5}{48} ql^2$

$M(x) = q\frac{l^2}{3}(\xi - 2\xi^3 + \xi^4)$

$\Delta = \frac{61\, ql^4}{5\,760\, EI}$

A-1.34 Simple Load — Moment M at One End



$\xi = \frac{x}{l}$, $\xi' = \frac{x'}{l}$

$A = -B = -\frac{M_a}{l}$

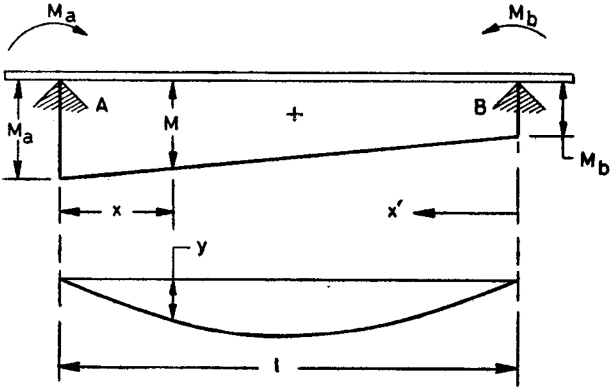
$M(x) = M_a\,\xi'$

$\Delta = \frac{M_a\, l^2}{9\sqrt{3}\, EI}$

at $x_1 = 0{\cdot}423\, l$

$y(x) = \frac{M_a\, l^2}{6\,EI}(\xi' - \xi'^3)$

A-1.35 Simple Beam — Unequal Moments at Ends



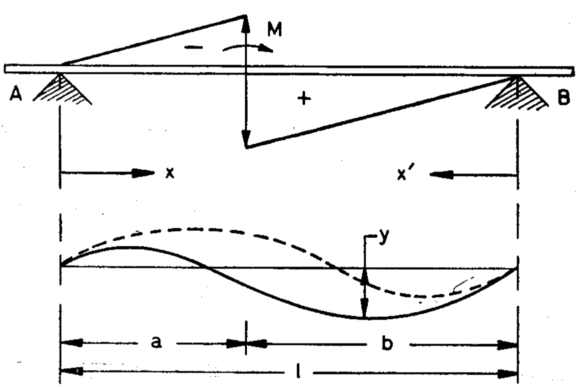
$$A = -B = \frac{M_b - M_a}{l}$$

$$M(x) = M_a\,\xi' + M_b\,\xi$$

$$y(x) = \frac{l^2}{6EI}\left[M_a(\xi' - \xi'^3) + M_b(\xi - \xi^3)\right]$$

$$\xi = \frac{x}{l} \qquad \xi' = \frac{x'}{l}$$

A-1.36 Simple Beam — Moment *M* at Any Point



$$\alpha = \frac{a}{l},\ \beta = \frac{b}{l},$$

$$\xi = \frac{x}{l},\quad \xi' = \frac{x'}{l}$$

$$A = -B = -\frac{M}{l}$$

$M(x) = -M\xi$ for $x \leqslant a$

$M(x') = M\xi'$ for $x \geqslant a$

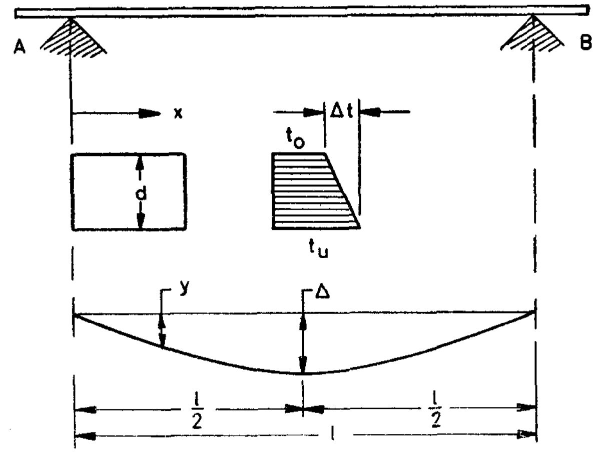
$$y(x) = \frac{Mlx}{6EI}(\xi^2 - \alpha^2 - 2\alpha^2\beta + 2\beta^3)$$

for $x \leqslant a$

$$y(x) = -\frac{Mlx'}{6EI}(\xi'^2 + 2\alpha^3 - 2\alpha\beta^2 - \beta^2)$$

for $x \geqslant a$

A-1.36 Simple Beam — Subjected to Unequal Temperature



Unequal temperature Δ_t

$A = B = M = 0$

$$y(x) = \frac{l^2}{2}\,\alpha_t\,\frac{\Delta t}{d}\,(\xi - \xi^2)$$

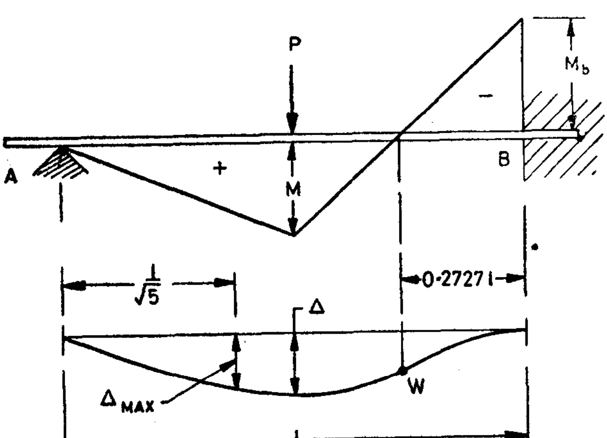
$$\Delta = \frac{l^2}{8}\,\alpha_t\,\frac{\Delta t}{d}$$

$$\xi = \frac{x}{l}$$

A-2. STATICALLY UNDETERMINATE CASES

A-2.1 Beams Fixed at One End and Free at the Other End

A-2.1.1 *Concentrated Load at the Centre*



P in the middle of the beam

$A = \frac{5}{16}P$, $B = \frac{11}{16}P$

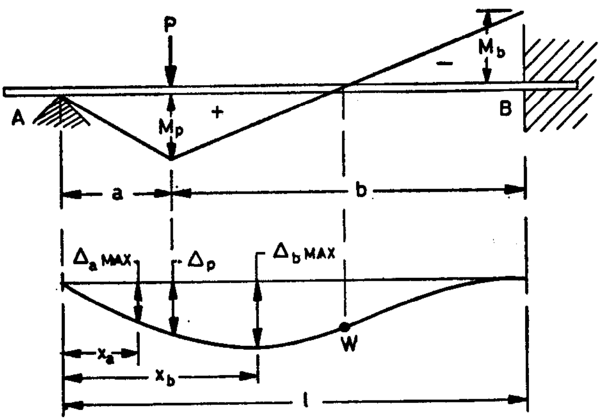
$M = \frac{5}{32}Pl$ (in the middle of the beam)

$M_b = M_{Max} = -\frac{3}{16}Pl$ (fixed end moment)

$\Delta = \frac{7\,Pl^3}{768\,EI}$ (under the load *P*)

$\Delta_{Max} = \frac{Pl^3}{48\sqrt{5}\,EI}$ at $x = 0{\cdot}447\,l$

A-2.1.2 *Concentrated at Any Point*



$$A = P\left(1 + \frac{a^3}{2\,l^3} - \frac{3\,a}{2\,l}\right)$$

$$B = P\left(-\frac{a^3}{2\,l^3} + \frac{3\,a}{2\,l}\right) = P - A$$

$$M_P = Pa\left(1 + \frac{a^3}{2\,l^3} - \frac{3\,a}{2\,l}\right)$$ under the load P

$$M_B = -\,Pa\,\frac{(l^2 - a^2)}{2\,l^2}$$ fixed end moment

$$\Delta_P = P\,\frac{a^2\,b^3}{12\,l^3\,EI}\,(4\,a + 3\,l)$$ under the load P

$M_{PMax} = 0{\cdot}174\ Pl$ for $a = 0{\cdot}366\ l$

$M_{BMax} = -\,0{\cdot}192\,5\ Pl$ for $a = 0{\cdot}577\ l$

Δ_{aMax} at the distance $A - P$, if $a \leqslant 0{\cdot}414\ l$ at

$$x_a = l\sqrt{\frac{a}{a + 2\,l}},$$

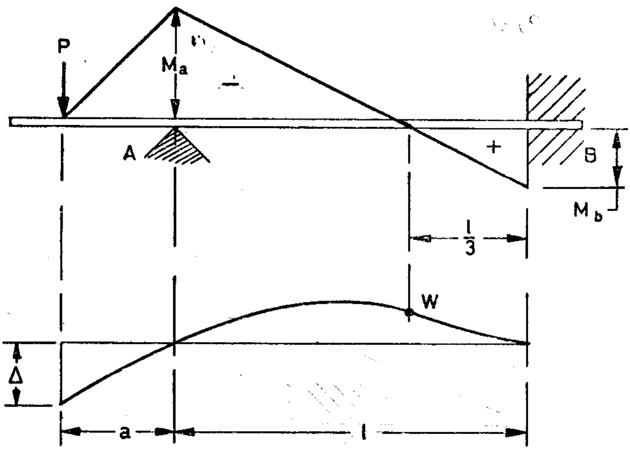
$$\Delta_{aMax} = \frac{Pab^2}{6\,EI}\sqrt{\frac{a}{a + 2l}}$$

Δ_{bMax} at the distance $P - B$, if $a \geqslant 0{\cdot}414\ l$ at

$$x_b = \frac{l^3 + a^2\,l}{3\,l^2\,a^2},$$

$$\Delta_{bMax} = \frac{Pa}{3\,EI}\,\frac{(l^2 - a^2)^3}{(3\,l^2 - a^2)^2}$$

A-2.1.3 *Concentrated Load on the Overhang*



$$A = P\left(\frac{3\,a}{2\,l} + 1\right),$$

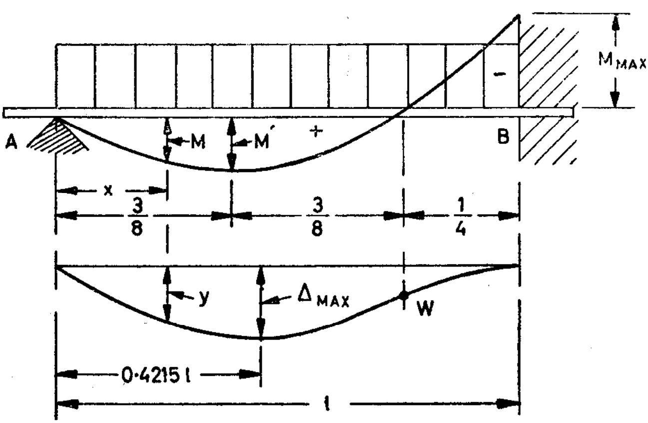
$$B = -\,P\,\frac{3\,a}{2\,l}$$

$$M_a = -\,Pa$$

$$M_b = \frac{Pa}{2}$$

$$\triangle = \frac{Pa^2\,(4\,a + 3\,l)}{12\,EI}$$

A-2.1.4 *Uniformly Distributed Load*



Total load $P = ql$

$$A = \frac{3}{8}\,P,\ B = \frac{5}{8}\,P$$

$$M = \frac{Px}{2}\left(\frac{3}{4} - \frac{x}{l}\right)$$

$$M_{\text{Max}} = -\,\frac{Pl}{8}$$ (fixed end moment)

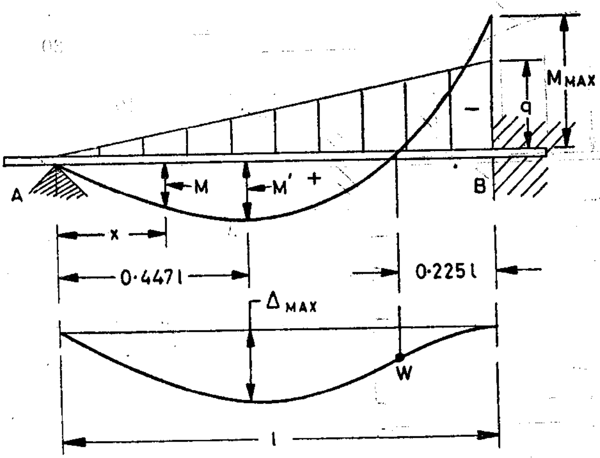
$$M' = \frac{9}{128}\,Pl$$ for

$$x = \frac{3}{8}\,l$$ (*Max* field moment)

$$y = \frac{Px(l^3 - 3\,lx^2 + 2\,x^3)}{48\,EI\,l}$$

$$\triangle_{\text{Max}} = \frac{Pl^3}{185\,EI}$$ for $x = 0{\cdot}421\,5\,l$

A-2.1.5 *Load Increasing Uniformly to Fixed End*



Total load $P = \frac{ql}{2}$

$A = \frac{P}{5}$, $B = \frac{4}{5} P$

$M = Px \left(\frac{1}{5} - \frac{x^2}{3l^2}\right)$

$M_b = M_{Max} = -\frac{Pl}{7 \cdot 5}$

(fixed end moment)

$M' = 0{\cdot}0596\ Pl$ for x

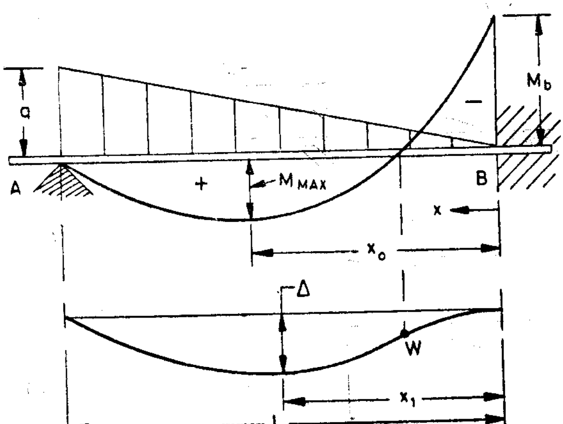
$= 0{\cdot}447\ l = \frac{l}{\sqrt{5}}$

(*Max* field moment)

$\Delta_{Max} = \frac{Pl^3}{210\ EI}$

for $x = 0{\cdot}447\ l$

A-2.1.6 *Load Increasing Uniformly to Free End*



$A = \frac{11}{40} ql$, $B = \frac{9}{40} ql$

$M_b = -\frac{7}{120} ql^2$

$M_{Max} = \frac{ql^2}{23 \cdot 6}$ at x_0

$= 0{\cdot}671\ l$

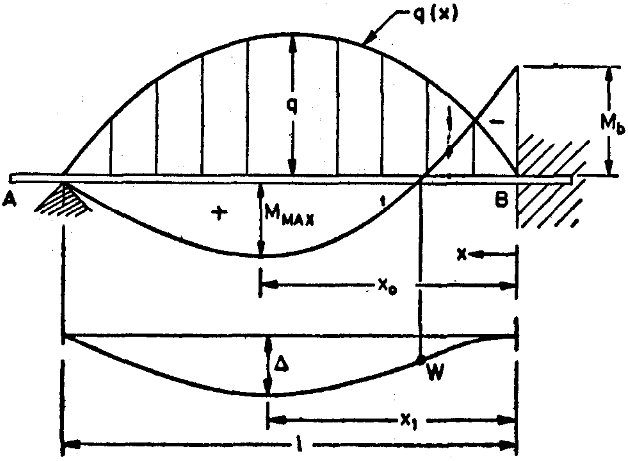
$M(x) = -\frac{ql^2}{120}(7 - 27\xi + 20\xi^3)$

$\Delta = \frac{ql^4}{328 \cdot 1\ EI}$ at x_1

$= 0{\cdot}598\ l$

$\xi = \frac{x}{l}$

A-2.1.7 *Parabolic Load Parabola of Second Order*



$A = \dfrac{7}{30}\, ql,\ B = \dfrac{13}{30}\, ql$

$M_b = -\dfrac{ql^2}{10}$

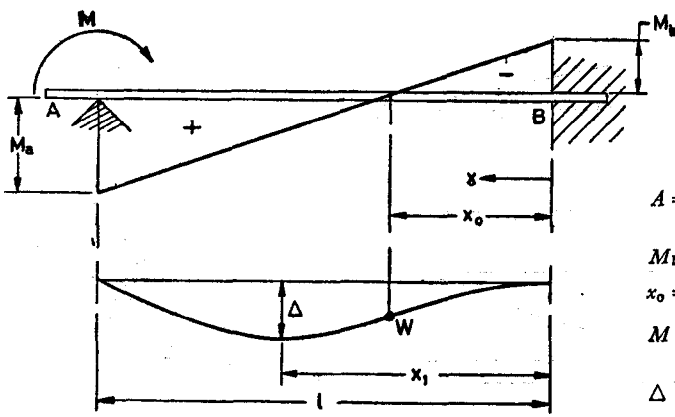
$M_{Max} = \dfrac{ql^2}{16 \cdot 89}$ at x_o
$= 0 \cdot 601\ l$

$M\,(x) = \dfrac{ql^2}{30}\ (-3 + 13\,\xi - 20\,\xi^3 + 10\,\xi^4)$

$\Delta = \dfrac{ql^4}{222 \cdot 5\, EI}$ at x_1
$= 0 \cdot 573\ l$

$\xi = \dfrac{x}{l}$

A-2.1.8 *Moment M at Free End*



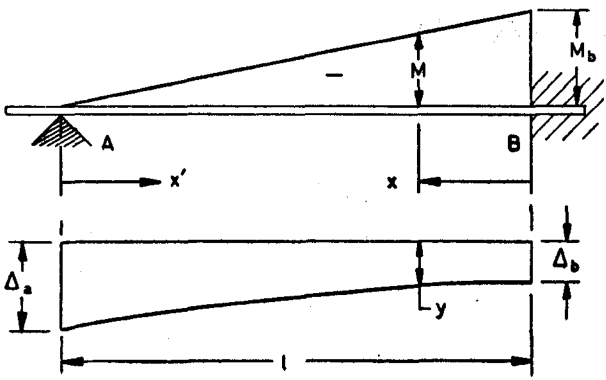
$A = -B = -\dfrac{3\ M}{2l}$

$M_b = -\dfrac{M}{2}$, $M = 0$ at $x_o = \frac{1}{3}\ l$

$M\,(x) = \dfrac{M}{2}\ (3\,\xi - 1)$

$\Delta = \dfrac{Ml^2}{27\ EI}$ at $x_1 = \frac{2}{3}\ l$

A-2.1.9 *Support Displacement*



$$A = -B = -\frac{3\,EJ}{l^3}\,(\Delta_a - \Delta_b)$$

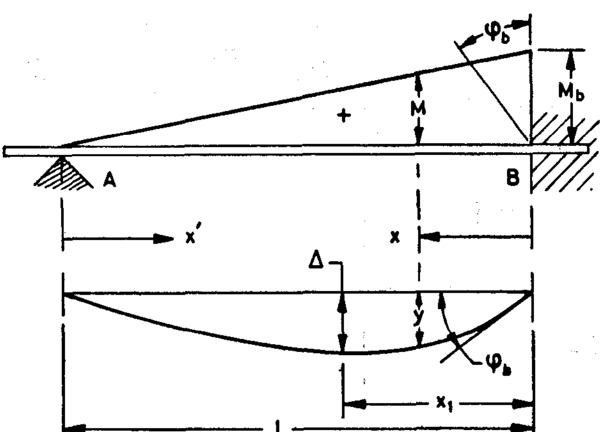
$$M_b = -\frac{3\,EI}{l^2}\,(\Delta_a - \Delta_b)$$

$$M\,(x) = M_b\,(1 - \xi)$$

$$y\,(x) = \Delta_b{}'\xi' + \Delta_a\,\xi - \tfrac{1}{2}\,(\Delta_a - \Delta_b)\,(\xi' - \xi'^3)$$

$$\xi = \frac{x}{l}, \quad \xi' = \frac{x'}{l}$$

A-2.1.10 *Support Rotation*



$$A = -B = \frac{3\,EI}{l^2}\,\varphi_b$$

$$M_b = \frac{3\,EJ}{l}\,\varphi_b$$

$$M\,(x) = M_b\,(1 - \xi)$$

$$\Delta = \frac{\varphi_b\,l}{3\sqrt{3}} \text{ at}$$

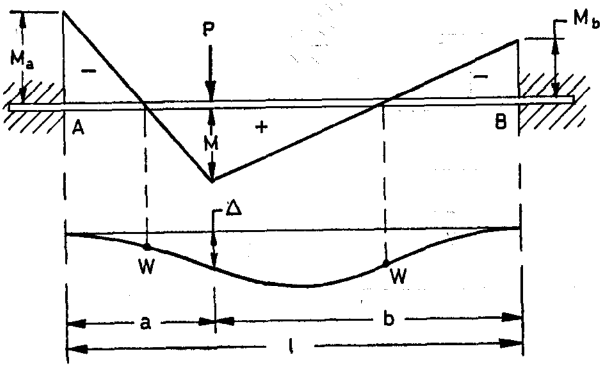
$$x_1 = 0{\cdot}423\,l$$

$$y\,(x) = \varphi_b\,\frac{l}{2}\,(\xi' - \xi^3)$$

$$\xi = \frac{x}{l}, \quad \xi' = \frac{x'}{l}$$

A-2.2 Fixed Beams at Both Ends

A-2.2.1 *Concentrated Load at Any Point*



P at distance *a* from *A*

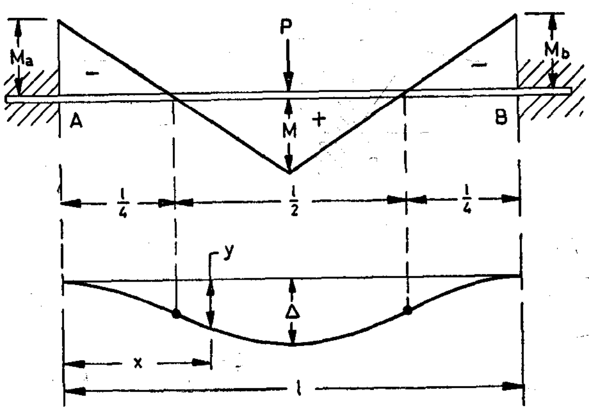
$$A = \frac{Pb\,(\,l^2 - a^2 + ab\,)}{l^3}$$

$$B = \frac{Pa\,(\,l^2 - b^2 + ab\,)}{l^3}$$

$$\left.\begin{aligned} M_a &= -\frac{Pab^2}{l^2} \\ M_b &= -\frac{Pba^2}{l^2} \end{aligned}\right\}\text{fixed end moment}$$

$$\left.\begin{aligned} M &= \frac{2\,Pa^2\,b^2}{l^3} \\ \Delta &= \frac{Pa^3\,b^3}{3\,EI\,l^3} \end{aligned}\right\}\text{field moment under } P$$

A-2.2.2 *Concentrated Load at Centre*



P in the middle of the beam

$$A = B = \frac{P}{2}$$

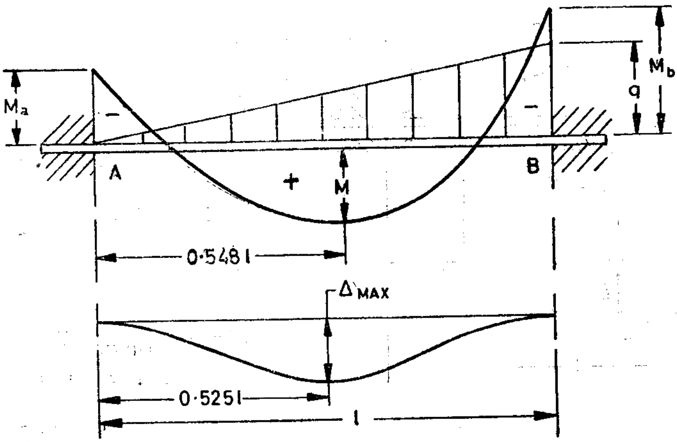
$$M = \frac{Pl}{8}\ (\text{field moment})$$

$$M_a = M_b = -\frac{Pl}{8}\ (\text{fixed end moment})$$

$$y = \frac{Px^2\,(\,3\,l - 4\,x\,)}{48\,EI}$$

$$\Delta = \frac{Pl^3}{192\,EI}$$

A-2.2.3 *Load Increasing Uniformly to End*



Total load $P = \frac{ql}{2}$

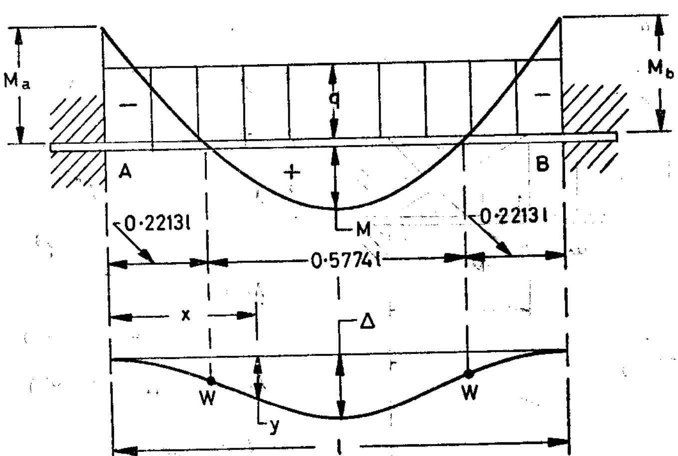
$A = \frac{3}{20} ql \quad B = \frac{7}{20} ql$

$M_a = -\frac{ql^2}{30}$, $M_b = -\frac{ql^2}{20}$ } fixed end moment

$M' = \frac{ql^2}{46 \cdot 6}$ at $x = 0 \cdot 548\, l$
(*Max* field moment)

$\Delta \mathrm{Max} = \frac{ql^4}{764\ EI}$ at $x = 0 \cdot 525\, l$

A-2.2.4 *Uniformly Distributed Load*



Total load $P = q$

$A = B = \frac{P}{2}$

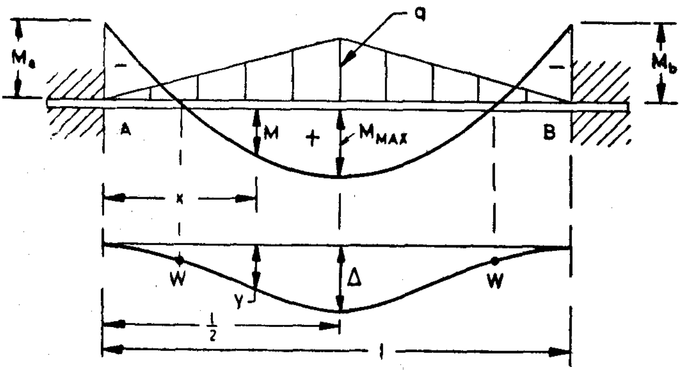
$M = \frac{Pl}{24}$ (field moment)

$M_a = M_b = M_{\mathrm{Max}} = -\frac{Pl}{12}$
(fixed end moment)

$y = \frac{Px^2 (l - x)^2}{24\ EI\ l}$

$\Delta = \frac{Pl^3}{384\ EI}$

A-2.2.5 *Load Increasing Uniformly to Centre*



$A = B = \frac{ql}{4}, \quad \xi = \frac{x}{l}$

$M_a = M_b = -\frac{5}{96}\, ql^2$

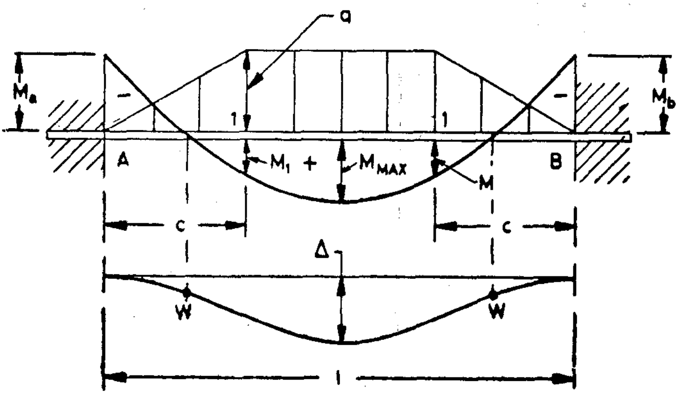
$M_{Max} = \frac{ql^2}{32}$

$M(x) = -\frac{ql^2}{96}(5 - 24\xi + 32\xi^3)$ for $x < \frac{l}{2}$

$\Delta = \frac{7\, ql^4}{3840\, EI}$

$y(x) = \frac{ql^2 x^2}{960\, EI}(25 - 40\xi + 16\xi^2)$ for $x < \frac{l}{2}$

A-2.2.6 *Trapezoidal Load*



$A = B = \frac{q}{2}(l - c)$,

$\gamma = \frac{c}{l}$

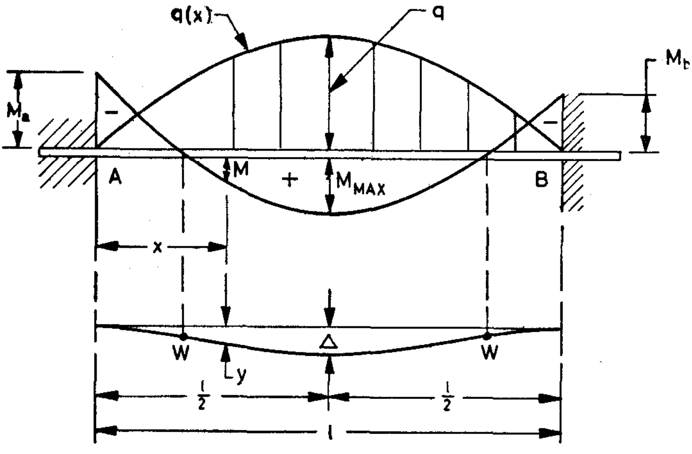
$M_a = M_b = -\frac{ql^2}{12}(1 - 2\gamma^2 + \gamma^3)$

$M_1 = -\frac{ql^2}{12}(1 - 6\gamma + 6\gamma^2 + \gamma^3)$

$M_{Max} = \frac{ql^2}{24}(1 - 2\gamma^3)$

$\Delta = \frac{ql^4}{1920\, EI}(5 - 20\gamma^2 + 16\gamma^4)$

A-2.2.7 *Parabolic Load*



$$A = B = \frac{1}{3}\, ql$$

$$M_a = M_b = -\frac{1}{15}\, ql^2$$

$$M_{Max} = \frac{3}{80}\, ql^2$$

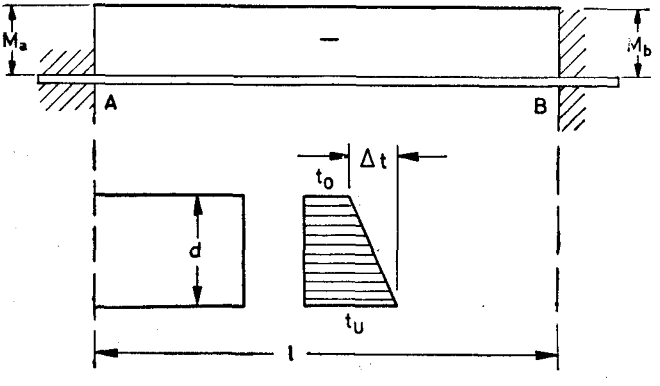
$$M(x) = -\frac{ql^2}{15}\,(1 - 5\,\xi + 10\,\xi^3 - 5\,\xi^4)$$

$$\Delta = \frac{13\, ql^4}{5\,760\; EI}$$

$$y(x) = \frac{ql^2\, x^2}{90\; EI}\,(3 - 5\,\xi + 3\,\xi^3 - \xi^4)$$

$$\xi = \frac{x}{l}$$

A-2.2.8 *Unequal Temperatures*



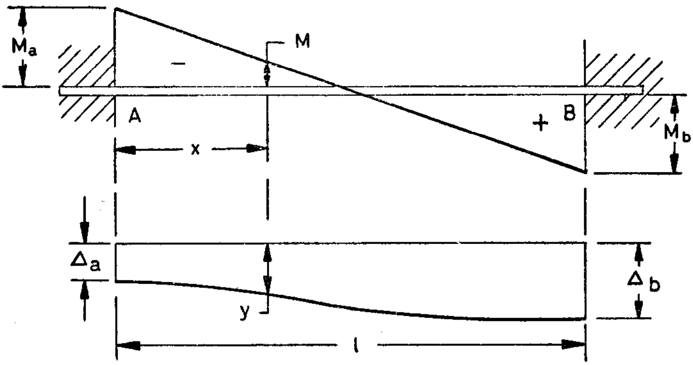
Unequal temperature Δt

$A = B = 0$

$$M_a = M_b = -EI\,\alpha_t = \frac{\Delta\, t}{d}$$

$y(x) = 0$

A-2.2.9 *Support Displacement* Δ



$$A = -B = \frac{12\,EI}{l^3}\,(\Delta_b - \Delta_a)$$

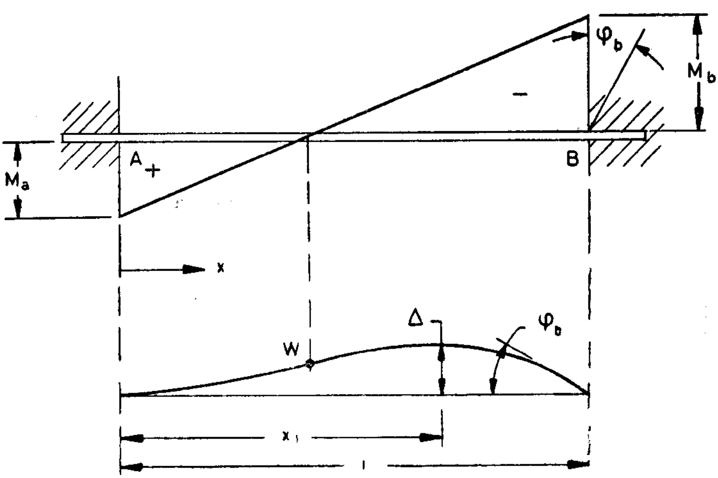
$$M_a = -M_b = -\frac{6\,EI}{l^2}\,(\Delta_b - \Delta_a)$$

$$M(x) = -\frac{6\,EI}{l^2}\,(\Delta_b - \Delta_a)\,(1 - 2\xi)$$

$$y(x) = \Delta_a + (\Delta_b - \Delta_a)\,(3\xi^2 - \xi^3)$$

$$\xi = \frac{x}{l}$$

A-2.2.10 *Support Displacement* φ



$$A = -B = \frac{6\,EI}{l^2}\,\varphi_b$$

$$M_a = 2\,\frac{EI}{l}\,\varphi_b$$

$$M_b = -4\,\frac{EI}{l}\,\varphi_b$$

$$M(x) = 2\,\frac{EI}{l}\,\varphi_b\,(1 - 3\xi)$$

$$\Delta = -\frac{4}{27}\,l\,\varphi_b, \text{ at } x_1 = \frac{2}{3}\,l$$

$$y(x) = -l\,\varphi_b\,(\xi^2 - \xi^3)$$

$$\xi = \frac{x}{l}$$

APPENDIX B

COMPOSITION OF STRUCTURAL WELDING SECTIONAL COMMITTEE, SMDC 15

and

AD HOC PANEL FOR HANDBOOK FOR WELDING ENGINEERS, SMDC 15 : P15

Structural Welding Sectional Committee, SMDC 15

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SHRI N. C. BAGCHI	National Test House, Calcutta
SHRI S. BALASUBRAHMANYAM	Braithwaite Burn & Jessop Construction Co Ltd, Calcutta
SHRI D. P. CHATTERJEE	Directorate General of Supplies & Disposals (Inspection Wing), New Delhi
EXECUTIVE ENGINEER, MACHINERY AND STORES DIVISION, ALIYARNAGAR	Public Works Department, Government of Tamil Nadu
GENERAL SUPERINTENDENT, PUBLIC WORKS WORKSHOPS AND STORE (*Alternate*)	
SHRI C. P. GHOSH	Engineer-in-Chief's Branch, Army Headquarters
DR J. JAIN	Tata Engineering & Locomotive Co Ltd, Jamshedpur
SHRI H. D. WAGHULADE (*Alternate*)	
SHRI J. C. KAPUR	Indian Engineering Association, Calcutta
SHRI S. K. MAJUMDAR	Indian Institute of Welding, Calcutta
SHRI P. K. MALLICK	Burn & Co Ltd, Howrah
SHRI SAMBHU NATH MOOKERJI	ACC-Vickers-Babcock Ltd, Durgapur
SHRI J. A. MULIYIL	Indian Oxygen Ltd, Calcutta
SHRI V. R. SUBRAMANIAN (*Alternate*)	
SHRI S. V. NADKARNI	Advani-Oerlikon Private Ltd, Bombay
SHRI R. K. THARIANI (*Alternate*)	
PROF R. NARAYANAN	Institution of Engineers (India), Calcutta
SHRI S. K. PATHAK	Braithwaite & Co (India) Ltd, Calcutta
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DEPUTY DIRECTOR STANDARDS (B & S)-II RDSO, LUCKNOW (*Alternate*)	

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SHRI S. SWAROOP (*Alternate*)	
TECHNICAL ADVISER (BOILERS)	Central Boilers Board, New Delhi
SHRI K. VEERARAGHAVACHARY	Bharat Heavy Electricals Ltd, Tiruchirapalli
SHRI T. N. VELU	Hindustan Shipyard Ltd, Visakhapatnam
SHRI R. K. SRIVASTAVA, Deputy Director (Struc & Met)	Director General, ISI (*Ex-officio Member*)

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SHRI M. S. NAGARAJ
Assistant Director (Struc & Met), ISI

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SHRI K. VEERARAGHVACHARY	Bharat Heavy Electricals Ltd, Tiruchirapalli
SHRI B. S. KRISHNAMACHAR, Deputy Director General	Director General, ISI (*Ex-officio Member*)

APPENDIX C

(*Clause* 1.2)

INDIAN STANDARDS RELEVANT TO THIS HANDBOOK

C-1. MATERIALS AND SECTIONS

IS : 226-1969	Specification for structural steel (standard quality) (*fourth revision*)
IS : 808-1964	Specification for rolled steel beam, channel and angle sections (*revised*)
IS : 811-1965	Specification for cold formed light gauge structural steel sections (*revised*)
IS : 961-1962	Specification for structural steel (high tensile) (*revised*)
IS : 1161-1968	Specification for steel tubes for structural purposes (*second revision*)
IS : 1173-1967	Specification for hot rolled and slit steel tee bars (*first revision*)
IS : 1252-1958	Specification for rolled steel sections bulb angles
IS : 1570-1961	Schedule for wrought steels for general engineering purposes
IS : 1730-1961	Dimensions for steel plate, sheet and strip for structural and general engineering purposes
IS : 1731-1971	Dimensions for steel flats for structural and general engineering purposes (*first revision*)
IS : 1732-1971	Dimensions for round and square steel bars for structural and general engineering purposes (*first revision*)
IS : 1863-1961	Dimensions for rolled steel bulb plates
IS : 1864-1963	Dimensions for angle sections with legs of unequal width and thickness
IS : 1870-1965	Comparison of Indian and overseas standards for wrought steels for general engineering purposes
IS : 1977-1969	Specification for structural steel (ordinary quality) (*first revision*)
IS : 2002-1962	Specification for steel plates for boilers
IS : 2062-1969	Specification for structural steel (fusion welding quality) (*first revision*)
IS : 2314-1963	Specification for steel piling sections
IS : 3443-1966	Specification for crane rail sections

IS : 3954-1966	Specification for hot rolled steel channel sections for general engineering purposes
IS : 3964-1967	Specification for light rails
SP : 6 (1) - 1964	ISI Handbook for structural engineers : 1 Structural steel sections (*revised*)

C-2. WELDING

IS : 812-1957	Glossary of terms relating to welding and cutting of metals
IS : 813-1961	Scheme of symbols for welding (*amended*)
IS : 814-1970	Specification for covered electrodes for metal arc welding of structural steel (*third revision*)
IS : 815-1966	Classification and coding for covered electrodes for metal arc welding of mild steel and low alloy high-tensile steel (*revised*)
IS : 816-1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
IS : 817-1966	Code of practice for training and testing of metal arc welders (*revised*)
IS : 818-1968	Code of practice for safety and health requirements in electric and gas welding and cutting operations (*first revision*)
IS : 819-1957	Code of practice for resistance spot welding for light assemblies in mild steel
IS : 823-1964	Code of procedure for manual metal arc welding of mild steel
IS : 1024-1968	Code of practice for use of welding in bridges and structures subject to dynamic loading
IS : 1179-1967	Specification for equipment for eye and face protection during welding (*first revision*)
IS : 1181-1967	Qualifying tests for metal arc welders (engaged in welding structures other than pipes) (*first revision*)
IS : 1261-1959	Code of practice for seam welding in mild steel
IS : 1278-1967	Specification for filler rods and wires for gas welding (*first revision*)
IS : 1323-1966	Code of practice for oxy-acetylene welding for structural work in mild steel (*revised*)
IS : 1393-1961	Code of practice for training and testing of oxy-acetylene welders
IS : 1395-1964	Specification for molybdenum and chromium molybdenum low alloy steel electrodes for metal-arc welding (*revised*)

IS : 1442-1964	Specification for covered electrodes for the metal arc welding of high tensile structural steel (*revised*)
IS : 2680-1964	Specification for filler rods and wires for inert gas tungsten arc welding
IS : 2751-1966	Code of practice for welding of mild steel bars used for reinforced concrete construction
IS : 2811-1964	Recommendations for manual tungsten inert-gas arc welding of stainless steel
IS : 2879-1967	Specification for mild steel for metal arc welding electrode core wire (*first revision*)
IS : 3016-1965	Code of practice for fire precautions in welding and cutting operations
IS : 3600-1966	Code of procedure for testing of fusion welded joints and weld metal in steel
IS : 3613-1966	Acceptance test for wire flux combinations for submerged arc welding
IS : 4353-1967	Recommendations for submerged arc welding of mild steel and low alloy steels
IS : 4943-1968	Assessment of butt and fillet fusion welds in steel sheet, plate and pipe
IS : 4944-1968	Code of procedure for welding at low ambient temperatures
IS : 4972-1968	Specification for resistance spot welding electrodes
IS : 5206-1969	Specification for corrosion-resisting chromium and chromium-nickel steel covered electrodes for manual metal arc welding

ISI Handbook of manual metal-arc welding for welders

C-3. DESIGN, FABRICATION

IS : 800-1962	Code of practice for use of structural steel in general building construction (*revised*)
IS : 801-1958	Code of practice for use of cold formed light gauge steel structural members in general building construction
IS : 803-1962	Code of practice for design, fabrication and erection of vertical mild steel cylindrical welded oil storage tanks
IS : 805-1968	Code of practice for use of steel in gravity water tanks
IS : 806-1968	Code of practice for use of steel tubes in general building construction (*first revision*)

IS : 807-1963	Code of practice for design, manufacture, erection and testing (structural portion) of cranes and hoists
IS : 875-1964	Code of practice for structural safety of buildings: Loading standards (*revised*)
IS : 2713-1964	Specification for tubular steel poles for overhead power lines
IS : 3177-1965	Code of practice for design of overhead travelling cranes and gantry cranes other than steel work cranes
IS : 4000-1967	Code of practice for assembly of structural joints using high tensile friction grip fasteners
IS : 4014 (Part I)-1967	Code of practice for steel tubular scaffolding: Part I Definitions and materials
IS : 4014 (Part II)-1967	Code of practice for steel tubular scaffolding: Part II Safety regulations for scaffolding
IS : 4137-1967	Code of practice for heavy duty electric overhead travelling cranes including special service machines for use in steel works
IS : 4573-1968	Code of practice for design of mobile cranes (all types)
IS : 4594-1968	Code of practice for design of portal and semi-portal wharf cranes (electrical)
SP : 6(2)-1962	Handbook for structural engineers : 2 Steel beams and plate girders
SP : 6(3)-1962	Handbook for structural engineers : 3 Steel columns and struts

C-4. MISCELLANEOUS

IS : 2812-1964	Recommendations for manual tungsten inert-gas arc welding of aluminium and aluminium alloys
IS : 2927-1964	Specification for brazing alloys
IS : 3023-1965	Recommended practice for building-up by metal spraying
IS : 3039-1965	Specification for structural steel (shipbuilding quality)
IS : 3525-1966	Code of practice for use of metal arc welding for hull construction of merchant ships in mild steel
IS : 3908-1966	Specification for aluminium equal leg angles
IS : 3909-1966	Specification for aluminium unequal leg angles
IS : 3921-1966	Specification for aluminium channels
IS : 5139-1969	Recommended procedure for repair of grey iron castings by oxy-acetylene and manual metal arc welding