SOFTWARE FOR ANALYSIS & DESIGN OF INDUSTRIAL STRUCTURE

By

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SOFTWARE FOR ANALYSIS & DESIGN OF INDUSTRIAL STRUCTURE

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Nilesh S. Kitey
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Guide
Prof. G.N. Patel

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CERTIFICATE

This is to certify that the Major Project entitled "Software for Analysis & Design of Industrial Structure" submitted by Mr. Kitey Nilesh S. (07MCL007), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Software plays a vital role in analysis and design of different types of structures with different configurations. It is reducing the work as well the time for the analysis and design of Structure. In the last four decades there has been a great expansion in the programming and designing procedures. This has opened a new era in analysis and design for wide range of problems. Most of building material used in the industrial building, steel is perhaps the most universally acceptable as a versatile material for construction.

The primary aim of the present work is to Analysis and design of Gantry girder, Purlins, different roof trusses, side rail with sag rod, columns and beams of industrial structures as per new IS 800-2007 and prepare a software that analyze and design all this components of Steel Industrial building Structures based on various loading conditions. Visual Basic is used as the programming and designing language for Software design & SAP 2000 is used for analysis.

The prepared software is very user friendly. By providing suitable input data it will give a most economical section design by considering all necessary checks. It gives the complete solution for analysis and design of Gantry girder up to 250 kN capacity crane.

The prepared software produce a complete 3D model of industrial building structure in a SAP2000 software and apply all necessary loads such as Dead loads, live load, wind loads, crane loads and also generates all required load combination and importing all analysis result value from SAP 2000. On that analysis the design is carried out with all required checks and details. Standard Steel Sections, which are given in steel table and approved by I.S., are used. And these Standard Sections are easily available and widely used in practice.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOMENCLATURE

A_g	Gross area of cross section
A_n	Net effective area
A_{tg}	Minimum gross in tension
A_{tn}	Net area in tension
A_{vg}	Minimum gross area in shear along a line of transmitted force
A_{vn}	Net area in shear along a line of transmitted force
b	Width
b_s	Shear lag width
CL	Crane load
C_{pe}	External pressure coefficient
C_{pi}	Internal pressure coefficient
d	Depth
d_h	Diameter of the bolt hole
DL	Dead load
E	Modulus of elasticity for steel
ER	Erection load.
F	Net wind force
$f_{cr.b}$	Extreme fibre compression elastic lateral buckling stress
f_{db}	Design bending stress in compression
f_u	Ultimate stress of the material
f_y	Yield strength
g	Gauge length between the bolt holes
I	Moment of inertia of member
I_t	Torsional constant
I_w	Warping constant
I_y	Moment of inertia about the weak axis
IL	Imposed load
K_a	Area averaging factor
K_c	Combination factor
K_d	Wind directionality factor
K_1	Risk coefficient
K_2	Terrain, height, and structure size factor

K_3	Topography (ground contours) factor
K_4	Importance factor for the cyclonic region
L_c	Length of the end connection
n	Number of bolt holes in the critical section
M_{cr}	Elastic critical moment corresponding to lateral torsional buckling
M_{dz}	design bending strength as governed by overall buckling about minor axis
M_{dz}	design bending strength as governed by overall buckling about major axis
M_u	Ultimate Moment
M_{ndy},M_{ndz}	design strength under combined axial force and the respective uniaxial moment acting alone.
P_{cr}	Elastic Eular buckling load
P_d	Design wind speed
P_s	Staggered pitch length between line of bolt holes
P_z	Wind pressure
r	Appropriate radius of gyration
r_y	Radius of gyration of the section about the weak axis
SL	Snow load
t	Thickness
t_f	Thickness of the flange
T	Factored design tension
T_d	Design strength of the member
T_{db}	Design Strength Due to Block Shear
T_{dg}	Design Strength Due to Yielding of Gross Section
T_{dn}	Design strength in tension
V_{dz},V_{dy}	Design Shear strength
V_z	Design wind speed
w	Outstand leg width
WL	Wind load
Z	Section modulus
Z_e	Elastic section modulus
Z_p	Plastic section modulus
ρ	Mass density of air
Υ_{m0}	Partial safety factor

Υ_{m1}	Partial safety factor for failure at ultimate stress
χ_{LT}	Strength reduction factor for lateral torsional buckling of a beam
ϕ_{LT}	Strength or resistance reduction factor
λ_{LT}	Non-dimensional slenderness ration
α_{LT}	Imperfection factor

1. INTRODUCTION

1.1 GENERAL

In recent year most of the designers taking interest in software for Analysis and Design. In the last four decades there has been a great development in the programming and designing procedures. This has opened a new era in analysis and design for wide range of problems.

Most of building material used in the industrial building, steel is perhaps the most universally acceptable as a versatile material for construction. This is of course, the result of its many fine qualities so eminently suited to modern engineering structure. The primary function of all the structure is to withstand stresses due to loads such as dead load, live load, wind loads, earthquake loads, etc. without failure or undue distress such as excessive deflections, dangerous vibrations etc. The task of the structural engineer is to propose a suitable system for the specified purpose, to examine its overall stability and finally, to assess structural viability for the applied loads. The term structural design therefore signifies a process by which a structural engineer puts together a functionally efficient, economically affordable and structurally safe system for a set of given applied loads.

In general the frames for Industrial Buildings such as steel mills for manufacturing of machine parts, workshops, consumer's goods factories, gymnasium etc. are usually made single story. A mezzanine floor or basement is sometimes added for housing the lockers, restrooms and cafeteria for the workman instead of keeping these services at the shop floor. So, the software which deals with all objectives is very essential for efficient and economical design which can be done in less time with more accuracy.

1.2 Objective of Study:

The objective of study is to develop software for Analysis through SAP-2000 (Direct interfacing with SAP2000) and Design of an Industrial Structure as per new Indian Standard (IS 800-2007) provisions. Visual Basic is used as a Programming language.

Standard Steel Sections widely used in practice are selected in the project. The project also consist the preparation of Design Aids for the Truss, Gable Frame and Gantry Girder which gives the design of different members of these structures.

1.3 Scope of work:

Literature survey.

1. Study of VBA, SAP 2000 and AutoCAD coding software.
2. Problem formulation.
3. Industrial building.
    a. General guidelines of industrial building.
    b. Roofing cladding and other material.
    c. Loads.
    d. Selection of bay width.
    e. Structural framing system.
    f. Roofing system.
    g. Analysis and design of gantry girder, column, base plates and foundation.
    h. Connections.
4. Preparation of software tool in VBA interfacing with SAP 2000 and AutoCAD.
5. Use of AutoCAD for detailing.

1.4 Layout of the Report

Chapter-2 includes the literature available and study done in past on steel structure especially on behaviour of industrial structure under various loading condition.

Chapter-3 introduces, the basic guidelines required for planning, building configuration, various framing system and different elements of industrial building.

Chapter-4 includes the detailed information about different types of roofing materials and system currently available in market, various types of loads acts on the roof truss and design of compression and tension members of truss.

Chpater-5 includes the detailed information about the gantry girder and Design step for design of gantry girder.

Chpater-6 includes detailed information about the design of beams, purlins and side rails of industrial building.

Chpater-7 includes the detailed information about the various criteria for design of columns, crane columns and gives the design steps for design of columns.

Finally **Chpater-8**, gives summary and conclusion of the present work, which is followed by future scope of work.

2. LITERATURE SURVEY

2.1 General:

The analysis and design of industrial steel building is discussed by many authors. From the analytical as well as experimental work they framed some important conclusion for the increasing the efficiency of the structural system. All possible aspects such as fire condition, wind pressure distribution, Non linear behavior of structure. Seismic assessment and dynamic nonlinear analysis, elastic buckling of pitched roof steel frames etc are considered in detail.

2.2 Literature Review:

John A. Rolfes and Robert A. Macc [1]: The author had given the brief discussion on current seismic provision in the United State and Canada for the design and construction of steel-framed industrial building and non building structures also design codes for it. They give the strategies for design of steel framed industrial building using the building code in the U.S (IBC 2006) and Canadian building code (NBCC 2005).

S. L. Chan [2]: The author had discussed the work done by different researchers related to non-linear analysis technique. Theoretical background and advances in the non linear analysis and design of steel frames with practical application and finally concluded the merits and limitation of the different methods of non linear analysis.

Pal Tomka. [3]: The author had discussed about the lateral stability of parallel beam connected by an element with concentrated force acting on the upper flange. The disadvantageous effect of this load position is not so severe when the load is taken by two parallel beams that are connected by an element. This is the case of the crane girders at which the gantry yields such a connection and derives the same equilibrium equation for that and also did the parametric study on simply supported beams. The results of study were approximated by simple formulae. Practical application was also shown on bases of his study and concluded that the performance of these beams considerably higher than that of stand-alone beam and load bearing capacity exceeds up to 20%.

Nuno Silvestre, Dinar Camotim.[4]: The author had discussed about the results of study concerning the elastic in plane stability and second order behavior of unbraced single bay pitched roof steel frames and developed the design application with an efficient methodology. Characterizing the relevant frame buckling modes and p-Δ second order effect, and also addressed the exact and approximate calculation of the associated bifurcation loads and secondary bending moments of unbraced single bay pitched roof steel frames. The paper briefly addressed the fundamental concepts and procedure involved in the second analysis and design of orthogonal beam and column frames. This works dealt only with in plane global analyses of pitch roof frames and did not address issues related to the second- order behavior and safety checking of the individual frame members (p-δ effects).

Ben Young. [5]: In this paper the author had discussed about Design and numerical investigations of cold-formed steel channel columns with inclined edge stiffeners. The edge stiffeners of the channel sections consisted of simple lips inclined at different angles both outwards and inwards. For numerical investigation Finite element analysis was used. A non-linear finite element model was developed and verified against fixed-ended column test. Geometric and material non-linearities were included in the finite element model. It is shown that the finite element model closely predicted the experimental ultimate loads of the cold-formed steel channel columns with inclined edge stiffeners. Hence, the model was used for the parametric study of cross-section geometries. The column strength predicted by the finite element analysis was compared with the design column strength calculated using the American and Australian/New Zealand specifications for cold-formed steel structures.

Alexandre Landesmann, Eduardo de M. Batista [6]: This paper is regarding with the development of an advanced analysis numerical tool capable of estimating the inelastic large-displacement behavior of plane steel-framed structures under fire conditions. A non-linear transient heat transfer analysis is performed on the basis of the finite element method (FEM). The computational analysis program is used to assess the structural load-bearing functions and to estimate the structural behavior and the corresponding time-resistance period. The results obtained are compared with those from FEM computational program

J.G. Yang, G.Y. Lee [7]: This paper gives an idea to converting the complicated model to a simplified analytical model that is only applicable in the preliminary design stage. Using the simplified analytical model, it can easily predict the storey drift, maximum allowable loads, and bracing stiffness of a single-storey multi-bay steel frame. The application feasibility of the simplified model is verified by performing a two-dimensional finite element analysis.

S K Duggal (2000) [8] Describes the industrial steel structures. Different forms of tension, compression and flexure elements are discussed. Different roof supporting tension systems and tension roof structures also presented. Information about Gantry Girder also discussed about it. This book also gives new trends in Industrial Building Design.

Pasala Dayaratanam (2000) [9]: This book includes the design of Industrial Steel Structures. It also includes the design of Gantry Girder. This book describes all the different members of Industrial Building as Compression Member, Tension Member.

N Subramanium(2008)[10]: This book explain the behaviour of various elements of structures and to provide the basis for the codal rules. It gives the design of the steel structure members (such as gantry girder, compression member, tension members, beams, columns, etc...) as per new codes IS 800-2007

Hand Book for Structural Engineering, Steel Beams and Plate Girders (1999) [11]: these deals with the design of Rolled Beams and Plate Girder. Numerical Analysis for Bending Moment and Deflection in Beams. And Different Problems related to Rolled Beams and Plate Girder.

Hand Book for Structural Engineering, Steel Columns and Struts (1998) [12] This deals with the design of Centrally loaded columns, columns in Multi-Storey Buildings Crane Girder, Mill Building Column with Crane Girder, Concluding Remarks Concerning Column Design.

Hand Book of Typified Designs for Structures with Steel Roof Trusses(1998) [13]: This deals with the analysis of roof truss, design of roof truss, different roof configurations, fabrication details and design examples related to roof truss.

Hand Book on structure with Steel Portal Frames (1988) [14]: these deals with Portal frame analysis, Design foundation forces, Fabrication details, design examples.

3. GENERAL CONSIDERATION

3.1 INTRODUCTION

Any building structure used by the industry to store raw materials or for manufacturing products is known as an industrial building. Industrial buildings may be categorized as Normal type and special type. Normal types of industrial building are shed type buildings with simple roof structures on open frames. These buildings are used for workshop, warehouses etc. These building require large and clear unobstructed area by the columns. The large floor area provides sufficient flexibility and facility for future change in the productivity area without major building alterations. The industrial buildings are constructed with adequate headroom for the use of an overhead traveling crane. Special types of industrial buildings are steel mill buildings used for manufacture of heavy machines, production of power etc. The function of the industrial building dictates the degree of sophistication.

3.2 PLANNING

The planning of an industrial building is based on functional requirements, i.e. on the operations to be performed inside the building. In the planning of a particular type of industrial building due consideration should be given to factors such as wide area free of columns, large height, large doors and openings, large span of trusses, consistency of operation, a compromise with all the listed factors to give minimum weight of trusses, purlins, columns, etc. and lighting and sanitary arrangements.

Generally, the site for a proposed plant is pre-selected, before it comes for design. Preferably, for the preliminary planning, if discussion is carried out with designer, it will give him an opportunity to choose a suitable site for future developments. Some of the factors governing the site selection are as listed below:

1. The site should be located on an arterial road.
2. Local availability of raw materials.
3. Facilities like water, electricity, telephone etc.

4. Topography and water drainage.
5. Soil condition with reference to foundation design
6. Sufficient space should be available for storage of raw materials and finished products.
7. Sufficient space should be available for transportation facilities to deliver raw materials and collect the finished product.
8. Waste disposal facilities.

3.3 BUILDING CONFIGURATION

Typically the bays in industrial buildings have frames spanning the width direction. Several such frames are arranged at suitable spacing to get the required length (Fig. 3.1). Depending upon the requirement, several bays may be constructed adjoining each other. The choice of structural configuration depends upon the span between the rows of columns, head room or clearance required the nature of roofing material and type of lighting. If span is less, portal frames such as steel bents (Fig. 3.2a) or gable frames (Fig. 3.2b) can be used but if span is large then buildings with trusses (Fig. 3.3 & 3.4) are used.

The horizontal and vertical bracings, employed in single and multi-storey buildings, the trusses used primarily to resist wind and other lateral loads. These bracings minimize the differential deflection between the different frames due to crane surge in industrial buildings. They also provide lateral support to columns in small and tall buildings, thus increasing the buckling strength.

3.4 STRUCTURAL FRAMING:

A structural framework consists of steel trusses supported over columns making a transverse bent, whereas a longitudinal bent is formed by joining transverse bents. Structural framing consists of the following steps:

1. Knowing the production layout, the column length can be thought of. The column rows can be located according to clearance requirements. Column rows at wide distance and with large trusses are economical as compared to closely spaced more number of rows of column with short span trusses.
2. The columns in each row are fixed so as not to interfere with the mechanical layout. Generally, a column spacing of 4 m to 8 m is provided. If larger column spacing is required in some portion of a multi-bay system

industrial building, the trusses may be supported over a girders or supporting trusses.

3. A sketch of the members, openings, lintels, doors windows, girts and end wall columns is made.
4. As the column spacing is fixed by this time, the pitch of the truss is selected and the type of truss to be provided is decided. The panel lengths are worked out and purlin locations are marked.
5. A plan is prepared to show the lateral bracing system. It may be done either in the plane of the bottom chord or in the plane of the top chord and some time in both the planes.
6. Various materials to be used for the floor, roof walls and partition walls are decided.

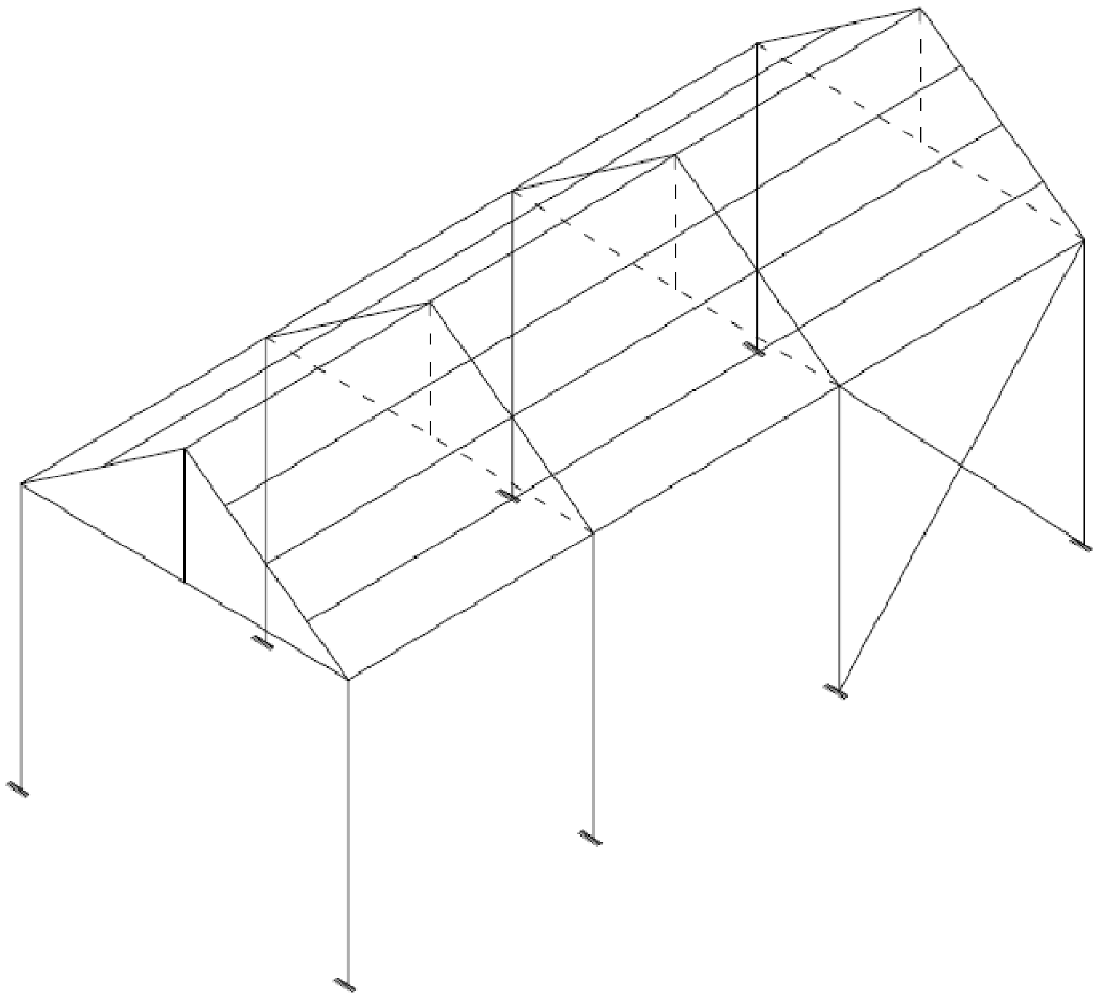


Fig 3.1 Typical structural layout of an industrial

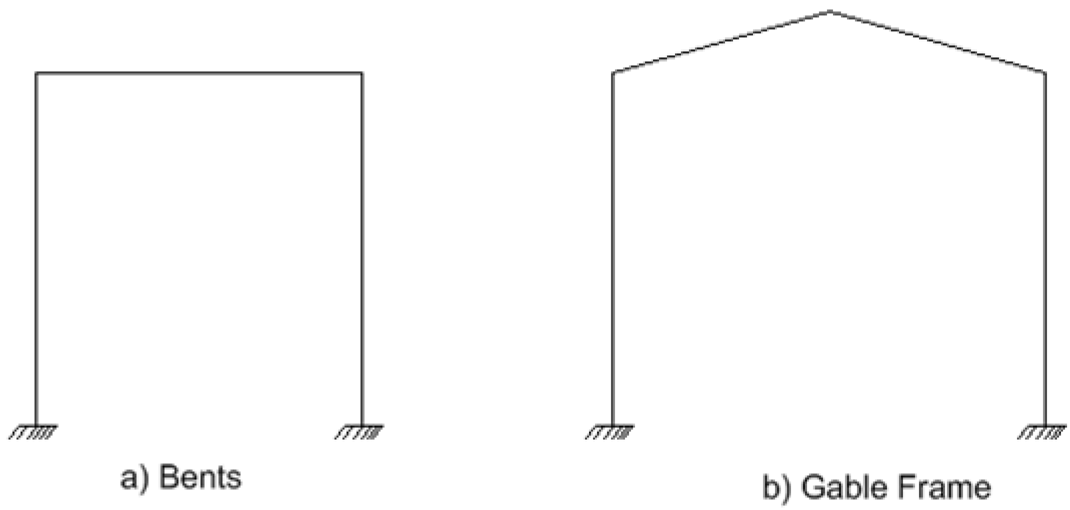


Fig 3.2 Typical structural layout of an industrial (Bents & gable frame)

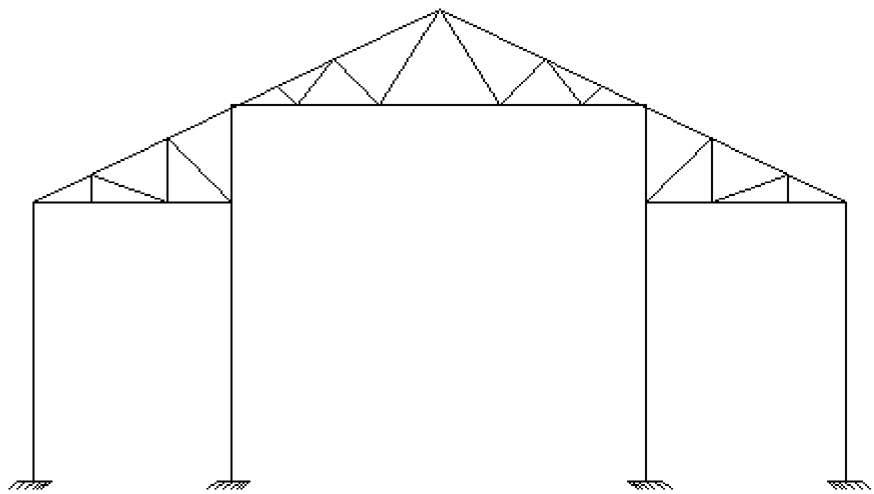


Fig 3.3 Industrial building with side span

3.5 ELEMENTS OF AN INDUSTRIAL BUILDING:

The elements of an industrial building are as listed below. These are discussed one by one in the articles to follow.

1. Purlins. 2. Sag rods. 3. Principal rafter. 4. Roof truss.
5. Gantry girder. 6. Bracket. 7. Column and column base.
8. Girt. 9. Bracing.

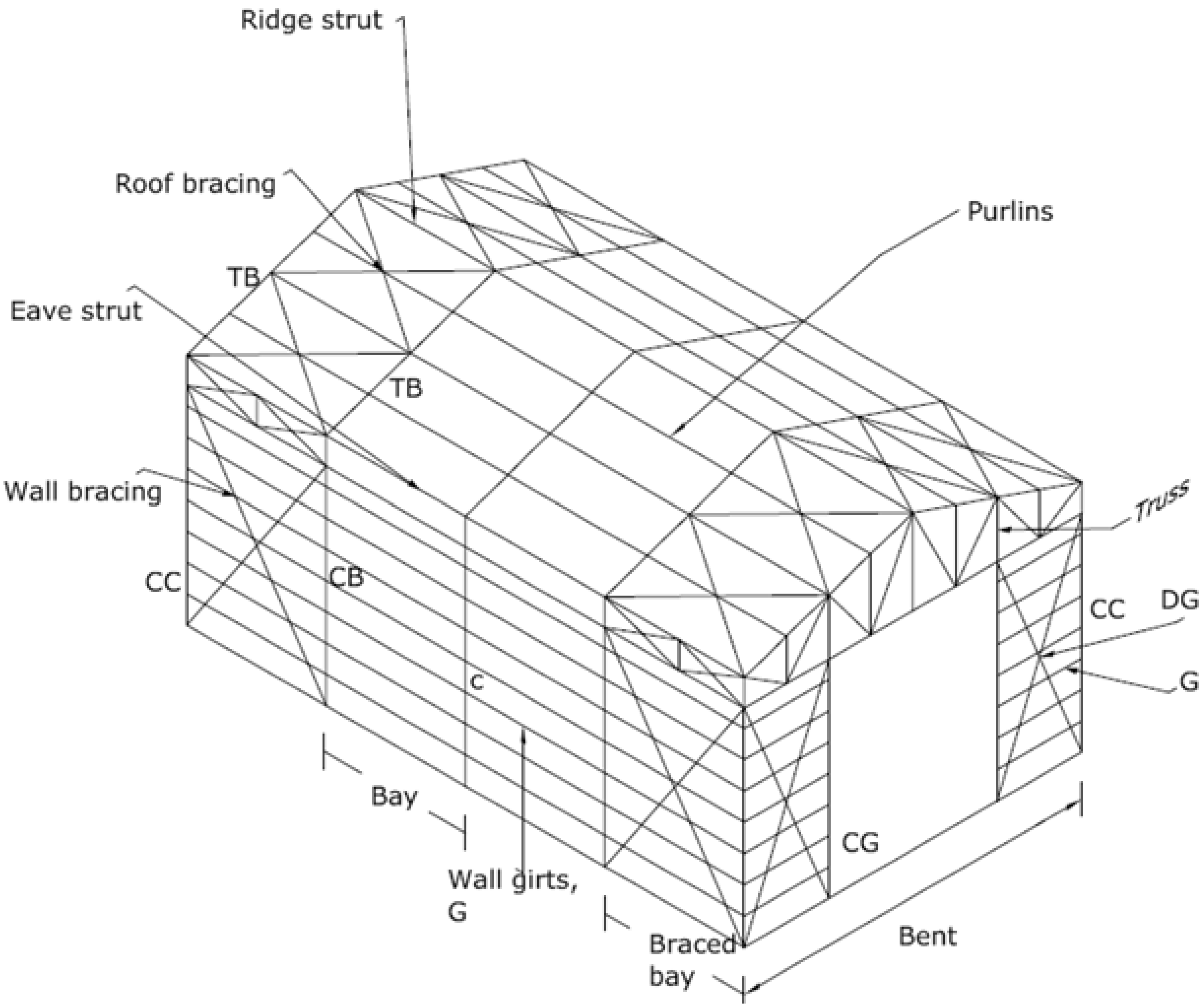


Fig3.4 Structural Frame of an Industrial Building

C – Column, *CC*- corner column, *CB* –column in braced bay, *CG*–column in gable end, *DG*- Diagonal bracing in gable end, *TB*- truss top chord in braced bay

3.5.1 Purlins:

Purlins are beams which are provided over roof trusses to support the roof coverings. Channels, angle sections, and cold formed C- or Z sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points. For larger trusses, however, it is more economical to space purlins at closer intervals. In India, where asbestos cement (AC) sheets are used, the maximum spacing of purlins is also restricted by the length of these sheets. AC sheets provide better insulation to sun's heat (compared to GI sheets), which can be further improved by painting them white on the top surface. The maximum permissible span for these sheets is 1.68 m. A longitudinal overlap of not less than 150 mm is provided for AC sheets. The purlin spacing is so adjusted with lengths of the sheets that the longitudinal overlaps fall on the purlins to which they are directly bolted. Spacing of purlins should be so fixed that the cutting of sheets is avoided. Hence in practice when AC sheets are used, the Purlin spacing is kept between 1.35 to 1.40 m. But in general, purlins are spaced from 0.6 m to about 2 m and their most desirable depth to span ratio is about 1/24.

3.5.2 Sag Rods:

These are round section rods and are fastened to the web of the purlins. The roof coverings in industrial buildings are not rigid and do not provide proper support. Therefore, sag rods are provided between adjacent purlins to extend lateral support for the purlins in their weaker directions.

A sag rod is designed as a tension member to resist the tangential component of the resultant of the roof load and purlin dead load. The tangential component of the roof load is considered to be acting at the top flange of purlins, whereas the normal component and purlins dead load is assumed to act at its centroid. Therefore, the sag rods should be theoretically placed at the point where the resultant of these forces act. But this is not practicable and sag rods are placed at the minimum gauge distance below the top.

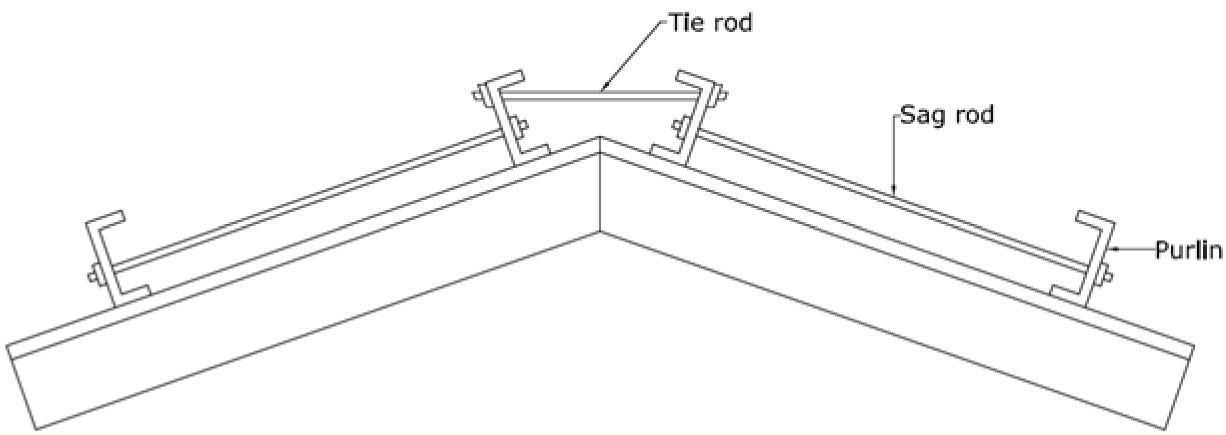


Fig 3.5 Sag Rod Connections

3.5.3 Principal Rafter:

The principal rafter of a roof truss is designed as a continuous strut. Generally, a double unequal angle section with long legs back-to-back is most suited for an industrial building structure. This is due to two reasons. A rafter section should have the same radius of gyration about the main axes ($r_{xx} = r_{yy}$), so as to achieve the same bending strength about the two axes and to have the value of the least radius of gyration as large as possible. Also, the double angle section provides extra strength which is useful during erection when the trusses are unbraced.

The heel of the rafter angle is placed on top due to the following two reasons:

1. The rafter angle acts as a beam in carrying its self weight. Therefore, the compression edge should be wide.
2. If the heel is kept down the rafter angle will act as a gutter and in exposed conditions will lead rain water down to the connections.

Although the design forces in various panels of the rafter vary considerably, they are still made of uniform cross-section. There are two reasons for this:

1. The saving achieved by changing the sizes in various panels is offset by the cost of splice and connections.
2. There is a loss of lateral stiffness of the structure otherwise.

3.5.4 Roof Trusses:

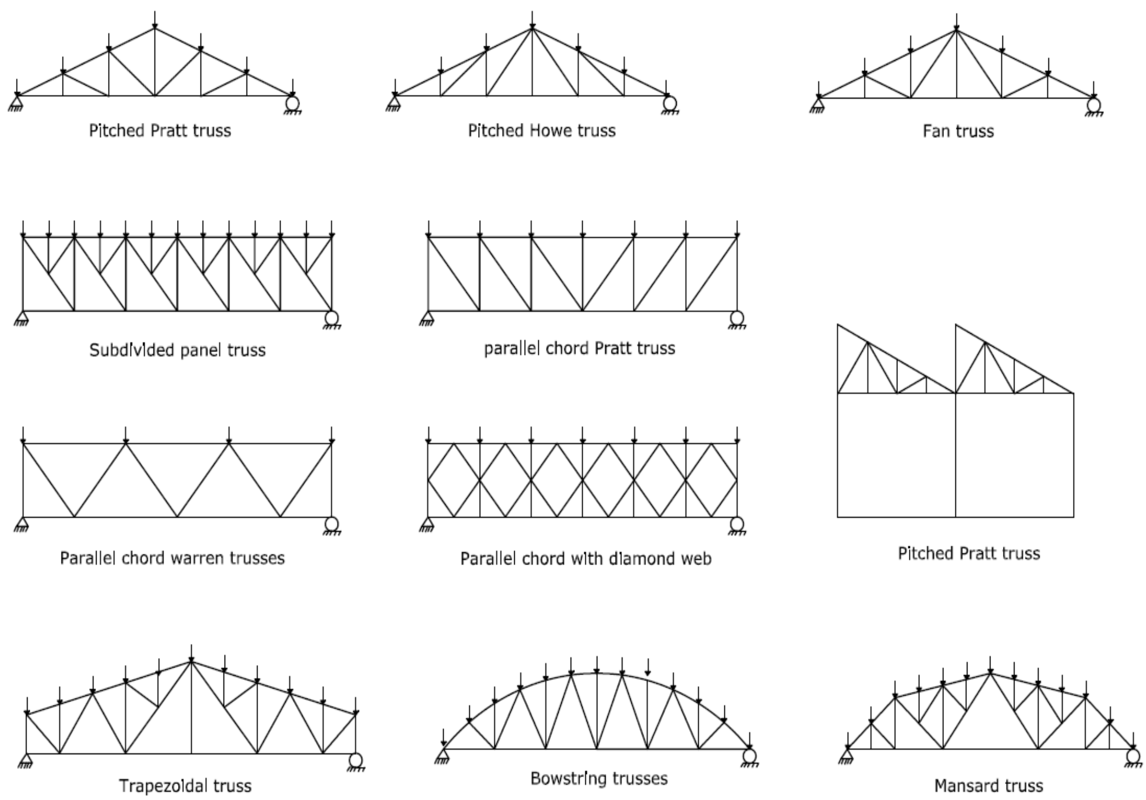


Fig 3.6 Common type of truss

Roof trusses are elements of the structure composed of members subjected to direct stresses. Sometimes the truss is also called an *open web beam.* It consists of a triangular network of compression and tension members. On the *basis* of structural behavior, roof trusses can be classified as simple roof trusses supported over masonry walls, and roof trusses supported over columns and connected to it with knee braces.

3.5.5 Gantry Girder:

Gantry girders are laterally unsupported beams in industrial buildings. Overhead travelling cranes are used in industrial buildings to lift and transport heavy jobs, machines, and so on, from one place to another. The crane consists of a bridge made up of two truss girders. The bridge is termed a *crane bridge, crane girder* or *crab girder.* It spans the bay of the shop and moves in a longitudinal direction. To facilitate movement, wheels are attached to the ends of crane girders. These wheels move over rails placed centrally over the girders which are called *gantry* or *crane gantry girders.* Figure 3.7 shows the front view of the typical arrangement of a crane girder, gantry girder and column in a workshop.

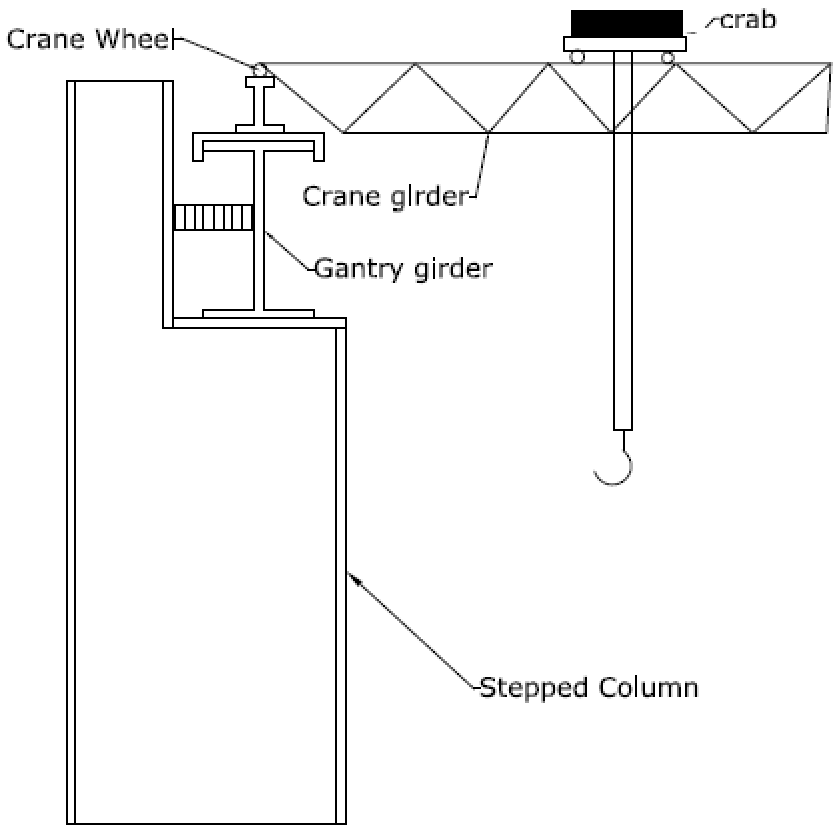


Fig 3.7 Front view of Gantry Girder arrangement

3.5.6 Brackets:

Brackets are used to support the gantry girder. The bracket type connections are more rigid compared to any type moment connection. The fabrication cost is very high and hence they are not adopted in practice, except some special cases.

3.5.7 Crane Columns:

The columns which support the gantry girders are in general, called gantry columns or crane columns. Depending upon the arrangement of the connection and crane load to be transmitted the columns are divided into three categories:

1. Column with bracket.*(fig 3.8 (a))*
2. Stepped column or notched column. *(fig 3.8 (b))*
3. Double column. *(fig 3.9)*

An industrial building column is subjected to the following loads in addition to its self-weight.

1. Dead loads from roof truss.
2. Live loads on roof truss.
3. Crane load which consists of dead weight of gantry girder, rail, bracket, wheel load and impact load
4. *Loads* due to wind (i.e. the horizontal and vertical component of wind reaction) and knee brace loads. This can be found from the wind stress analysis of bents. The wind load is assumed to be uniformly distributed over the columns.

3.5.8 Girts:

These are beams subjected to unsymmetrical bending. These support vertical dead loads from the sheeting and horizontal wind loads. The main function of girts is to transfer wind loads from wall materials to the primary frame. Girts are positioned horizontally to span between the columns. When the space between primary columns is more than 9m, wind columns may be provided to reduce the girt span.

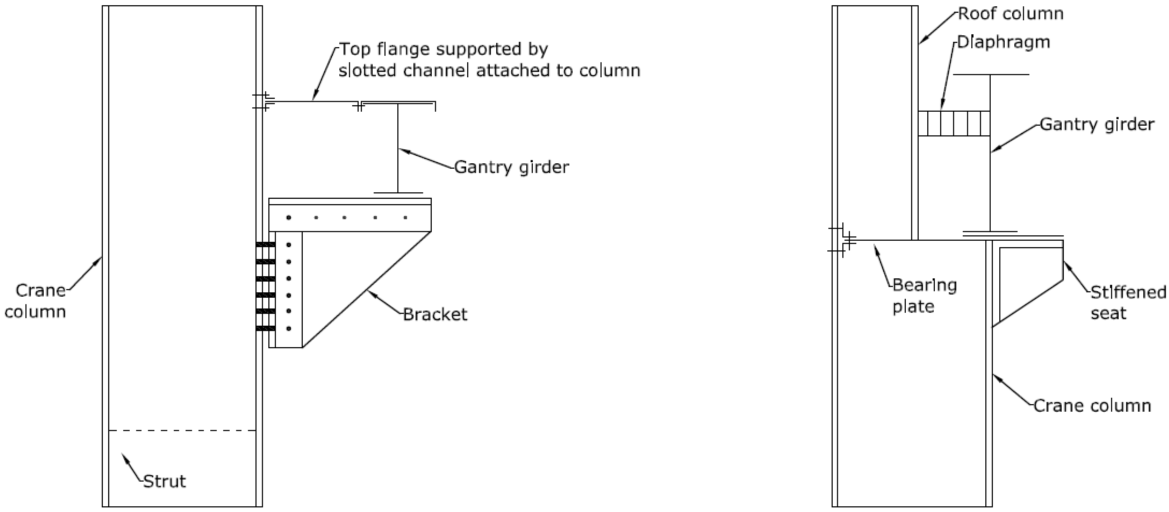


Fig 3.8 Column with bracket and Stepped column

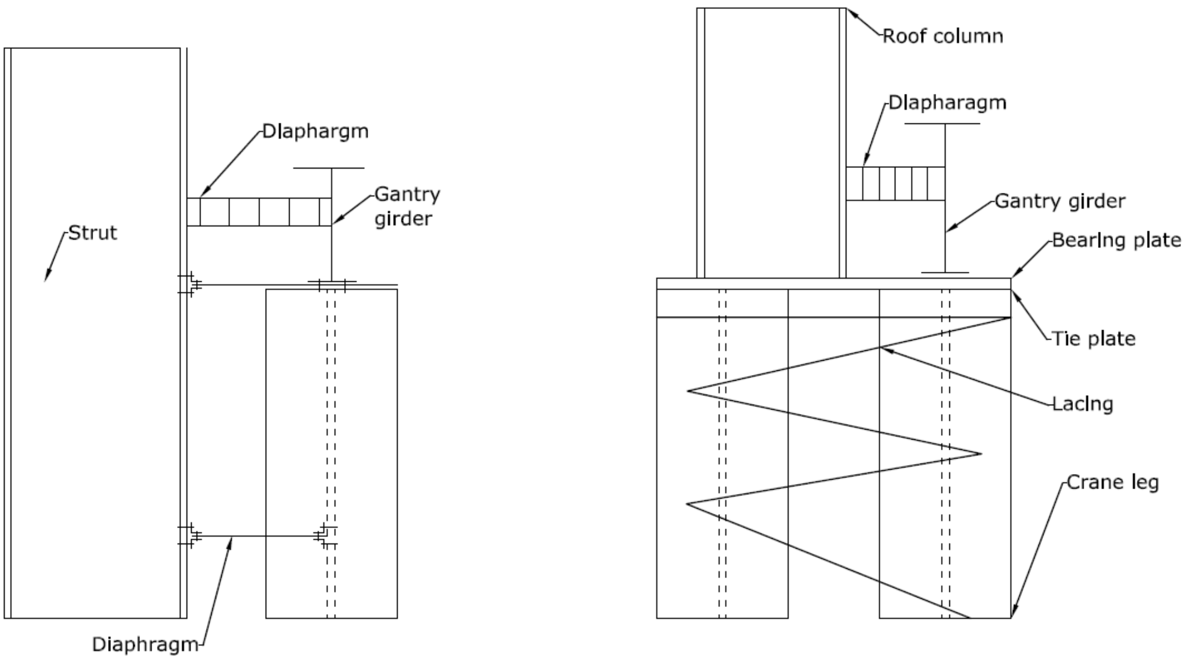


Fig 3.9 Double Column

3.5.9 Bracing:

A simple industrial building bent can effectively resist gravity loads but deforms under lateral loads. The bracings are so planned and provided that lateral forces due to wind, earthquake and crane surge, etc. are transmitted efficiently to the foundation of the building. An individual bent may be stable against lateral loads in its own plane. However, it has little resistance to the forces acting normal to its plane. Also a series of bents constituting the industrial building should be stable.

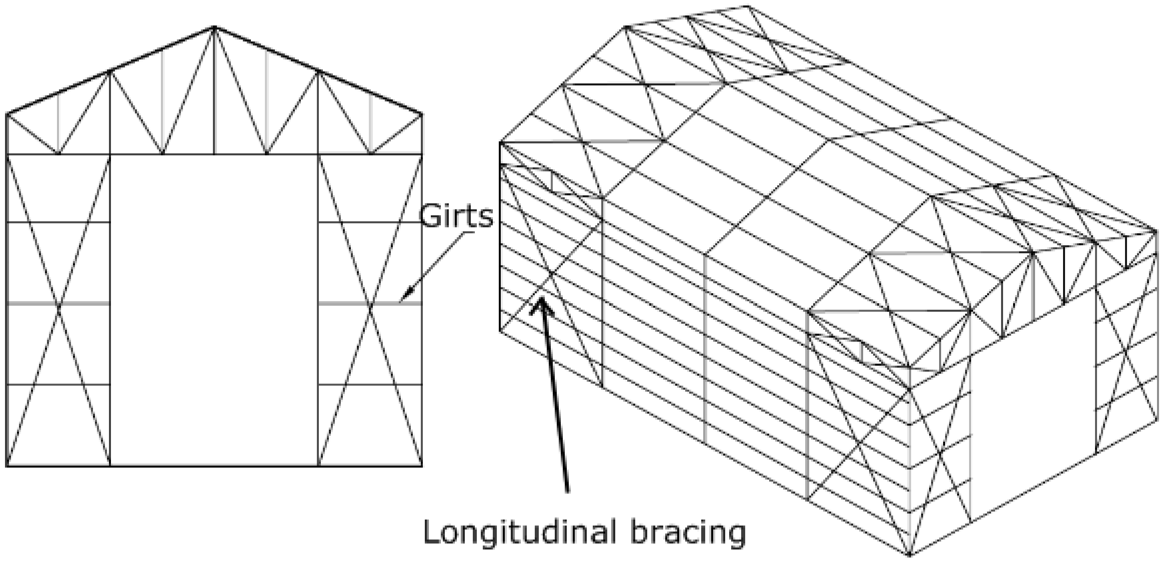


Fig 3.10 Types of Bracings

a) Transverse bracing: A bent should be braced transversely to preclude collapse even during erection. Each bent after erection should be stable in the transverse direction. The end conditions of the columns have large bearing over the stability of the bent.

b) Longitudinal bracing: the wind may be normal to the transverse bent. Therefore it becomes essential to brace the industrial building in the longitudinal direction. The diagonals in the plane of the top chord of truss are usually bolted to purlins wherever possible. This arrangement reduces the unsupported length of the diagonals and makes them more effective in compression.

4. ROOF STRUCTURE

4.1 ROOFING CLADDING AND OTHER MATERIAL:

The type of roof deck, type of purlin used, purlin spacing, deflections of structural members, roof pitch, and drainage requirements are all governed by the choice of roofing. The roof weight also affects the gravity load design of the roof system and in the case of seismic calculations, the lateral load design. Similar considerations apply to the cladding/wall systems. In selecting the cladding/wall system, the designer should consider the following:

a) Cost
b) Interior surface requirements
c) Aesthetic appearance (including colour)
d) Acoustics and dust control
e) Maintenance
f) Ease and speed of erection
g) Insulating properties
h) Fire resistance

Note that *cladding* carries only its own weight and the loads imposed by wind. In the case of roofs, the sheeting supports insulation and water proofing in addition to self weight and loads due to wind and/or Snow. Hence, it is often termed as *roof decking.* The cladding/wall system will have an impact on the design of girts, wall bracing, eave members, and foundation.

In India, corrugated galvanized iron (GI) sheets are usually adopted as coverings for roofs and sides of industrial buildings. Now light-gauge cold-formed ribbed steel or aluminum decking (manufactured by cold drawing flat steel or aluminium strips through dies to produce the required section) is also available. Sometimes asbestos cement (AC) sheets are also provided as roof coverings owing to their superior insulating properties. Their insulating properties may be enhanced by painting them white on the top surface.

4.1.1 Galvanized Iron (GI) Sheets

Most common sizes of corrugated GI sheets are as follows:

(a) 8 corrugations (75 mm wide and 19 mm deep) per sheet

(b) 10 or 11 corrugations (75 mm wide and 19 mm deep) per sheet

The available sizes of sheets are as follows:

(a) Length — 1.8, 2.2, 2.5, 2.8, and 3m

(b) Width — 0.75 m and 0.9 m

(c) Thickness — 0.63, 0.8, 1.0, 1.25, and 1 .6 mm

The weights of the sheets vary from 50—156 N/m^2 .When the sheets are installed, side laps and end laps should be provided to make the joint water proof. The following overlaps are normally used:

(a) For roof: Side overlap — 1½ to 2 corrugations

End overlap — 150 mm

(b) For side cladding: Side overlap — 1 corrugation

End overlap — 100 mm

The sheets are fastened to purlins or side girts by 8 mm-diameter J or L-type hook bolts with GI nuts along with GI and bituminous felt washers at a maximum pitch of 350 mm. Where laps do not occur over supports, 6-mm diameter bolts at a maximum pitch of 250 mm for roofs and 300-50 mm for sides are used.

Spacing of purlins and girts, which support the sheeting is governed by the length of the sheet, thickness of the sheet, and applied loading. The approximate section modulus of the corrugated GI sheeting may be taken as

$$Z = \frac{4}{15}\, b\, d\, t \qquad ..(4.1)$$

Where b is the curvilinear width (equal to 1.13 x covering width), d is the depth of the corrugation, and t is the thickness of the sheet.

4.1.2 Asbestos Cement Sheets

Asbestos cements sheets may be used to cover the roof as an alternative to corrugated steel sheets. (These sheets are banned in many countries due to the risk of lung cancer while working with them) AC sheets are manufactured in two shapes, corrugated and Trafford and are available in lengths of 1.75, 2.0, 2.5, and 3 m. They are manufactured in thicknesses of 6 mm or 7 mm. The maximum permissible spacing of purlins

(a) for 6-mm sheet— 1.4 m
(b) for 7-mm sheet— 1.6 m

For side cladding, the spacing may be increased by 300 mm. A side overlap of one corrugation is normally given. The end lap should not be less than 150 mm for slopes less than 18° and for flatter slopes this overlap may be increased. For side covering, an overlap of 100 mm is sufficient. The purlin spacing should be so adjusted with the lengths of the sheets that the longitudinal overlaps fall on the purlins to which they are directly fastened. Spacing of purlins should be so fixed that the cutting of sheets is avoided. The overhang near the eaves end should not be more than 400 mm.

The weight of asbestos sheets varies from 160 N/m^2 to 170 N/m^2. The load per square meter of the sheet on the slope may be generally increased by 30% to get the load per square meter of an area, to account for the large area on the slope and additional material in the side and end lapping. The sheets are fastened to purlins or grits by using 8 mm diameter hook bolts at a maximum spacing of 350mm.

4.1.3 Cool Metal Roofing

Cool metal roofing is a family of sustainable, energy efficient roofing products comprised of unpainted metal, repainted metal, and granular-coated metal. It is available in a wide variety of finishes, colors, textures, and profiles, for steep-slope and low-slope applications. Cool metal roofing products are part of an interdependent system of exterior roofing surfaces, substrates, underlayment’s, configurations, ventilation, and insulation.

With proper design, cool metal roofing systems save energy by reducing a building's cooling and/or heating load. Many metal roof systems are reflective, easily vented, and lend themselves well to insulated roof systems to help reduce heat gain into a building. Many products are also formed in ways that stop heat transfer through conduction by allowing only minimal contact between the metal and the underlying structure.

Mill-finish metal roof systems have very high solar reflectance, providing further reductions in heat gain. Metal roofs with oven-cured, pre-painted organic coatings that incorporate new "cool pigment" technology offer high total solar reflectance and high infrared emittance even with darker colors. Emissivity as high as 90%; can be achieved for painted and granular-coated roofing. Such pre-painted metal roofing products meet the reflectance requirements of all major energy code initiatives. Finally, unlike many roofing materials, metal's low thermal mass will not store heat and radiate it into a building in the evening hours. Painted metal roofs retain 95% of their initial reflectance and emittance over time.

Cool Metal Roofing typically has a minimum recycled content of 25% and is 100% recyclable at the end of a long, useful life. Most metal roofs are credibly proven to last over 30 years with minimal maintenance.

4.1.3.1 Characteristics of Cool Metal Roofing

The benefits of metal roofing include:

1. ***Durability***. Metal roofing products are not subjected to the degradation experienced by organic materials when they are exposed to the weather cycle. This provides metal roofing with a long life in terms of its ability to resist the elements and also to possess a low maintenance cost.

2. ***Low Weight.*** Metal roof systems typically vary from 20 to 64.5 N/m^2, making them among the lowest weight roofing products available. Low weight places fewer demands on a building's structure making metal roofing an excellent choice for retrofit projects. The light weight is also a benefit in locations prone to seismic activity.

3. ***Fire Resistance*.** Many metal roof systems have been tested to meet Class A, B, and C fire ratings.

4. ***Aesthetics.*** Due to its ability to accept coatings of various colors and patterns and its ability to be formed into a wide variety of functional

profiles, cool metal roofing products can be found to fit and enhance the aesthetic design of virtually any building. This gives architects extensive design flexibility.

5. ***Wind Resistance.*** The interlocking or active fastening of most metal roofing panels allows them to pass very severe wind and uplift tests including ASTM E1592, UL 580, and UL 1897. Many products carry approval for use in Dade County, Florida.

4.1.4 PROFLEX Roofing Structure:

PROFLEX SYSTEMS are self-supporting roofing panels made from colour-coated galvalume/ galvanized steel with the use of mobile manufacturing systems, also known as portable roofing and building devices. One of the most remarkable aspects of the technology is that it enables installation of the roofs and buildings at an incredible speed of covering over 2000 square meters in just 12 hours, something which none other alternative permanent roof can yet achieve.

This leads to a big time-gain for user, especially in time-bound construction projects where timely implementation plays a critical role! PROFLEX SYSTEMS, an American technology, proves to be an ideal cover for factories, warehouses and other industrial, agricultural & defense structures. With no requirement for any kind of ancillary support structures, trusses and purlins, these roofs offer economy in both, time and energy. The mechanical system of panel connection requires no holes, nuts, bolts, overlaps or sealants, again adding on to the economy of costs. The Proflex roof is full proofed against water seepage and extreme weather conditions, thus giving an enhanced protection to goods or machinery stored in the facility, which reduces the cost related to product failure or rejection. The non-combustible qualities and high tensile strength make them stronger against natural calamities like fire, earthquakes or hurricanes. Also, the unique panel seaming technology makes it needless of constant safeguarding & maintenance efforts.

A continuous roof of up to 40 meters span at a stretch can be easily and very effectively covered with these systems. A very distinguishing arch shape and the possibility of using colors in roofing along with basic design flexibility result in a strong aesthetic appeal for Proflex Systems which gives an interestingly attractive look to the buildings. It also rules out the possibilities of any bird nuisance, a common trouble faced in factories and warehouses. Proflex Systems' dome-type structures provide a perfect solution when it comes to the utility of vertical space. The unique structure facilitates storage up to much higher levels while maintaining lower wall heights when compared to conventional go-downs.

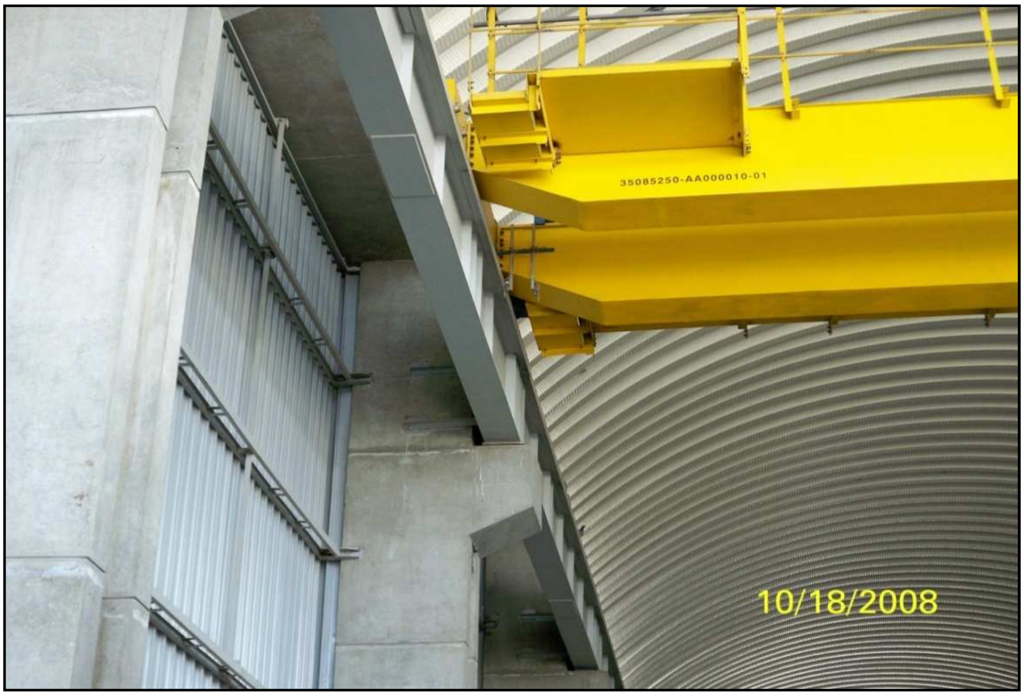


Fig 4.1 PROFLEX Roofing system
(Transformer & Rectifier Pvt. Ltd. at Ahmedabad)

Fig 4.2 PROFLEX Roofing system
(Kemrock Industries & Exports Pvt. Ltd. Vadodara
And Arvind Mills Ltd. Ahmedabad)

4.2 Loading and Load Combinations:

For design of any structure, designer must know the possibilities of various natural and manmade loads acting on that structure. There may be different types of loads and the designer must be evaluate the magnitude of each of the loads and combination of loadings condition which will result in the most unfavorable loading with respect to the stress in the member. The usual forms of loading include

- Dead loads (DL), also called fixed loads
- Imposed loads (IL), also called Live loads
- Wind loads (WL)
- Earthquake loads (EL)
- Snow loads
- Crane loads
- Temperature loads (loads due to contraction or expansion resulting from variation in temperature)
- Erection and fabrication loads
- Dynamic loads
- Blast and explosion loads (due to terrorist activities and accidents)
- Loading due to fire
- Foundation settlements
- Fatigue effects
- Dust loads (in certain industries and building in certain locations, especially in deserts)

A load on structure may be due to the following:

- Mass and gravitational effect (*mg)* – examples of such loads are dead loads, imposed loads, snow, ice, earth loads, etc.,
- Mass and its acceleration effect (*ma)* – examples of such loads are those caused by earthquake, wind, impact, blast etc.,
- Environmental effect : examples are loads due to temperature difference, settlement, shrinkage, etc.,

As per IS 800 – 2007 classifies loads as follows.

a) *Permanent actions (Qp)* Action due to the self – weight of structure and non-structural components, fixed equipments and machinery, etc.
b) *Variable actions (Qv)* Action due to construction and service stage loads such as imposed (live) loads (crane loads), wind loads, earthquake loads etc..

c) *Accidental action (Qa)* Action due to explosions, fire etc.

4.2.1 Dead Loads:

A load fixed in magnitude and in position is called a dead load. Determining the dead load of the structure requires the estimation of the weight of the structure together with its associated 'non-structure' components. The dead load of a steel structure is not known before it is design. Normally, an initial value is assumed or estimated based on experience.

The IS 800-2007 states that the self weight computed on the basis of nominal dimensions and unit weights as given in IS 875 (Part I) may be taken to represent the characteristic dead load.

4.2.2 Live Loads (Imposed loads):

Imposed loads are gravity loads other than dead loads and cover factors such as occupancy by people, movable equipment and furniture within buildings, stored materials such as books, machinery, and snow. Hence, they are different for different types of buildings residential, offices, warehouses, etc.

Imposed loads may be subdivided into two groups: (a) those which are applied gradually, in which case the static equivalent can be used, and (b) those that are dynamically (for example, repeated loads and impact loads).

Imposed loads are specified in the codes based on observation and measurements are generally expressed as static loads for convenience, although there may be minor dynamic forces involved. As per the Indian code [IS 875 (Part 2—Live loads), imposed loads are classified into the following groups:

a) Residential (dwelling. hotels. hostels, boarding and lodging houses, clubs, etc.),
b) Educational (classrooms, libraries. etc.),
c) Institutional (office rooms, bedrooms, kitchens, general storage, etc.),
d) Assembly halls (with and without fixed seating, etc.),
e) Office and business buildings (rooms with and without separate storage, computer rooms, filing/store rooms, strong rooms, dining rooms, etc.),
f) Mercantile buildings (shops),

g) Industrial (with light- /medium- /heavy-duty machinery/equipment), and
h) Storage buildings (warehouses, cold storages), etc.

Though live loads occur at random on any floor, they are often assumed to be uniformly distributed on the floor/roof area. The complete guidelines are given in IS 875 (Part 2). These live loads are assumed to be present on the floors and have to be transmitted to the foundation through beams (cross-beams) and columns. The code allows for a reduction in live load in the design of a single beam or girders by 5% for each $50m^2$ of floor area supported (for areas greater than $50m^2$), subject to a maximum reduction of 25%. However, this reduction should not be applied to floor of storage buildings.

Table 4.1 Imposed loads on various type of roofs

Types of roof	Uniformly distributed imposed load measured on plan area	Minimum imposed load measured on plan
Flat. sloping, or curved roof with slopes up to and including 10^o (a) Access provided (b) Access not provided (except ladder for maintenance)	$1.5\ kN/m^2$ $0.75\ kN/m^2$	3.75 kN uniformly distributed over any span of 1 m width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall. Half of case (a) above
Roof with slope greater than 10^o	For roof membrane sheets or purlins $0.75\ kN/m^2$; For every degree increase in slope over 10 degrees: reduce by $0.02\ kN/m^2$	$0.4\ kN/m^2$
Curved roof with slope of line obtained by joining Springing point to the crown with the horizontal, greater than10^o	$0.75 – 0.52\ r^2\ kN/m^2$, where r=h/l, h is the height of the highest point of the structure measured from its springing, and l is the chord width of the roof if the roof is singly curved and the shorter of the two sides if it is doubly curved.	$0.4\ kN/m^2$

4.2.3 Wind loads as per IS 875 (part 3) draft code:

IS *875* (Part 3): 1987 gives the basic wind speeds averaged over a short interval of 3 s and having a return period of *50* years at a height of 10 m above ground level in different parts of the country. The entire country is divided into six wind zones.

Wind pressure at any height of a structure depends on the following.

a) The velocity and density of the air,
b) The height above ground level,
c) The shape and aspect ratio of the building,
d) The topography of the surrounding ground surface,
e) The angle of wind attack, and
f) The solidity ratio or openings in the structure.

Depending on the above factors, the wind can create positive pressure or negative pressure (suction) on the sides of the building.

4.2.3.1 Design Wind Speed

The design wind speed is obtained from the basic wind speed after modifying it to include risk level, terrain roughness, height and size of structure, and local topography as (per draft IS 875)

$$V_z = V_b\, k_1\, k_2\, k_3\, k_4 \qquad ..(4.2)$$

Where V_z is the design wind speed at any height z in meters per second, V_b is the basic wind speed. k_1 is the probability factor or risk coefficient, k_2 is the terrain, height, and structure size factor, k_3 is the topography (ground contours) factor and k_4 is the importance factor for the cyclonic region.

Probability factor or risk coefficient (k_1):

The probability factor *(k_1)* is mainly based on basic wind speed and class of structure, that is, life of structure. The risk coefficient for different classes of structures is given in (Table 1, IS 875, part 3 -1987). For important structures, the risk coefficient is high and for temporary sheds, it is low.

Terrain and height factor (k_2):

IS 875 (Part 3) gives the values of k_2 (Table 2, IS 875, part 3 -1987) by which the basic wind speed has to be multiplied to obtain the wind speed at different heights in each terrain category. It has to be noted that the velocity profile for a given terrain category does not develop to full height immediately with the commencement of that terrain category but changes gradually in a terrain over *a fetch distance. Fetch* is the distance at which the wind profile stabilizes for a given terrain. The relation between fetch and developed height is given in (Table 3. IS 875, part 3 -1987). A method to calculate the velocity profile for structures exceeding the developed height is given in Appendix B of the code.

Topography factor (k_3):

The basic wind speed V_b does not take into account local topographic features such as hills, valleys, cliffs, escarpments, or ridges, which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs escarpments, or ridges, and decelerate the wind in valleys or the near the foot or cliffs, steep escarpments, or ridges. The value of k_3 for level ground or when upwind slope is less than 3° may be taken as 1.0. For slopes greater than 3^o. it is confined in the range 1.0-1.36. A method of evaluating the value of k_3 for the value grater then 1.0 is given is the Appendix C (IS 875, part3 -1987).

Importance factor for cyclonic regions (k_4):

In order to ensure greater safety of structures located within regions 60 km wide of the east coast as well as on the Gujarat coast (where the wind speeds given in are often exceeded (> 70 m/s) during cyclones), the following values of k_4 are given in the draft code.

a) For structures of post-cyclone importance (such as cyclone shelters, hospitals. schools, community buildings, communication towers, power plant structures, water tanks, etc.) $k_4 = 1.30$,
b) For industrial structures, $k_4 = 1.15$,
c) For all other structures, $k_4 = 1.0$, and
d) For non-cyclonic regions, $k_4 = 1.0$.

4.2.3.2 Design Wind Pressure:

Bernoulli's equation for streamline flow can be used to determine the local pressure at the stagnation point as a column of air strikes (at 90°) on an immovable body. Thus

$$p_z = 0.5\,\rho\,V_z^2 \qquad ..(4.3)$$

Where the p_z is the wind pressure in N/m^2 at height Z, ρ is the mass density of air (1.2 kg/m^3 at sea level and at 20°C, and V_z is the design wind speed in meters per second at height Z.

Substituting the value for mass density of air, we get

$$p_z = 0.6\,V_z^2 \qquad ..(4.4)$$

The design wind speed is obtained as [draft IS 875 (part 3)]

$$p_d = K_d\,K_a\,K_c\,p_z \qquad ..(4.5)$$

Where K_d is the wind directionality factor, K_a is the area averaging factor, K_c is the combination factor.

Wind directionality factor (K_d):

Considering the randomness in the directionality of wind and recognizing that the pressure or force coefficients are determined for specific directions, this factor has been specified and accounts for a reduced probability of

a) Maximum winds coming from any given direction, and
b) Maximum pressure coefficient occurring for any given direction

For circular, near-circular, and axisymmetric sections, which offer a uniform resistance. Irrespective of the wind, the value of K_d is taken as 1.0. For cyclone affected regions also, the same value of K_d is adopted. For all other buildings, solid or open signs, lattice frameworks, and trussed towers (triangular, square, and rectangular),K_d is taken as 0.9.

Area averaging factor (K_a):

Pressure coefficients are often obtained by averaging pressures over specified tributary areas. It is well known that the correlation decreases with increasing .area. Hence an area averaging factor K_a, which is a reduction factor, is specified, as given in Table 4.2. K_a is taken as 1 .0 while considering local pressure coefficients.

Table 4.2 area averaging factor (Ka)

Tributary area, A (m^2)	Area averaging factor
≤ 10	1.0
10-25	0.9
≥ 100	0.8

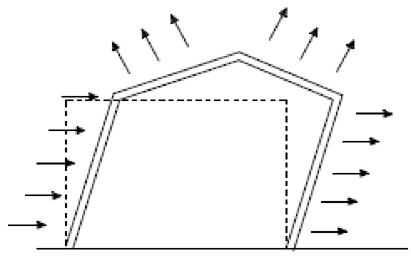
Combination factor *(K_c) :*

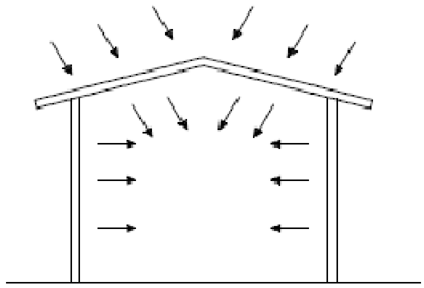
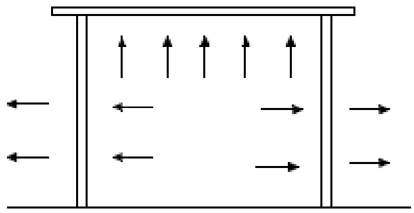
While calculating wind loads on frames of clad buildings, it is reasonable to assume that the pressures or suctions over the entire structure will not be fully correlated.

Thus, when considering the combined effect of wind loads on the frame, the forces obtained in the frame may be reduced by using the factor K_a. This factor should be taken as per table 4.3

The code recommends that for offshore structures situated at a distance of up to 200 km off the coast, the wind speed may be taken as 1.5 times the value on the neatest coast.

Table 4.3 combination factors for wind pressure contributed from two or more building surfaces.

Design case	Combination factor (K_c)	Example diagrams
a) Where wind action from any single surface contributes 75% or more to an action effect	1.0	-
b) pressure from wind-ward and leeward walls in combination with positive or negative roof pressures	0.8	

c) Positive pressures on roofs in combination with negative internal pressures (from a wall opening)	0.8	
d) Negative pressures on roofs or walls in combination with positive internal pressures.	0.95	
e) All other cases	1.0	

4.2.3.3 Wind Pressure on Roofs:

Wind pressure acting normal to the individual element or cladding unit is given by

$$F = \left(C_{pe} - C_{pi}\right) A\, p_d \qquad ..(4.6)$$

Where *F* is the net wind force on the element, *A* is the surface area of the element or cladding. C_{pc} is the external pressure coefficient, C_{pi} is the internal pressure coefficient, and p_d is the design wind pressure.

If the surface design pressure varies with height, the surface area of the structural element may be subdivided so that the specified pressures are taken over appropriate areas. The wind pressure coefficients depend on:

a) The shape of the building or the roof,
b) The slope of the roof,
c) The direction of the wind with respect to the building, and
d) The zone of the building

The external pressure coefficient (C_{pe}) for different roofs and internal pressure coefficient (C_{pi}) are given in IS 875 (part3) -1987.

4.2.4 Crane Loads:

Dynamic forces (vibration, shock, acceleration, retardation, and impact) are of importance in the design of overhead travelling cranes, or lifts, which are found in industrial buildings and tall buildings. The impact loads they cause, although not high, should be properly taken care of in the design. Impacts due to vertical crane loads are converted empirically into equivalent static loads through an impact factor, which is normally a percentage of the crane load. Table 4.4 shows the impact factors as suggested by the IS 875 code for cranes and lifts. Thus, if the impact 25%, the live load is multiplied in the calculation of the forces by 1.2

Table 4.4 Additional impact loads on buildings

Structure	Impact allowance in percentage
Lift and hoists	
a) Frames	100
b) Foundation	40
Reciprocating machinery – frames and foundation	50
Light machinery – structure and foundation	20
Electric overhead cranes	
a) Girder	25
b) Columns (class III and IV cranes)	25
c) Columns (class I and II cranes)	10
d) Foundation	0
Hand-operated cranes	
a) Girder	10
b) Columns and foundations	0

4.2.5 Load Combination for Design:

The earthquake loads are not critical in the design of industrial building, since the weight of the roof is not considerable. Hence the following combinations of loads are considered.

1) 1.5 (DL + IL) + 1.05(CL or SL),
2) 1.2 (DL + IL) + 1.05(CL or SL) ± 0.6 WL,
3) 1.2 (DL + IL ± WL) + 0.53 (CL or SL),

4) 1.5 (DL ± WL),
5) 0.9 DL ± 1.5 WL,
6) 1.2 (DL + ER),
7) 0.9 DL + 1.2 ER, and
8) DL + 0.35 (IL + CL + or SL)

Where DL = dead load, IL = imposed load, WL= wind load, SL = snow load, CL= crane load, ER= erection load.

4.3 Design of Truss Member:

The members of the trusses are made of either rolled steel sections or built-up sections depending upon the span length and intensity of loading. Rolled steel single or double angles, T-sections, hollow circular, square, or rectangular section are used in the roof trusses of industrial buildings. In long-span roof trusses and short span bridges, heavier rolled steel sections, such as channels and I-sections are used. Access to the surface of the members for inspection, cleaning, and repainting during service are important considerations while using built-up sections. Hence in highly corrosive environments, fully closed welded box-sections or hollow sections are used, with their ends fully sealed to reduce the maintenance cost and improve the durability of the trusses.

The various steps involved in the design of truss members are as follows:

1. Depending upon the span, required lighting, and available roofing material, the type of truss is selected and a line diagram of the truss is sketched.
2. Various loads acting over the truss are calculated using IS 875 (Parts 1-5).
3. The purlins are designed and the loads acting on the truss at the purlin points are computed.
4. The roof truss is analysed for the various load combinations using the graphical method or the method of sections or joints or by a computer program and the forces acting on the members for various combinations are tabulated.
5. Each member may experience a maximum compressive or tensile force (called the design force) under a particular combination of loads. Note that a member which is under tension in one loading combination may be

subjected to reversal of stresses under some other loading combination. Hence, the member has to be designed for both maximum compression and maximum tension and the size for the critical force has to be adopted. The principal rafter is designed as continuous strut and the other compression members are design a discontinuous struts.

6. When purlins are placed at intermediate points, i.e., the nodes of the top chord, the top chord will be subjected to bending moment in addition to axial compression. Since the rafter is a continuous member, the bending moments may be computed by any suitable method. Then the member is designed for combined bending and axial compression.
7. The members meeting at a joint are so proportioned that their centroidal axes intersect at the same point, in order to avoid eccentricity. Then the joints of the trusses are designed either as bolted or as welded joints. If the joint is constructed with eccentricity, then the members and fasteners must be designed to resist the moment that arises. The moment at the joint is divided between the members in proportion of their stiffness.
8. The maximum deflection of the truss may be computed by using either strain energy method or matrix stiffness analysis program. A computer analysis gives the value of deflection as part of the output. This deflection should be less than that specified in Table 4.5.
9. The detailed drawings and fabrication drawings are prepared and the material take-off is worked out.
10. The lateral bracing members are then designed. When a cross braced wind girder is used, it is necessary to use a computer analysis program, since the truss will be redundant.

4.3.1 Design Criteria for Tension Member :(Section 6, IS 800-2007)

Steel tension members are probably the most common and efficient members in the structural applications. They are those structural elements that are subjected to direct axial tensile loads, which tend to elongate the members. A member in pure tension can be stressed up to and beyond the yield limit and does not buckle locally or overall. Hence, their design is not affected by the classification of section. For example, compact, semi-compact, etc.

The factored design tension *T,* in the members shall satisfy the following requirement:

$$T < Td \qquad ..(4.7)$$

Where,

Td = design strength of the member.

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of gross section, T_{dg} rupture strength of critical section; T_{dn} and block shear T_{db}

Table 4.5 Deflection limits

<table>
<tr><th>Type of building</th><th>Deflection</th><th>Design load</th><th>Member</th><th>Supporting element</th><th>Maximum deflection</th></tr>
<tr><td rowspan="10">Industrial building</td><td rowspan="8">Vertical</td><td>Live load/ Wind load</td><td>Purlin and girts</td><td>Elastic cladding</td><td>Span/ 150</td></tr>
<tr><td>Live load/ Wind load</td><td>Purlin and girts</td><td>Brittle cladding</td><td>Span/ 180</td></tr>
<tr><td>Live load</td><td>Simple span</td><td>Brittle cladding</td><td>Span/300</td></tr>
<tr><td>Live load</td><td>Cantilever</td><td>Brittle cladding</td><td>Span/ 150</td></tr>
<tr><td>Live load</td><td>Simple span</td><td>Elastic cladding</td><td>Span/240</td></tr>
<tr><td>Live load</td><td>Cantilever</td><td>Elastic cladding</td><td>Span/ 120</td></tr>
<tr><td>Live load/ Wind load</td><td>Rafter supporting</td><td>Profiled metal sheeting</td><td>Span/ 180</td></tr>
<tr><td></td><td></td><td>Plastered sheeting</td><td>Span/240</td></tr>
<tr><td rowspan="2">Lateral</td><td>No cranes</td><td>Column</td><td>Elastic cladding</td><td>Height/150</td></tr>
<tr><td>No cranes</td><td>Column</td><td>Masonry /brittle cladding</td><td>Height/240</td></tr>
</table>

4.3.1.1 Design Strength Due to Yielding of Gross Section :(Cl 6.2, IS 800-2007)

Generally a tension member without bolt holes can resist loads up to the ultimate load without failure. But such a member will deform in the longitudinal direction considerably (nearly 10%-15% of its original length) before fracture. At such a large deformation a structure becomes unserviceable. Hence, one of the limiting values in design strength is the one corresponding to the yielding of the gross cross section, calculated as follows

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} \qquad ..(4.8)$$

Where A_g *is* the gross area of cross section in mm^2, f_y is the yield strength of the material (in MPa), and γ_{m0} is the partial safety factor for failure in tension by yielding *(*γ_{m0} = 1.10 as per IS 800).

4.3.1.2 Design Strength Due to Rupture of Critical Section :(Cl 6.3, IS 800-2007)

A tension member is often connected to the main or other member by bolts or welds. When connected using bolts, tension members have holes and hence reduced cross section, being referred to as the *net area.* Holes in the members cause stress concentration at service loads. From the theory of elasticity, we know that the tensile stress adjacent to a hole will be about two to three times the average stress on the net area, depending upon the ratio of the diameter of the hole to the width of the plate normal to the direction of stress. The ratio of the maximum elastic stress to the average stress (f_{max}/f_{avg}) is referred to as the stress-concentration factor and a plot of the stress-concentration factor with the ratio of the hole diameter r to the net width b_n. Stress concentration becomes very significant when repeated applications of load may lead to fatigue failure or when there is a possibility of a brittle fracture of a tension member under dynamic loads. Stress concentration may be minimized by providing suitable joint and member details.

a) Plates:

The design strength in tension of a plate, T_{dn} as governed by rupture of net cross-sectional area, A_n at the holes is given by

$$T_{dn} = 0.9\, A_n\ f_y/\gamma_{m1} \qquad ..(4.9)$$

Where

γ_{m1} = partial safety factor for failure at ultimate stress *(see* Table 5),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

$$A_n = \left[b - nd_h + \textstyle\sum_i \frac{P_{si}^2}{4g_i}\right] \qquad ..(4.10)$$

Where

b, t = width and thickness of the plate, respectively,

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),

g = gauge length between the bolt holes,

p_s = staggered pitch length between line of bolt holes,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

b) Single angle:

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn} as governed by rupture at net section is given by:

$$T_{dn} = 0.9\, A_{nc} \frac{f_u}{\gamma_{m1}} + \beta\, A_{go} \frac{f_y}{\gamma_{m0}} \qquad ..(4.11)$$

Where

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(f_u \frac{\gamma_{m0}}{f_y \gamma_{m1}}\right) \qquad ..(4.12)$$

$$\ge 0.7$$

Where

w = outstand leg width,

b_s = shear lag width, and

Lc = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction.

4.3.1.3 Design Strength Due to Block Shear :(Cl 6.4, IS 800-2007)

The block shear phenomenon becomes a possible mode of failure when the material bearing strength and bolt shear strength are higher. When high bearing strength of material and high-strength bolts are used, only a few bolts are required in the connection. Decreasing the number of bolts per connection results in a smaller connection length; however, the possibility of block shear failure increases. As indicated earlier, the appropriate model of the block shear failure is the rupturing of the net tension plane and yielding on the gross shear plane, which results in rupturing of the shear plane as the connection lengths become shorter. It is also possible to have rupture of the shear area and yielding of the tension area governing the block shear strength.

$$T_{db} = \left[A_{vg}\frac{f_y}{(\sqrt{3}\gamma_{m0})} + 0.9\,A_{tn}\frac{f_u}{\gamma_{m1}}\right] \qquad ..(4.13)$$

or

$$T_{db} = \left[0.9\,A_{vn}\frac{f_u}{(\sqrt{3}\,\gamma_{m1})} + A_{tg}\frac{f_y}{\gamma_{mo}}\right] \qquad ..(4.14)$$

where A_{vg} and A_{vn} are the minimum gross and net area in shear along a line of transmitted force, respectively; A_{tg} and A_{tn} are the minimum gross and net area in tension from the hole to the toe of the angle or next last row of bolt in plates, perpendicular to the line of force respectively and f_u *and* f_y are the ultimate and yield stress of the material, respectively.

4.3.2 Design of Compression Member: (Section 7, IS 800-2007)

The code (IS 800) gives the following recommendations for compression members in trusses.

a) In the case of bolted, riveted, or welded trusses, the effective length KL of the compression member will be taken as 0.7 to 1.0 times the distance between the centers of connection, depending on the degree of end restraint provided.

b) In the case of members of trusses, for buckling in the plane perpendicular to the plane of the truss, the effective length may be taken as the distance between the centers of intersection.
c) In the case of double angle discontinuous struts, connected back-to-back on opposite sides of the gusset by not less than two bolts/rivets in line along the length of the angle, at each end or by equivalent welding, the value of K may be taken as 0.7 to 0.85, depending on the restraint provided. In the plane perpendicular to the truss, a value of K = 1.0 is recommended.
d) For double angle discontinuous struts connected back-to-back to one side of a gusset or other members by one or more bolts/rivets in each angle or

by equivalent weld length, the same *K* values as above are recommended (i.e., *K* = 0.7-0.85 for in-plane and *K*= 1.0, for out-of-plane buckling).

e) For double angle continuous struts such as those forming the flanges, chords, or ties of trusses or trussed girders, or the legs of towers, values of *K* from 0.7 to 1.0 may be chosen depending on the end restraint provided.

For single angle discontinuous struts, the code does not give any recommendation for the effective length factor. Hence, the effective length factor of single angle discontinuous struts connected by single or more bolts/rivets or equivalent welding may be taken as 1.0 and the design be carried out.

4.3.2.1 Design of Compressive Members:

The strength of compression members is based on its gross area A_g. The strength is always a function of the effective slenderness ration KL/r, and for short columns the yield stress f_y of the steel. Since the radius of gyration *r* depends on the section selected, the design of compression members is an iterative process, unless column load tables are available. The usual design procedure involves the following steps.

1. The axial force in the member is determined by a rational frame analysis, or by statics for statically determinate structures. The factored load P_u is determined by summing up the specified loads multiplied by the appropriate partial load factors γ_f as given in (table 5, IS 800-2007).

2. Select a trial section. Note that the width/thickness limitations as given in (table-2, IS 800-2007) to prevent local buckling must be satisfied (most of the rolled sections satisfy the width-to-thickness ratios specified in Table 2). If it is not satisfied and a slender section is chosen, the reduced effective area A_{cff} should be used in the calculation. The trial section may be chosen by making initial guesses for A_{eff}/A, and f_{cd} (say 0.4-0.6 f_y.) and calculating the target area *A.*

The following member sizes may be used as a trial section:

a) *Single angle size* 1/30 of the length of compression member
b) *Double angle size* 1/35 of the length of compression member
c) *Circular hollow section* diameter = 1/40 of length

The slenderness ratios as given in Table 4.6 will help the designer to choose the trail sections.

Table 4.6 Slenderness ratios to be assumed while selecting the trial sections

Type of member	Slenderness ration (*L/r*)
Single angles	100-150
CHS, SHS, RHS	90-110
Single channels	90-150
Double angles	80-120
Double channels	40-80
Single I-section	80-150
Double I-sections	30-60

3. Compute KL/r for the section selected. The computed value *of* KL/r should be within the maximum limiting value given in (Table 3, IS 800-2007). Using (Fig.8, IS 800-2007).

4. Compare P_d with P_u. When the strength provided does not exceed the strength required by more than a few per cent, the design would be acceptable; otherwise repeat steps *2* through 4.

5. GANTRY GIRDER

5.1 Overview

This section deals with the crane buildings, and will include coverage of those aspects of industrial building peculiar to the existence of overhead and under hung cranes. The major difference between crane building and other industrial buildings is the frequency of loading

The function of the crane girders is to support the rails on which the travelling cranes move. These are subjected to vertical loads from crane, horizontal lateral loads due to surge of the crane, that is, the effect of acceleration and braking of the loaded crab and swinging of the suspended load in the transverse direction, and longitudinal force due to acceleration and braking of the crane as a whole. In addition to the weight of the crane, impact and horizontal surge must be considered. According to IS: 875, the values given in Table 13-9 may be taken for the design of crane gantry girders and columns. Both the horizontal forces, lateral and longitudinal, are assumed not to act together with the vertical loads simultaneously. Only one of them is to be considered acting with the vertical load at a time. Vertical load, of course, includes the additional load due to impact.

All classes of cranes are affected by the operating conditions so for the purpose of these definitions it is assumed that the crane will be operating in normal ambient temperatures and normal atmospheric conditions. The classifications for overhead traveling cranes are as below.

CLASS A (Standby or infrequent Service):
Precise handling of equipment at slow speeds with long idle periods between lifts. Typical installations are motor rooms, transformer stations, turbine rooms, and powerhouses. Rated-capacity loads may be handled for initial installation of equipment and for infrequent maintenance.

CLASS B (Light Service):
This service covers cranes which may be used in repair shops, light assembly operations, service buildings, and light warehouses, where service requirements are light and speed is slow. Loads may vary from no load to occasional full-rated load, with two to five lifts per hour.

CLASS C (Moderate Service):

Application in machine shops or other facilities where service is moderate. Crane handles loads that average 50 per cent of rated capacity. About 5 to 10 lifts are handled per hour, averaging 15 lift, with no more than 50 percent of the lifts being at rated capacity.

CLASS D (Heavy Service):

Lifting in heavy machine shops, foundries, fabrication plants, steel warehouses, and standard-duty bucket and magnet operations where heavy-duty production is required. Loads approaching 50 percent of the rated capacity are handled constantly during the working period. High speeds are involved, with 10 to 20 lifts than 65 percent of the lifts being are rated capacity.

CLASS E (Severe Service):

Loads approach rated capacity throughout the life of the crane. Applications include magnet, bucket, and magnet or bucket combination cranes for scrap yards, cement mills, lumber mills, and fertilizer plants. Application involves 20 or more lifts per hour, at or near rated capacity.

CLASS F (Continuous Severe Service):

Loads approach rated capacity continuously, under severe service conditions, throughout the life of the crane. Application include custom-designed specially cranes essential to performing critical work tasks affecting the total production facility. Cranes typically require high reliability critical work tasks affecting the total production facility. Cranes typically require high reliability, together with ease of maintenance. Cranes for steel mills are in this category.

Gantry girders are laterally unsupported beams. Overhead traveling cranes are used in industrial buildings to lift and transport heavy jobs, machines, and so on, from one place to another. The crane may be manually operated overhead traveling crane or an electrically operated overhead traveling crane. To facilitate movement, wheels are attached to the ends of crane girders. A trolley (crab) with wheels and a suspended hook is placed over the crane and this arrangement can move in the transverse direction. However, it should be noted that the two movements of the crane, the longitudinal and transverse cannot be had simultaneously.

The section of gantry girder shown in Figure.5.1 (a) is used for light cranes and moderate spans. In order to increase lateral stability and the torsional rigidity, the channel section is connected at the top as shown in Figure.5.1 (b). The channel sections are also attached with web instead of flange as shown in Figure.5.1 (c). In order to improve the torsional rigidity, bracket plates are also used. The plate girder section shown in Figure.5.1 (d) is used for long ' gantry girder. The plate girder section is also used for moderate span and for heavy loads.

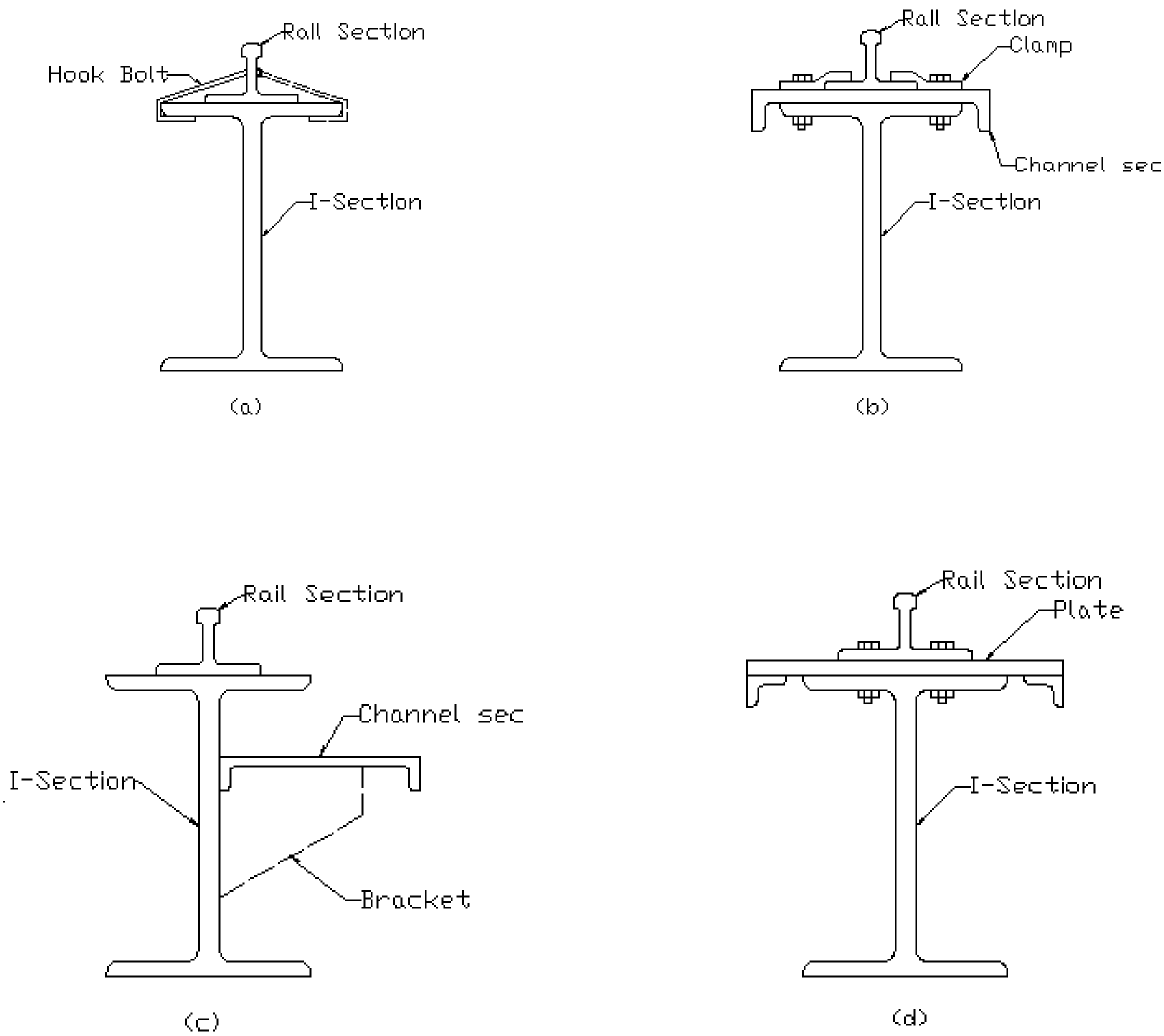


Fig 5.1 Various Cross Section of Gantry Girder

When two or more cranes are placed on one gantry, the gantry girder should be designed with the assumption that the crane will be running buffer to buffer, lifting or carrying their maximum loads simultaneously. If the load is applied above the neutral axis (at the top flange of the beam), lateral torsional buckling resistance is reduced. In addition, the lateral loads from the crane system are applied at the top flange level, generating a twisting moment on the beam. When vertical and lateral loads are applied simultaneously, these two effects are cumulative. To compensate for this, it is common practice to assume the lateral loads due to the twisting moment are resisted by only the top flange.

5.2 Various Loads on Gantry Girder:

The loads acting on the gantry girder are shown in figure 5.2 as follows.

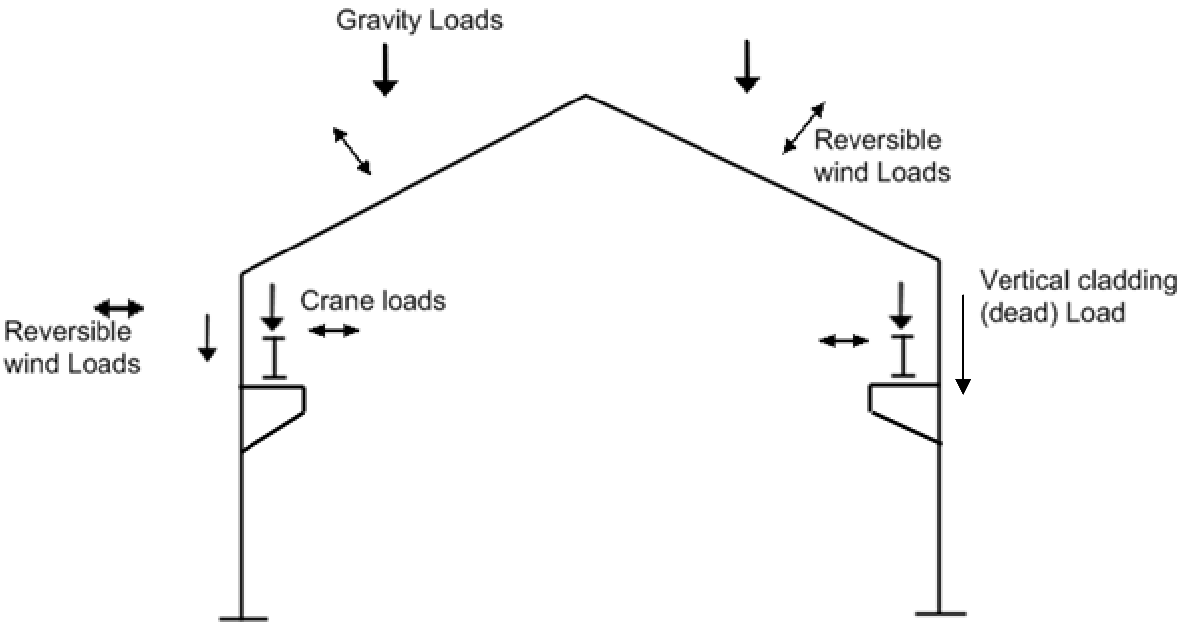


Fig 5.2 Various Loads on Gantry Girder

Gantry girders are laterally unsupported beams subjected to impact loads. The gantry girder undergoes asymmetrical bending due to the forces discussed as follows.

i. The reaction from the crane girder, acting vertically.
ii. The longitudinal thrust, due to staring or stopping of crane, acting in the longitudinal direction.
iii. The lateral thrust, due to starting or stopping of the crab acting horizontally, normal to the gantry girder.

Vertical Loads:

Vertical load acting over the gantry girder is the reaction from the crane girder and consists of the self weight of the crane, self weight of the crab and the crane capacity.

Lateral Loads:

Lateral loads are caused due to sudden stopping or starting of the crab. These cause a thrust applied to the top of the rail and normal to the rails direction and shall be distributed with due regard to the lateral stiffness of the structure supporting the rails. Horizontal forces exist in crane loading due to number of factors including:

i. Runway misalignment
ii. Crane skew
iii. Trolley acceleration
iv. Trolley braking
v. Crane steering

Longitudinal or Tractive Loads:

Longitudinal loads are caused due to the stopping or starting of the crane girders, and produce a thrust along the rails. The lateral and longitudinal thrusts are transferred at the top of the rail level. Therefore, gantry girders are also subjected to bending moment due to these loads.

Load effect due to Impact and Vibration:

The crane loads to be considered under imposed loads shall include by vertical load, impact factors, lateral and longitudinal braking forces acting across and along the crane rails respectively.

Impact allowance for Lifts, Hoists and Machinery:

The imposed loads shall be assumed to be including adequate allowance for ordinary impact conditions. For structures carrying loads which induce impact or vibration, as far as possible, calculations shall be made for increase in the imposed load, due to impact or vibration.

Composed Imposed Loads with Impact and Vibration:

Concentrated imposed loads with impact and vibration which may be due to installed machinery shall be provided for in the design. The impact factor shall not be less than 20 percent which is the amount allowable for light machinery.

Impact Allowance for Crane Girders:

For crane gantry girders and supporting columns, the following allowances shall be deemed to cover all forces set up by vibration, shock from slipping or slings, kinetic action of acceleration, and retardation and impact of wheel loads. The impact allowances for various loads are shown in Table 5.2.

Table 5.1 Impact Allowance for Crane Girders

Type of Load	Additional Load
a) Vertical loads for electric overhead cranes	25 percent of maximum static loads for crane girders for all classes of cranes 25 percent for columns supporting Class C and class D cranes 10 percent for columns supporting Class A and Class B cranes No additional load for design of foundations
b) Vertical loads for hand operated cranes	10 percent of maximum wheel loads for crane girders only
c) Horizontal forces transverse to rails:	
1) For electric overhead cranes with trolley having rigid mast for suspension of lifted weight (such as soaker crane, stripper crane, etc.)	-10 percent of weight of crab and the weight lifted by the cranes, acting on any one crane track rail. Acting in either direction and equally distributed amongst all the wheels on one side of rail track. For frame analysis this force shall be applied on one side of the frame at a time in either direction.
2) For all other electric overhead cranes and hand operated cranes	-5 percent of weight of crab and the weight lifted by the cranes, acting on anyone crane track rail, acting in either direction and equally distributed

	amongst the wheels on one side of rail track For the frame analysis, this force shall be applied on one side of the frame at a time in either direction
d) Horizontal traction forces along the rails for overhead cranes, either electrically operated or hand operated.	-5 percent of all static wheel loads

Lateral surge:

For design of columns and foundation, supporting crane girders, the following crane combination shall be considered.

a. Effect of one crane in the bay giving the worst effect shall be considered for calculation of surge force in case of single bay frame.
b. Effect of two cranes working one each in any of two bays in the cross section to give the worst effect shall be considered for calculation of surge force in case of multi bay frames.

Tractive force:

a. Where one crane is in operation with no provision for future crane, tractive force from only one crane shall be taken.
b. Where more than one crane is in operation or there is provision for future crane, tractive force from two cranes giving maximum effect shall be considered.

Lateral surge force and longitudinal tractive force acting across and along the crane rail respectively, shall not be assumed to act simultaneously. However, if there is only one crane in the bay, the lateral and longitudinal forces may act together with vertical loads.

5.3 Design of Gantry Girder:

The design of gantry girder subjected to lateral loads is a trial and error process. The various steps involved in the design are given below.

1. The first step is to find the maximum wheel load. This load is maximum when the trolley is closest to the gantry girder.

2. The maximum bending moment in the gantry girder due to vertical loads needs to be computed. This consists of the bending moment due to the maximum wheel loads (including impact) and the bending moment due to the dead load of the girder and rails. The bending moment due to dead loads is maximum at the centre of the girder, whereas the bending moment due to the wheel load is maximum below one of the wheels.
3. Next the maximum shear force is calculated. This consists of the shear force due to wheel loads and dead loads from the gantry girder and rail. Generally an I-section with a channel section is chosen with a plate at the top flange may be used for light cranes. When the gantry is not laterally supported, the following may be used to select a trail section:

$$Z_p \ = \ M_u \ / \ f_y \qquad \text{... (5.1)}$$

$$Z_p \ (trial) \ = \ kZ_p \ (k = \ 1.40 - 1.50) \qquad \text{... (5.2)}$$

   Generally, the economic depth of a gantry girder is about (1/12)th of the span. The width of the flange is chosen to be between (1/40) and (1/30)th of the span to prevent excessive lateral deflection.
4. The plastic section modulus of the assumed combined section is found out by considering a neutral axis which divides the area in two equal parts, at distance *y* to the area centroid from the neutral axis. Thus

$$M_p = \ 2f_y A/2\bar{y} \ = A\bar{y}f_y \qquad \text{... (5.3)}$$

   Where *Ay* is equal to the plastic modulus Z_p.
5. When lateral support is provided at the compression (top) flange, the chosen section should be checked for the moment capacity of the whole section (clause 8.2.1.2 of IS 800):

$$M_{dz} = \beta_b Z_p f_y/\gamma_{mo} \leq 1.2 \ Z_e \ \ f_y/\gamma_{mo} \qquad \text{... (5.4)}$$

   The above value should be greater than the applied bending moment. The top flange should be checked for bending in both the axes using the interaction equation

$$(M_y/M_{ndy}) \ + \ (M_z/M_{ndz}) \ \leq \ 1.0 \qquad \text{... (5.5)}$$

6. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local buckling and, if necessary, load-carrying stiffeners must be introduced to prevent local buckling of the web.

7. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local crushing, if necessary, bearing stiffeners should be introduced to prevent local crushing of the web.
8. The maximum deflection under working loads has to be checked.

Table 5.2 Deflection Criteria for Cranes

Types of cranes	Limiting deflection
Where the cranes are manually operated	$\frac{L}{500}$
Where the cranes are traveling overhead and operated electrically up to 500 kN	$\frac{L}{750}$
Where the cranes are traveling overhead and operated electrically over 500 kN	$\frac{L}{1000}$
Other moving loads, such as charging cars, etc.	$\frac{L}{600}$

5.4 Crane Load combination:

Case 1. This case applies to load combinations for members designed for repeated loads. The stress range shall be based on one crane (in one aisle only-where aisle represents the zone of travel of a crane parallel to its runway beams) including full vertical impact, eccentric effects and 50% of the side thrust.

Case 2. All dead and live loads, including roof live loads plus maximum side thrust of crane. If specific conditions warrants, longitudinal traction from one crane, plus all eccentric effects and one of the following vertical crane loadings:

i. Vertical load from one crane including full impact.
ii. Vertical load induced by as many cranes as may be positioned to affect the member under consideration, not including impact.

Case 3. All dead and live loads including impact from one crane plus one of the following:

i. Full wind with no side thrust but with one crane positioned for maximum vertical load effects.
ii. Fifty percent of full wind loads with maximum side thrust and vertical load effects from one crane.

iii. Full wind with no live load or crane load.
iv. Bumper impact at end of runway from one crane.
v. Seismic effect resulting from dead loads of the cranes parked in each aisle positioned for maximum seismic effects.

Other load combinations that have been used by engineers include:

i. Two adjacent cranes working in tandem with full load and with impact allowance of crane girders for vertical load of electric overhead cranes.
ii. for long span gantries, where more than one crane can come in the span, the girder shall be designed for one crane fully loaded with impact allowance of crane girders for vertical load of electric overhead cranes plus as many loaded cranes as can be accommodated on the span but without taking into account overloading to give maximum effect.

6. DESIGN OF BEAM

6.1 Introductions

One of the frequently used structural members is a beam whose main function is to transfer load principally by means of flexural or bending action. In a structural framework, it forms the main horizontal member spanning between adjacent columns or as a secondary member transmitting floor loading to the main beams. Normally only bending effects are predominant in a beam except in special cases such as crane girders, where effects of torsion in addition to bending have to be specifically considered.

The types of responses of a beam subjected to simple uniaxial bending are shown in Table 6.1. The response in a particular case depends upon the proportions of the beam, the form of the applied loading and the type of support provided. In addition to satisfying various strength limits as given in the Table 6.1, the beam should also not deflect too much under the working loads i.e. it has to satisfy the serviceability limit state also.

Recently, IS: 800, the structural steel code has been revised and the limit state method of design has been adopted in tune with other international codes of practice such as BS, EURO, and AISC. This chapter attempts to throw light on the provisions for bending members in this code.

6.2 Limit state design of beams

In the working stress or allowable stress method of design, the emphasis is on limiting a particular stress in a component to a fraction of the specified strength of the material of the component. The magnitude of the factor for a structural action depends upon the degree of safety required. Further, elastic behaviour of the material is assumed. The main limitation to the permissible stress method is that the stress safety factor relating the permissible stress to the strength of the material is not usually the same as the ratio of the strength to the design load. Thus it does not give the degree of safety based on collapse load.

In the limit state method, both collapse condition and serviceability condition are considered. In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur throughout its life. Designs should ensure that the structure does not become unfit for the use for which it is required. The state at which the unfitness occurs is called a limit state. Special features of limit state design method are:

1. It is possible to take into account a number of limit states depending upon the particular instance
2. This method is more general in comparison to the working stress method. In this method, different safety factors can be applied to different limit states, which is more rational than applying one common factor (load factor) as in the plastic design method.
3. This concept of design is appropriate for the design of structures since any new knowledge of the structural behaviour, loading and materials can be readily incorporated.

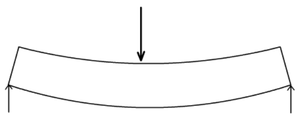
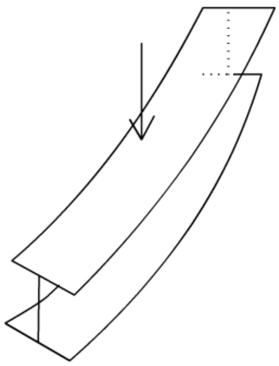
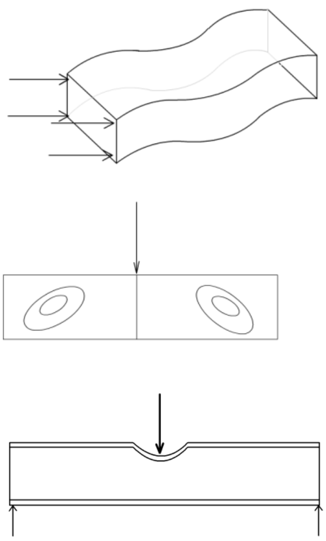
The limit state design method is essentially based on the concept of probability. Its basic feature is to consider the possibility and probability of the collapse load. In this respect, it is necessary to consider the possibility of reduced strength and increased load.

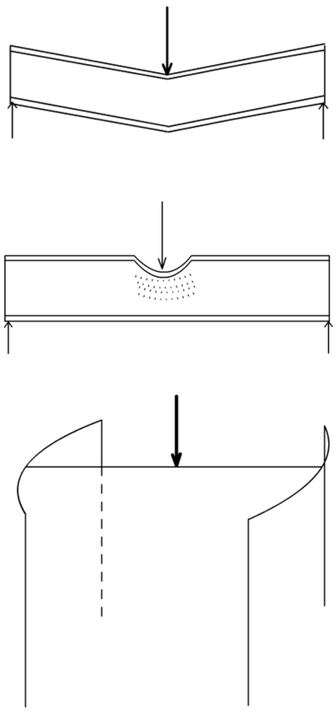
The object of design is to keep an acceptable level the probability of any limit state not being exceeded. This is achieved by taking account of the variation in strength and properties of materials to be used and the variations in the loads to be supported by the structure, by using the characteristic values of the strength of materials as well as the loads to be applied. The deviations from the characteristic values in the actual structures are allowed by using their design values. The characteristic values should be based on statistical evidence where necessary data are available; where such data are not available they should be based on an appraisal of experience. The design values are derived from the characteristic values through the use of partial safety factors, one for material strengths and the other for loads and load effects.

6.3 Beam Types

There are various forms of beam cross sections used in practice. The selection of a section would depend upon the use for which it is intended and on the overall economy. The beam chosen should possess required strength and it should not deflect beyond a limit. Generally thin-walled open sections are used as beams (e.g., rolled sections). This is mainly due to their capability for forming easy and convenient connections to adjacent members. If suitable rolled sections are not available, they may be fabricated out of rolled plates. Such members are called *plate girders.* Compound sections may also be fabricated out of rolled sections for special purposes. Similarly, in *hybrid sections,* the flanges of higher yield stress compared to the web can also be formed.

Table 6.1 Main failure modes of hot-rolled beams

Category	**Mode**		**Comments**
1	Excessive bending triggering collapse		This is the basic failure mode provided (1) the beam is prevented from buckling laterally,(2) the component elements are at least compact, so that they do not buckle locally. Such "stocky" beams will collapse by plastic hinge formation.
2	Lateral torsional buckling of long beams which are not suitably braced in the lateral direction.(i.e. "un restrained" beams)		Failure occurs by a combination of lateral deflection and twist. The proportions of the beam, support conditions and the way the load is applied are all factors, which affect failure by lateral torsional buckling.
3	Failure by local buckling of a flange in compression or web due to shear or web under compression due to concentrated loads		Unlikely for hot rolled sections, which are generally stocky. Fabricated box sections may require flange stiffening to prevent premature collapse. Load bearing stiffeners are sometimes needed under point loads to resist web buckling.

4	Local failure by (1) Shear yield of web (2) local crushing of web (3) Buckling of thin flanges.		Shear yield can only occur in very short spans and suitable web stiffeners will have to be designed. Local crushing is possible when concentrated loads act on unstiffened thin webs. Suitable stiffeners can be designed. This is a problem only when very wide flanges are employed. Welding of additional flange plates will reduce the plate b / t ratio and thus flange buckling failure can be avoided.

Other economical types of beams that can be fabricated out of rolled sections are the *castellated* and *tapered beams.* Table 6.2 gives the important types of steel beams used in practice with their optimum span range. (Note that open web joists and trusses do not strictly come under the classification of beams.)

Table 6.2 Beam Types

Type of beam	Optimum span range (m)	Application
Angles	3-6	For lightly loaded beams such as roof purlin and sheeting rail
Rolled I-sections	1-30	Most frequently used as a beam
Castellated beams	6-60	Long spans and light loads
Plate girders	10-100	Long spans with heavy loads such as bridge girders
Box girders	15-200	Long spans and heavy loads such as bridge girders
Open web joist (see Fig. 10.37)	4-40	Fabricated for large spans, using angles or tubes as chords and round bars for web diagonals
Trusses	10-100	Long spans and moderate loads such as industrial roofing

6.4 Effective Length

The concept of effective length of the compression flange incorporates the various types of support conditions. For the beam with simply supported end conditions and no intermediate lateral restraint, the effective length is equal to the actual length between the supports. When a greater amount of lateral and torsional restraints is provided at the supports, the effective length is less than the actual length and alternatively, the length becomes more when there is less restraint. The effective length factor would indirectly account for the increased lateral and torsional rigidities provided by the restraints. Thus, there are two K factors, K_b (restraint against lateral bending) and K_w (restraint against warping). The values of K_b and K_w are one for free bending and free warping (simply supported) and 0.5 for bending and warping prevented (fixed) support condition. The values of K_b and K_w vary with the proportions of beams and the accurate assessment of the degree of restraint provided by practical forms of connection is difficult. Hence in the code, this problem is treated in an approximate way by considering $K_b = K_w$ and treating the effective length as $L = KL$. The effective lengths KL of the compression flange for different end restraints according to IS 800:2007 are given in Table 6.3 (see also Table 15 of the code, which is based on BS 5950-1: 2000).

Table 6.3 Effective length of compression flanges

Effective length *KL* for beams, between supports	
Condition at supports	Effective length, *KL*
Compression flange at the ends unrestrained against lateral bending (free to rotate in plan)	L*
Compression flange partially restrained against lateral bending (Partially free to rotate in plane at the bearings)	0.85L*
Compression flange restrained fully against lateral bending (rotation fully restrained in plan)	0.7L*

*When the ends of the beam are not restrained against torsion, or where the loading condition is destabilizing or when flanges are free to move laterally, these values have to be increased as per Table 15 of the code.

6.5 Behaviour of steel beams

Laterally stable steel beams can fail only by (a) Flexure (b) Shear or (c) Bearing, assuming the local buckling of slender components does not occur. These three conditions are the criteria for limit state design of steel beams. Steel beams would also become unserviceable due to excessive deflection and this is classified as a limit state of serviceability.
The factored design moment, M at any section, in a beam due to external actions shall satisfy
$M \leq M_d$

Where M_d = design bending strength of the section

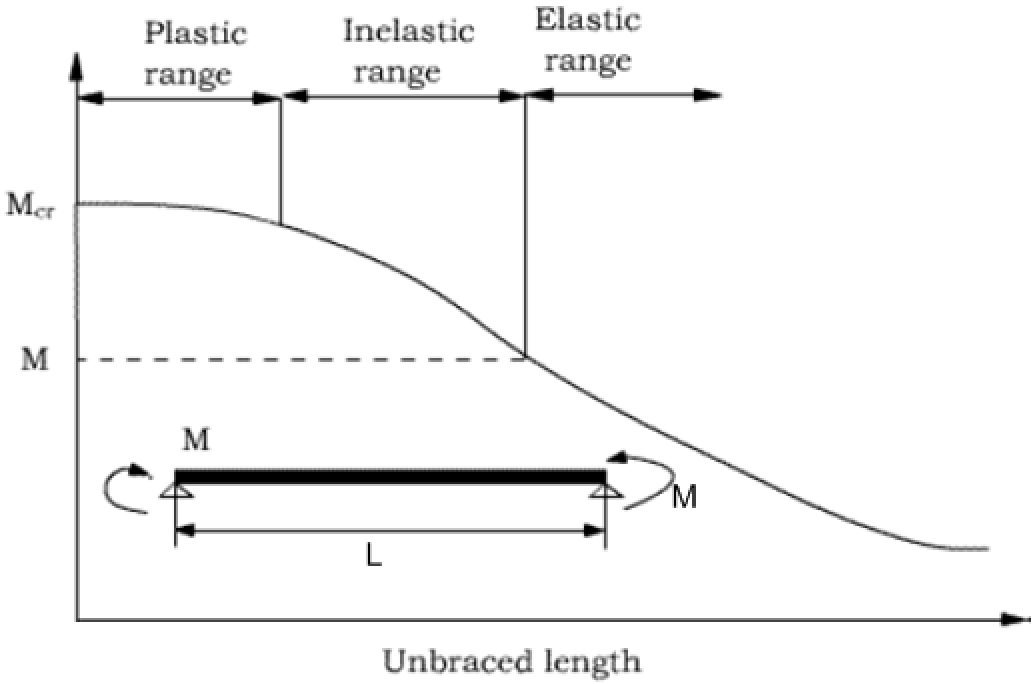


Fig 6.1 Beam buckling behaviour

6.5.1 Design strength in bending (Flexure)

This behaviour can be classified under two parts:

- When the beam is adequately supported against lateral buckling, the beam failure occurs by yielding of the material at the point of maximum moment. The beam is thus capable of reaching its plastic moment capacity under the applied loads. Thus the design strength is governed by yield stress and the beam is classified as laterally supported beam.

- Beam experiencing bending about major axis and when its compression flange is not restrained against lateral buckling, may fail by lateral torsional buckling before attaining its bending strength.Such beams are classified as laterally unsupported beam.

The behaviour of members subjected to bending demonstrated in Fig 6.1

6.5.2 Laterally supported beam

For laterally supported beams, the factored design moment M at any section in beam, due to external actions, satisfy the relationship $M< M_d$, where M_d is the design bending strength of the section. The design bending strength of a laterally supported beam is governed by the yield stress. For a laterally unsupported beam, the design strength is most often controlled by the lateral torsional buckling strength. The above relationship is obtained with the assumption that the beam web is stocky. When the flanges are plastic, compact, or semi-compact but the web is slender (i.e., $d/t_w > 67\varepsilon$), the design bending Strength may be calculated using one of the following methods.

- The flanges resist the bending moment and the axial force acting on the section and the web resists only the shear.
- The whole section resists the bending moment and the axial force and therefore the web has to be designed for combined shear and its share of normal stresses. This is done by using simple elastic theory in the case of semi-compact flanges and simple plastic theory in the case of compact and plastic flanges.

Shear force does not have any influence on the bending moment for values of shear up to 0.6 V_d (called the low shear load), where V_d is the design shear strength. When the design shear force V is less than 0.6 V_d, the design bending strength M_d will be taken as

$$M_d = \beta_b Z_p f_y/\gamma_{mo} \leq 1.2 Z_e f_y/\gamma_{mo} \quad \text{... (6.1)}$$

Where β_b = 1.0, for plastic and compact sections and $\beta_b = Z_e/Z_p$ for semi-compact sections.

Thus, for class 3 semi-compact sections,

$$M_d = Z_e f_y / \gamma_{mo} \qquad \text{... (6.2)}$$

For class 4 slender sections, $M_d = f_y' Z_e$, where f_y' is the reduced design strength for slender sections. Z_p and Z_e are the plastic and elastic section moduli of the cross section, respectively, f_y is the yield stress of the material, and is the material partial safety factor = 1.10 as per IS 800 : 2007.

The additional check $(M_d < 1.2\ Z_e\ f_y / \gamma_{mo})$ is provided to prevent the onset of plasticity under unfactored dead, imposed, and wind loads. For most of the I-beams and channels given in IS 808, Z_{pz}/Z_{ez} is less than 1.2 and hence the plastic moment capacity governs the design. For sections where $Z_{pz}/Z_{ez} > 1.2$, the constant 1.2 may be replaced by the ratio of factored load/unfactored load (γ_f). Thus the limitation $1.2 Z_e f_y$ is purely notional and becomes in practice $\gamma_f\ Z_e\ f_y$.

If the design value of the shear force is greater than 60% of the plastic design resistance in shear, a member subject to co-existing bending and shear has to use its web to resist the shear force as well as to assist the flanges in resisting moment.

6.5.3 Laterally unsupported beam

Beam experiencing bending about major axis and when its compression flange is not restrained against lateral buckling, may fail by lateral torsional buckling before attaining its bending strength. The effect of lateral torsional buckling on flexural strength need not be considered when $\lambda_{LT} < 0.4$ where λ_{LT} *is* the non-dimensional slenderness ratio for lateral torsional buckling. The design bending strength of a laterally unsupported beam as governed by lateral torsional buckling as per the Indian code (IS 800: 2007) is given by

$$M_d = \beta_b Z_p f_{bd} \qquad \text{... (6.3)}$$

With

β_b = 1.0 for plastic and compact sections.

$\beta_b = Ze/Z_p$ for semi-compact sections.

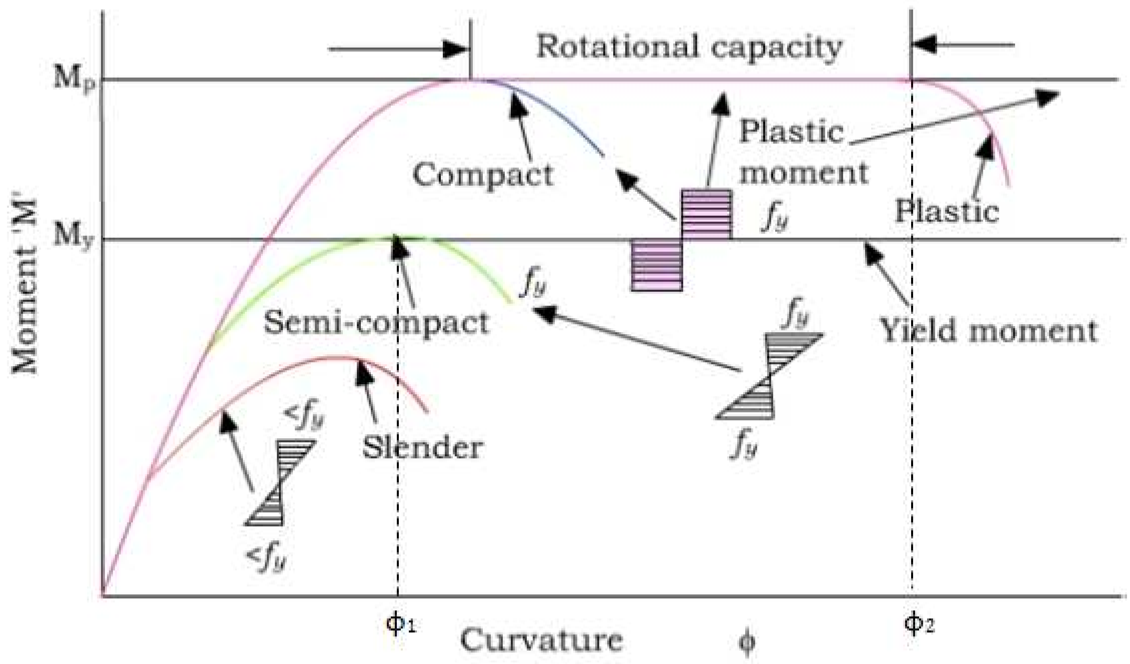


Fig 6.2 Flexual member performance using section classification

Where Z_p and Z_e are the plastic section modulus and elastic section modulus with respect to extreme compression fibre and f_{bd} is the design bending compressive stress.

The design bending compressive stress is given by

$$f_{bd} = \chi_{LT} f_y / \gamma_{mo} \quad \text{... (6.4)}$$

Where χ_{LT} is the reduction factor to account for lateral torsional buckling given by

$$\chi_{LT} = \frac{1}{\left(\phi_{LT} + \left(\phi_{LT}^2 - \lambda_{LT}^2\right)^{0.5}\right)} \leq 1.0 \quad \text{... (6.5)}$$

In which $\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

The value of imperfection factor α_{LT} for lateral torsional buckling of beam is given by

α_{LT} = 0.21 for rolled section

α_{LT} = 0.49 for welded section

γ_{m0} = partial safety factor for material=1.10

The non-dimensional slenderness ration λ_{LT} , is given by

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{\frac{1.2 Z_p f_y}{M_{cr}}} \qquad \text{...(6.6)}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}} \qquad \text{... (6.7)}$$

Where M_{cr} is the elastic critical moment and $f_{cr,b}$ is the extreme fibre compressive elastic lateral buckling stress.

The Eqns (6.3) to (6.6) have been adopted from Eurocode 3. However, Eurocode gives multiple beam curves (similar to multiple column curves). Thus, for lateral-torsional buckling curves are provided in the Eurocode (selected on the basis of the overall height-to-width ratio of the cross section, the type of cross section, and whether the cross section is rolled or welded), whereas the Indian code offers only two curves (only making a distinction between rolled and welded sections). Eurocode 3 also defines lateral torsional buckling curves for two cases: (a) the general case and (b) rolled sections or equivalent welded sections.

The elastic lateral buckling moment M_{cr} is given by

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(KL)^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(KL)^2}\right]\right\}} \qquad \text{... (6.8)}$$

Kerensky et al. (1956) simplified this expression by introducing the following approximations for doubly symmetric sections.

$$I_y = \frac{b_f^3 t_f}{6}\ ,\ I_w = \frac{I_y h^2}{4}\ ,\ I_t = 0.9 b_f t_f^3\ ,\ b_f = 4.2 r_y \ and\ E = 2.6\ G$$

Thus the equ (6.8) is reduced to

$$M_{cr} = \beta_{LT}\pi^2 EI_y h/2(KL)^2 \left[1 + 1/20\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5} \qquad \text{... (6.9)}$$

$$= \beta_b Z_p f_{cr,b}$$

where I_t is the torsional constant, I_w is the warping constant, I_y is the moment of inertia about the weak axis, r_y is the radius of gyration of the section about the weak axis, *KL* is the effective laterally unsupported *length* of the member, *h* is the overall depth of the section, t_f is the thickness of the flange, and β_{LT}=

1.20 for plastic and compact sections with $t_f/t_W \leq 2.0$ and 1.00 for semi-compact sections or sections with $t_f/t_W > 2.0$.

Using the same approximations, the extreme fibre compressive elastic buckling stress may be obtained as (with E=2.0x 10^5 MPa and Z_z=1.1b_f d_f h)

$$f_{cr,b} = \left(\frac{1473.5}{KL/r_y}\right)^2 \left[1 + 1/20\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5} \qquad \ldots (6.10)$$

The value of $f_{cr,b}$ for various values of KL/r_y and h/t_f based on IS 800:2007. Table 10.10 gives the values of design bending compressive strength corresponding to lateral torsional buckling [based on Eqn (6.4)] for α_{LT}= 0.21 and α_{LT} = 0.49 for f_y = 250 MPa. Intermediate values may be obtained by interpolating the values given in these tables.

6.6 Purlins

Purlins are beams used on trusses to support the sloping roof system between the adjacent trusses. Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points. For larger trusses, however, it is more economical to space purlins at closer intervals. In India, where asbestos cement (AC) sheets are used, the maximum spacing of purlins is also restricted by the length of these sheets. AC sheets (though banned in many countries for the risk of lung cancer while working with them) provide better insulation to sun's heat (compared to GI sheets), which can be further improved by painting them white on the top surface. The maximum permissible span for these sheets is 1.68 m. A longitudinal overlap of not less than 150 mm is provided for AC sheets. The purlin spacing is so adjusted with lengths of the sheets that the longitudinal overlaps fall on the purlins to which they are directly bolted. Spacing of purlins should be so fixed that the cutting of sheets is avoided. Hence in practice when AC sheets are used, the purlin spacing is kept between 1.35 to 1.40 m. But in general, purlins are spaced from 0.6 m to about 2 m and their most desirable depth to span ratio is about 1/24. While the dead loads act through the centre of gravity of the purlin section, the wind loads act normal to the roof trusses. Thus, the purlin section is subjected to bending and twisting resulting in unsymmetrical bending.

Purlins may be designed as simple, continuous, and cantilever beams. The simple beam design yields the largest moments and deflections. For simply supported purlins the maximum moment will be $WL^2/8$ and if they are assumed as continuous, the moment will be $WL^2/10$. While erecting angle, channel- or I-section purlins, it is desirable that they are erected over the rafter with their flange facing up slope [see Fig 6.3]. In this position, the twisting moment does not cause instability. If the purlins are kept in such a way that the flanges face the downward slope, then the twisting moment will cause instability.

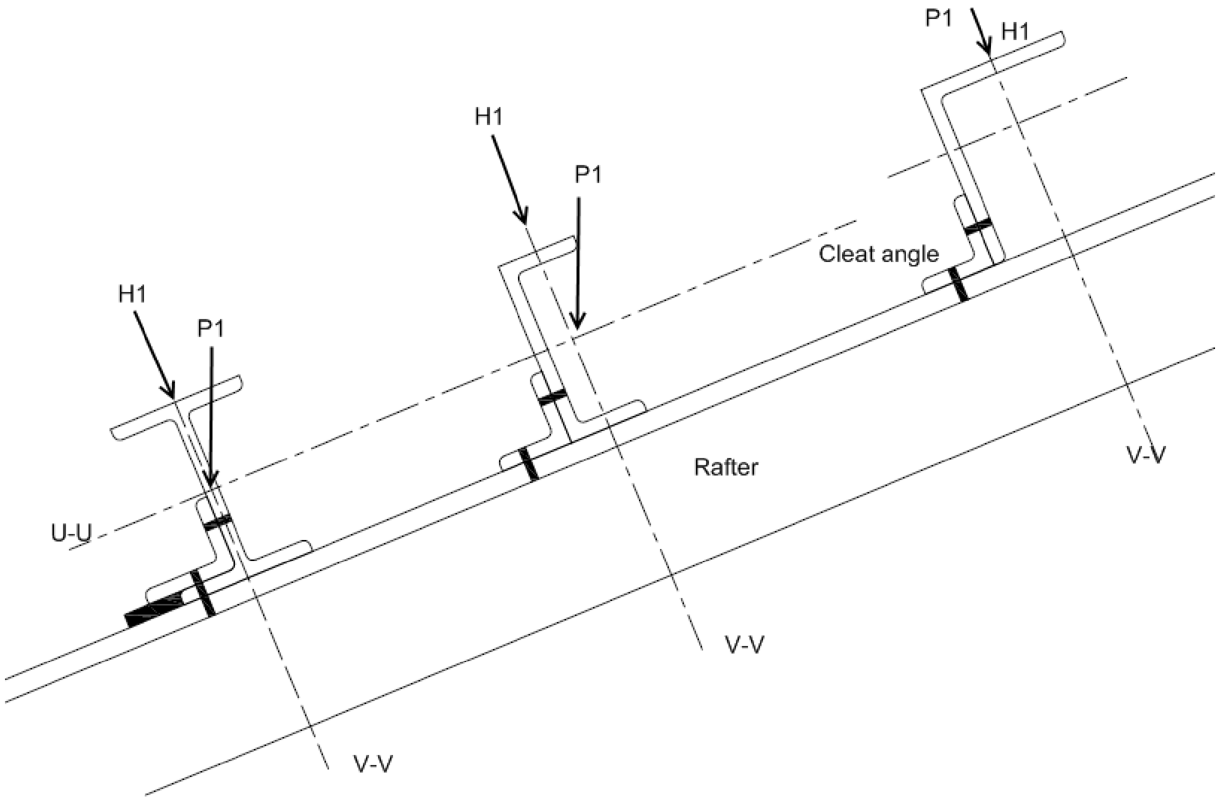


Fig 6.3 Orientation of purlins

As we know channel are very weak about their web axes and have a tendency to sag in the direction of the sloping roof and often sag rods are provided midway or at one-third points between the roof trusses to take up the sag. If sag rods are used they will provide lateral support with respect to y-axis bending. Consequently, the moment is reduced and thereby the required purlin section is smaller. It is further assumed that the purlins are simply supported at the trusses. This is a conservative assumption, since purlins are often continuous over two or more trusses and appreciable continuity will be achieved at their splices. The code also permits to take advantage of the continuity of purlins over supports (clause 8.9.1). Note that if sag rods are not used, the maximum moment about the web axis would be $w_{uy}L^2 / 8$. Thus, when one sag rod is used the moments are reduced by 75% and two sag rods are used as one-third points, the moments are reduced by 91%. In addition to providing lateral supports to purlins, sag rods also help to keep the purlins in proper alignment during erection until the roof deck is installed and connected to the purlins. Sag rods are often used with channel and I-section purlins, but are very rarely used with angle purlins.

6.7 Design Procedure for Channel/I-section Purlins

The design of purlins is a trial and error procedure and the various steps involved in the design are as follows:

1. The span of the purlin is taken as the centre-to-centre distance of adjacent trusses.
2. The gravity loads P, due to sheeting and live load, and the load *H* due to wind are computed. The components of these loads in the direction perpendicular and parallel to the sheeting are determined. These loads are multiplied with partial safety factors γ_f *for* loads (see Table 4 of the code) for the various load combinations.
3. The maximum bending moments (M_z and M_y) and shear forces (F_z and F_y) using the factored loads are determined.
4. The required value of plastic section modulus of the section may be determined by using the following equations.

$$z_{pz} = \frac{M_z \gamma_{m0}}{f_y} + 2.5(d/b)\left[M_y \gamma_{m0}/f_y\right] \qquad \dots (6.11)$$

   where γ_{m0} is the partial safety factor for material = 1.1, *d* is the depth of the trial section, *b* is the breadth of the trial section, M_z and M_y are the factored bending moments about the Z and *Y* axes, respectively, and f_y, is the yield stress of steel.

Since the above equation involves *b* and *d* of .a section, we must use a trial section and from the above equation find out whether the chosen section is adequate or not.

5. Check for the section classification (Table 2 of the code).
6. Check for the shear capacity of the section for both the z and y axes (for purlins shear capacity will always be high and may not govern the design.

$$V_{dz} = f_y/\left(\sqrt{2}\gamma_{m0}\right)A_{vz} \quad \text{... (6.12)}$$

$$V_{dy} = f_y/\left(\sqrt{2}\gamma_{m0}\right)A_{vy} \quad \text{... (6.13)}$$

And $A_{vz} = ht_w$

$A_{vy} = 2b_f t_f$

Where *h* is the height, t_w is the thickness of the web, b_f is the breadth of the flange, and t_f is the thickness of the flange of I-channel section, respectively.

7. Compute the design capacity of the section in both the axes.

$$M_{dz} = Z_{pz}f_y/\gamma_{m0} \leq 1.2Z_{ez}f_y/\gamma_{m0} \quad \text{... (6.14)}$$

$$M_{dy} = Z_{py}f_y/\gamma_{m0} \leq \gamma_f Z_{ey}f_y/\gamma_{m0} \quad \text{... (6.15)}$$

Note that in the second equation 1.2 is replaced by γ_f - It is because, in the y-direction, the shape factor Z_p/Z_e will be greater than 1.2 and hence if we use the factor as 1.2 we cannot prevent the onset of yielding under unfactored loads.

8. Check for local capacity by using the interaction equation

$$(M_z/M_{dz}) + \left(M_y/M_{dy}\right) \leq 1.0 \quad \text{... (6.16)}$$

9. Check whether the deflection is under permissible limits (Table 6 of the code).

7. DESIGN OF COLUMN

7.1 Introductions

Most columns are subjected to bending in addition to the axial load; considerable care should be taken in a practical situation to load a column under axial load only. When significant bending is present in addition to an axial load in a member, the member is termed as a *beam-column.* The bending moments on a column may be due to any of the following effects.

(a) *Eccentricity of axial force:* Figure 7.1 shows the disposition of beams and girders at interior, corner, and exterior columns. The simple shear connection is generally used to connect beams or girders to columns in braced frames (see Fig. 7.2) and is assumed to transmit only reactions from the beams/girders to the columns. Thus, the beam B given in Fig. 7.1(b) transfers its reaction at the face of the column, that is, with an eccentricity of $e = D/2$, where D is the depth of the column section. However, since the eccentricity of the reaction of beam A given in Fig. 7.1(b), framing into the web of the column, is small, its effect is often neglected. Thus, when beams having equal reactions frame into the column opposite each other as in Fig. 7.1 (a) (interior column), the column loads may be assumed to be applied along the axis of the column. However even in an interior column. If the reactions from the beams {say beam B and D given in Fig. 7.1(a)} are not equal, then the difference in the value of the reactions will create a moment on the column section. The column shown in Fig. 7.1(b) will be subjected to biaxial bending and axial load whereas the column shown in Fig. 7.1(c) will be subjected to bending moment about the major axis, in addition to axial load (provided the spans of beam *A* and *C* are equal).

(b) *Portal or gable frame action:* Another common example of a column with bending moments occurs in a portal frame where the column and rafters are subjected to relatively light axial loads combined with bending.

(c) *Load from brackets:* In industrial buildings, column brackets may be used to carry gantry girders on which the cranes move. The resulting eccentricity produces bending moments In addition to the axial loads in the columns. In this case, the column moment is not at the column ends {see Fig. 7.2}.

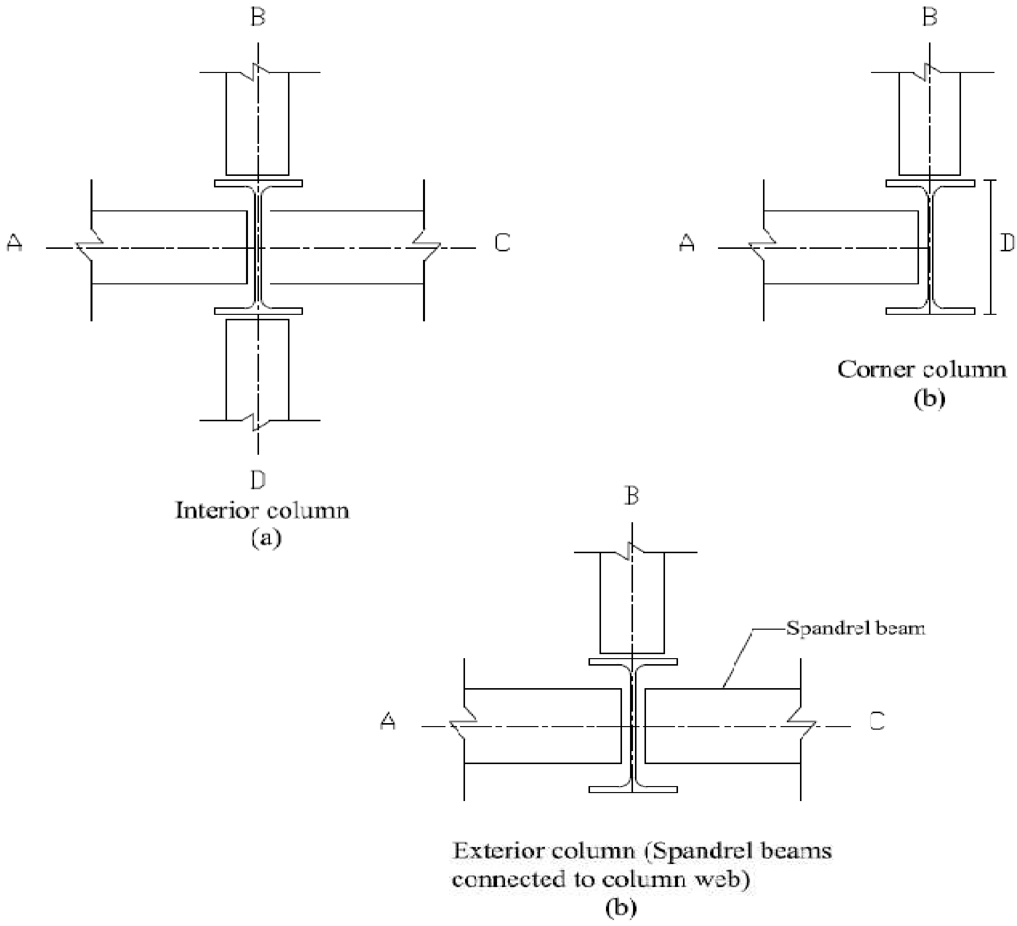


Fig 7.1 Eccentric loads on columns (in plan)

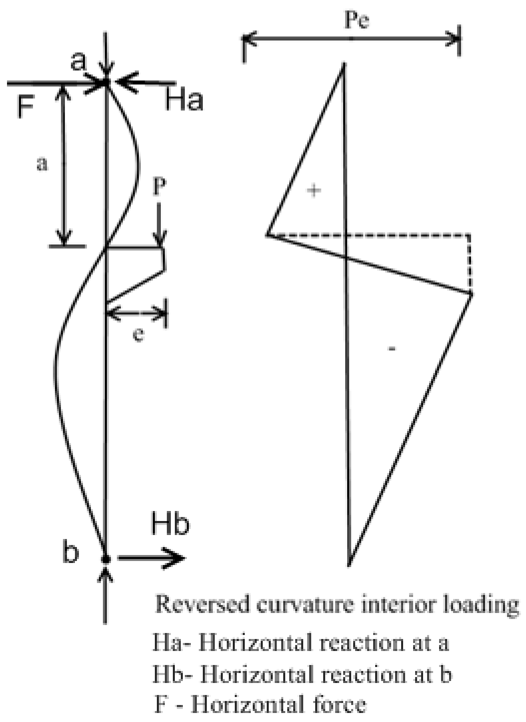


Fig 7.2 Reversed curvature bending in beam-columns

d) *Transverse loads:* The wind pressure on long vertical members may produce bending moments. Similarly earthquakes also produce bending in the columns. Other design conditions also produce bending in addition to axial forces. For example, the top chords of roof and bridge trusses are often considered as pin jointed compression members, but the self weight of these members will produce bending as well. Purlins placed between panels joints of a rafter of roof trusses (in order to reduce sheeting deflection and the size of purlin or to accommodate maximum size of roofing sheets) will produce bending in rafters.

(e) *Fixed base condition:* If the base of the column is fixed due to piles, rafts, or grillage foundation, bending moments will be present at the base of the columns, even though their top ends may be hinged.

Beam columns in steel structures are often subjected to biaxial bending moments, acting in two principal planes, due to the space action of the framing system. The column cross section is usually oriented in such a way to resist significant bending about the major axis of the member. When I-shaped cross sections are used for the columns, the minor axis bending may also become significant, since the minor axis bending resistance of I-section is small compared to the major axis bending resistance.

7.2 Cross Section Classification

In the code (IS 800), cross sections are placed into four behavioral classes depending upon the material yield strength, the width-to-thickness ratios of the individual components (e.g., webs and flanges) within the cross section, and the loading arrangement. The four classes of sections are defined as follows.

1. *Plastic or class 1* Cross sections which can develop plastic hinges and have the rotation capacity required for the failure of the structure by the formation of a plastic mechanism (only these sections are used in plastic analysis and design).
2. *Compact or class 2* Cross sections which can develop their plastic moment resistance, but have inadequate plastic hinge rotation capacity because of local buckling.
3. *Semi-compact or class 3* Cross sections in which the elastically calculated stress in the extreme compression fibre of the steel member, assuming an

elastic distribution of stresses, can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance.

4. *Slender or class 4* Cross sections in which local buckling will occur even before the attainment of yield stress in one or more parts of the cross section. In such cases, the effective sections for design are calculated by deducting the width of the compression plate element in excess of the semi-compact section limit.

The moment-rotation characteristics of these four classes of cross sections are shown in Fig. 6.2. As seen from this figure, class 1 (plastic) cross sections are fully effective under pure compression, and capable of reaching and maintaining their full plastic moment in bending and hence used in plastic design. These sections will exhibit sufficient ductility ($\phi_2 > 6\ \phi_1$ where ϕ_1 is the rotation at the onset of plasticity and ϕ_2 is the lower limit of rotation for treatment as a plastic section).

Class 2 (compact) cross sections have lower deformation capacity, but are also fully effective in pure compression and are capable of reaching their full plastic moment in bending (they have ductility in the range $\phi_1 < \phi_2 < 6\ \phi_1$).

Class 3 (semi-compact) cross sections are fully effective in pure compression but local buckling prevents the attainment of the full plastic moment in bending; bending moment resistance in these cross sections is limited to the (elastic) yield moment only.

For class 4 (slender) cross sections, the local or lateral buckling of the member occurs in the elastic range. An effective cross section is therefore defined based on the width-to-thickness ratios of the individual plate elements and this is used to determine the resistance of the cross section. The majority of the hot-rolled cross sections belong to class 1, 2, or 3, and hence their resistances may be based on the gross cross-section properties obtained from section tables (IS 808).

7.3 Loads on Column Member

The axial loading on columns in buildings is due to loads from roofs, floors, and walls transmitted to the column through beams and also due to its own self weight.

In industrial buildings, loads from crane and wind cause moments in columns as shown in Fig. 7.2 In such cases, wind load is applied to the column through the sheeting rails and may be taken as uniformly distributed throughout the length of the column.

The strength of a column depends on the following parameters:

- Material of the column
- Cross-sectional configuration
- Length of the column
- Support conditions at the ends (called restraint conditions)
- Residual stresses
- Imperfections

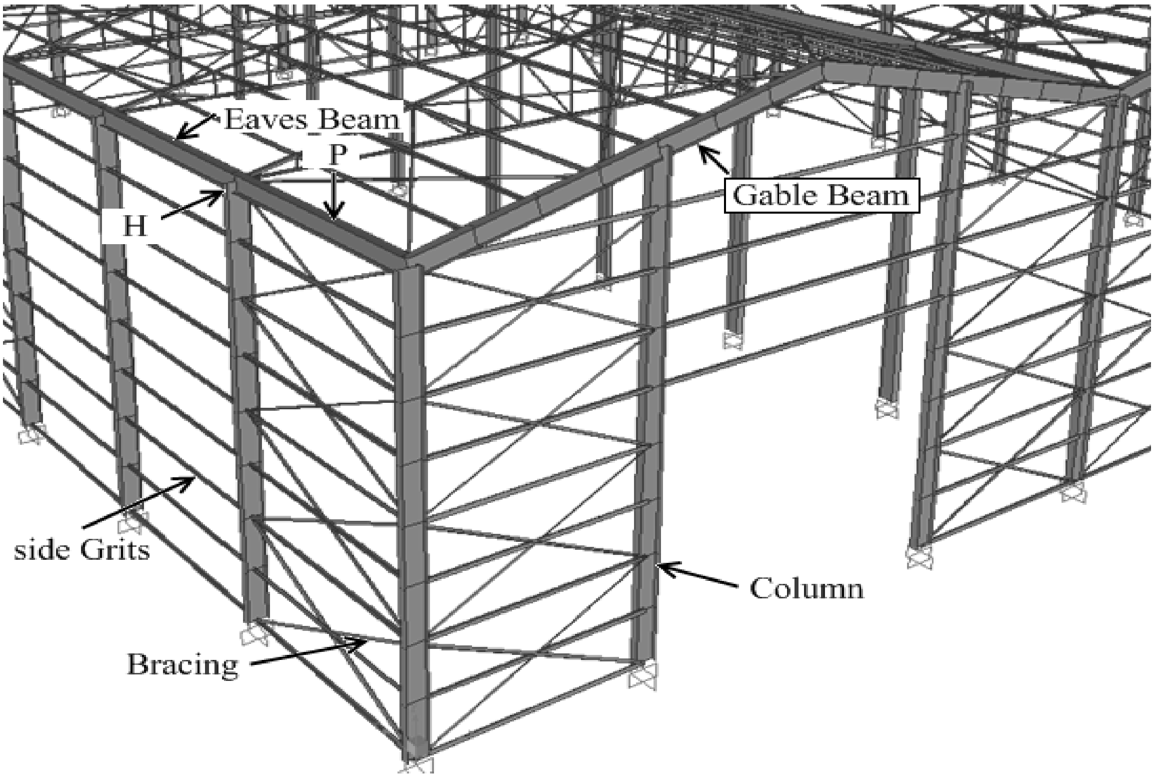


Fig 7.3 Beam-Column as a part of the braced frame.

The imperfections include the following:

- The material not being isotropic and homogenous
- Geometric variations of columns
- Eccentricity of load

It is difficult to assess the residual stress acting on each column cross section and also to assess the degree of support condition offered by a variety of connection details adopted in practice. Due to the large number of variables, which influence the strength of columns and beam-columns in the elastic and inelastic ranges, several researchers throughout the world have done extensive experimental and theoretical investigations on the behavior of columns due to these variables.

7.4 Possible Failure Modes

The possible failure modes of an axially loaded column are as follows:

1. *Local buckling:* Failure occurs by buckling of one or more individual plate elements, e.g., flange or web, with no overall deflection in the direction normal to the applied load. This failure mode may be prevented by selecting suitable width-to-thickness ratios of component plates. Alternatively, when slender plates are used, the design strength may be reduced.

2. *Squashing:* When the length is relatively small (solid column) and its component plate elements are prevented from local buckling, then the column will be able to attain its full strength or 'squash load' (yield stress x area of cross section).

3. *Overall flexural buckling:* This mode of failure normally controls the design of most compression members. In this mode, failure of the member occurs by excessive deflection in the plane of the weaker principal axis. An increase in the length of the column, results in the column resisting progressively less loads.

4. *Torsional and flexural-torsional buckling:* Torsional buckling failure occurs by twisting about the shear centre in the longitudinal axis. A combination of flexure and twisting, called *flexural-torsional buckling* is also possible. Torsional buckling is a possible mode of failure for point symmetric sections. Flexural torsional buckling must be checked for open sections that are singly symmetric and for

sections that have no symmetry. Note that open sections that are doubly symmetric or point symmetric are not subjected to flexural-torsional buckling, since their shear centre and centroid coincide. Closed sections are also immune to flexural-torsional buckling.

In addition to the above failure modes, in compound members (two or more shapes joined together to form a lattice cross section), failure of a component member may occur, if the joints between members are sparsely placed. Codes and specifications usually have rules to prevent such failures.

7.5 Behaviour of Compression Member

7.5.1 Long, Short, and Intermediate Compression Members

Compression members are sometimes classified as being long, short, or intermediate. A brief discussion about this classification is as follows:

Short compression members: For very short compression members the failure stress will equal the yield stress and no buckling will occur. Note that for a compression member to fall into this classification, it has to be so short (for an initially straight column L < 88.85r, for f_y = 250 MPa) that it will not have any practical application.

Long compression members: For these compression members, the Euler formula predicts the strength of long compression members very well, where the axial buckling stress remains below proportional limit. Such compression members will buckle elastically.

Intermediate length compression members: For intermediate length compression members, some fibers would have yielded and some fibers will still be elastic. These compression members will fail both by yielding and buckling and their behavior is said to be 'inelastic'. For the Euler formula to be applicable to these compression members, it should be modified according to the reduced modulus concept or the tangent modulus concept (as is done in AISC code formula) to account for the presence of residual stresses.

7.5.2 Short Compression Members

Consider an axially compressed member of short length which is initially straight and made of material having the ideal rigid—plastic stress-strain relationship as shown in Fig. 7.4. At low values of external load *P.* there will be no visible deformation—neither lateral nor axial. Since *P* is applied at the centroid of the section, apart from possible localized effects at the ends of the member, all parts of the member will experience the same value of compressive stress f_c = *P A.* Large deformation is possible only when f_c reaches the yield stress fy*.* At this stage the member deforms axially. The value of the axial force at which this happens is termed as the 'squash load' and is given by

$$P_y = f_y A \qquad ..(7.1)$$

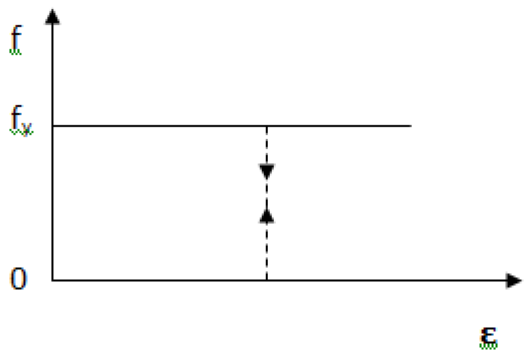


Fig 7.4 Idealized material behavior

7.5.3 Slender Compression Member

The strength of the compression member decreases as its length increases, in contrast to the axially loaded tension member whose strength is independent of its length. Thus, the compressive strength of a very slender member may be much less than its tensile strength. This decrease in strength is due to the following parameters, which are often grouped under the heading of imperfections: the initial lack of straightness, accidental eccentricities of loading, residual stresses, and variation of material properties over the cross section.

7.4.3.1 Elastic Buckling of Slender Compression members

Euler considered an ideal column with the following attributes.

- Material is isotropic and homogenous and is assumed to be perfectly elastic.
- The column is initially straight and the load acts along the centroidal axis (i.e., no eccentricity of loads).
- Column has no imperfections.
- Column ends are hinged.

Such a column is also known as an Eulercolumn and is shown in Fig. 7.5. A concentric load P is applied to the upper end of the member, which remains straight until buckling occurs, when it is slightly bent as shown in Fig. 7.5.

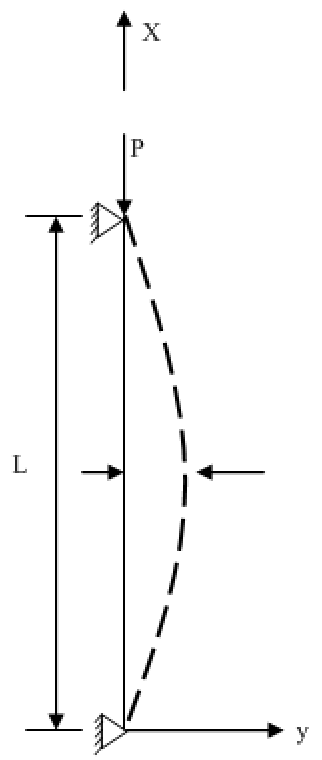


Fig 7.5 Behavior of a perfectly straight elastic pin-ended column

At any location x, the bending moment M on the member bent slightly about the principal axis is

$$M = Py \qquad \dots (7.2)$$

And since

$$\frac{d^2y}{dx^2} = -(M/EI) \qquad \dots (7.3)$$

Where E is young's modulus and I is the moment of inertia; the differential equation for the members becomes

$$\frac{d^2y}{dx^2} + \left(\frac{P}{EI}\right)y = 0 \qquad \text{... (7.4)}$$

After letting k^2=P/EI, the solution of this sevond order differential equation may be expreesed as

$$y = A_1 \sin kx + A_2 \cos kx \qquad \text{... (7.5)}$$

Where A_1 and A_2 are unknown coefficients. Applying the boundary conditions (a)y=0 at x=0 and (b)y=0 at x=L, one may obtain for condition(a),

A_2=0

And for condition (b),

0=A_1 sin kx

The above euation is satisfied if kL=Nπ thus we get

$$P = \frac{\pi^2 EI}{L^2}, \frac{4\pi^2 EI}{L^2}, \dots, \frac{n^2\pi^2 EI}{L^2}, \qquad \text{... (7.6)}$$

The fundamental buckling mode, with a single curvature deflection (y=A_1sin πx/L) will occur when N=1. Thus, the Eular critical load for a column with both ends hinged is

$$P_{cr} = \pi^2 EI/L^2 \qquad \text{... (7.7)}$$

Or in terms of average critical sterss, using I=$A_g r^2$

$$f_{cr} = \frac{P_{cr}}{A_g} = \frac{\pi^2 E}{\lambda^2} \qquad \text{... (7.8)}$$

Where the λ is the slenderness ration defined by

$$\lambda = L/r \qquad \text{... (7.9)}$$

Buckling phenomenon is associate with the stiffness of the member. A member with low stiffness will buckle early than one with high stiffness. Increasing member lengths causes reduction in stiffness. The stiffness of the member is strongly influenced by the amount and distribution of the material in the cross section of the column;the value of r reflects the way in which the material is distributed.

7.6 Effective Length of Column Member

The effect of end restraints on column strength is usually incorporated in the design by the concept of effective length. Under ideal conditions, the boundary conditions of a column may be idealized in one of the following ways.

- Both ends pinned
- Both ends fixed
- One end fixed and the other end pinned
- One end fixed and the other end free

For all these conditions, the differential equations as given in Eqn (7.3) can be set up and the appropriate boundary conditions applied to get the following critical loads (Timoshenko & Gere 1961; Allen & Bulson 1980).

1. For column with both ends fixed:

$$P_{cr} = \frac{4\pi^2 EI}{L^2} = \frac{4\pi^2 EA_g}{\left(\frac{L}{r}\right)^2} \qquad ..(7.10)$$

2. For column with one end fixed and the other end pinned:

$$P_{cr} = \frac{2\pi^2 EA_g}{\left(\frac{L}{r}\right)^2} \qquad ..(7.11)$$

3. For columns with one end fixed and the other end free:

$$P_{cr} = \frac{\pi^2 EA_g}{\left[4\left(\frac{L}{r}\right)^2\right]} \qquad ..(7.12)$$

Using the length of the pin-ended column, as the basis for comparison, critical load in the three cases mentioned earlier can be obtained by employing the concept *of effective length KL* where *K* is called the *effective length ratio or* effective length coefficient. (It is the ratio of the effective length to the

unsupported length of the columns.) The value of *K* depends on the degree of rotational and translation restraints at the column end. The *unsupported length* L is taken as the distance between lateral connections, or actual length in the case of a cantilever. In a conventional framed construction, *L* is taken as the clear distance between the floor and the shallower beam framing into the column in each direction at the next higher floor level.
In other words, the *effective length* of a column in a given plane may be defined as the distance between the points of inflection (zero moment) in the buckled configuration of the column in that plane (see Fig: 7.6).

Using KL, Eqa (7.10),(7.11) and (7.12) can be written as

$$P_{cr} = \pi^2 E A_g/(KL/r^2) \qquad ..(7.13)$$

Where K= 1 for column with both ends pinned

K = 0.5 for columns with both ends fixed

K = 0.707 for columns with one end fixed and the other end pinned

K = 2.0 for columns with one end fixed and the other end free

K ≤ 1.0 for columns partially restrained at each end, and

K ≥ 2.0 for columns with one end unrestrained and the other end rotation partially restrained.

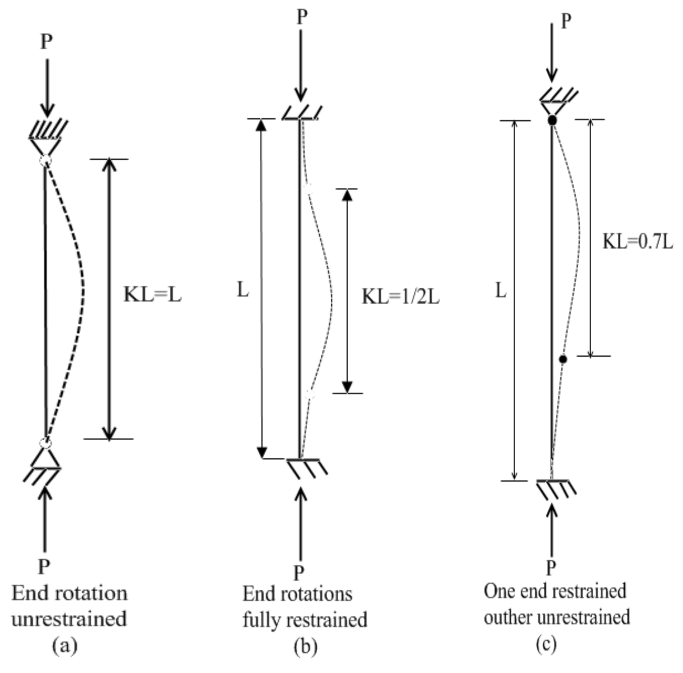


Fig 7.6 Effective length KL when there is no joint translation at the ends

7.6.1 Effective length of columns in industrial Buildings:

The effective lengths of columns for single storey buildings of simple design may be determined by reference to the typical cases illustrated in Figs 7.7 & 7.8, provided that the following conditions are satisfied (BS 5950-1:2000).

a) In the y-y plane, the columns act as cantilevers tied together by the roof trusses, but in this plane, the tops of columns are not otherwise held in position or restrained in direction,
b) In the perpendicular plane (Z-Z), the tops of the columns are effectively held in position by members connecting them to a braced bay, or by other suitable means. In Figs 7.6, it is assumed that the braced bay also holds columns in position at crane girder level.
c) The bases of columns are effectively held in position and restrained in direction in both the planes.
d) The foundations provide restraint equal to that provided by the bases of columns.

If there are different end conditions, then the following modifications are suggested by the British code.

a) If the roof truss or other roof member is connected to the columns in such a way that they transfer appreciable moment to the column, the effective length may be obtained by using the Wood's curves.(fig 27 and fig 28 in IS 800:2007)
b) If in the Z-Z plane, the base of the column is not effectively restrained in direction, the effective lengths 0.85L or $0.85L_1$ in fig 7.7 & 7.8 should be increased to 1.0L or $1.0L_1$ respectively.

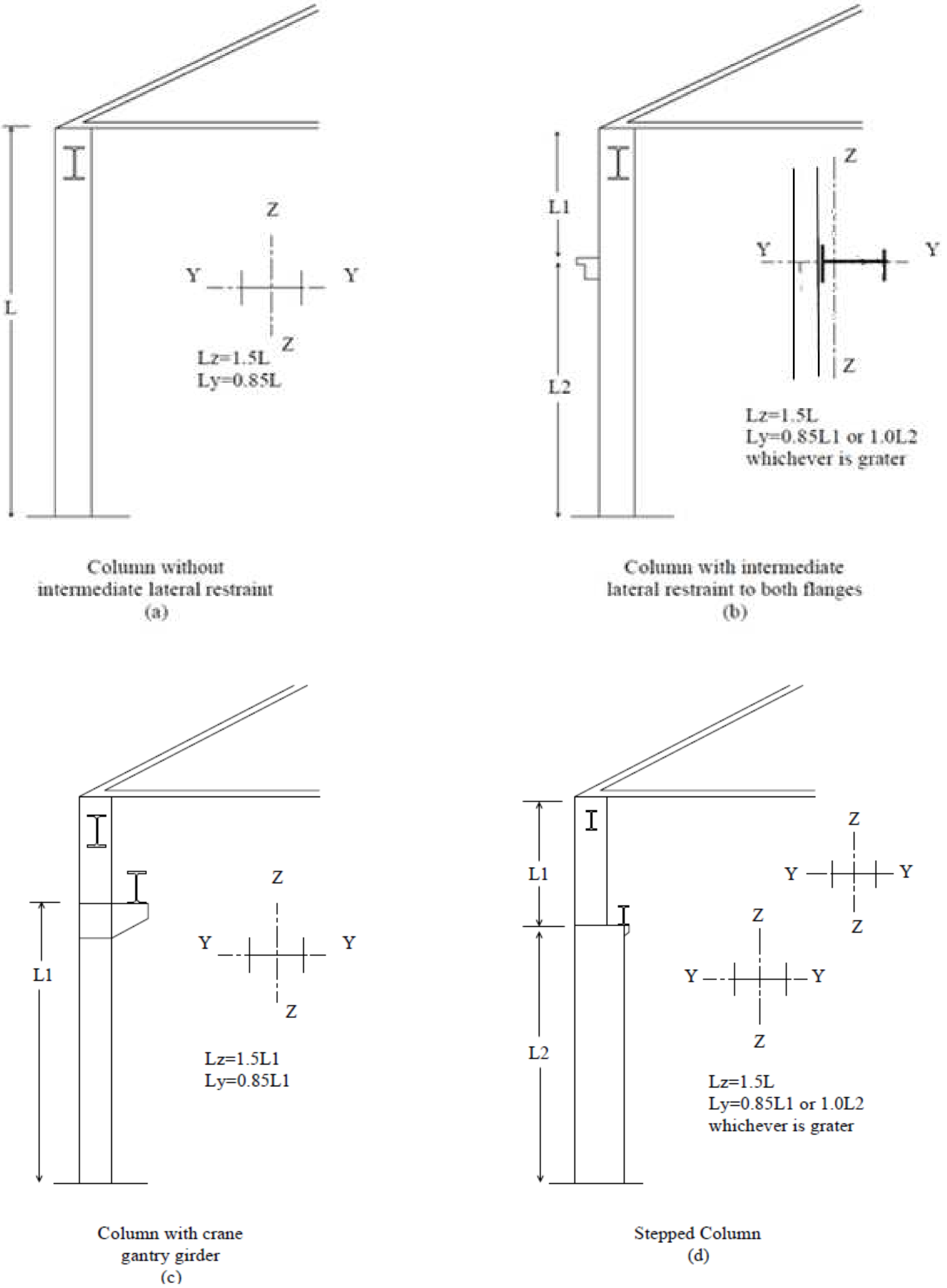


Fig 7.7 Effective length for side stanchions in single storey building

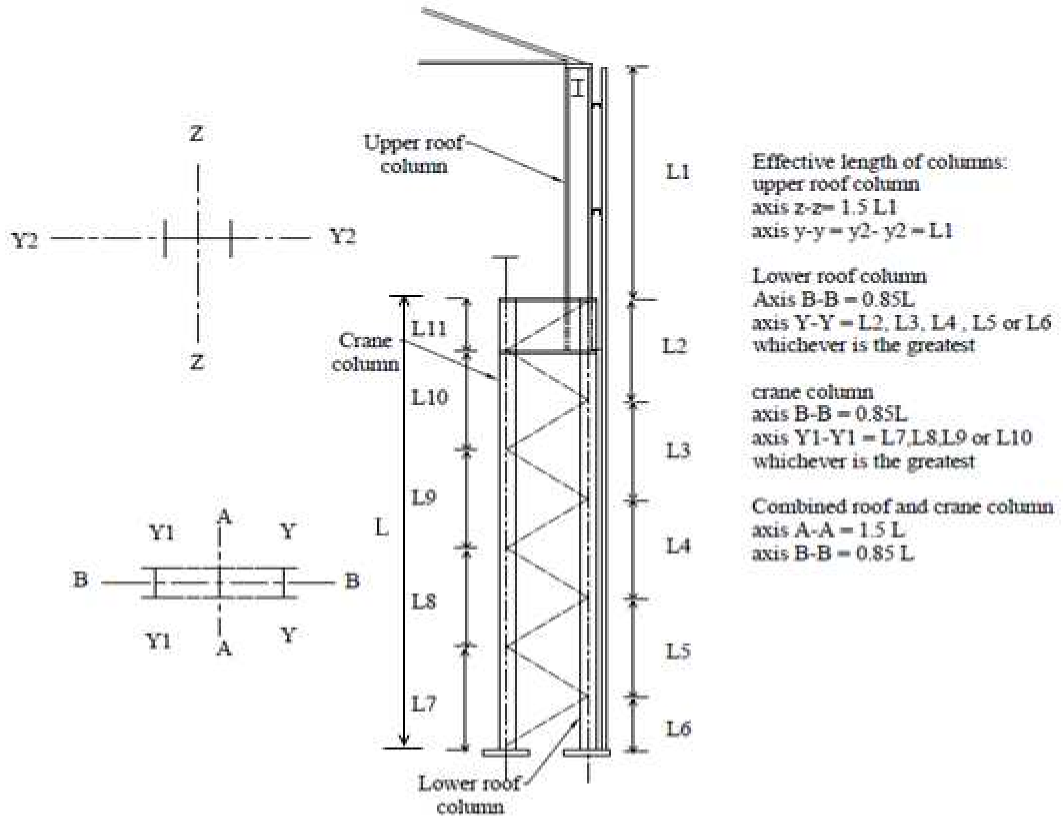


Fig 7.8 Compound side column with crane gantry beams

7.7 Types of column sections

7.7.1 Rolled Steel Sections

Generally for column, rolled steel prismatic section is used for light weight industrial building structure. When the available sections are not suitable, a suitable section may be built-up either by welding or by lacing or battening two sections separated by a suitable distance.

7.7.2 Built-Up Column or Fabricated Compression Members

The cross section consists of two channel sections connected on their open sides with some type of lacing or latticing (dotted lines) to hold the parts together and ensure that they act together as one unit. The ends of these members are connected with "batten plates" which tie the ends together. Box sections of the type shown in Fig. 7.9(a) or 7.9(b) are sometimes connected by such solid plates either at intervals (battened) or continuously along the length.

A pair of channels connected by cover plates on one side and latticing on the other is sometimes used as top chords of bridge trusses. The gussets at joints can be conveniently connected to the inside of the channels. Plated I sections or built-up I sections are used when the available rolled I sections do not have sufficient strengths to resist column loads [Fig 7.9(c)]. Flange plates or channels may be used in combination with rolled sections to enhance the load resistance of the commonly available sections, which are directly welded or bolted to each other. The lateral dimension of the column is generally chosen at around 1/10 to 1/15 of the height of the column. For purposes of detailing the connection between the flange cover plates or the outer rolled sections to the flanges of the main rolled section, it is customary to design the fasteners for a transverse shear force equal to 2.5% of the compressive load of the column. Columns with open webs may be classified as laced columns or battened columns. In Fig. 7.8, the two channel sections of the column are connected together by batten plates or laces which are shown by dotted lines. A typical lacing or batten plate is shown in Fig. 7.10

The Code gives simple guidelines for the design of laced (Cl.7.6) and battened columns (Cl.7.7). One of the guidelines is that such columns should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing (Cl. 7.6.1.1). To account for the inherent flexibility of laced and battened columns, the Code suggests that the effective slenderness ratio be taken as 5 and 10% respectively more than the calculated values (Cl.7.7.1.4). All columns should be tied at the ends by tie plates or end battens to ensure a satisfactory performance.

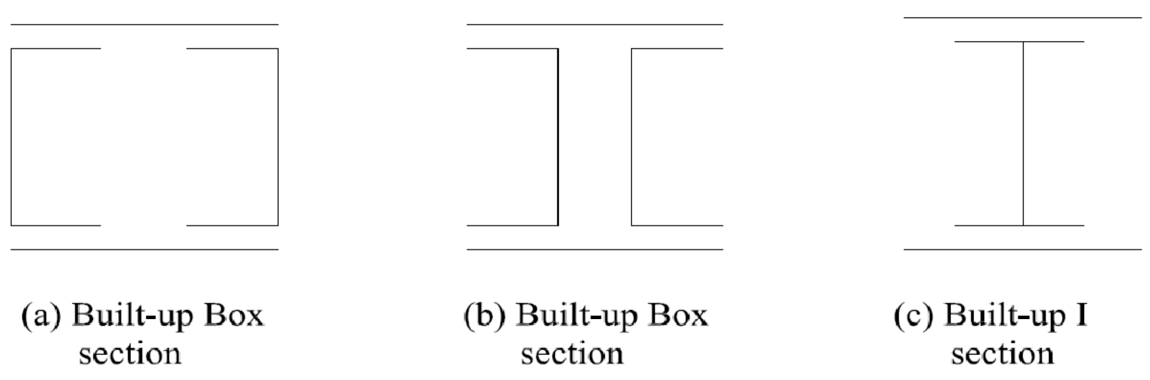


Fig 7.9 Cross Section Shapes for Built - up or fabricated Compression Members

In laced columns, the lacing should be symmetrical in any two opposing faces to avoid torsion. Lacings and battens are not combined in the same column. The inclination of lacing bars from the axis of the column should not be less than 40^o nor more than 70^o. (Cl. 7.6.5) The slenderness ratio of the lacing bars should not exceed 145 (Cl. 7.6.3). The effective length of lacing bars is the length between bolts for single lacing and 0.7 of this length for double lacing. The width of the lacing bar should be at least 3 times the diameter of the bolt (Cl. 7.6.3). Thickness of lacing bars should be at least 1/40th of the length between bolts for single lacing and 1/60 of this length for double lacing (both for welded and bolted connections) (Cl. 7.6.4).

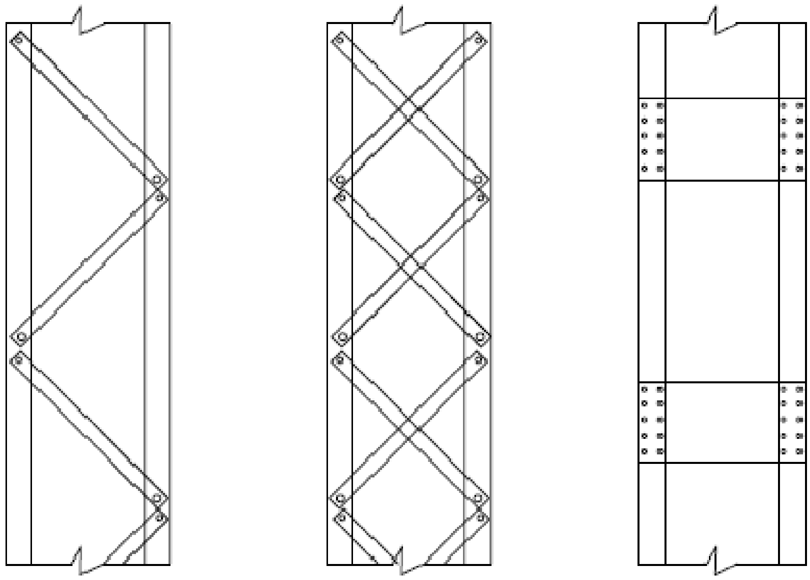


Fig7.10 Built-up column members

In battened columns, the Battens plates at their ends shall be riveted or welded to the main components so as to resist simultaneously a shear Vb = Vt C/N S along the column axis and a moment M = Vt C / 2 N at each connection (Cl.7.7). Where,

Vt = the transverse shear force

C = the distance between centre-to-centre of battens, longitudinally

N = the number of parallel planes of battens (usually 2)

S = the minimum transverse distance between the centroid of the bolt group/welding connecting the batten to the main member.

7.8 Crane Column

Three common types of crane columns, used in single-storey industrial buildings, are shown in fig 7.7(c), 7.7(d) and 7.8. They are as follows:

1. A column of uniform section for the entire length, with the gantry girder supported by column brackets [see fig 7.7(c)].
2. A stepped column, with a heavy wide flange section at the bottom and a lighter rolled I beam or wide flange section at the top, supporting the roof structure [see fig 7.7(d)]. The gantry girder is seated on the top of the lower column. A small bracket (usually stiffened) is used when the flange of the gantry girder is overhanging as shown in Fig. 7.7(d). To transmit the bending moments from the upper column, the junction of the top and lower column should be strongly spliced and stiffened. The effective length of such stepped columns can be determined by using Annex D2 of IS 800: 2007.
3. For very heavy loads, a laced or battened column as shown in Fig. 7.8 is used, which contains two separate columns, one supporting the load from the crane and the other supporting the roof load. The two sections may be designed independently of each other. The building column may be proportioned to take the gravity and wind loads from the building and the lateral thrust from the crane. The crane column may be designed to support the vertical loads from the gantry girder and a horizontal longitudinal load at the top in the direction of the gantry girder. The two sections are separated sufficiently to provide necessary crane clearance. The advantage of having two separate columns is as follows. The crane loads can be applied concentrically on the crane column and may be oriented to obtain maximum resistance to longitudinal forces (e.g., column web can be placed parallel to the plane of the gantry girder). Similarly, the building column can be oriented independently, so that its strong axis can resist wind loads on the building and lateral forces from the crane.

7.8.1 Loading

The different loads that act on a crane column are as follows (see Fig. 7.11):

W_A - Vertical load from roof truss, taken as applied concentrically.

W_B - Horizontal loads due to wind, applied to side rails.
W_c and W_{HC} = Crane gantry loads applied through a bracket at a known eccentricity.
W_D = Self weight of the column/sheeting.
W_E = Resultant force carried through the truss bottom chord.

In single-storey industrial buildings, the load W_E occurs whenever the columns carry unequal horizontal loads or unequal moments. Values of the resultant force for different arrangement of horizontal loading are shown in Fig. 7.12 (Morris & Plum 1996).
For the design of a column under a load system such as that shown in Fig. 7.11, load factors γ_f-must be included in the calculation. Unlike simple load cases, it will no longer be clear which combination produces the highest axial forces and moments. It may be necessary to examine all the possible combinations, although with experience three or four worst combinations may be selected.

7.9 Design of Column

The design of columns involves a trial-and-error procedure. A trial section is selected by some process and is then checked with the appropriate interaction formula. If the section does not satisfy the equation(7.20) (LHS > 1.0) or if it is too much on the safer side, indicated by LHS much less than 1.0 (that is, if it is over designed), a different section is selected and the calculations are repeated till a satisfactory section is found. Thus, the different steps involved in the design of beam columns are as follows.

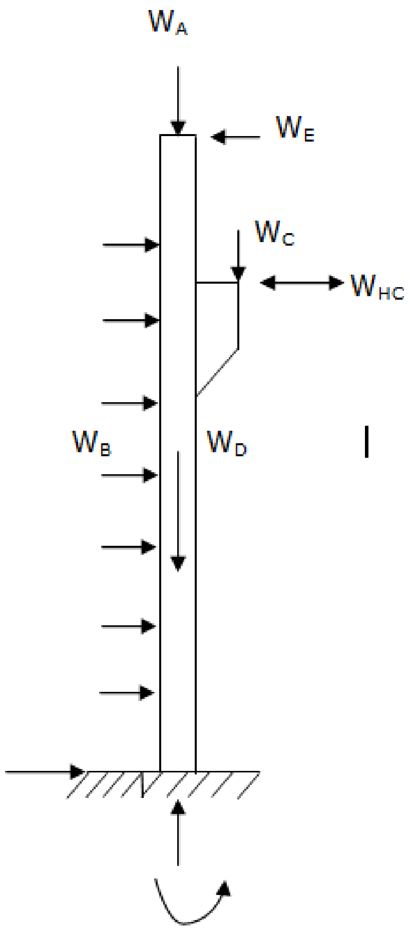


Fig 7.11 loads acting on a crane column

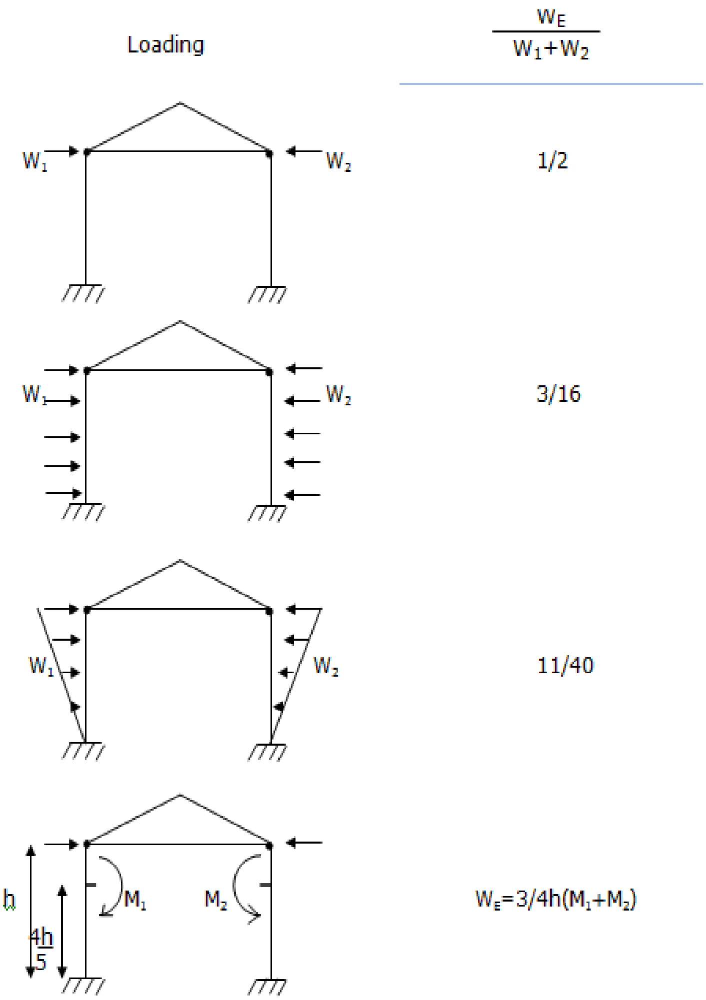


Fig 7.12 Resultant force carried through the bottom chord of roof truss

1. Determine the factored loads and moments acting on the column using a first-order elastic analysis.
2. Choose an initial section and calculate the necessary section properties.
3. Classify the cross section (plastic, compact, or semi-compact) as per clause 3.7 of the code.
4. Find out the bending strength of the cross section about the major and minor axis of the member (clause 8.2.1.2).

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \qquad \text{... (7.14)}$$

Where,

β_b = 1.0 for plastic and compact sections

β_b = Z_e/Z_p for semi-compact sections.

Z_e, Z_p = plastic and elastic section modulii of the cross section

f_y = yield stress of the material.

γ_{m0} = partial safety factor.

5. Determine the shear resistance of the cross section (clause 8.4.1). When the design shear force exceeds $0.6V_d$, then the design bending strength must be reduced as given in clause 9.2.2 of the code,

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}\gamma_{m0}} \quad \text{... (7.15)}$$

Where,

V_d = design shear strength

A_v = Shear area

f_{yw} = yield strength of web

6. Check whether shear buckling has to be taken into account (clause 8.4.2).

$d/t_w > 67\varepsilon$ For the web without stiffeners, and

$> 67\varepsilon\sqrt{\frac{K_v}{5.35}}$ For a web with stiffeners

Where,

K_v = shear buckling coefficient and

$\varepsilon = \sqrt{250/f_y}$

7. Calculate the reduced plastic flexural strength (clause 9.3.1.2), if the section is plastic or compact.

For standard I of H section.

For $n \leq 0.2$ $M_{ndy} = M_{dy}$ ... (7.16)

For $n > 0.2$ $M_{ndy} = 1.56\ M_{dy}(1-n)(n+0.6)$ ... (7.17)

$$M_{ndz} = 1.11\ M_{dz}(1-n) \leq M_{dz} \quad \text{... (7.18)}$$

Where

M_{ndy} , M_{ndz} = design reduced flexural strength under combination axial force and the respective uniaxial moment acting along.

M_{dy}, M_{dz} = design strength under corresponding moment acting along. [See equation (7.14).]

8. Check the interaction equation for cross-section resistance for biaxial bending (clause 9.3.1.1 for plastic and compact section and clause 9.3.1.3 for semi-compact section). If not satisfied go to step 2.

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{… (7.19)}$$

Where,

N = factored applied axial force (Tension, T or compression C)

N_d = design strength in tension T_d or in compression due to yielding.

9. Calculate the design compressive strength P_{dz} and P_{dy} (clause 7.1.2) due to axial force.
10. Calculate the design bending strength governed by lateral-torsional buckling (clause 8.2.2).
11. Calculate the moment amplification factors (clause 9.3.2.2).
12. Check with the interaction equation for buckling resistance (clause 9.3.2.2). If the interaction equation is not satisfied (LHS > 1.0) or when it is over design (LHS < 1.0), go to step 2.

$$\frac{P}{P_{dy}} + K_y \frac{C_{my} M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} \leq 1.0 \qquad \text{… (7.20)}$$

$$\frac{P}{P_{dz}} + 0.6 K_y \frac{C_{my} M_y}{M_{dy}} + K_z \frac{C_{mz} M_z}{M_{dz}} \leq 1.0 \qquad \text{… (7.21)}$$

Where,

C_{my}, C_{mz} = equivalent uniform moment factor as per table 18 of IS800

P = applied axial compression under factored load.

M_y, M_z = maximum factored applied bending moment about y and z axis of the member.

P_{dy}, P_{dz} = design strength under axial and compression as governed by buckling about minor and major axis.

M_{dy}, M_{dz} = design bending strength about minor and major axis considering laterally unsupported length of column.

$K_y = 1 + (\lambda_y - 0.2) n_y \leq 1 + 0.8\, n_y$

$$K_z = 1 + (\lambda_z - 0.2)n_z \leq 1 + 0.8\, n_z$$

n_z, n_y = ration of actual applied axial force to the design axial strength for buckling about the y and z axis, respectively.

7.10 Column Bases and Caps

For transmitting the load from columns to its foundations, *base plates* are used Base plates assist in reducing the intensity of loading and distributing it over the foundations. The area of base plate is so chosen that the intensity of load distributed is less than the bearing capacity of concrete on which it rests.

The safety of a column and thus of a structure depends mainly upon the stability of the foundations and consequently on the bases, in the case of steel columns Hence, column base plates should be designed with great care. The design of a base plate is generally assumed to be on the condition that the distribution of load under the base is uniform and the outstanding portions of the base plate are treated as cantilevers.

The main types of bases used are shown in Fig. 7.13. These are as follows:

1. Slab base
2. Gussetted base; and
3. Pocket base

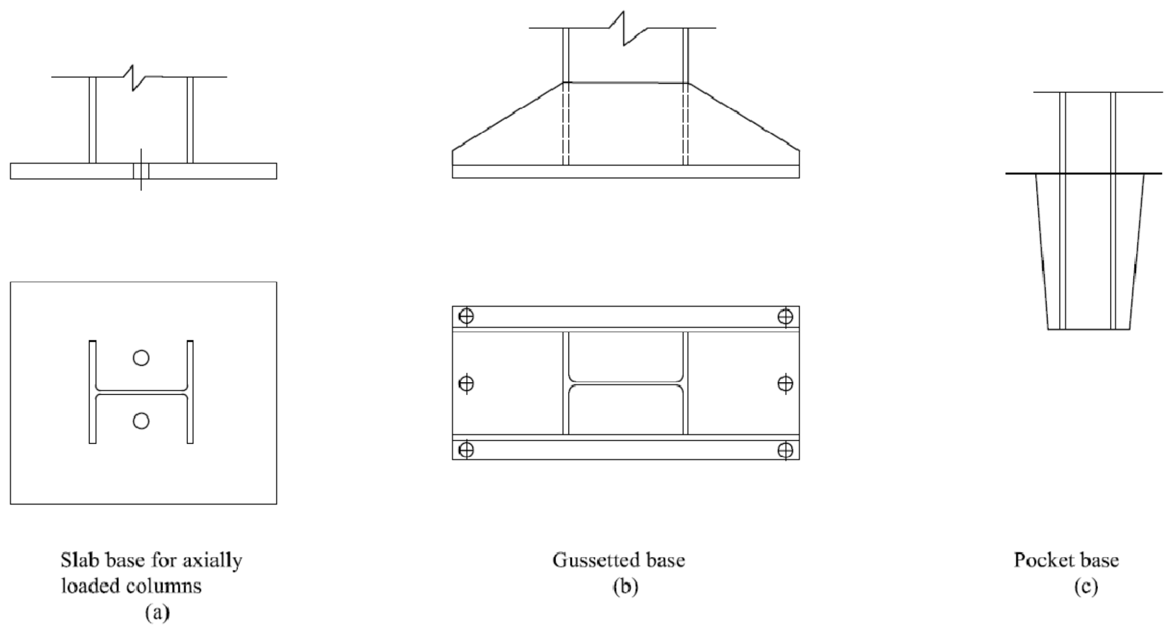


Fig 7.13 Column bases

With respect to slab and gussetted bases, depending on the values of axial load and moment, there may be compression over the whole base or compression over part of the base and tension in the holding-down bolts. Horizontal loads are restricted by shear in the weld between column and base plates, friction, and bond between the base and the concrete. If the base plate has a grout pad of any substantial thickness and the anchor rod does not bear against the base plate (the base plate holes will be larger than the anchor rod and hence in many cases the base connection will not bear against the side of the hole), then bending will be introduced in the rod in addition to shear. The bending capacity of the anchor rods is limited and hence the AISC code does not allow shear transfer through anchor rods (AISC design guide 1-2005). Hence AISC suggests the use of a shear key or lug or embedded plate with welded side plates to transfer a large horizontal shear force from the column base to the foundation (see Fig. 7.14). Note that the horizontal loads will be substantial for earthquake loading or wind loading. We will consider only the design of base plates with concentric loading here.

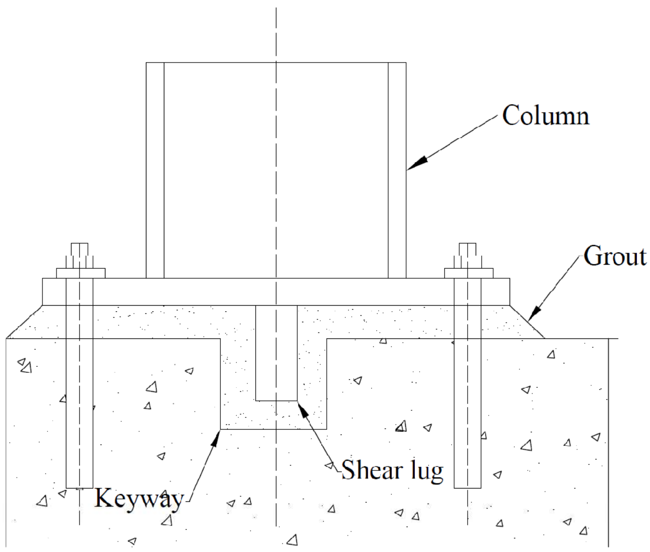


Fig 7.14 Use of shear lug to transfer heavy shear force

Lightly loaded columns are provided with thick slab bases. The slab base is free from pockets where corrosion may start. Base plates with especially large loads require more than a simple plate. This may result in a double layer of plates, a grillage system, or the use of stiffeners to reduce the plate thickness. The *column caps* serve similar purpose except that they act as a link between load coming on the columns and the column itself.

The design of slab bases with concentric load is covered in Section 7.4.3 of IS 800: 2007. This states that where the rectangular plate is loaded by I-, *H-,* channel, box, or rectangular hollow sections, the minimum thickness of base plate t_s should be

$$t_s = \left[2.5w(a^2 - 0.3b^2)\gamma_{mo}/f_y\right]^{0.5} > t_f \qquad \text{... (7.22)}$$

Here *w* is the pressure on the underside of the slab base due to the factored compressive load on the column (assumed as uniformly distributed over the area of the slab base), *a* and *b* are the larger and smaller projections of the slab base beyond the rectangle, circumscribing the column, respectively, fy is the yield strength of the base plate, t_f the flange thickness of the compression member, and γ_{mQ} is the partial safety factor for material =1.10.

Equation (7.22) takes into account plate bending in two directions. The moment in the direction of the greater projection is reduced by the co-existence moment at right angles. Poisson's ratio of 0.3 is used to allow for this effect.

7.11 Eccentrically Loaded Base Plate

The forces in the base plates may be axial loads, shear force, and moments about either axis or any combination of them. The main function of the base plate is to distribute the loads to the weaker material.

The common design deals with axial load and moment about major axis. With respect to slab and gusseted base (see Fig. 7.13), there may be two separate cases (a) compression over the whole base or compression over part of the base and (b) tension in the *holding-down bolts* (also called *anchor bolts).* The relative values of moment and axial load determine which case will occur in a given instance. Horizontal loads are resisted by shear in the weld between column and base plates, friction and bond between the base plate and concrete, and shear in the holding down bolts. Though ANSI/AISC code does not allow the anchor rods to transfer substantial shear, ACI-318 code, Appendix D gives the limit states to be checked in the anchor bolts, including the shear strength of anchor bolts. Hence ANSI/AISC code suggests the use of a shear key or lug to transfer a large horizontal force from the column to the foundation (see Fig. 7.14). Note that the horizontal loads are generally small except when earthquake loads are considered.

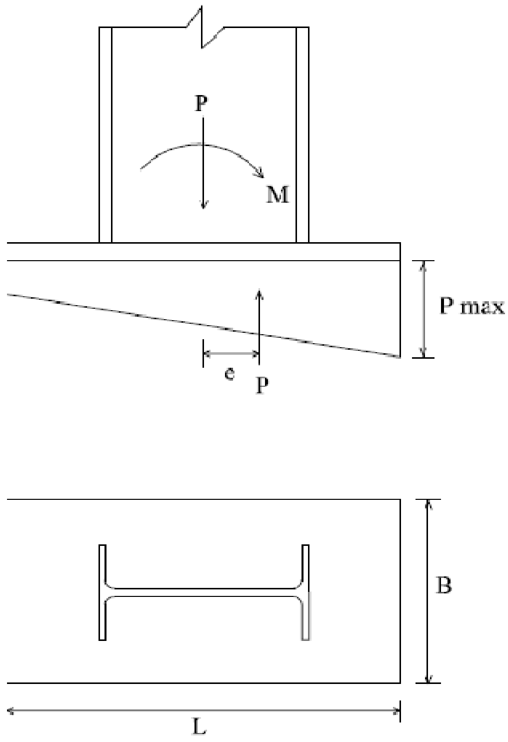


Fig 7.15 Base Plate subjected to axial load and moment

7.12 Design of Base Plate

The design of base plate consists of finding out its size and thickness. The following are the design steps to be followed:

1. Assuming the grade of concrete, calculate the bearing strength of concrete, which is given by $0.45f_{ck}$ as per clause 34.4 of IS 456-2000.
2. Required area of slab base may be computed by
   Find the eccentricity e of applied axial load and moment of the column. [See fig 7.15]

   $$e = p/M \qquad \dots (7.23)$$

   Base plate length by considering there will be compressive pressure over the whole of the base

   $$L = 6e \qquad \dots (7.24)$$

   The required breadth to limit the bearing pressure to 0.45 f_{ck}

   $$b = 2P/(L * 0.45f_{ck}) \qquad \dots (7.25)$$

3. From the above, choose a suitable size for the base plate *L* x *B,* so that *L* x *B* is greater than the required area. Though a few designers prefer to have a square base plate, it is advisable to keep the projections of the base plate beyond column edges.
4. Calculate the maximum pressure intensity, acting below the base plate using

$$P_{max} = \frac{P}{A} + \frac{M_z}{Z_z} \quad \text{... (7.26)}$$

When bi-axial moment then

$$P_{max} = \frac{P}{A} + \frac{M_z}{Z_z} + \frac{M_y}{Z_y} \quad \text{... (7.27)}$$

Where *A* is the provided area of the base plate *(L* x *B)* and M_y and M_z is the minor and major axis moment on the plate and Z_y and Z_z id the modulus about y-y and z-z axis.
5. Calculate the minimum thickness of slab base as per Eqn (7.14). If it is less than t_f thickness of flange of column, provide the thickness of slab base = thickness of flange of column [when only axial force is acting].

   And for axial load and moment consideration, find the pressure intensity at the face of the column and also find the moment at that point then we know that

   Moment capacity of plate= $\frac{1.2 f_y Z_e}{\gamma_{m0}}$ ... (7.28)

   Where $Z_e = t^2/6$

   From that we find the thickness of the base plate.
6. Provide nominal two or four 20-mm holding-down bolts
7. Check the weld length connecting the base plate with the column (this check is required only for fillet welds).

8. SUMMARY AND CONCLUSION

8.1 Summary

The study presented here is based on the Analysis and Design of Industrial Steel Structure as per IS 800-2007. The analysis and design of Gantry girder, purlins, different roof truss, Gable frame, side rail with sag rod, column of Industrial building is carried out. The software is prepared, that directly generate required 3D model of industrial structure in SAP2000. The analysis result is further used for design calculation. The software gives the most suitable section that satisfies all design check. Standard Steel Sections, which are given in steel table, widely used in practice and approved by I.S., are used. Based on the study, programs are prepared and compiled in Visual Basic.

Some silent features of software.

- It is an easy user interface.
- In a form of comment, it provides a help to provide a proper input data.
- It provides a save function for saving all project input data.
- By providing some primary data for industrial building it produce a complete 3D SAP2000 Model.
- It gives a complete solution for analysis and design Gantry girder.
- It takes care of all possible loads acting on an industrial structure.
- It generates all possible load combination for all different loads.
- In a SAP2000 3D model every member has its specific name.
- It provides a member wise filter for showing proper analysis result for all load combinations.
- It gives the suitable design section for every member of the industrial building.
- Finally it produces a report in a form of Excel sheet which gives details of each and every member of structure.

8.2 Conclusion

On the basis of above study the following conclusion can be made:

1) The study gives basic understanding and guidelines of design of different structural components of industrial building.

2) As consult to truss design, it shows that limit state design procedure gives economical section as compare to IS 800-1984.

3) From study of software development, came to know the usefulness and importance of programming. As through programming time consuming process can be reduces and final result produce in couple of seconds.

8.3 Future Scope

The present study will be extended future as follows:

1) Different type of roof truss profiles in software tool.
2) Design the connection for different member.
3) Different types of column section profile in software tool.
4) Prepare a tool that directly produces a detailed drawing of industrial building in AutoCAD.

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3) John A. Rolfes & Robert A. MacCrimmon, *Industrial Building Design - Seismic Issues*, industrial and regional projects, Hatch Acres, Niagara Falls, Ont., Canada.

4) S.L. Chan, *Non-linear behavior and design of steel Structures*, Journal of constructional Steel Research 57 (2001) 1217–1231.

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9) N. Subramanian, *Design of steel structure,* Oxford university press.

10) P. Dayaratnam, *Design of Steel Structures,* 2002.

11) A.S.Arya & J.L.Ajmani, *Design of Steel Structures*, Nemchand & Bros.,Roorkee, 5th Edition, 1996.

12) CODE OF PRACTICE FOR DESIGN LOADS (OTHER THAN EARTHQUAKE) FOR BUILDINGS AND STRUCTURES', Part – 1 Dead Loads, IS: 875 (I) – 1987.

13) CODE OF PRACTICE FOR DESIGN LOADS (OTHER THAN EARTHQUAKE) FOR BUILDINGS AND STRUCTURES' , Part -2 Imposed Loads , IS:875 (II) – 1987.

14) CODE OF PRACTICE FOR DESIGN LOADS (OTHER THAN EARTHQUAKE) FOR BUILDINGS AND STRUCTURES' , Part -3 Wind Loads , IS:875 (III) – 1987(Draft code).

15) CODE OF PRACTICE FOR DESIGN LOADS (OTHER THAN EARTHQUAKE) FOR BUILDINGS AND STRUCTURES' , Part -5 Special Loads and Combinations , IS:875 (V) – 1987.

16) CODE OF PRACTICE FOR GENERAL CONSTRUCTION IN STEEL', IS: 800-2007.

Useful web sites:

1. http://www.sciencedirect.com
2. http://www.vb6.us for VB6 coding
3. http://pages.cpsc.ucalgary.ca
4. http://www.sevillaonline.com/ActiveX/
5. http://www.kxcad.net for SAP2000 API documentation
6. http://www.sefindia.org/

APPENDIX I

SOFTWARE APPLICATION

Input data: (Fig A-1)

1. Length of building = 50 m
2. No. of span = 2
3. No. of Bay = 8
4. Bay Width = 6.25 m
5. Height of building = 12 m
6. Location = Ahmedabad
7. Wind speed= 39 m/sec
8. Design life of structure = 25 years
9. Terrain category = 3
10. Permeability of building = 11%

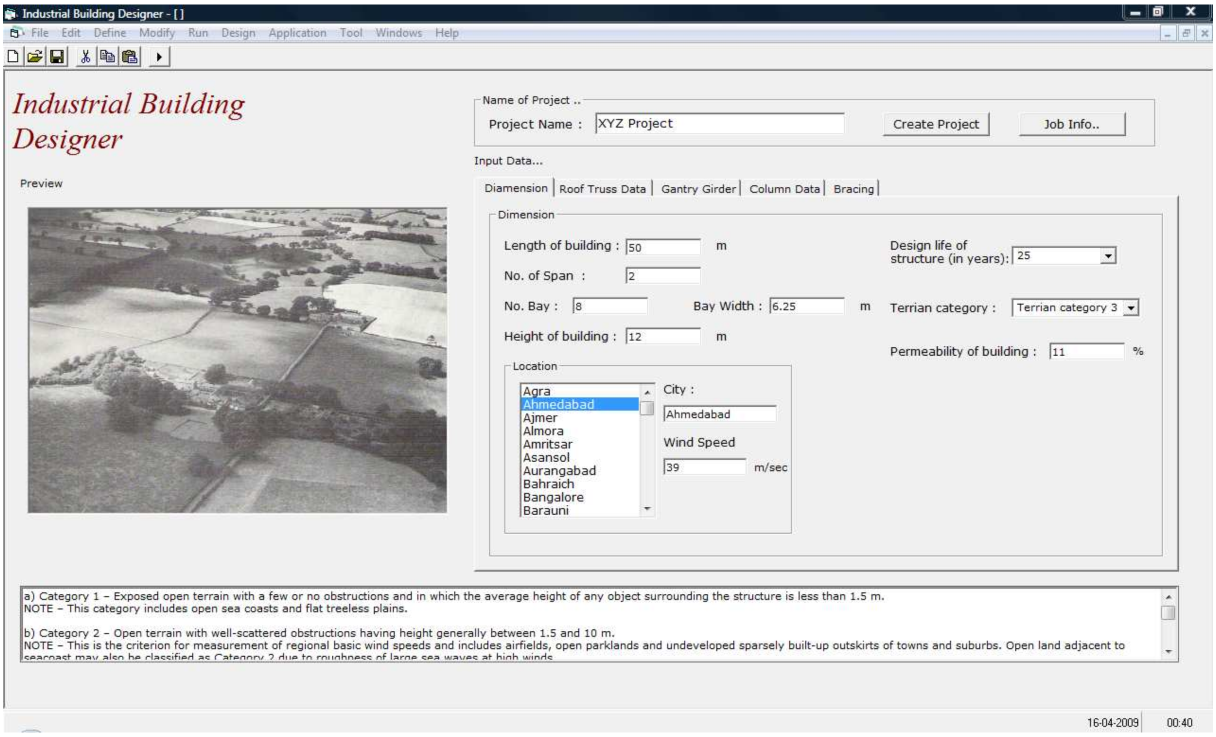

Fig A-1 Screen-Shot of Software Input 1

Roof Truss Data (Fig A-2)

1. Roof material = Asbestos sheets
    a. Length = 1.75 m
    b. Width = 1.01 m

c. Thickness = 6 mm

2. Dead loads
   a. Sheeting = 0.16 kN/m^2
   b. Fixing = 0.025 kN/m^2
   c. Services = 0.01 kN/m^2
3. Roof dimension Span 1(for Gable truss)
   a. Span length = 20 m
   b. Type of roof = Gable frame
   c. Span length L1 = 10 m
   d. No. of division N1 = 8
   e. Span length L2 = 10 m
   f. No. of division N2 = 8
   g. Height H1 = 2.5 m
   h. No. of purlins(left Span) = 8
   i. No. of Purlins(Right Span) = 8
4. Roof dimension Span 1(for Main truss) (fig A-3)
   a. Span length = 20 m
   b. Type of roof = Gable frame
   c. Span length L1 = 10 m
   d. No. of division N1 = 8
   e. Span length L2 = 10 m
   f. No. of division N2 = 8
   g. Height H1 = 2.5 m
   h. No. of purlins(left Span) = 8
   i. No. of Purlins(Right Span) = 8
5. Roof dimension Span 2(for Gable truss)
   a. Span length = 20 m
   b. Type of roof = Gable frame
   c. Span length L1 = 10 m
   d. No. of division N1 = 8
   e. Span length L2 = 10 m
   f. No. of division N2 = 8
   g. Height H1 = 2.5 m
   h. No. of purlins(left Span) = 8
   i. No. of Purlins(Right Span) = 8

6. Roof dimension Span 2(for Main truss)
    a. Span length = 20 m
    b. Type of roof = Gable frame
    c. Span length L1 = 10 m
    d. No. of division N1 = 8
    e. Span length L2 = 10 m
    f. No. of division N2 = 8
    g. Height H1 = 2.5 m
    h. No. of purlins(left Span) = 8
    i. No. of Purlins(Right Span) = 8

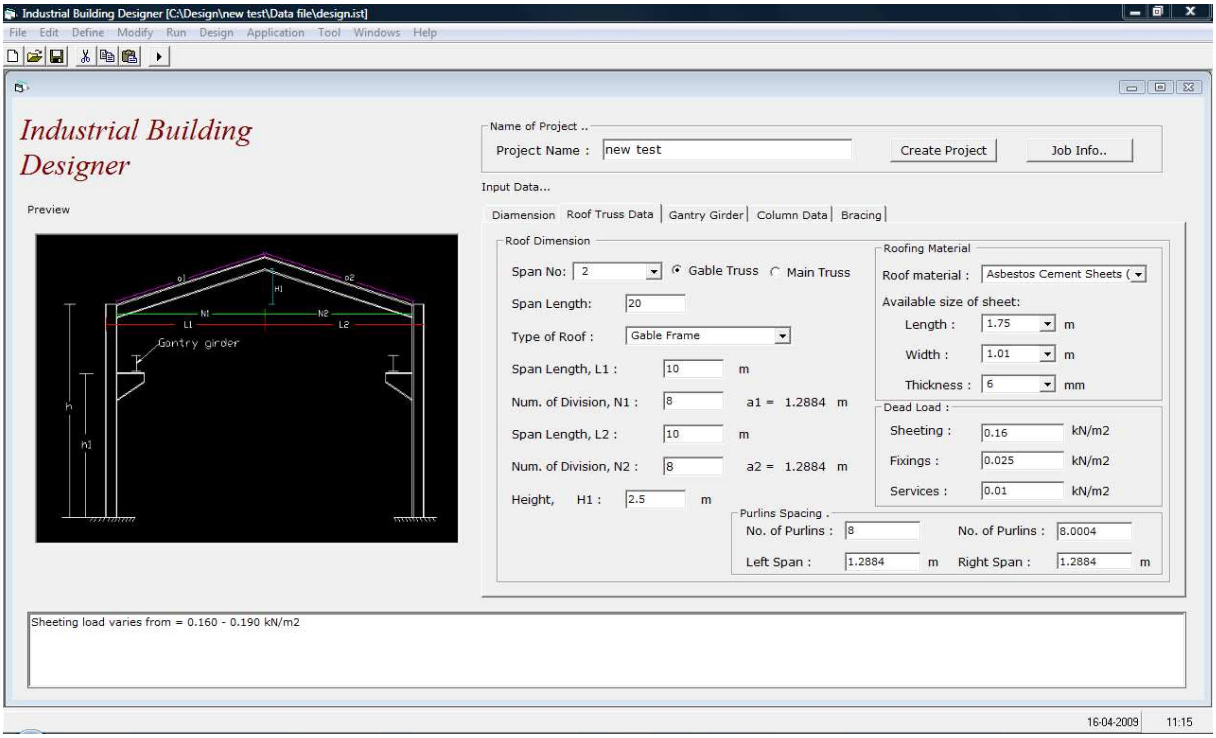

Fig A-2 Screen-Shot of Roof Truss Data Input(Gable Truss)

Gantry Girder Data Input (Fig A-4)

1. No. of Gantry Girder = 2
2. For gantry girder 1
    a. No. of crane = 2
    b. Crane capacity (crane 1) = 50 kN
    c. Crane capacity (crane 2) = 100 kN
    d. Minimum hook approach (crane 1) = 1m
    e. Minimum hook approach (crane 2) = 1m

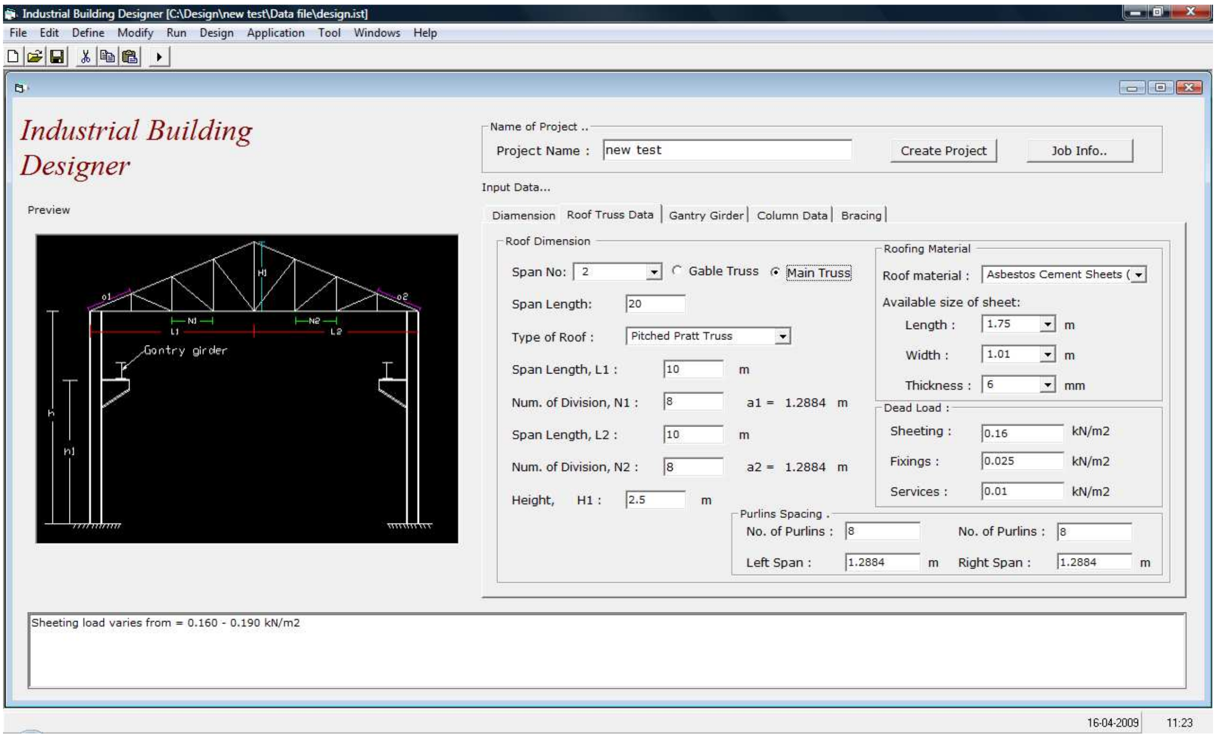

Fig A-3 Screen-Shot of Roof Truss Data Input(Main Truss)

f. Centre to centre distance bet^n gantry rail = 19 m
g. Distance between wheel centre (crane 1) = 3 m
h. Distance between wheel centre (crane 2) = 3.2 m
i. Distance between two cranes = 1 m
j. Self weight of crane girder 1 = 50 kN
k. Self weight of crane girder 1 = 100 kN
l. Self weight of trolley (crane 1) = 15 kN
m. Self weight of trolley (crane 2) = 35 kN
n. Self weight of rail section = 0.4 kN

3. For gantry girder 2 (Fig A-5)
   a. No. of crane = 1
   b. Crane capacity (crane 1) = 150 kN
   c. Minimum hook approach (crane 1) = 1m
   d. Centre to centre distance bet^n gantry rail = 19 m
   e. Distance between wheel centre (crane 2) = 3.5 m
   f. Self weight of crane girder 1 = 200 kN
   g. Self weight of trolley (crane 1) = 60 kN
   h. Self weight of rail section = 0.4 kN

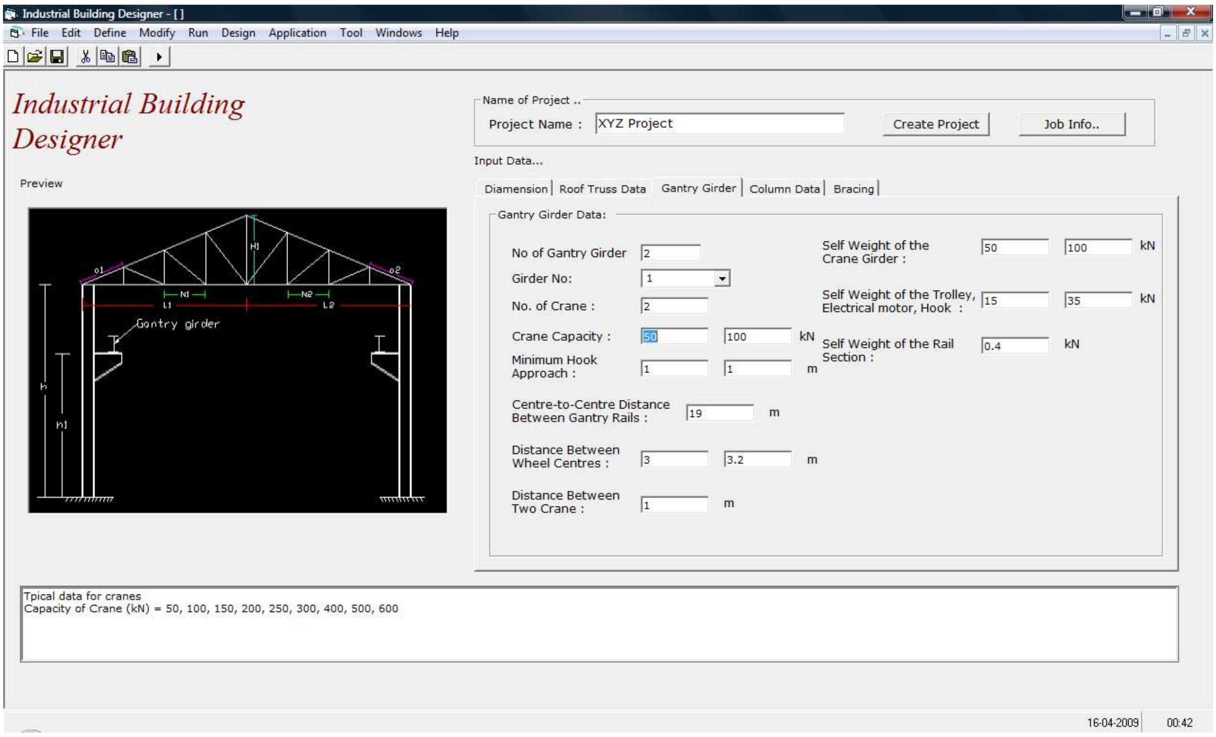

Fig A-4 Screen-Shot of Gantry Girder Input Data for Girder 1

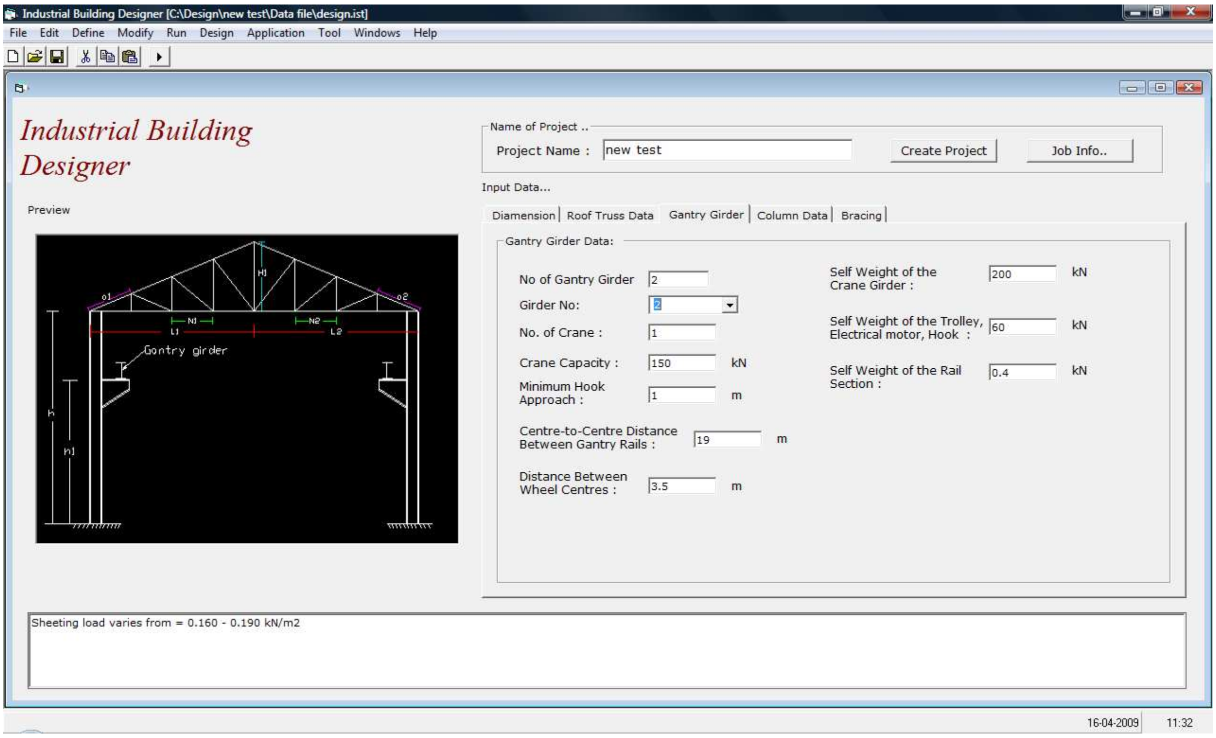

Fig A-5 Screen-Shot of Gantry Girder Input Data for Girder 2

Column Data Input (fig A-6)

1. For Span 1
    a. Column section
        i. Left Side Column = Prismatic section with left G.G
        ii. Middle Column = Prismatic section
        iii. Right side Column = Prismatic section with right G.G
    b. Gantry girder no = 1
    c. Height of gantry girder = 9 m
    d. No. of column (Gable Truss) = 4
        i. Spacing 1 = 5.714 m
        ii. Spacing 2 = 8.571 m
        iii. Spacing 3 = 5.714 m
    e. No of column (main truss) = 2
        i. Spacing 1 = 20 m
2. For Span 2
    a. Column section
        i. Left Side Column = Prismatic section
        ii. Middle Column = Prismatic section
        iii. Right side Column = Prismatic section
    b. No. of column (Gable Truss) = 4
        i. Spacing 1 = 5.714
        ii. Spacing 2 = 8.571
        iii. Spacing 3 = 5.714
    c. No of column (main truss) = 2
        i. Spacing 1 = 20 m
3. No. of grits = 10
4. Grits Spacing =1.2 m
5. C_{f}, Surface coefficient = 0.02

Bracing Data Input (Fig A-7)

1. Side column
    a. Bay 1 = true
    b. Bay 4 = true
    c. Bay 5 = true

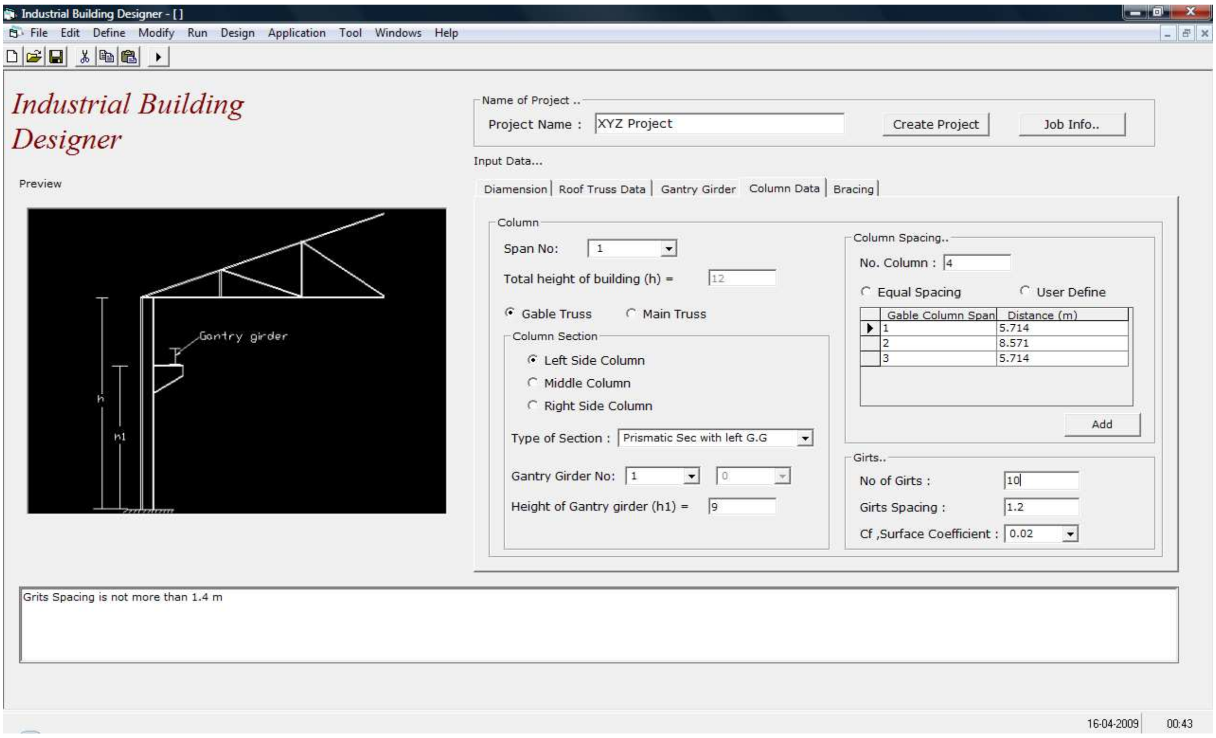

Fig A-6 Screen-Shot of Column Data Input

d. Bay 8 = true

2. Front Column Bracing
   a. Span 1
      i. Space 1 = true
      ii. Space 3 = true
   b. Span 2
      i. Space 1 = true
      ii. Space 3 = true
3. No. of Cross Bracing = 5
4. Roof Bracing
   a. Bay 1 = true
   b. Bay 4 = true
   c. Bay 5 = true
   d. Bay 8 = true
   e. No. node N1 = 3
   f. Distance from edge = 4.417
   g. No. Node N2 = 3
   h. Distance from edge = 4.417

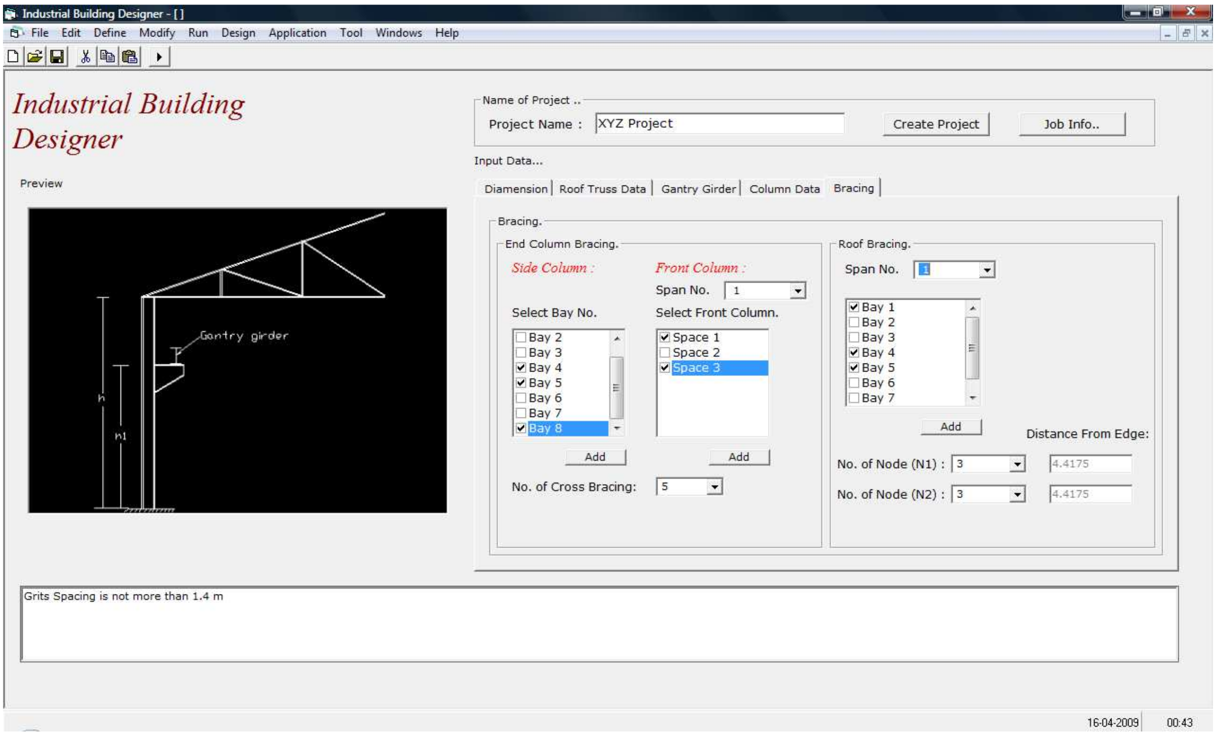

Fig A-7 Screen-Shot of Bracing Data Input

Design of Gantry Girder (Fig A-8)

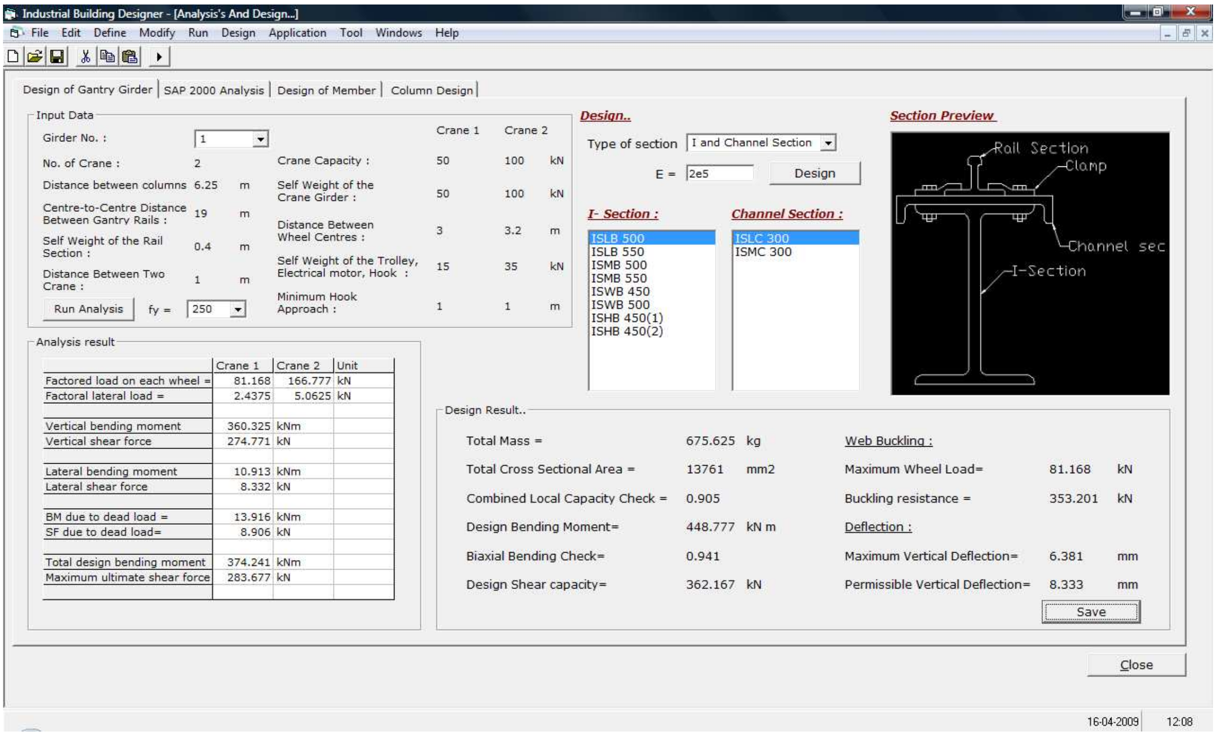

Fig A-8 Screen-Shot of Gantry Girder Design

Fig A-8 shows the analysis and design result of the gantry girder, the software application design in a such a way that it can analysis any combination cranes load(up to 250 kN) and give the accurate required bending moments and shear force. On that base the software suggests the most suitable combination of I-section and channel section. By selecting required I-section and channel section, then it will shows the all design checks such as design bending moments, biaxial bending checks, design shear capacity, buckling resistance, maximum vertical deflection ,permissible vertical deflection and total weight of section.

Generating the SAP 2000 Model (Fig A-9)

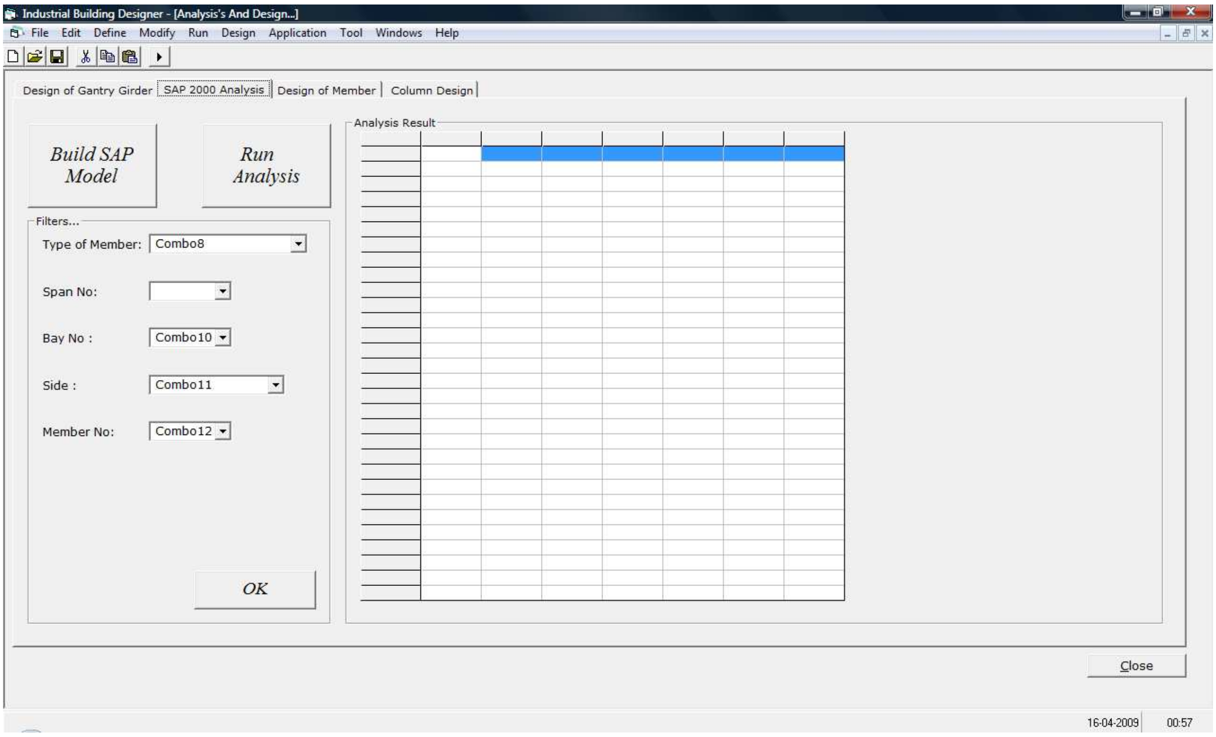

Fig A-9 Screen-Shot of SAP 2000 Analysis

Fig A-9 shows the SAP2000 Analysis screen. In this screen there is "Build SAP Model" tab when we click that tab then the SAP2000 (fig A-10) software start to build a required industrial building 3D model (fig A-11). Complete 3D Model show in (Fig A-12) and apply all calculated Dead loads, Live loads, winds loads and cranes loads, show in (fig A-13). Then it will run SAP2000 Analysis (fig A-15) and generates all analysis result data and by applying member wise filters we can get a proper required analysis data show in (fig A-16). Then it will calculate the member wise maximum analysis data such as maximum bending moment, shear force, axial force on that bases it will suggest the suitable design section with all required possible checks. In case, the designer wants to provide some different section then he can be provided by clicking "Redesign" tab (fig A-18).

Fig A-10 Screen-Shot of SAP 2000

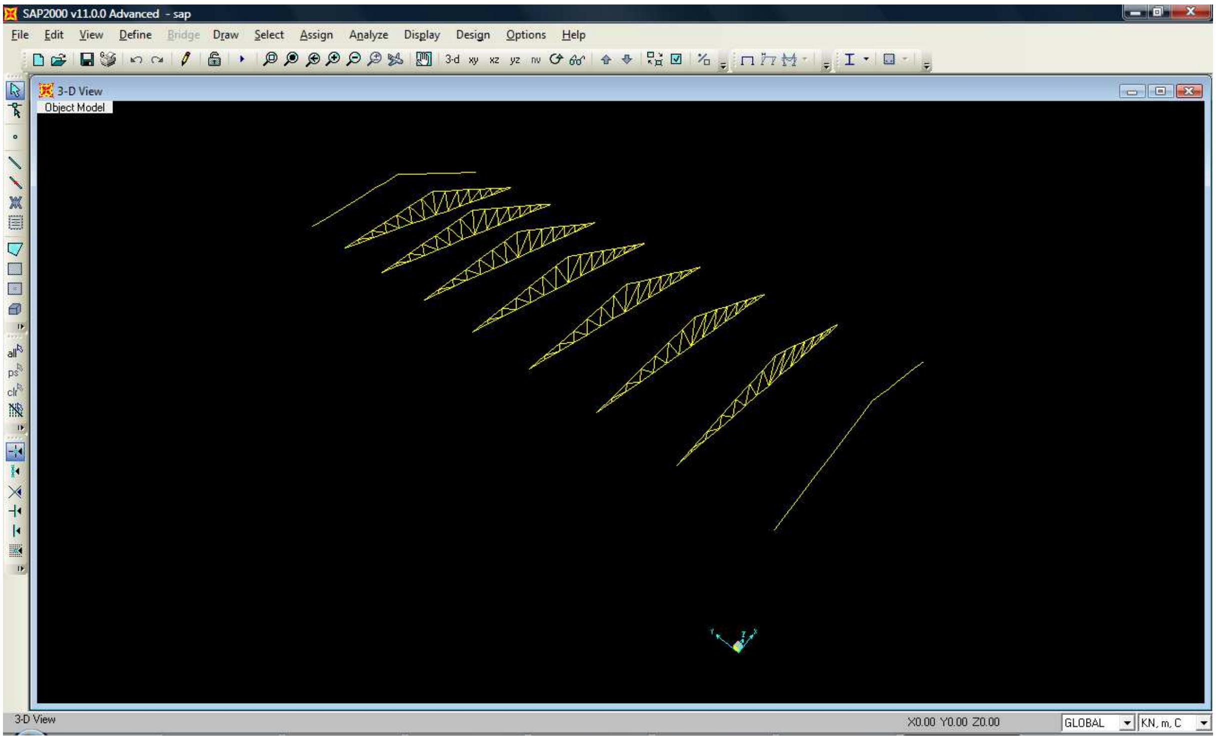

Fig A-11 Screen-Shot of SAP 2000 Generating Model

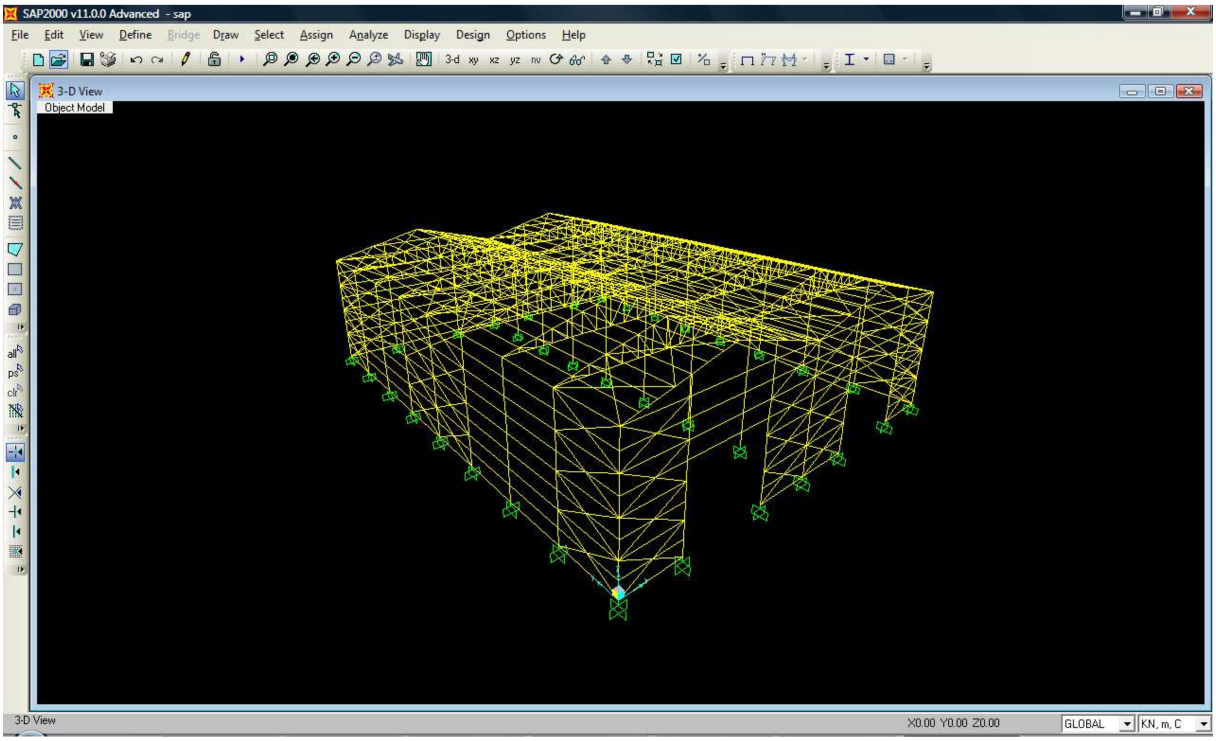

Fig A-12 Screen-Shot of Complete SAP 2000 3D Model

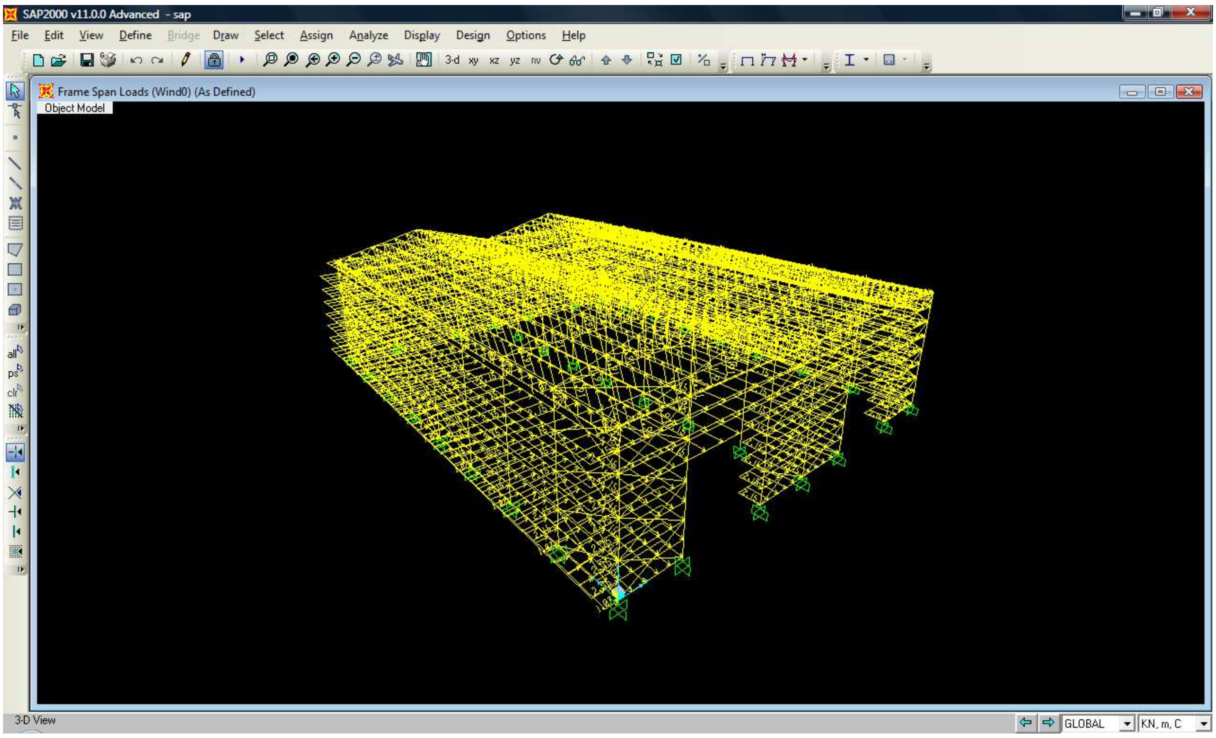

Fig A-13 Screen-Shot of Complete SAP 2000 3D Model with all Loads

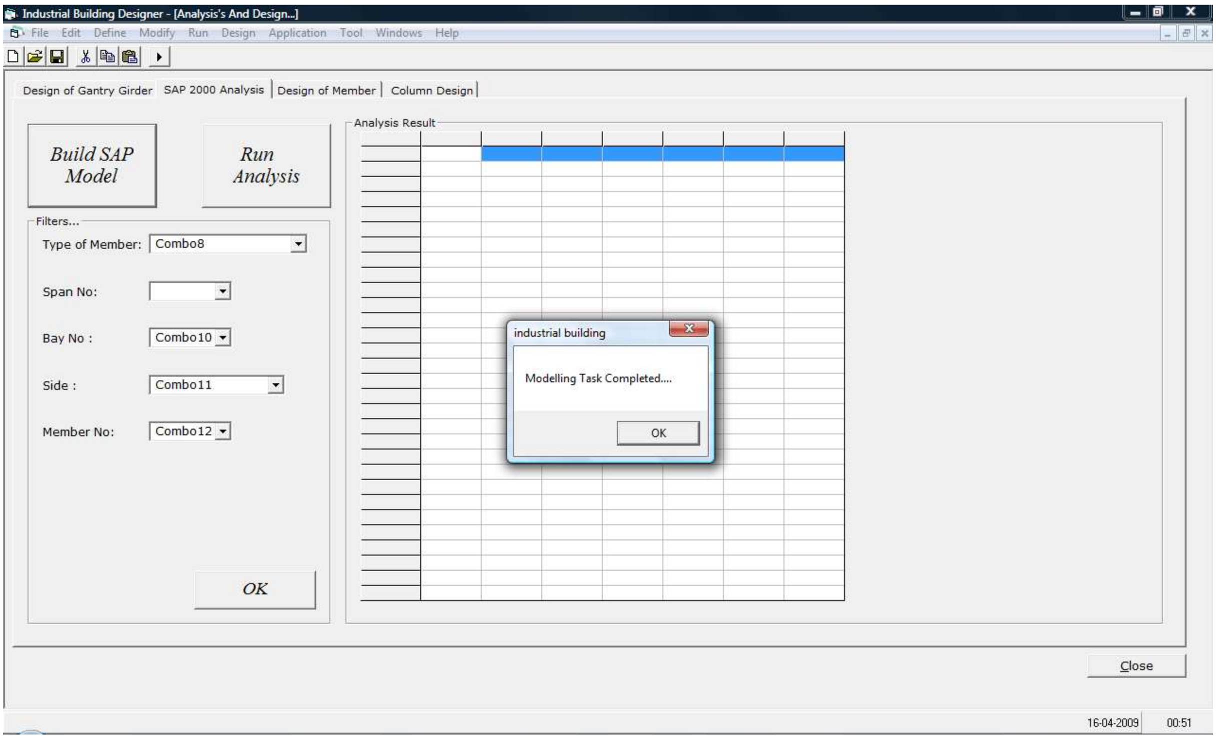

Fig A-14 Screen-Shot of Complete SAP 2000 3D Model

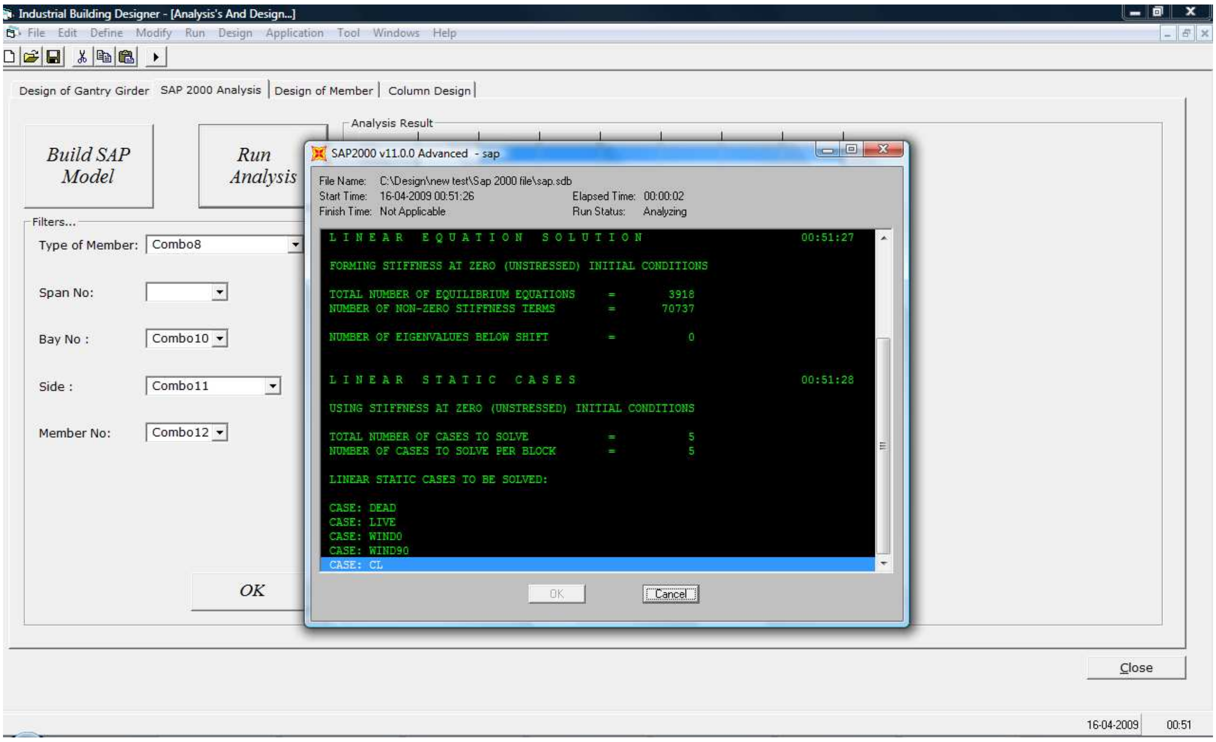

Fig A-15 Screen-Shot of Run SAP 2000 Analysis

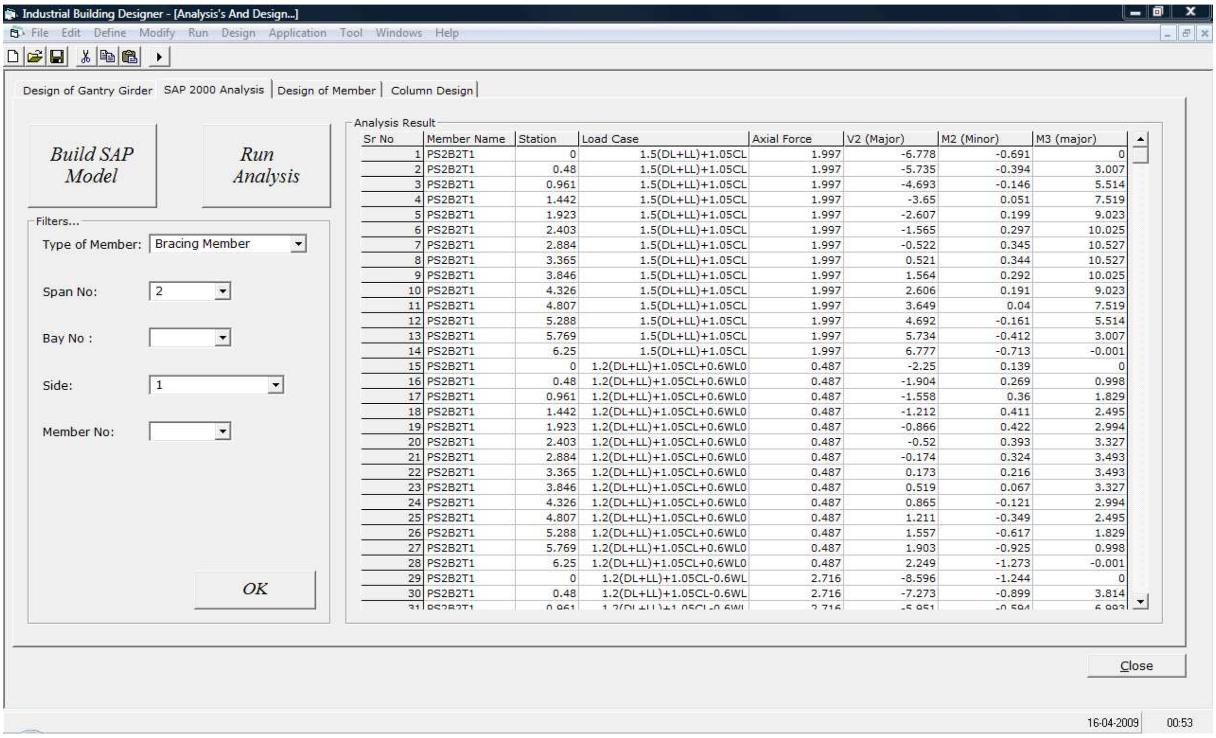

Fig A-16 Screen-Shot of Member wise Analysis result

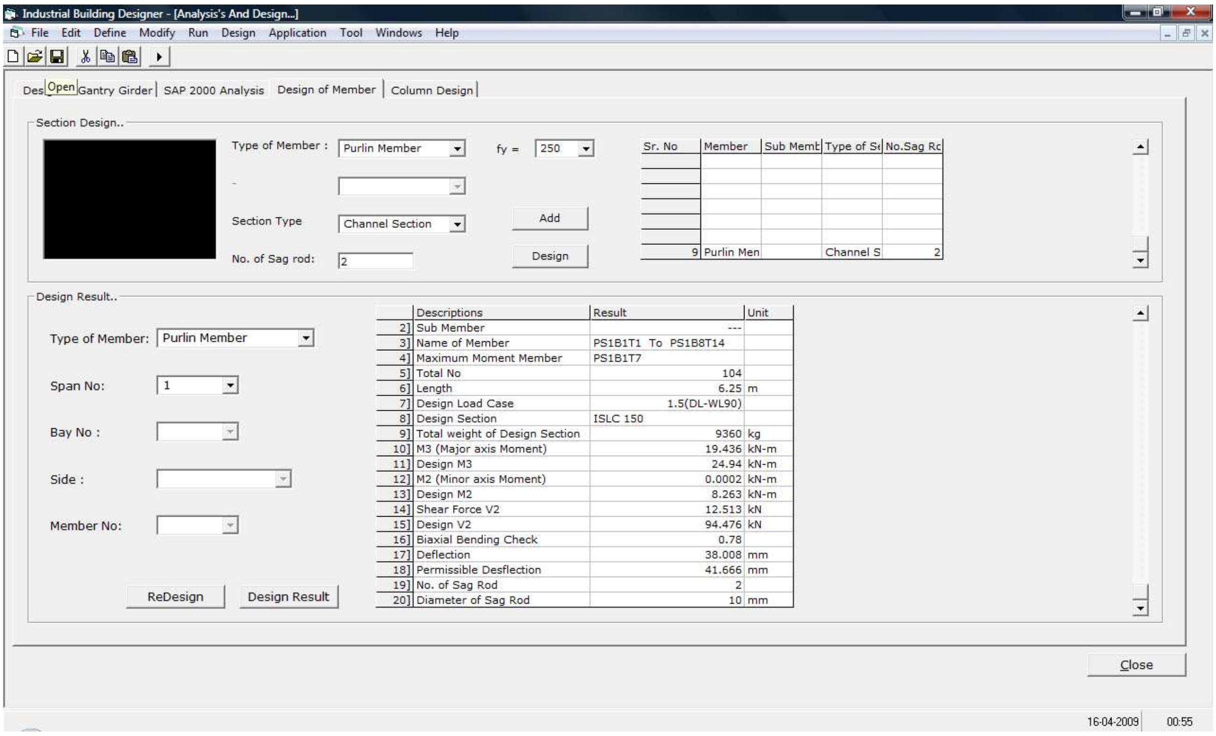

Fig A-17 Screen-Shot of Member design (Purlin member design result)

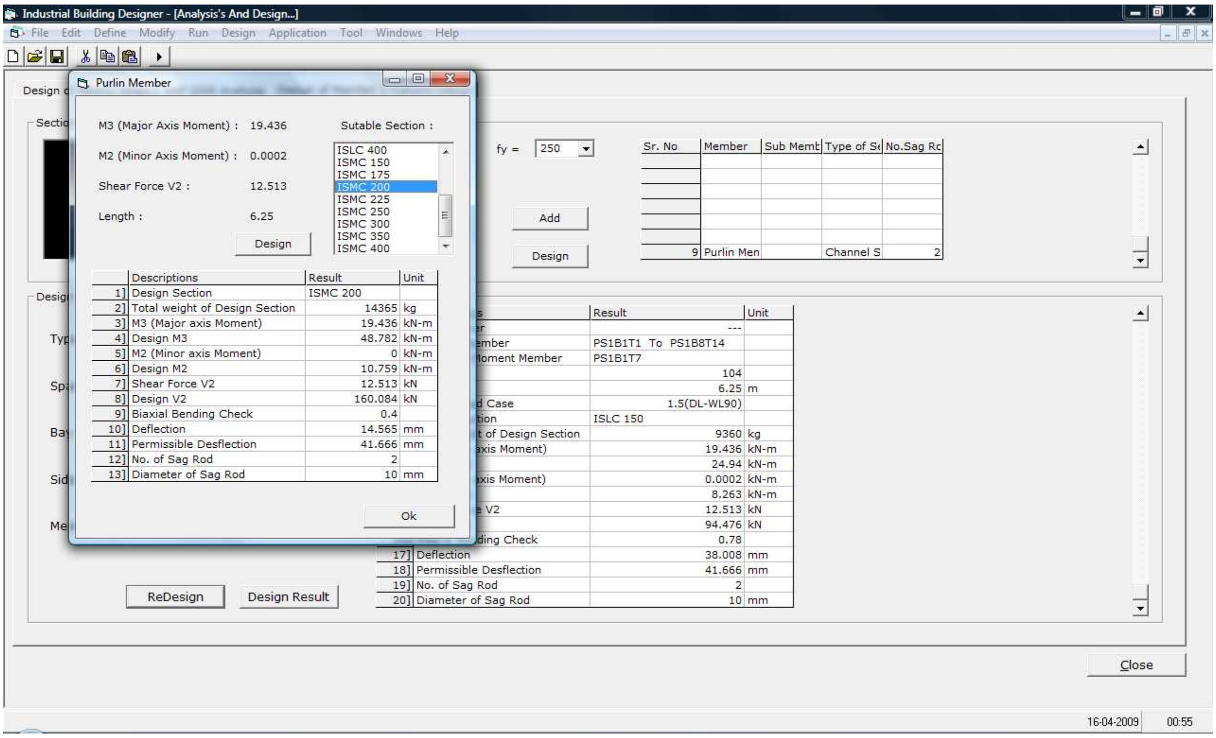

Fig A-18 Screen-Shot of Modifying Member design result (Purlin member design result)