OPTIMAL DESIGN OF INDUSTRIAL BUILDING

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I. INTRODUCTION

1.1 GENERAL

The term industrial building has come to mean any building structure used by industry where at least a part of the enclosed area is of one-story height. Included are buildings for steel mills, structural shops, train sheds, automotive assembly plants, and air- craft factories. These diverse structures all have the common requirement of large open floor areas frequently requiring roof trusses that provide adequate headroom for the use of an overhead travelling crane.

1.2 CLASSIFICATION OF INDUSTRIAL BUILDING

Regarding the details of loading conditions and method of calculations the cycle as mentioned here in under for classifying an industrial building, reference shall made to IS: 8640-1977

Group A: Cover the industrial buildings where certain members may experience 500000 to 2000000 repetitions of loading condition 3 or 2 million and above repetition of loading condition in the estimated life span of building of 50 years. After considering the service, determination of loading conditions shall be decided. The main industries that will fall under this category will be batch annealing building, billet yard, continuous casting building, foundries, mixer building, and mould conditioning building. Scraping yards, steel making building and other building based on predicted operational requirements.

Group B: Cover the industrial buildings where certain members may experience a repetition of 100000 to 600000 cycle of specified loading condition in the estimated life span of about 50 year. The industries consider under this group are: Metal industries for manufacturing equipment like heavy machinery, boiler, ships, locomotives, aircrafts and other building based on predicted operational requirement.

Group C: Cover the industrial buildings where certain members may experience repetition of 20000 to 100000 cycle of specified loading condition in the estimated life span of about 50 years. The industries consider under this group are: Industries for manufacturing cars, scooters, inch moving equipment, machine shops and other building based on predicted operational requirements.

Group D: Cover the industrial buildings where certain members may experience below 20000 repetition of specified loading condition in one estimated life span of about 50 years. The industries consider under this group are: Generally all the light, utility and process industries.

Group E: Cover industrial structure that requires special consideration based on the process or utility and which may not be provided with cranes or if provided with cranes, they may be used only for maintenance. In such cases in addition to the dead loads, wind/seismic forces, live loads/superimposed loads as required for each individual situation shall be considered. In addition, stresses due to temperature caused by the process, airborne vibration, special needs of height, etc. may have to be considered. Typical structures under this group are: Thermal power stations, fertilizer units, petrochemical units, transformer test stations. Compressor house, textile mills, paper mills, etc.

Group F: Structure which are not provided with cranes and that which do not come under group E. These structures generally are of simplest type involving live, dead and wind/seismic load. Typical structures under this group are: Storage building garages, repairs shop without cranes, consumer goods manufacturing units, small scale industries where use of crane is not required

1.3 OBJECTIVE

1. In a developing country like India the capital outlay under each Five Year Plan towards setting up of industries and consequently construction of industrial building is very high.
2. In addition the quantity of steel produced in the country is not sufficient to meet the requirement of the industries.
3. It is therefore necessary that the parameters of the industrial building have to be optimizing by considering economy, stress and safety.
4. To Check The Economy Of Industrial Building By Plastic Design
5. To Check The Utilisation Of Material Strength By Conventional Elastic Design
6. To Check The Defection Of Industrial Building By Plastic And Elastic Design

1.4 STRUCTURAL STEEL FOR INDUSTRIAL BUILDING

Compared to other materials, particularly reinforced or prestressed concrete, steel has major advantages. Its high electricity-to-weight ratio and its excessive tensile and compressive power allow steel homes to be of relatively light construction. Steel is consequently the most suitable material for long-span roofs, wherein self-weight is of top importance. Steel buildings can also be modified by connecting steel sections to existing work.

1.5 CHOICE OF INDUSTRIAL BUILDING

Initial options in respect of preferred location, site acquisition and environmental needs must first be decided. Then main dimensions, process operation, plant layout. Foundation needs, handling systems, day lighting, environmental control, service routes, staffing level and access all require definition. The location of internal columns and the internal headroom are always important, and considerations of these requirements alone may determine the choice. The advantage of freedom to plan the building to suit requirements closely and allow for future development is very valuable.

II. LITERATURE REVIEW

1) Amin Saleh Aly "Stability of pitched roof frames in national building codes" journal of structural engg. Vol.119 No.10, October, 1993 pp 30903093

The buckling loads, buckling lengths, and P-delta effects for pitched roof frames with different loading arrangements are examined. Results are in comparison to some not unusual and the world over known code results It was found that the load distribution significantly affects the buckling loads and hence the buckling lengths and the magnified (or added) moments. This paper clarifies a defect in some national building codes and encourages the use of computers in determining the buckling loads and the magnified moments. It is worth noting that despite the fact that the program suggested is simple, it's miles sturdy and will be used to hint the huge deformation characteristics of frames.

2) B. Somashekar Rao, C. Thiagarajan, Md. Tafseeruddin "Design and Construction Feature of a Tall Single Bay Steel Structure" Advanced Design of Steel Structure ppt 148-155

This chapter describes the planning, design and construction features of a Tall Single Bay Single Storey Steel Structure built at Valiamala, Trivandrum for Department of Space. costing about Rs.62.14 lakhs. Details of analysis and the cost of various components of the structure have been discussed in terms of percentage. The typical twin column with knee- braced truss was analysed using the general purpose Structural Analysis Program (SAP-IV). The structure was described by means of 3D beam element and 3D truss element. The structure was analyzed as a plane frame suppressing Z-displacements. The entire descretised structure comprises of 64 nodes, 46 beam elements and 79 truss elements with 4 types of section properties for beam element and 6 types of section properties for truss elements.

3) D.E.Grierson "Optimal Design of Structural Steel Framework" Journal of computing system in Engg. Volume 2 No.4 1991 pp 409-420

This paper reviews work conducted at the University of Waterloo during the 1980s concerning the computer-automated design of least-weight structural steel frameworks. First, design under static loads is considered whereby the members of the structure are Automatically sized using commercial steel sections in full conformance with design standard provisions for elastic strength/stability and stiffness. This problem is illustrated for the least-weight design of a steel mill crane framework comprised of a variety of member extended to the least-weight design of structural steel framework We both service and ultimate loading conditions. Here, acceptable elastic stresses and displacements are ensured at the service-load level while, simultaneously, adequate safety against plastic collapse is ensured at the ultimate-load level. This design problem is illustrated for the least-weight design of an industrial steel mill framework for which plastic behaviour is governed by conservative piecewise linear yield conditions. Finally, - computer-based design methodology is extended to the least-weight design of structural steel framework subjected to dynamic loading Constraints are located on dynamic d i stress, herbal frequencies and member sizes. The design hassle is illustrated est-weight design of a metallic trussed arch subjected to non-structural loads and an impulse force.

4) Donald W. White "Plastic Hinge Method for Advanced Analysis of Steel Frame" journal of Construction Steel research 24(1993) pp 121-152 A number of recent research efforts have focused on he expand Advanced analysis techniques and their possible application in limit-states design of steel structures. The new (Australian Standard) AS 41001990 lets in the usage of this kind of evaluation for the design of frames in which the individuals are of compact segment and are sufficiently restrained towards lateral-tensional buckling to increase the system's ane cancel The erm superior' is intended to indicate any approach of evaluation that sufficiently captures the restrict states encompassed through specification equations for member proportioning such that the checking of such equations isn't always required. The first a part of this paper presents an in depth research of the adequacy of second-order plastic-hinge based totally techniques for use as superior evaluation techniques. This is observed by way of a discussion of one possible method for attention of geometric imperfection results in advanced evaluation/layoute erm advanced' is intended to indicate any method of analysis that sufficiently captures the limit states encompassed by specification equations for member proportioning such that the checking of such equations is not required.

5) Gregory A. Kopp, Christian Mans, David Surry "Wind effect of parapets on low building: part 2 structural loads" Journal of wind Engg. And industrial Aerodynamics 93(2005) pp 843-855

The present paper, Part 2 in a four part series, focuses on the effects of solid. perimetric parapets on the wind-induced structural loads on low-rise buildings. Roof and wall pressures were measured more than 500 locations simultaneously for five parapet heights (h 0, 0.46, 0.9, 1.8 and 2.7 m in equivalent full-scale dimensions) and three building heights (H 4.6, 9.1 and 18.3 m) with plan dimensions 31.1 by 61.6m and a on 12 gable roof slope.

6) H. Randolph Thomas, Daniel M. Brown "Optimum least cost design of A truss roofsystem" Journal of computer and structure, Volume 7 1977 pp 13-22

An algorithm is presented encompassing the application of optimization methods to the least cost elastic design of roof systems composed of rigid steel trusses, web joists and Steel roof deck. The method is capable of designing rigid trusses that can be fabricated from Various grades of steel and several types of standard sections. The selection of open web joist is presently limited to standard H-series, and decking material is standard 22 gages.

7) J.D. Ginger, J.D. Holmes "Effect of building length on wind load on low-rise building with a steep roof pitch" journal of wind engineering and industrial aerodynamics Vol.91.2003.pp1377-1400

A wind tunnel model study was carried out on long, low-rise building with a steep roof pitch to determine the effect of the length to span aspect ratio on the external wind pressure distribution. The study showed a significant increase in the magnitude of the negative pressure coefficient on the leeward roof and wall, with an increase in aspect ratio, for oblique approach winds. These large suction pressure also general large design wind load effect on the frames near the gable end. The 1989 edition of the Australian standard for wind load as 117021989 was found to underestimate the wind loads on steep pitch gable roof building of aspect ratio greater than 3.

8) J.Isenberg, V.Pereyra, D. Lawyer "Optimal design of steel frame structure" journal of Applied Numerical Mathematics, Vol40, 2002 pp59-71.

He considers the optimal design of steel structure. Given a preliminary design, he attempt to minimize the total weight of the structure subject to various loads and constraints. He combine finite element structural analysis codes with nonlinear programming optimization ones. Several examples are given.

9) Lip H. Teh, Marray J. Clarke. "Plastic zone analysis of 3D steel frames using beam elements" journal of structural engineering, Vol. 125. Nov 1999, 1328-1337

This paper presents a co-rotational formulation of spatial hemant for the purpose of 3D plastic zone analysis of steel frames composed of compact tubular section and section with no significant torsional wiring. The special feature of this paper which have not been discussed in detailed by other researchers, include the proportional of a torsional strain-displacement relationship for rectangular hollow section in the elucidation of the proper force recovery procedure for displacement based plastic zone beam element hues convening of the meta model and the use of a consistent lanzini operator in geometrically nonlinear analysis are briefly discussed.

10)MahenMahendran, Costin Moor "Three Dimensional Modeling of Steel Portal Frame Building "Journal of structural Engg. Vol.125 No. 8 Aug1999 pp 870-878.

The realistic strength, and deflection behavior of industrial and commercial steel portal frame buildings are understood only if the effects of rigidity of end frames and rolled steel cladding are included. The conventional designs for these effects and are very much based on idealized two-dimensional (2D) frame behavior. Full-scale test of a 12 X 12 m steel portal frame building under a range of design load cases indicated that the observed deflections and bending moments in the portal frame were considerably different from those obtained from a 2D analysis of frames ignoring these effects.

/11) Michael Kasperski “Design wind load for low-rise building: A critical review of wind load specification for industrial building" Journal of wind Engg. And Industrial Aerodynamic 61(1996) pp 169-179.

From extensive wind tunnel tests on low-rise buildings with flat roofs. two wind load distributions have been identified to be decisive for the design of the steel frames. Based on these results. the Euro code introduces a novel alternative wind load distribution with a positive roof pressure. In many design-practical , mon of the code reviewed do not sufficiently cover this wind action and its effects, leading to a possible under-design of the cross-sections of up to 35%. The recent trend to lighter construction and non-linear design med further investigations of possible resonant effect. First results indicate that dynamic effects may have some additional influence on the design

12) M.N.Chandrasekaran "Structural design of crane and gantry girder-certain discrepancies in IS code provision Advanced steel structure pp 63-67.

This article deals with the discrepancies observed among different codes in some of the norms adopted in structural design of EOT Cranes and highlights areas where guidance is missing in IS Codes. This calls for review of code provisions and for advocating uniform standards where possible end inclusion of additional guidelines in IS Codes wherever required.

13) M.P. Saka "Optimum design of pitched roof steel frame with haunched rafter by, genetic algorithm" journal of computer and structure, Vol 81,2003,1967-1978.

The pitched roof steel frames appear to be the simplest structure form used in single single storey building. However design is necessitates condideration of many different structural criteria that are required in the design of complex structure. In this paper a genetic algorithm is used to develop an optimum design method for pitch steel frames with haunched for the rafter in the eaves. The algorithm selects the optimum universal beam section for column and rafter from the available steel section table.

14) Raymond H. Plaut," "Requirement for lateral bracing of columns with two spans" journal of structural engineering,Vol.119.No.10,Oct. 199329132931 VolI19,No.10,0ct. 199329132931 No.10,Oct. 1993,2913-2931

Lateral bracing requirements for two span elastic column are examined. The braced are represented by elastic translation springs. The column have pinned supported at the base, an internal braced at an arbitrary point and either a pinned support or a braced at the top,where -the axial load is applied. They may be perfect or may have an initial deflection.

III. INTRODUCTION TO BRACED INDUSTRIAL BUILDING

The industrial buildings are planned on basis of operation to be performed during the manufacturing processes. Industrial building may be classified into two categories:

1. Normal or simple industrial building
2. Sophisticated industrial building.

Normal industrial building consists of single storey industrial shed, with or without gantry girder, to house workshop, warehouses or factories, and do not contain Internal columns Sophisticated industrial buildings, usually called steel mill buildings, are used to house big industries in which some manufacturing processes, need spaces with specific and controlled environmental condition

3.1 MAJOR COMPONENT OF INDUSTRIAL BUILDING

The major component of industrial building in general as shown consist of following

a) Roof Truss
b) Gantry girder
c) Side rails (or girt) with Claddings
d) Gable rafter
e) Gable column
f) Rafter bracing
g) Vertical bracing in longitudinal side
h) Gable wind girder at eaves level
i) Eaves girder
j) Main column
k) Column brackets

IV. ANALYSIS AND DESIGN OF BRACED INDUSTRIAL BUILDING

Design an Industrial building for following data
Plan area 24.2 x 60m Span of one truss-12.Im
Height of column at eaves level -13m
Height of column at gantry level 8.5m
Longitudinal spacing of column-7.5m
Type of roof truss = compound fink type
Rise of truss= ¼ of span.
Capacity of gantry = 200 KN
Building is situated at Pune MIDC.

4.1. Roof Truss

4.1.1 Arrangement of purlins:

Span of truss-12.1m
Assume pitch =1/4 of span,

Pitch=1/4 x 6.05m=1.5125m

Slope of roof truss

$\phi=\tan^{-1}(1.5125/6.05)$

$=14.03$, $\sin\phi= 0.2424$, $\cos\phi=0.9701$.

Length of top chord member = $\sqrt{(6.052+1.5122\ 2)}$ = 6.23m.

Dividing the length into four equal panel, first two panel length is 1610mm and rest panel length is 1510mm. to achieve economy and according to length sheet purline are provi at intermediated panel points. The arrangement of purline as shown. Let us provide five purline at each rafter leaving gap of 150mm at apex.

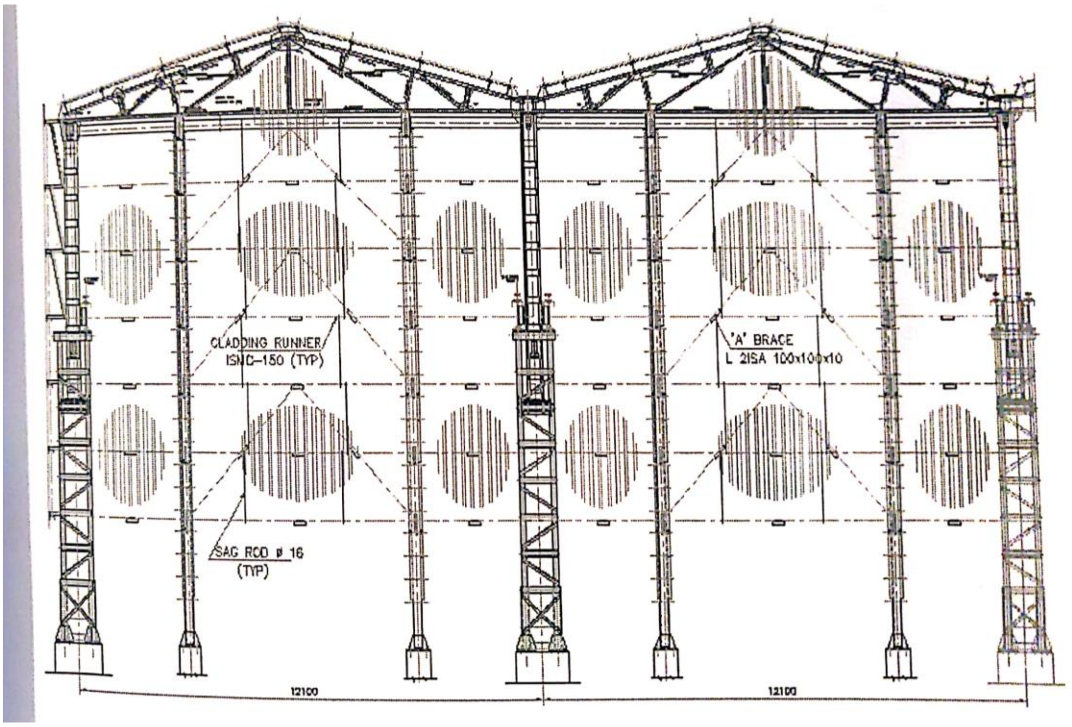

FIG 11 ELEVATION OF BUILDING

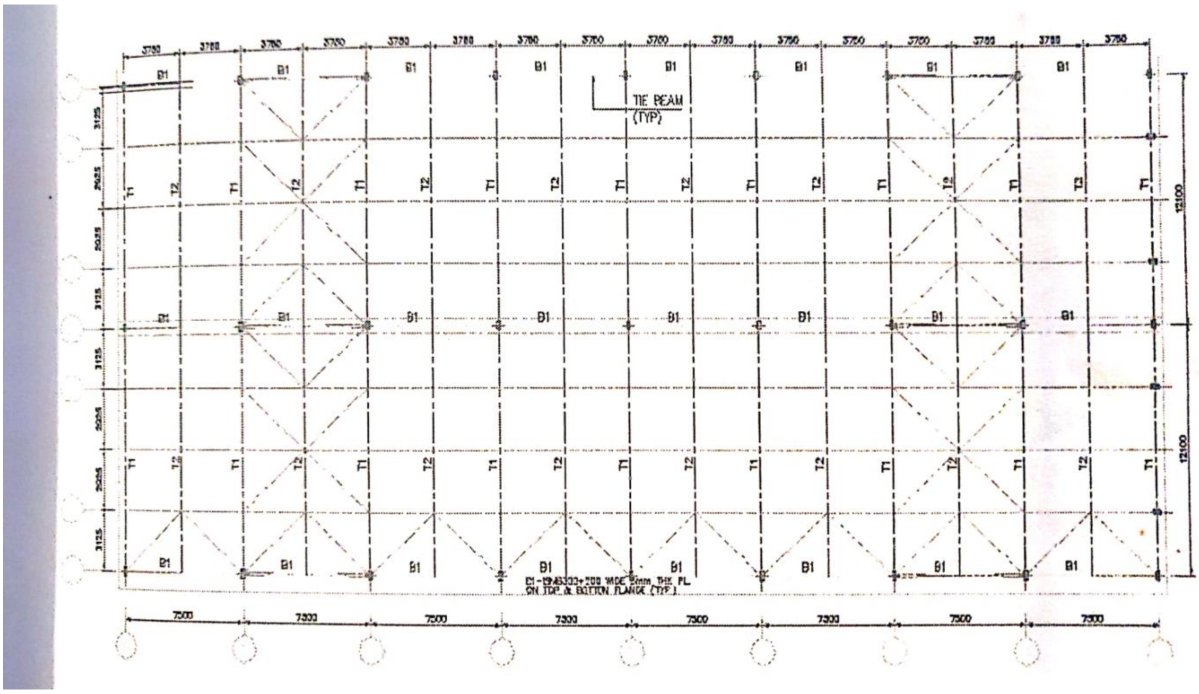

FIG 12 PLAN OF BUILDING

4.1.1.1 Computation of Dead and Live Load

Mass of galvanized sheets (plain) = 5.55 Kg/m^2
Weight of sheet = 5.55 x 9.81
= 55 N/m^2
Weight of sheet per meter length of purlin = 55 x 1.80
= 99 N/m
Assume weight of purlin per meter length = 100 N/m
Dead load (Wd) per meter length = 99+100
= 199 N/m
Total dead load on purlin Wd = 199 x 3.75
= 746.25 N
Live load on purlin = 750 –(ϕ-10) 20
= 670 N/m^2
Live load for design of roof truss = 2/3 of L.L. on purlin
= 2/3 x 670
= 446.67 N/m^2
Total live load on purlin WL = 670 x 1.8 cos 14.03 x 3.75
= 4095.61N

4.1.1.2 Computation of Wind Load

Average height of truss above the ground level = 13+1.5/2
=13.75m
Basic wind speed at Pune (Vb) =39m/s
K1=Probability factor (risk coefficient) =1 (IS 875-III clause 5.3.1)
K2 Topography factor =1 (IS 875-III clause 5.3.3)
K3 Terrain, Height, and structural size factor (IS 875-III clause 5.3.3)
Terrain category-2
Height of structure = 13.75m
Class-B structure
From IS875K3 = 1.01
Design wind speed (V2) =Vb x KI x K2x K3
=39x1x1x1.01
= 39 m/s
Design wind pressure (P2) = $0.6(Vz)^2$
= 0.6 x 392
= 912.6 N/m2

Eaves height, h-13m
Width 12.1 + 2 x 0.3
=12.7m (assuming that eaves girder projected 0.3m beyond the centreline.)
H/d = 13/12.7 = 1.02 1/2<h/d<3/2
Hence external pressure coefficient are taken from table-5 (IS 875:1987) and ϕ=14.03

	Windward		Leeward	
Wind angle	0°	90°	0°	90°
Face	EF	GH	EG	FH
Cpe	-0.8	-0.4	-0.75	-0.6

TABLE TAMARI EXTERNAL WIND COEFFICIENT FOR ROOF

Assuming that building with large opening
Internal pressure coefficient Cpi= ±0.7
(Cpe-Cpi) = -0.8+0.7
= -0.1
=-0.8-0.7
=-1.5
Wind load intensity = (Cpe-Cpi) x Pz

= -1.5 x 912.6

= -1368.90 N/m^2

Design wind pressure on wall

Cpe is obtained by referring table-4 (IS-875-11I 6.7.1)

Here L =64m

w = 24.2.m

h = 13m

h/w 13/24.2= 0.53> 0.5 but <1.5

L/W =64/24.2= 2.64 > 1.5 but <4

Wind angle	Value of Cpe	Cpe-Cpi for surface			
		A	B	C	D
0°	0.7	0.7	-0.3	-0.7	-0.7
90°		-0.5	-0.5	0.7	0.1

TABLE 2 EXTERNAL WIND COEFFICIENT FOR WALL

The internal pressure coefficients Cpi = ±0.7

F =(Cpe-Cpi) Pz x A

=(0.7+0.7) 912.6 x A

F = 1277 x A N/m2

4.1.1.3 Design of purlins

Design of channel purlin with sag rod. Let us provide one sag rod at the mid-span of purlin.

Total wind load on building F =(Ope-Cpi) x A x Pz

=1368.9A

Where,

A= Effective exposed area

=Spacing Of Purlin X Length Of Purlins

=1.8 X3.75

= 6.75 m2

F =-1368.9 X 6.75

=-9240N

Load Combinations Are

1. Dead Load +Live Load

Wd+Wl = WDL

=974.25+4095.61

= 4841.86N

2. Dead Load + Wind Load

Wd=746.25N

Which act vertically due to the purlin is subjected to biaxial bending

F= -9240N

Which acts normal to the roof and due to which purlin is subjected to uniaxial bending

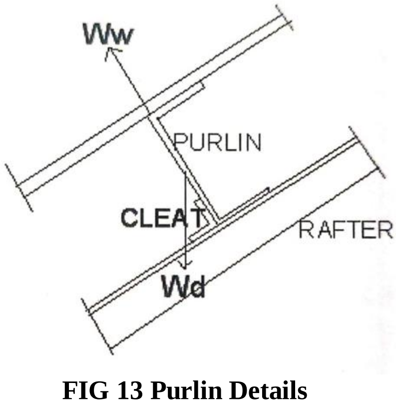

FIG 13 Purlin Details

4.1.1.4 Design Of channel Purlins

Load Combinations:

1. Dead Load + Live Load

W_{dL} = 4841.86: Biaxial bending

W_{dlx}=Load Normal To The Slope
= 4841.86 cos 14.03
= 4697.42N
W_{dLy}=Load Parallel to the Slope
= 4841.86 sin 14.03
= 117.82N
M_{ax}=WL/10
=4697.42 x 1.8/10
= 845.53 N-m
M_{dLy}=1083.66 X 1.55 /10
=211.28 N-m
Assume Zx/Z =8 for channel section
Required= (845.53 +8 x 211.28) x10^3/165
= 15.36x 103 mm^3
Let us try ISMC100@9.2kg/m having
Z_x=37.3 X 10^3 mm^3, Zy= 7.5 X10^3 mm3
Ix=186.7 X 10^4 mm4 ,ly =25.9X 10^4 mm4
f_{bt} =M_{dLx}/Z_{xx} +M_{dLy}/Z_{yy}
=845.53x1000/37.3 x1000+ 211.28 x 1000/7.5 x 1000
= 50.83 <165 N/mm2
Deflection along X-direction= 5/384 (WL^3 /Elx)
=5/384 X (4697.42 x$1800^{3)}$/(2x10^5 x186.7 x 10^4)
=0.95mm
Deflection along Y-direction 5/384 x (1173.42X1800^3)/ 2x10^5x 25.9 x10^4
= 1.72
Total deflection =√(0.562+1.722)
= 1.15 mm
Permissible deflection =L/200 =1800/200
=9.0mm Hence OK

Load combination:
2. Dead Load + Wind Load (Wd=746.25N, F=-9240N)
$W_{dLx=}$ Load Normal To The Slope
=9240+746.25cos 14.03
=-8516.01N
W_{dLy} =Load Parallel to the Slope
=746.25 sin 14.03
=180.91N

M_{dLx} =-8516.01x1.5/10
= -1532.88N-m
MdLy =168.41xI.55/10
=32.56N-m
f_{bt} = M_x/Z_x + M_y/Z_y
= 1512.88/37.3+32.56/ 7.5
=45.43N/mm2 <165N/mm^2
Deflection along x-direction: diction s
=5/384x8516.01 x1800^3(2x10^5x186.7x10^4)
=1.73mm
Deflection along Y-direction:
=5/384x180.91X1800^3(2x10^5x25.9x10^4)
=0.26mm
Total deflection = √(1.732+0.262)
=1.75mm<7.75mm Hence OK

V. ANALYSIS OF STRUCTURE FOR ULTIMATE LOAD

5.I GENERAL

Plastic design is an advantageous replacement for conventional elastic design as to statically loaded structural steel frames of certain types. These are rigid-jointed continuous or restrained beams and girders, and statically indeterminate structures in frames continuous or rest which are stressed primarily (although not exclusively) in bending. Plastic design is upon the ultimate load carrying capacity (maximum strength) of the structure. The plastic is derived from the fact that the ultimate load is computed from knowledge of the strength of steel in plastic range.

5.2 EARLY DEVELOPMENT OF PLASTIC ANALYSIS AND DESIGN

The concept of design based upon ultimate load as the design criterion is more than 40 years old. The function of plastic examination to structural invent was started by Dr. Gabor Kazinczy, Hungarian, who published results of his tests of clamped girders as early as 1914. He also suggested analytical procedures similar to those now current, and designs of apartment-type buildings were actually carried out.

. IS 800-1984 permits the use of plastic theory in the design of steel structure.

5.3 WHY PLASTIC DESIGN?

What is the justification for plastic design? One could reverse the question by asking.why use elastic design. If the structure will support the load and otherwise meet its intended action, are the magnitudes of the stresses really important.

5.4 THE PLASTIC HINGE

The cause a shape will help the computed closing load is that plastic hinges are fashioned at certain critical sections. What is the plastic hinge? What factors influence its formation? What is its importance? The M - curve is characteristic of the plastic hinge.

Two features are particularly important:

The rapid approach to M - Mp -σ_y Z and

The indefinite increase in at constant M.

5.5 FUNDAMENTAL PRINCIPLE OF PLASTIC ANALYSIS

Whatever method of plastic analysis is used it should satisfied the fallowing three condition

a) Mechanism condition (ultimate load is reached when a mechanism is form).
b) Equilibrium condition (the structure must be in equilibrium).
c) Plastic moment condition(the moment may no where be greater than Mp)

5.5.1 Virtual displacements:

The principal of virtual displacement is as follows:

If a system of forces in equilibrium is subjected to a virtual displacement, the work did by the external forces equals the work done by the internal forces. This is simply a means of expressing an equilibrium condition. If the internal work is called W, and the external work is called W_E we may write:

$$W_E=W_I$$

5.5. 2 Upper and lower bound theorems:

The important upper and lower bound theorems ar principles were proved by Greenberg and Prager. When both theorems have been satisfied in any given problem, then the solution is in fact the correct one. The two principles will now be stated and illustrated

Upper Bound Theorem

The load computed on the basis of an assumed mechanism will always be greater than or at best equal to ultimate load

Lower Bound Theorem

A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than M, is less than or at best equal to the true ultimate load.Thus,if the problem is approached from the point of view of assuming a mechanism, an upper bound to the correct load will be obtained. But this could violate the plastic moment condition. On the other hand, if we approach it from the aspect of making arbitrary assumptions as to the moment diagram, then the load might not be sufficiently great to create a mechanism.

5.6 THE ASSUMPTIONS AND CONDITIONS

The assumptions and conditions used in the following development are:

a) Strains are proportional to the distance from the neutral axis
   (Plane sections under bending remain plane after deformation)

b) The stress-strain relationship is idealized to consist of two straight lines.

5.7 STATICAL METHOD OF ANALYSIS

The statically method of analysis is based on the Lower Bound Principle. The procedure is fully described as follows

a) Select redundant(s).

b) Draw moment diagram for determinate structure,

c) Draw moment diagram for structure loaded by redundant (s).

d) Sketch composite moment diagram in such a way that a mechanism is formed (sketch mechanism),
e) Compute value of ultimate load by solving equilibrium equation and Check to see that M<M,

VI. PLASTIC DESIGN OF INDUSTRIAL BUILDING

Two span gable frames is design with fixed column based and will illustrated of the application of the mechanism method. Design of industrial building as a gable frame of span Im slope at rafter %. The height of column at knee level is 13m. the longitudinal spacing feeble column is 7.5m. The plan of building is 24.2 x 60m. and the building is assume to be located at pune.

Step 1: Span of gable frame -12.Im
Pitch 1/4
Roof rise= ¼ x 6.05
= 1.5125m
Slope of the gable rafter $\Theta = \tan^{-1}(1.5125/6.05)$
$= 14.03^0$
Length of gable rafter $= \sqrt{(6.05^2 + 1.5125^2)}$
$= 6.235$m

Step 2: Computation of load for gable frame:
Computation of dead load and live load

Wt. of sheet	= 55 N/m^2
	= 55 x 75
	=412.5 N/m
Assume self weight of purlins	= 100 N/m
Therefore total dead load	= 512.5N/m
Live load on purlin	= 750-(Θ-10)20
	= 750-(14.3-10)20
	=670 N/m^2
Live load for gable rafter	= 2/3 of L.L. on purlin
	= 2/3 x 669.4
	= 446.27 N/m^2
Live load for gable rafter	= 446.67 x 7.5
	= 3350 N/m

Computation of Wind Load on slopping roof

Average height of truss above the ground level	= 13+1.5/2
	=13.75 m
Basic wind speed at Pune (Vb)	=39m/s
K1 =Probability factor (risk coefficient)	= 1
K2 = Topography factor	=1
K3 = Terrain, Height, and structural size factor	
Terrain category - 2	
Height of structure	=13.75 m
Class-B structure	
From IS 875 K3	= 1.01
Design wind speed (Vz)	= Vb x KI x K2 K3
	= 39 x 1 x1x1.01
	=39m/s
Design wind pressure (Pz)	= 0.6[Vz]^2
	= 0.6 x 39^2
	= 912.6 N/m^2

Eaves height, h-13m

Width =12.1 + 2 x 0.3 = 12.7m assuming that caves girder projected 0.3m beyond the centerline.
Hd - 13/12.7 = 1.02 ½<h/d <3/2
Hence external pressure coefficient are taken from table -5(IS 875-1987) and Θ=14.03

	Windward		Leeward	
Wind angle(degree)	0	90	0	90
Face	EF	GH	EG	FH
Cpe	-0.8	0.4	-0.75	0.6

TABLE 1 EXTERNAL WIND COEFFICIENT

Assuming that building with large opening

Internal pressure coefficient Cpi =±0.7

(Cpe-Cpi) =-0.8+0.7

=-0.1

=-0.8-0.7

=-1.5

Wind load intensity = (Cpe-Cpi) x Pz

= -1.5 x 912.6

=-1368.9 N/m^2

= -1368.9 x 7.5

= - 10267 N/m

= 10.266KN/m

Calculation of wind load on vertical cladding:

Cpe is obtained by referring Table-4 (IS 875-III 6.2.1)

Here L= 64m, w-24.2m, h-13m

b/w = 13/24.2 =0.53 >0.5 but less than 1.5

w=64/24.2 =2.64>1.5 and less than of 4

Wind angle(degree)	Value of Cpe	Cpe-Cpi for surface			
		A	B	C	D
0'	0.7	0.7	-0.3	-0.7	-0.7
90'		-0.5	-0.5	0.7	-0.1

TABLE 2 WIND COEFFICIENT FOR WALL

On wind word cladding:

Cpe =+0.7 for angle of incidence =0°

= -0.5 for angle of incidence = 90°

The internal pressure coefficient are Cpi = +0.7

F =(Cpe-Cpi) Pz A

= (0.7 +0.7) x 912.6 x A

F =1.277A KN/m^2

= 1.277 x 7.5

= 9.57 N/m

Load from gantry girder

Horizontal thrust at gantry level =8.62 KN

Vehicle maximum reaction at one end = 218 KN

Vertical minimum reaction at other end = 54 KN

Detail of structure of loading as shown in fig. no. 25

VII. DESIGN RESULT OF SOFTWARE

7.1 2D ANALYSIS AND DESIGN

Design of Industrial building for following data:

Plan area 24.2 x 60 m

Span of one truss = 12.1m

Height of column at eaves level = 13 m

Rise of truss ¼ of span

Capacity of gantry = 200 KN

Building is situated at Pune MIDC
Direct load is taken as we calculated previously. 2D model are shown below

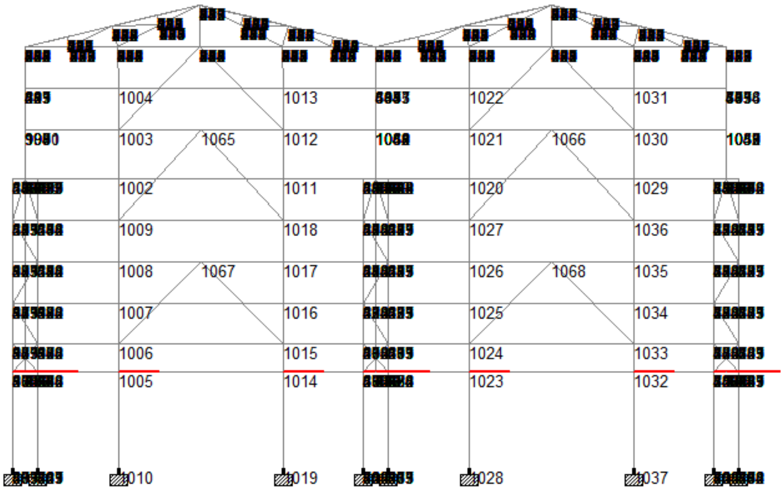

Fig.33 2D MODEL WITH NODE NUMBER

7.1.1 NODE DISPLACEMENT (SUMMARY)

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	63	14 DL+WL2+GANTRY	52.886	-0.384	11.42	54.107	0.005	-0.001	0.01
Min X	63	VIND PAR TO RIDGE (PRE	-21.145	0.331	-2.408	21.285	0	0	-0.008
Max Y	898	VIND PAR TO RIDGE (PRE	-13.275	30.592	-0.285	33.35	0.001	0	-0.001
Min Y	884	8 DL+LL+GANTRY	10.789	-34.194	1.248	35.878	0	0	-0.001
Max Z	1029	13 DL+WL1+GANTRY	4.866	-0.465	283.922	283.964	-0.005	-0.008	0
Min Z	1058	12 DL+WL3	-12.931	0.018	-4.637	13.737	0.001	0	0.004
Max rX	1068	13 DL+WL1+GANTRY	1.668	-0.069	208.533	208.54	0.065	0	0
Min rX	698	10 DL+WL1	2.642	0.186	0.633	2.723	-0.091	0	-0.001
Max rY	1009	10 DL+WL1	1.509	0.105	261.625	261.629	0.029	0.028	0
Min rY	1036	10 DL+WL1	1.431	-0.245	263.797	263.801	0.029	-0.029	0
Max rZ	143	8 DL+LL+GANTRY	22.767	-0.641	6.519	23.691	-0.001	0	0.012
Min rZ	31	8 DL+LL+GANTRY	17.905	-1.374	0.934	17.982	0.001	-0.001	-0.013
Max Rst	1029	13 DL+WL1+GANTRY	4.866	-0.465	283.922	283.964	-0.005	-0.008	0

7.1.2 BEAM

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	234	14 DL+WL2+GANTRY	137	910.515	25.844	2.715	0.14	-8.528	89.935
Min Fx	326	14 DL+WL2+GANTRY	177	-435.252	0.364	-2.069	0.004	0.156	0.41
Max Fy	1061	13 DL+WL1+GANTRY	598	-6.937	89.608	-14.094	0.008	3.07	20.231
Min Fy	120	14 DL+WL2+GANTRY	87	0.168	-90.21	0.159	0.018	0	-0.147
Max Fz	1207	14 DL+WL2+GANTRY	681	-15.276	-2.49	18.867	-21.22	-9.364	-6.031
Min Fz	1250	13 DL+WL1+GANTRY	712	151.646	11.427	-22.621	-3.565	66.47	25.646
Max Mx	1206	13 DL+WL1+GANTRY	680	-13.151	33.128	-10.919	24.475	8.113	8.335
Min Mx	1207	13 DL+WL1+GANTRY	681	-13.226	-5.004	7.393	-26.243	3.494	-6.378
Max My	2161	13 DL+WL1+GANTRY	1037	205.514	16.245	-16.817	-5.35	103.854	30.973
Min My	1186	15 DL+WL3+GANTRY	664	170.036	-0.917	-16.603	4.994	-29.067	2.322
Max Mz	2399	VIND PAR TO RIDGE (PRE	967	12.202	28.835	0.32	0.002	-0.775	108.132
Min Mz	2399	8 DL+LL+GANTRY	967	-18.469	-28.945	-0.255	-0.001	0.616	-117.476

DISPLACEMENT DETAILS (SUMMARY)

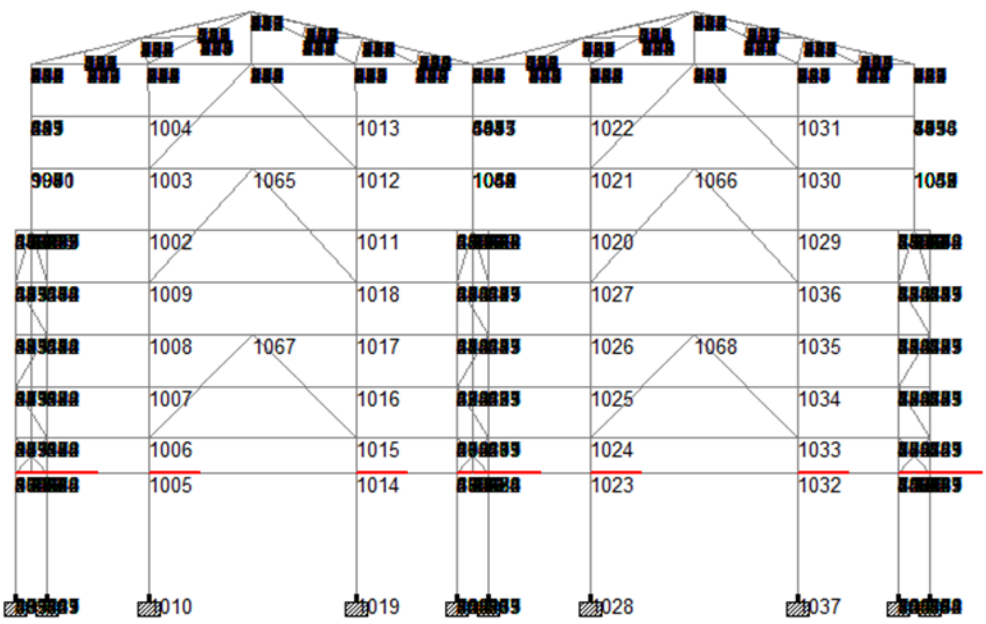

FIG 33, 2D MODEL WITH BEAM NUMBER

VIII. DISCUSSION AND CONCLUSION

A) Discussion of result

Table 9 Comparison of result from 2D and 3D analysis for building

Frame	Result from	Deflection mm		
		Windward knee	Ridge	Leeward knee
Fixed bases	3D analysis	49	50	46
	2D analysis	77	97	50

Table 10 Comparison of Max. moment result at base of column:

Result from	**Left side column**	**Central column**	**Right side column**
2D analysis	360.89	631.141	280.515
3d analysis	44.682	129.34	142.84
Elastic analysis	832	871	832
Plastic analysys	252	504	252

Tables from 9 and 10 the shows the results of 2D, 3D, elastic and plastic method of analysis of steel frame building. The result showed significant difference in maximum moment and deflection for frame comparing to the 2D and 3D Elastic and Plastic method of analysis of steel frame building. The analysis and design of steel framed building based on a 2D analysis was done completely ignoring the behaviour of claddings, and thus the resulting spread of moment and deflection from windward and leeward location and from internal frames to end gable frames, and thus are not based on 3D behaviour of entire steel frame building system. Thus 2D analysis and design do not derived the advantage of reduction in moment and joint deflection. This investigation clearly indicated that it is very important that a 3D analysis taking in to account the effect of cladding and end frame rigidity is used in the design of steel framed buildings, particularly for lateral loading due to crosswind

IX. CONCLUSION

On the basis of Elastic, Plastic, 2D and 3D analysis and design of industrial building various loading condition the fallowing conclusion appear to be warranted.

1) Plastic design gives promise of economy in the use of steel, of saving time in by virtue of its simplicity and of building frame more logically design for greater overall strength.

2) The reserve in strength above the working load computed by conventional considerable in indeterminate steel structure. Indeed in some nc design, as much load carrying capacity is disregarded as is used.

3) Use of ultimate load as the design criterion design criterion provides at least the same margin of safety as is presently afforded in the classic design of a simple beam.

4) In most cases a structure design by the plastic method will deflect no more at working load than will a simply supported beam design by elastic method to support the same load.

5) Effects of roof cladding, end frame rigidity, roof purlins and side rails include in 3D computer modelling under a range of various load cases.

6) 2D and 3D analysis result clearly showed the significant differences between behaviour of structure.

7) The maximum moments and deflections were significantly reduced by the addition of claddings and end frame bracing, particularly under lateral loads.

8) 3D modelling of steel portal frame buildings, including the effects of both the structural and non-structural components, is necessary for a realistic and efficient design from both strength and deflection points of view.

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