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DESIGN AND ANALYSIS OF RCC BUILDING UNDER DIFFERENT LOADING CONDITIONS

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ABSTRACT: One of the major developments in seismic design over the past 10 years has been increased emphasis on limit states design, now generally termed Performance Based Engineering. Three techniques – the capacity spectrum approach, the N2 method and direct displacement-based design have now matured to the stage where seismic assessment of existing structures or design of new structures can be carried out to ensure that particular deformation-based criteria are met. The paper will outline and compare the three methods, and discuss them in the context of traditional force-based seismic design and earlier design approaches which contained some elements of performance based design. Factors defining different performance states will be discussed, including the need, not yet achieved, to include residual displacement as a key performance limit. Some emphasis will be placed on soil-related problems, and the incorporation of soil/structure interaction into performance-based design. It will be shown that this is relatively straightforward and results in consistent design solutions not readily available with force-based designs using force-reduction factors.

Over the past several years, federal guidelines were published which help to facilitate the implementation of Performance Based Design with respect to existing structures. FEMA 273, Guidelines for the Seismic Rehabilitation of Buildings, which has subsequently been updated as FEMA 356, provides specific performance objectives for both the building under consideration and the nonstructural components associated with the building. While written for use with existing structures, the Guidelines may also be used as the basis for the design of the seismic force-resisting system for new structures.

Performance Based Seismic Design has the following distinguishing characteristics.

I. INTRODUCTION

1.1. Purpose

Structural and geotechnical engineers and researchers associated with the Earthquake Engineering Research Center developed these Guidelines for Performance- Based Seismic Design of Buildings as a recommended alternative to the prescriptive procedures for seismic design of buildings contained in the IS: 456-2000 Concrete Designand other standards incorporated by reference into the International Building Code (IBC). These Guidelines may be used as:

• Basis for the seismic design of individual tall buildings under the Building Code alternative (non-prescriptive) design provisions; or

• Basis for development and adoption of future Building Code provisions governing the design of tall buildings.

Properly executed, the Guidelines are intended to result in buildings that are capable of achieving the seismic performance objectives for Occupancy Category II buildings intended by IS: 1893 (Part-1). Alternatively, individual users may adapt and modify these guidelines to serve as the basis for designs intended to achieve higher seismic performance objectives.

These Guidelines are intended to serve as a reference source for design engineers, building officials, peer reviewers, and developers of building codes and standards.

1.2 Commentary

This document intentionally contains both requirements, which are stated in mandatory language (for example, using "shall") and recommendations, which use non-mandatory language (for example, using "should").

An alternative or non-prescriptive seismic design is one that takes exception to one or more of the requirements of the IBC by invoking Section 104.11 of the Building Code, which reads as follows:

1.3. Alternate materials, design and methods of construction and equipment.

The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed in this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability, and safety.

1.4. Scope

The design recommendations contained herein are applicable to the seismic design of structures that generally have the unique seismic response characteristics of tall buildings including:

• A fundamental translational period of vibration significantly in excess of 1 second

• Significant mass participation and lateral response in higher modes of vibration

• A seismic-force-resisting system with a slender aspect ratio such that significant portions of the lateral drift result from axial deformation of the walls and/or columns as compared to shearing deformation of the frames or walls.

The Earthquake Engineering Research Center developed these Guidelines as an alternative means of compliance with the strength requirements for structural resistance to seismic loads specified in IS:1893(Part-1) for Risk Category II structures considering the seismic hazard typical in the Western United States. Such structures are intended to resist strong earthquake motion through inelastic response of their structural components. These recommendations may be applicable to the seismic design of structures that do not exhibit substantial inelastic response or that are located in regions with seismicity somewhat different than the Western United States. However, some modification may be appropriate.

Structural design for resistance to loadings other than that associated with earthquakes is beyond the scope of this document. Design of nonstructural components other than exterior cladding for seismic resistance is also not included within the scope of this document. Design for these loadings and systems should conform to the applicable requirements of the Building Code or other suitable alternatives that consider the unique response characteristics of tall building structures.

II. LITERATURE REVIEW

Chavan, Jadhav (2014) studied seismic analysis of reinforced concrete with different bracing arrangements by equivalent static method using ETABS 2015. software. The arrangements considered were diagonal, V-type, inverted V-type and X-type. It was observed that lateral displacement reduced by 50% to 60% and maximum displacement reduced by using X-type bracing. Base shear of the building was also found to increase from the bare frame, by use of X-type bracing, indicating increase in stiffness.

Esmaili et al. (2008) studied the structural aspect of a 56 stories high tower, located in a high seismic zone in Tehran. Seismic evaluation of the building was done by non-linear dynamic analysis. The existing building had main walls and its side walls as shear walls, connected to the main wall by coupling of beams. The conclusion was to consider the time-dependency of concrete. Steel bracing system should be provided for energy absorption for ductility, but axial load can have adverse effect on their performance. It is both conceptually and economically unacceptable to use shear wall as both gravity and bracing system. Confinement of concrete in shear walls is good option for providing ductility and stability.

Kappos,Manafpour (2000) presented new methodology for seismic design of RC building based on feasible partial inelastic model of the structure and performance criteria for two distinct limit states. The procedure is developed in a format that can be incorporated in design codes like Eurocode 8. Time-History (Non-linear dynamic) analysis and Pushover analysis (Non-linear Static analysis) were explored. The adopted method showed better seismic performance than standard code procedure; at least in case of regular RC frame building. It was found that behavior under “life-safety” was easier to control than under serviceability earthquake because of the adoption of performance criteria involving ductility requirements of members for “life-safety” earthquake.

Yamada et al. studied, experimentally as well as analytically, deformation and fracture characteristics of lateral load resisting systems-shear wall for RC frame- and –steel bracing for steel multi-storey frame- under earthquake, considering models having 3 different spans and 3, 6 and 9 storeys. Deformations and facture results for all the three cases are compared and differences are clarified by normalization of proposed horizontal resisting ratios.

RESEARCH METHODOLOGY

To gather various types of work on seismic analysis of high-rise structures and increasing lateral stiffness of the system various papers, thesis and research articles were studied thoroughly and referred. The idea behind doing literature review was to collect data and have understanding on different methods and approaches that can be used, to clear understand the software requirement of the project. Literature review was done to have thorough guidelines during the entire project work.

Data collection

Various Indian standard codes were collected from the department of civil engineering NIT Rourkela. The earthquake data’s were obtained from the site Peer.berkeley.edu. The earthquakes considered in this work are time history of ground motion as per IS 1893:2002 (Part-I), Imperial Valley and San Francisco.

III. METHODOLOGY ADOPTED

As discussed in the scope of the work, the entire work is divided into three parts:

Analysis of bare frame in all the above three mentioned ground motions

Analysis of the braced frames.

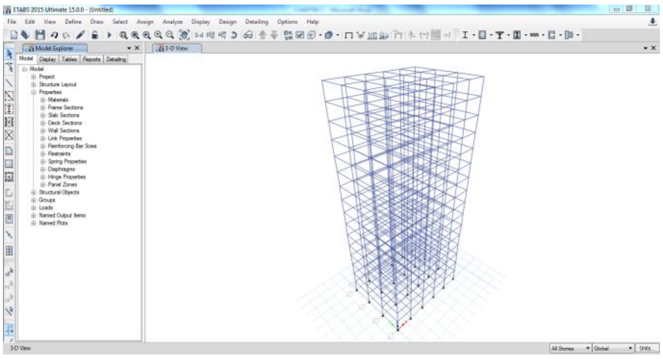
Analysis of the frame with shear wall

For analysis a 12 stories high building is modeled in Staad Pro as a space frame. The building is does not represent any real existing building. The building is unsymmetrical with the span more along Z direction than along X direction. The building rises up to 42m along Y direction and spans 15m along X direction and 20 m along Z direction .The building is analyzed by Response Spectrum Analysis, which is a linear dynamic analysis. Dynamic Analysis is adopted since it gives better results than static analysis. The specifications of the frame are given in Table 1. and the plan and the model of the building is shown in Fig. 4 and Fig.5 respectively. In the entire course work X and Z are taken as the horizontal axes and Y as the vertical axes.

Table 1: Specifications of the building

Specifications of Building	Data
Storey Height	3.5m
No. of bays along X direction	3
Bay Length along X direction	4 m
Bay Length along Z direction	5 m

No of Bay along Z-Direction	5
Concrete grade used	M 30
Columns	0.40m X 0.40m
Beams	0.30m X 0.45m
Slab Thickness	0.15m
External Wall Thick ness	0.23 m
Internal Wall Thick ness	0.120 m
Unit Weight of Concrete	25 kN/m3
Live Load	3.5 kN/m3
Zone	IV
Soil Conditions	Hard Soil
Damping Ratio	5%
No of Floors or Stories	G+15


(G+15) Three Dimensional View of the Structure

IV. ANALYSIS AND STRUCTURAL MODELLING

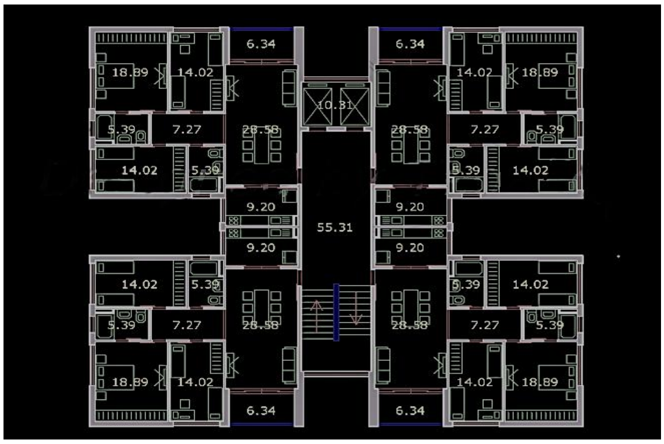
4.1 BUILDING DESCRIPTION

An RC framed building located at Guwahati, India (Seismic Zone -V) is selected for the present study. The building is fairly symmetric in plan and in elevation.

No. of Floors of Building – G+3
Slab Thickness – 150 m
Each Floor Height – 3.0 m
Total Height of the Building – 12 m
External Wall Thick – 450 mm
Internal Thickness – 230 mm
For Live Load – 2.5 kN/m
Column Sizes – 350 x 300 mm
Beam Sizes – 450 x 400 mm

The cross sections of the structural members (columns 230 mm×380 mm and beams 300 x 450 mm) are equal in all frames and all stories. Storey masses to 295 and 237 tonnes in the bottom storyes and at the roof level, respectively. The design base shear was equal to 0.15 times the total weight.

AUTO CADD PLAN



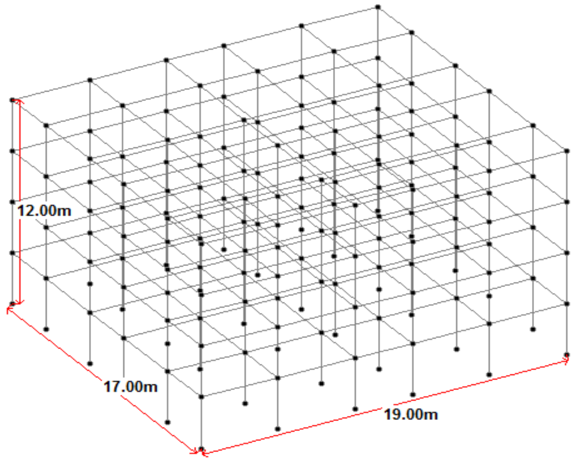
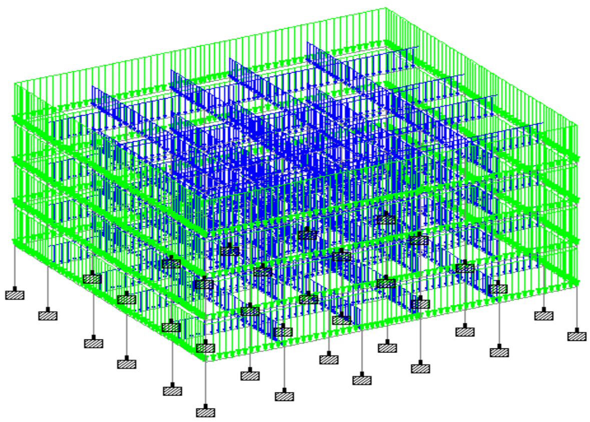
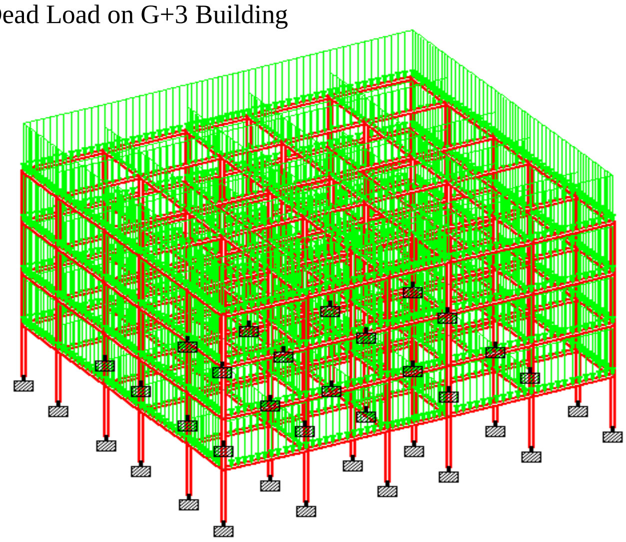


Fig. 3D View of the G+3 RC Framed Building

For Calculation of Dead Load:
Self- weight- 1 kn/Sq.m
Floor load -2 kN/Sq.m
External wall Thickness – 450mm
For Density of Brick Wall = 20 kN/ m^2
= 20 x 0.45 x 3
= -27kN/m^3
Internal wall Thickness – 120mm
For Density of Brick Wall = 20 kN/ m^2
= 20 x 0.23 x 3
= -13.8kN/m^3
For Considering of Floor Load -1.8 kN/m^2
Live Load –2.5kN/ m



Dead Load on G+3 Building



Self -Weight of G+3 Building

SCOPE

1.1 This code (Part I) covers unit weight/mass of nationals, and parts or components In a building that apply to the determination of dead loads In the design of buildings. 1.1. J The unit weight/mass of maternal that arc likely to be stored in a building are also specified for the purpose of load calculations along with angles of maternal canon as appropriate

Live Load (IS875-PART-2)

1.1 This standard (Part 2) covers imposed loads* (live loads) to be assumed in the design of buildings.

The imposed loads, specified herein, are minimum loads which should be taken into consideration for the purpose of structural safety of buildings.

1.2 This Code does not cover detailed provisions for loads incidental to construction and special cases of vibration, such as moving machinery, heavy acceleration from cranes, hoists and the like. Such loads shall be dealt with individually in each case.

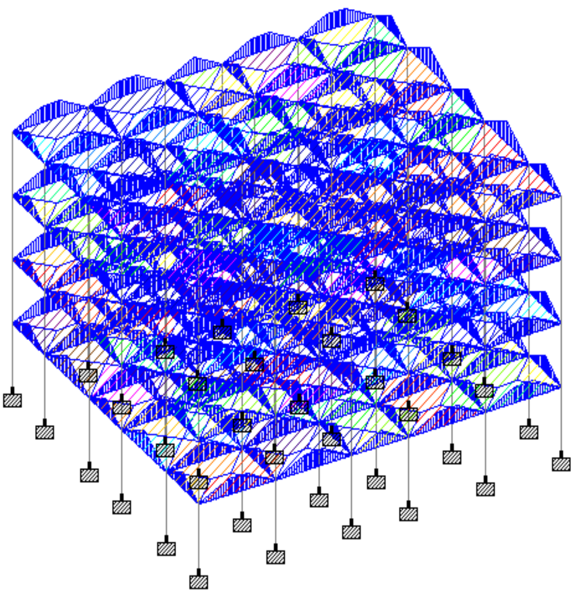
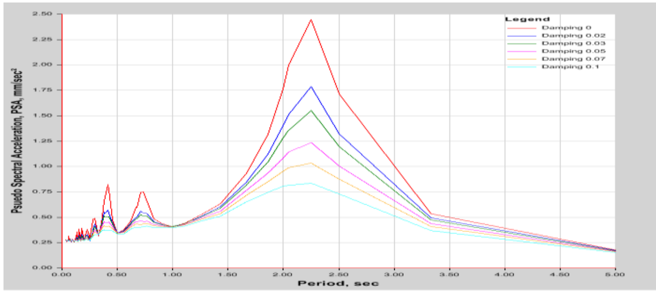


Fig. Live Load on G+3 Building

Input Data

Name	RSFromTH1		
Load Case	EL+Z	Coordinate System	Modal
Story	Story4	Response Direction	X
Point	1	Spectrum Widening	0 %

Plot



Seismic Mass

It is the seismic weight divided by acceleration due to gravity.

Seismic Weight (W)

It is the total dead load plus appropriate amounts of specified imposed load.

Structural Response Factors (S,/g)

It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.

Tectonic Features

The nature of geological formation of the bedrock in the earth's crust revealing regions characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes with their age of formation, which are directly involved in the earth movement or quake resulting in the above consequences.

Zone Factor (Z)

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate

of effective peak ground acceleration.

Zero Period Acceleration (ZPA)

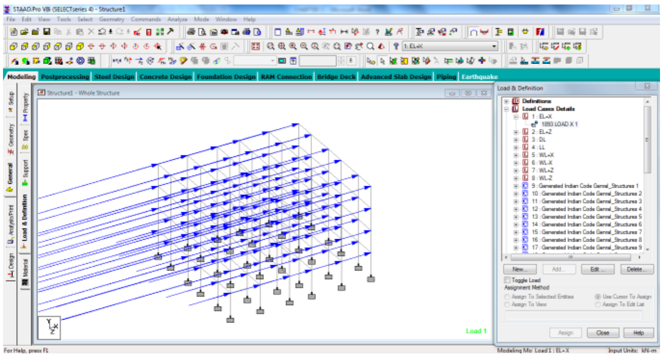
It is the value of acceleration response spectrum for period below 0.03 s (frequencies above 33 Hz).

Base Dimensions (d)

Base dimension of the building along a direction is the dimension at its base, in meter, along that direction.

Centre of Mass

The point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.



Earthquake Load Application of G+3 Building

Storey

It is the space between two adjacent floors.

Storey Drift

It is the displacement of one level relative to the other level above or below.

StoreyShear (~)

It is the sum of design lateral forces at all levels above the storey under consideration.

Story Response - Maximum Story Displacement

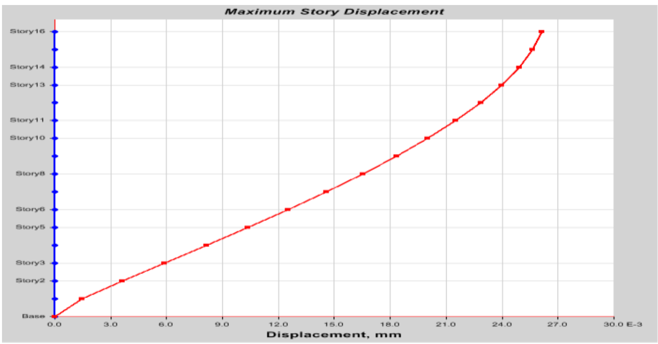
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story display	Story Range	All Stories
Modal Case	Modal	Top Story	Story4
Mode Number	1	Bottom Story	Base

Plot



Tabulated Plot Coordinates

Story Response Values

Story	1.0	Location	X-Dir	Y-Dir
	M		mm	Mm
Story4	12	Top	1.254E-07	8.142E-03
Story3	9	Top	9.802E-08	5.889E-03
Story2	6	Top	6.918E-08	3.628E-03
Story1	3	Top	2.785E-07	1.453E-03

Story	1.0	Location	X-Dir	Y-Dir
	M		mm	Mm
Base	0	Top	0	0

V. DESIGN OF G+3 BUILDING

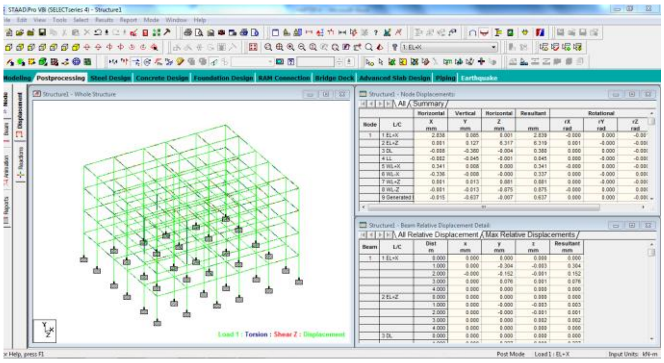


Fig. Showing the Beam and Column Analysis results

BEAMS:

Beams transfer load from slabs to columns .beams are designed for bending. In general we have two types of beam: single and double. Similar to columns geometry and perimeters of the beams are assigned. Design beam command is assigned and analysis is carried out, now reinforcement details are taken.

Beam Design:

A reinforced concrete beam should be able to resist tensile, compressive and shear stress induced in it by loads on the beam.

There are three types of reinforced concrete beams

1.) Single reinforced beams

2.) Double reinforced concrete

3.) Flanged beams

Singly reinforced beams:

In singly reinforced simply supported beams steel bars are placed near the bottom of the beam where they are more effective in resisting in the tensile bending stress. I cantilever beams reinforcing bars placed near the top of the beam, for the same reason as in the case of simply supported beam.

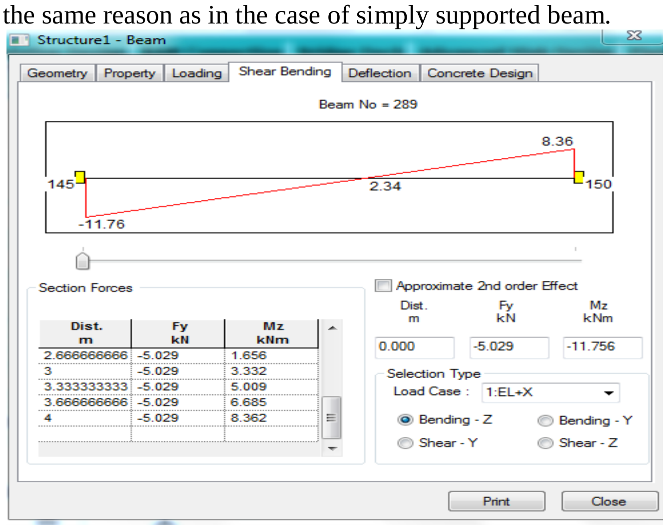


Fig.5.1. Bending moment of RC Beams

Doubly reinforced concrete beams:
It is reinforced under compression tension regions. The necessity of steel of compression region arises due to two reasons. When depth of beam is restricted. The strength availability singly reinforced beam is in adequate.
At a support of continuous beam where bending moment changes sign such as situation may also arise in design of a beam circular in plan. Figure shows the bottom and top reinforcement details at three different sections. These calculations are interpreted manual.

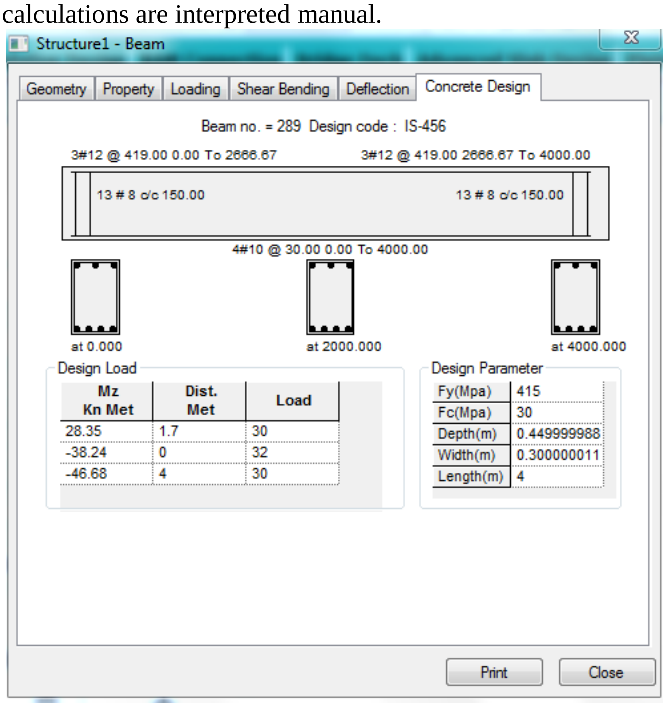
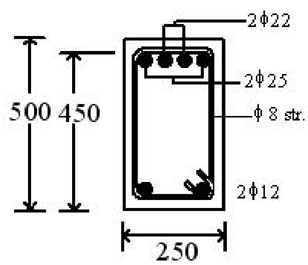
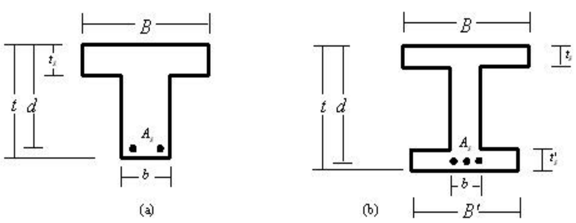


Fig5.2a Diagram of the Reinforcement Details of Beam





Reinforcement Details in Beams
5.3 Check for the design of a beam (no. 230):
Given data:
Cross section of beam: b x d = 300mm x450 mm
Vertical shear force = v_u =145.93 KN
τ_c= 0.29 N/mm^2(from table 19 of IS 456 200)
Minimum Shear Reinforcement:
When $\tau_v$is less than τ_c, given in Table 19, minimum shear reinforcement shall -be provided
Design of Shear Reinforcement:
When $\tau_v$exceeds τ_c, given in Table 19, shear reinforcement shall be provided in any of the following forms:
a) Vertical stirrups,
b) Bent-up bars along with stirrups, and c) Inclined stirrups,
τ_v=v_u/(bxd) (Asperclause40.1ofIS456-2000)
=145.93x10^3/(400x300)
=1.216N/mm^2
$\tau_v \geq \tau_c$
Design reinforcement
V_{us}= V_u-$\tau_c$xbxd(Asperclause40.4ofIS456-2000)
= 145.93x10^3-0.29x400x300
= 111100N
Shear reinforcement shall be provided to carry a shear equal to Vu – $\tau_c$bd. The strength of shear reinforcement Vus, shall be calculated as below:
For vertical stirrups:
V_{us}= 0.87$f_y A_{sv}$d/S_v(Asperclause40.4ofIS456-2000)
A_{sv}=total cross-sectional area of stirrup legs or bent-up bars within a distance S_v.S_v = spacing of the stirrups or bent-up bars along the length of the member,
τ_v = nominal shear stress
τ_c =design shear strength of the concrete,
b = breadth of the member which for flanged beams, shall be taken as the breadth of the web
fy = characteristic strength of the stirrup or bent-up reinforcement which shall not be taken greater than 415 N/mm^2,
α = angle between the inclined stirrup or bent- up bar and the axis of the member, not less than45”, and
d = effective depth.
111130 N= 0.87x415x2xπx8^{2}x400/Sv
S_v = 140 mm
S_v should not be more than the following
1. 0.75xd = 0.75 x 400 = 300 mm
2. 300mm
3. Minimumshear reinforcement spacing = S_{vmi}
Minimum shear reinforcement:
Minimum shear reinforcement in the form of stirrups shall be provided such that: $A_{sv}/bS_v \geq$ 0.4/ 0.87fy (Asperclause26.5.1.6ofIS456-2000)
A_{sv} = total cross-sectional area of stir rup legs effective in shear,
S_v = stirrup spacing along the length ofthe member,
b = breadth of the beamor breadth of the web of flanged beam, and
fy = characteristic strength of the stirrup reinforcement in N/mm* which shall not be taken greater than 415 N/mn^2
S_v=2x(π/4)x8^{2}x0.87x415/(0.4x300)=302 mm.
Provided 2 legged 8mm @140 mmstirrups.
Hencematched with staad output.

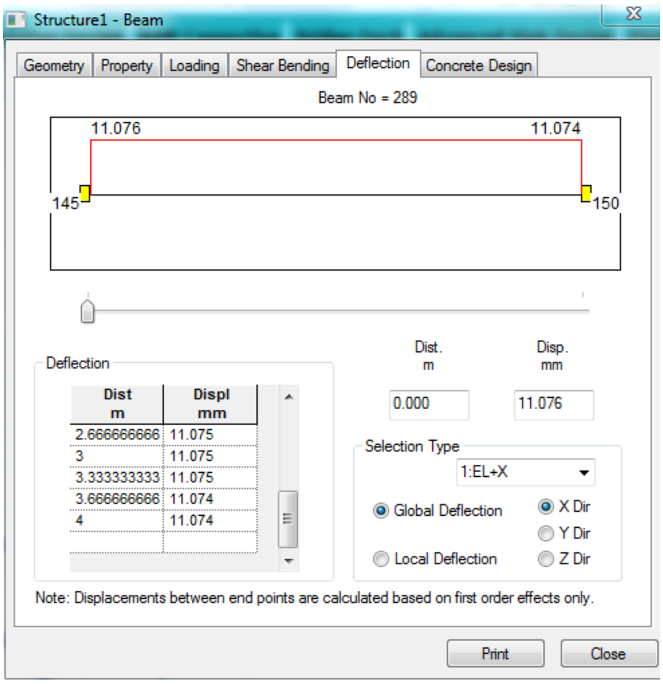


Fig. Showing the Deflection in Beams

OUT PUT OF STAAD PRO
B E A M N O. 2 D E S I G N R E S U L T S
M30 Fe415 (Main) Fe415 (Sec.)
LENGTH: 3000.0 mm SIZE: 300.0 mm X 450.0 mm
COVER: 25.0 mm
STAAD SPACE -- PAGE NO. 8

SUMMARY OF REINF. AREA (Sq.mm)
SECTION 0.0 mm 750.0 mm 1500.0 mm 2250.0 mm 3000.0 mm
TOP 618.19 270.99 258.07 258.07 600.73
REINF. (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm)
BOTTOM 513.73 374.38 258.07 258.07 275.02
REINF. (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm)
SUMMARY OF PROVIDED REINF. AREA
SECTION 0.0 mm 750.0 mm 1500.0 mm 2250.0 mm 3000.0 mm
TOP 3-20í 3-20í3-20í3-20í3-20í
REINF. 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s)
BOTTOM 7-10í 5-10í 4-10í 4-10í4-10í
REINF. 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s) 1 layer(s)
SHEAR 2 legged 8í 2 legged 8í 2 legged 8í 2 legged 8í 2 legged 8í
REINF. @ 150 mm c/c @ 150 mm c/c @ 150 mm c/c @ 150 mm c/c @ 150 mm c/c
SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPORT
SHEAR DESIGN RESULTS AT 530.0 mm AWAY FROM START SUPPORT
VY = 62.60 MX = 0.93 LD= 33
Provide 2 Legged 8í @ 150 mm c/c
SHEAR DESIGN RESULTS AT 530.0 mm AWAY FROM END SUPPORT
VY = -75.42 MX = 0.93 LD= 31
Provide 2 Legged 8í @ 150 mm c/c

DESIGN OF COLUMN:

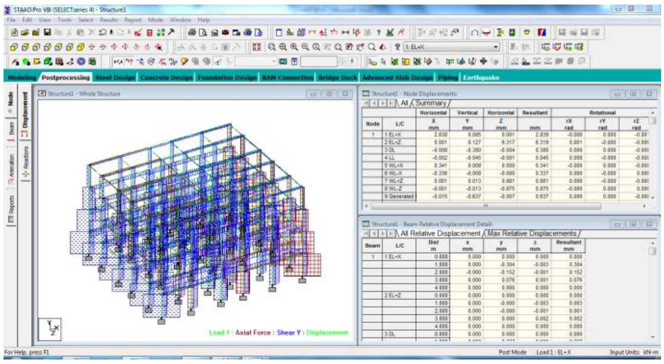


Fig. Showing the Axial and Shear Force in G+3 Building

Column design:

A column may be defined as an element usedprimary to support axial compressive loads and with a height of a least three times its lateral dimension. The strength of column depends upon the strength of materials, shapeand size of cross section, lengthand degree of proportional and dedicational restrains at its ends.

A column may be classify based on deferent criteria such as
1.)Shape of the section
2.)Slendernessratio (a=l+d)
3.)Type of loading, land
4.)Pattern oflateral reinforcement.

The ratio of effective column length to least lateral dimension is released to as slendernessratio.

In our structure we have 3 types of columns.
Column with beams on two sides
Columns with beams on three sides
Columns with beams on four sides

So we require three types of column sections. So create three types of column sections and assign to the respective columns depending on the connection. But in these structure we adopted same cross section throughout the structure witha rectangular cross section .In foundations we generally do not have circular columns if circular column is given it makes a circle by creating many lines to increase accuracy.

The column design is done by selecting the column and from geometry page assigns the dimensions of the columns. Now analyze the column for loads to see the reactions and total loads on the column by seeing the loads design column by giving appropriateparameters like
1. Minimum reinforcement, max, bar sizes, maximumand minimum spicing.
2.Select the appropriate design code and input design columncommand to all the column.
3. Now run analysis andselect any column to collect the reinforcement details

The following figure shows the reinforcement details of a beamin staad. The figure represents details regarding

1. Transverse reinforcement
2. Longitudinal reinforcement

The type of bars to be used, amount of steeland loading on the column is represented in the below figure.

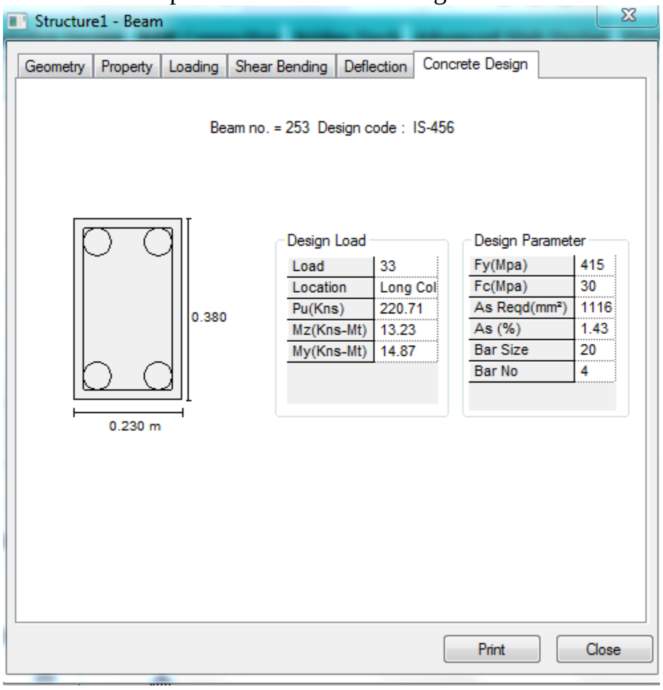


Fig 5.3a Reinforcement details of a column

Output:

Due to very huge and detailed explanation of staad output for each and every column we have shown a column design results below showing the amount of load, moments, amount of steel required, section adopted etc.

The main problemwith staad is it takes all columns also as beams initially before design and continue the same. so here outputof column 1 which os actually 131stbeam as most of beams are used in drawing the plan.

Check for Column design: Short axially Loaded columns: Given data

f_{ck} =30 N/mm^2

f_y=415N/mm^2 p_{uz}=2734 N b=350 d=45

DesignofreinforcementArea: (As per clause 39.6 of IS 456 2000)

P_{uz}=0.45$f_{ck}A_c$+0.75f_yA_{sc}

2734=0.45x30x (350x450-Asc) +0.75x415xA_{SC}

On solving the above equation we get A_{sc}=2041.15 Sq.mm.((Matched with Output)

Design of Main (Longitudinal)reinforcement:

(As per clause 26.5.3.1 of IS 456-2000)

1. The cross sectional area of longitudinal reinforcement shall not be less 0.8% , not more than 6% of the gross crosssectional area of the column.
2. The bars shall not be less than 12 mm in diameter.
3. Spacing of longitudinal bars measured along the periphery of the column shall not exceed 300 mm.

Provided main reinforcement: 20 - 12 dia

(1.44%, 2261.95 Sq.mm.)

Check for Transversere in for cement:

(As per clause 26.5.3.2 of IS 456-2000)

A)Pitch:

Shall not be more than the least of the following

1) Least lateral dimension of the compression member (350mm).

2) 16 x diameter of longitudinal reinforcement bar = 16x 12 = 192 mm

3) 300 mm

B)Diameter:

1) Shall not be less than one fourth of the diameter of main reinforcement.

2) Not less than 6 mm.

PROVIDED TIE REINFORCEMENT:

Provide 8 mmdia. rectangular ties @ 190 mm c/c

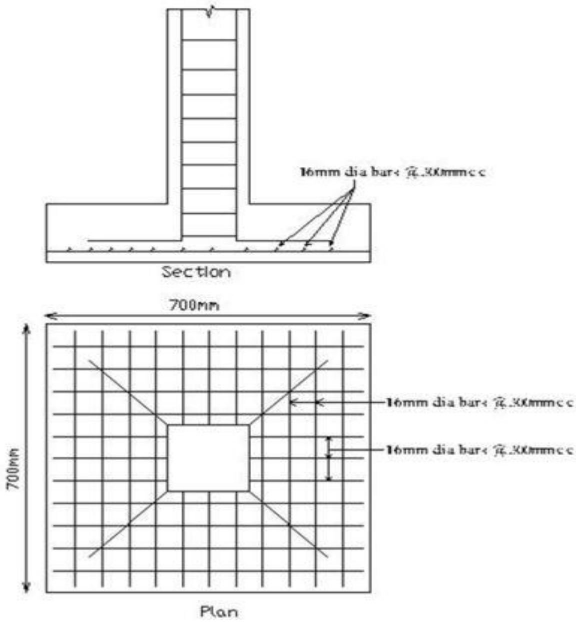


Fig. Reinforcement details of the columns

DESIGN OF ONEWAY SLABS:

Design of slabs:

End conditions for slab:

Adjacent long and short sides are continuous and other edges discontinuous. Assuming the thickness of slab as 120 mm

Live load:

For residential building live load is usually taken as 2 kN/sq.m. (in accordance with 875 part II)

Dead load:

Self -weight of slab = 1x1x0.12x25 = 3.0 KN/m^2

Weight of flooring (75mm thick) = 1x1x0.005x20 = 1.0 KN/m^2

Accidental loads = 1.0 KN/m^2 = 1.0 KN/m^2

= 5.0 KN/m^2

Live load:

Live load is taken = 2.0 KN/m^2

Total load = 2 + 5.0 KN/m^2

Factored load = 1.5x7.0 KN/m^2

Design load = 10.5 KN/m^2

Calculation of moments:

(As per Table 12 of IS 456-2000)

Bending moment coefficients for slab:

Dead load and super imposed load

Near the middle

End of span +1/12

At support next to

End support -1/10

Positive bending moment at mid span = + $wl^2/12$

Mu = 10.5x $(3.88)^2/12$
= 13.17KN

Calculation of loads:

Live load:

For residential building live load is usually taken as 2 kN/sq.m. (in accordance with 875 part II)

Dead load:

Self -weight of slab = 1x1x0.12x25 = 3.0 KN/m^2

Weight of flooring (75mm thick) = 1x1x0.005x20 = 1.0 KN/m^2

Accidental loads = 1.0 KN/m^2 = 1.0 KN/m^2

5.0KN/m^2

Live load:

Live load is taken = 2.0 KN/m^2

Total load = 2 + 5.0 KN/m^2

Factored load = 1.5x7.0 KN/m^2

Design load = 10.5 KN/m^2

Calculationofmoments:

(As per Table 12 of IS 456-2000)

Bending moment coefficients for slab:

Dead load and super imposed load

Near the middle

End of span +1/12

At support next to

End support -1/10

Positive bending moment at mid span = + $wl^2/12$

Mu = 10.5x $(3.88)^2/12$ = 13.17KNm

Adopting 8-mm dia bars as reinforcement

Effective cover = 15+10/2 =20mm

Over all depth = D =61.852+20=81.852

Therefore providing overall depth D = 120mm

Effective depth d = 120-20=100mm

Calculation of steel:

(MAINREINFORCEMENT)

Form IS 456-2000(Annexure G)

Mu =0.87xf_yx AstXD (1-f_yx Ast/bd x f_{ck})

=0.87x415x100xAst (1-415xAst/ (1000x100x30)

= 15.8 m

Ast= 437.6mm^2

Providing minimumsteel of= 0.12%xbxD=144mm^2

Spacing of 10mmdia bars = (a_{st}x1000)/A_{st}

= (∏x10^2x1000)/(4x437.6)

=179.47mm c/c

As per IS 456 2000, clause 26.3.3b, the spacing of Reinforcement should be not more than least of following

1. 3xeffectivedepth =3x100 = 300mm
2. Provide10mm Φ bars @ 175 mm.

<u>Distribution reinforcement:</u>

As per IS 456-2000(clause: 26.5.2.1)

Providing 0.12% of gross area as distribution reinforcement

Area of steel = (0.12x120x1000)/100=144mm^2

Adopting 6mm Φ bars as distribution reinforcement

Spacing = (a_{st}x1000)/A_{st}

= (∏/4x6^2x1000)/144

=196.35mm c/c

Provide 6mm Φ bars @ 180mmc/c

Check for development length:

As per IS 456-2000(clause 26.2.1)

The development length L_d is given by

L_d= Φσ_{st}/4^{t}bd

= (10x0.87x415)/(4x1.2x1.6)

= 470.11 mm (req.) L_d(available) = MI/V+L_0

M_1= 0.87xf_yxAstxd (1-f_yxAst / bdf_{ck})

= 0.87x415x437x100 (1-437x415/(1000x100x30)

= 14.82x10^6N-mm

Shear force at the section due to design loads

V =W_1/2 = 10.5x3.88/2

= 20.37

M_1/V+L_0 = 14.82/20.37 +L_0

= 0.727m+L_0

=727mm+L_0

$L_{d(available)}$>$L_{d(req'd)}$ safe

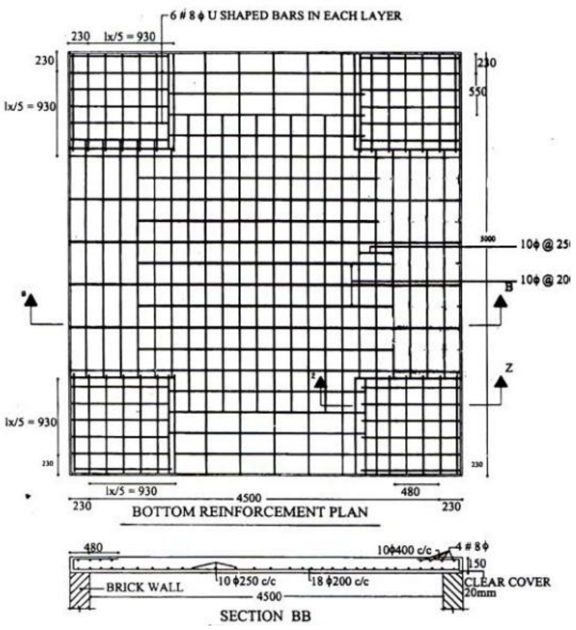


Fig. Reinforcement details of the slab

DESIGN OF FOOTINGS:

Footings are structural elements that transfer loads fromthe building or individual column to the earth .If these loads are to beproperly transmitted, foundations must be designed to prevent excessive settlement or rotation, tominimize differential settlement and to provide adequate safety against sliding and overturning.

GENERAL:

1.)Footing shall be designed to sustain the applied loads, moments and forces and the induced reactions and to assurethat any settlements which may occur will be as nearly uniformas possible and the safe bearing capacity of soil is not exceeded.

2.)Thickness at the edge of the footing: in reinforced and plain concrete footing at the edge shall be not less than 150 mm for footing on the neither soil nor less than 300mm above the tops of the pile for footing on piles.

BEARING CAPACITY OF SOIL:

The size foundation depends on permissible bearing capacity of soil. The total load per unit area under the footing must be less than the permissible bearing capacity of soil to the excessive settlements.

Foundation design:

Foundations are structure elements that transfer loads from building or individual column to earth this loads are to be properly transmitted foundations must be designed to prevent excessive settlement are rotation to minimize differential settlements and to provide adequate safety isolated footings for multi storey buildings. These may be square rectangle are circular in plan that the choice of type of foundation to be used in a given situation depends on a number of factors.

1.)Bearing capacity of soil
2.)Type of structure
3.)Type of loads
4.)Permissible differential settlements
5.)Economy

A footing is the bottommost part of the structure and last member to transfer the load. In order to design footings we used staad foundation software.

These are the types of foundations the software can deal.

<u>Shallow Foundation (D<B)</u>

1. Isolated (Spread) Footing
2. Combined (Strip) Footing
3. Mat (Raft) Foundation

<u>Deep foundation (D>B)</u>

1. Pile Cap
2. Driller Pier

The advantage of this software is even after the analysis of staad we can update the following prosperities if required.

The following Parameters can be updated:

Column Position
Column Shape
Column Size
Load Cases
Support List

After the analysis of structure at first we has toimport the reactions of the columns from staad pro using import button. After we import the loads the placement ofcolumns is indicated.

After importing the reactions inthe staad foundation the following input data is required regarding materials, Soil type, Type of foundation, safety factors.

Type of foundation: ISOLATED.
Unit weight of concrete: 25kn/m^3
Minimumbar spacing: 50mm
Maximumbar spacing: 500mm
Strengthofconcrete: 30 N/mm^2
Yield strength of steel: 415 n/mm^2
Minimumbar size: 6mm
Maximumbar size: 40mm
Bottomclear cover: 50mm
Unit weight of soil: 22 kn/m^3
Soil bearing capacity: 300 kn/m^3
Minimum length: 1000mm
Minimumwidth: 1000mm
Minimumthickness: 500mm
Maximumlength: 12000mm
Maximumwidth: 12000mm
Maximumthickness: 1500mm
Plan dimension: 50mm
Aspect ratio:1
Safety against friction, overturning, sliding: 0.5, 1.5, 1.5

After this input various properties of the structure and click on design. After the analysis detailed calculation of each and every footing is given with plan and elevation of footing including the manual calculation.

The following tables show the dimensions and reinforcement details of all the footings:

Footing	Group	Foundation Geometry

No.	ID	Length	Width	Thickness
2	8	2.600m	2.600m	0.852m
8	15	3.050m	3.050m	0.551m
14	16	4.100m	4.100m	0.852m
18	17	3.750m	3.750m	0.551m
22	18	3.500m	3.500m	0.652m
23	19	3.350m	3.350m	0.652m
24	20	3.200m	3.200m	0.752m
25	21	2.650m	2.650m	0.501m
26	22	3.500m	3.500m	0.501m
42	36	2.300m	2.300m	0.852m

The Reinforcement of the footings are :
After the design is complete the calculations isobtained for each and every column and a sample column calculations is shown below.

Table no 8.2 reinforcement details of footing

F.NO	Footing Reinforcement			
	Bottom Reinforcement (Mz)	Bottom Reinforcement (Mx)	Top Reinforcement (Mz)	Top Reinforcement (Mx)
2	#10 @ 65 mm c/c	#10 @ 65 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
8	#12 @ 70 mm c/c	#12 @ 70 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
14	#10 @ 70 mm c/c	#10 @ 70 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
18	#10 @ 60 mm c/c	#10 @ 60 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
22	#8 @ 65 mm c/c	#8 @ 50 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
23	#8 @ 60 mm c/c	#8 @ 55 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
24	#10 @ 65 mm c/c	#10 @ 55 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
25	#8 @ 50 mm c/c	#8 @ 50 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
26	#8 @ 80 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c
42	#8 @ 65 mm c/c	#8 @ 65 mm c/c	#8 @ 80 mm c/c	#8 @ 80 mm c/c

Isolated Footing

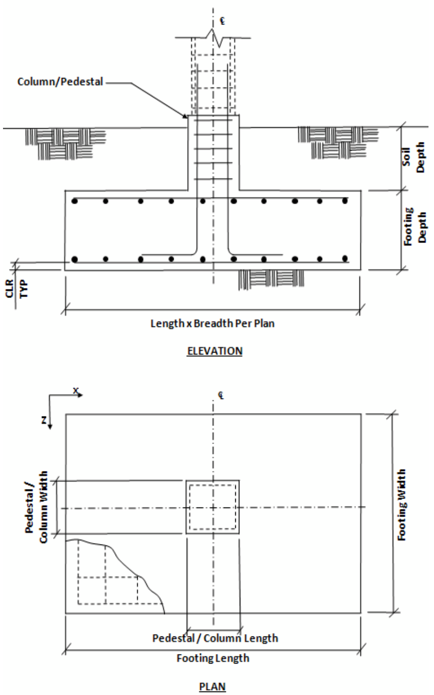


Fig8.1.a Elevation and Plan of Isolated Footing

Footing Geometry
Footing Thickness (Ft) : 500.00 mm
Footing Length – X (Fl) : 1000.00 mm
Footing Width – Z (Fw) : 1000.00 mm

Column Dimensions

ColumnShape :	Rectangular
Column Length–X (Pl) :	0.40m
Column Width –Z (Pw) :	0.40m

Pedestal
Pedestal Length – X :N/A
Pedestal Width – Z :N/A

Design Parameters
Concrete and Rebar Properties
Unit Weight of Concrete: 25.000 kN/m3
Strength of Concrete: 30.000 N/mm2
Yield Strength of Steel: 415.00 N/mm^2
Minimum Bar Size: 25 Dia.
Maximum Bar Size: # 40
Minimum Bar Spacing: 50.00 mm
Maximum Bar Spacing: 500.00 mm
Footing Clear Cover (F, CL): 50.00 mm
SoilProperties:
Soil Type: UN Drained
Unit Weight: 22.00 kN/m3
Soil Bearing Capacity: 300.00 kN/m2
Soil Surcharge: 0.00 kN/m2
Depth of Soil above Footing: 0.00 mm
Untrained Shear Strength: 0.00 N/mm2

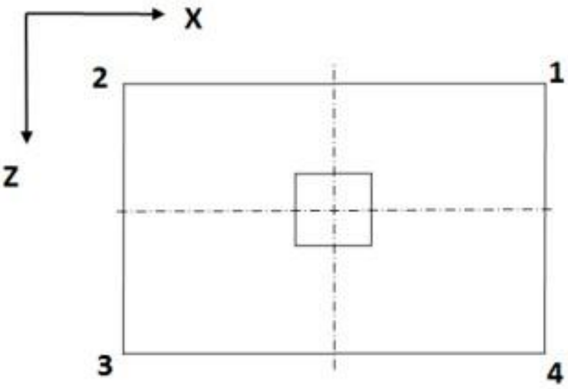
Sliding and Overturning:
Coefficient of Friction:0.50
Factor of Safety against Sliding:1.50
Factor of Safety against Overturning:1.50

Applied Loads – Allowable Stress Level					
Load Case	Axial (kN)	Shear X (kN)	Shear Z (kN)	Moment X (kNm)	Moment Z (kNm)
1	168.123	-1.837	0.275	1.491	1.441
2	140.638	-3.797	-0.289	0.593	3.370
3	842.201	-16.764	-1.784	2.831	14.560
4	-116.948	20.364	19.053	32.167	-52.030
5	1726.443	-33.597	-2.696	7.373	29.057
6	1240.817	-2.441	20.708	44.499	-39.191
7	1521.493	-51.314	-25.020	-32.702	85.682
8	1381.155	-26.878	-2.156	5.898	23.245
9	76.762	27.790	28.993	50.487	-75.884
10	427.606	-33.301	-28.168	-46.014	80.207
11	252.184	-2.755	0.413	2.237	2.162
12	151.310	-1.653	0.248	1.342	1.297

Table no. 8.3 Applied Loads and Allowable Stress Level

Design of Calculations:
Footing Size
Initial Length (L_o)=1.00 m
Initial Width (W_o)=1.00 m
Uplift force due to buoyancy =0.00KN
Effect due to adhesion =0.00 Kn
Min. footing area required from
Bearing pressure, A_{min}= P / q_{max}= 5 . 7 9 6 m^2
Footing area frominitial length and
Width, A_o= L_o+W_o=1.00KN

Final Footing Size
Length (L_2)= 3.80 M Governing Load Case: # 4
Width (W_2)= 3.80 M Governing Load Case: # 4
Depth (D_2)= 0.50 m Governing Load Case: # 4
Area (A_2)= 14.44 m^2
Pressures at Four Corners



Load Case	Pressure at corner 1 (q1) (kN/m^2)	Pressure at corner 2 (q2) (kN/m^2)	Pressure at corner 3 (q3) (kN/m^2)	Pressure at corner 4 (q4) (kN/m^2)	Area of footing in uplift(A_u) (m^2)
5	136.4151	126.3869	127.7045	137.7327	0.00
5	136.4151	126.3869	127.7045	137.7327	0.00
5	136.4151	126.3869	127.7045	137.7327	0.00
5	136.4151	126.3869	127.7045	137.7327	0.00

If $A_u$is zero, there is no uplift and no pressure adjustment is necessary. Otherwise, to account for uplift, areas of negative pressure will be set to zero and the pressure will be redistributed to remaining corners.

Table no 8.4 Pressure Calculation Four Cases

Load Case	Pressure at corner 1 (q1) (kN/m^2)	Pressure at corner 2 (q2) (kN/m^2)	Pressure at corner 3 (q3) (kN/m^2)	Pressure at corner 4 (q4) (kN/m^2)
5	136.4151	126.3869	127.7045	137.7327
5	136.4151	126.3869	127.7045	137.7327
5	136.4151	126.3869	127.7045	137.7327
5	136.4151	126.3869	127.7045	137.7327

Adjust footing size if necessary.
Details of Out-of-Contact Area
(If Any) Governing load case =N/A
Plan area of footing =14.44 sq.m
Area not in contact with soil =0.00 sq.m
% of total area notincontact=0.00%

Check for Stability against Overturning and Sliding

-	Factor of safety against sliding		Factor of safety against overturning	
Load Case No.	Along X-Direction	AlongZ-Direction	About X- Direction	About Z- Direction
1	94.894	633.851	406.729	280.722
2	42.284	556.554	1358.999	115.812
3	30.503	286.709	1001.994	84.696
4	1.560	1.668	2.896	1.941
5	28.379	353.724	601.339	79.013
6	291.089	34.319	49.231	71.120
7	16.584	34.012	71.523	29.044
8	29.051	362.094	615.569	80.883
9	4.629	4.437	7.522	5.444
10	9.130	10.794	19.225	11.929
11	78.517	524.459	336.534	232.274
12	100.353	670.316	430.128	296.871

Table no 8.5 Check for Stability against Overturning and Sliding

Critical Load Case and the Governing FactorOfSafety For Overturningand Sliding X Direction:
Critical Load Case for SlidingalongX-Direction:4
Governing Disturbing Force:20.364 kN
GoverningRestoringForce:31.776kN
MinimumSlidingRatiofortheCriticalLoadCase:1.560
CriticalLoadCaseforOverturningaboutX-Direction:4
GoverningOverturningMoment:41.693kNm
Governing Resisting Moment: 120.746kN
MinimumOverturningRatiofortheCriticalLoadCase:
Critical Load and the Governing Factor of Safety for Case Over turning and sliding Direction
Critical LoadCaseforSlidingalongZ-Direction:4
GoverningDisturbingForce:19.053kN
GoverningRestoringForce:31.776kN
MinimumSlidingRatiofortheCriticalLoadCase:1.668
CriticalLoadCaseforOverturningaboutZ-Direction:4
GoverningOverturningMoment:-62.212kNm
GoverningResistingMoment:120.746kNm
MinimumOverturningRatiofortheCritical LoadCase1.941
MomentCalculation:
Check Trial Depth against moment Critical Load Case = # (w.r.t. Axis)
Governingmoment(M_u) = 678.540753kNm
AsPerIS4562000ANNEXG G-1.1C
LimitingFactor1 (K_{umax}=700/(1100+0.87xfy)=0.479107
Limiting Factor2 (R_{umax}) =0.36xf_{ck}xk_{umax}x (1-0.42xkumax)=4133.149375kN/m^2
Limit Moment Of Resistance= (M_{umax})=R_{umax}x Bxd =3138.136379kN
Staircase
A stair is a series of steps arranged in such a manner as to connect different floors of a building. Stairs are designed to provide an easy and quick access to different floors. The stairs should be thoughtfully located, carefully planned, tastefully designed, serving its purpose and at the same time being economical in construction.
Stairs are provided in a building to afford a means of communication between various. Since they have to perform the very important function, the slab over which the steps rest should be designed properly to provide maximum

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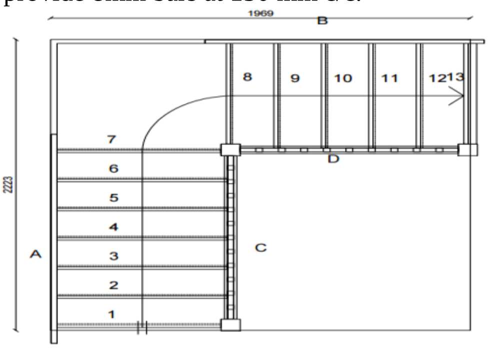
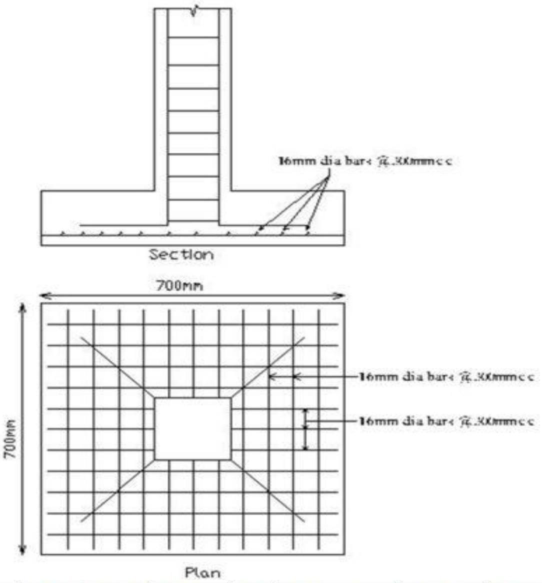
comfort, easy and safety. The most important aspect in providing staircase is its location. The location of staircase should be such as to provide an easy access so that in case of any casualty, occupants should be placed in the centre or to the side of a building. The location depends upon the positions of the rooms and the type of approach needed. In the commercial buildings, it should be placed centrally so as to:
Provide easy access to all shops/offices.
Maintain privacy.
The inclined slab of a stair is known as height of stair while the straight portion other than the floor level is known as landing. While going on flight, one travels vertically. The landing is provided mid-way either to turn the position and lore to relax while going up. Vertical height to a stair is known as Rise and available horizontal distance on stair is known as Tread.
Rise of the steps - 150mm to 180mm
Tread of the step - 200mm to 300mm
The width of the staircase – 1m in residential buildings to 2m in public buildings.
Classification of Stairs:
According to structural aspect:
Stairs spanning horizontally.
According to arrangement aspect:
Straight Stairs.
Quarter landing Stairs.
Dog-legged Stairs.
Open well Stairs.
Guidelines for fixing the dimensions of component parts of Stairs:
The rise should be between 150mm to 180mm and tread between 220mm to 250mm for the residential buildings. The riser should be between 120mm to 150mm and tread between 250mm to 300mm in public buildings.
The sum of the tread and twice the rise (T+2R) should be given between 500mm to 650mm.
The width of the stairs should be between 0.8m to 1m for residential buildings. The width of the stairs should be between 1.8m to 2m for public buildings.
The width of landing should not be less than the width of stairs.
The number of steps in each flight should not be greater than 12.
The pitch of the stairway should be greater than 38degrees.
The head room measured vertically above any step or below mid landing shall not be less than 2.1m.
Avoid winders as far as possible.
Design of Staircase
Proportioning of stairs:
Dimensions of stair hall = 6.096 m x 2.7432 m
Height of the floor = 3 m
Height of one flight = (3/2) = 1.5 m
Rise, R = 0.15 m
Tread, T = 0.28m
Number of risers= (1.5/0.150)=10
Hence, number of tread = 10-1=9
Adopt width of the stair = (9'/2) = (2.74/2)= 1.37 m
For 9 treads, the length required = 9 x 0.28 = 2.52 m.
Width of landing = 1.867 m.
Effective span:
As the stair slab is spanning longitudinally,
Effective span = c/c distance of walls
= 6.096 m
Thickness of slab:
Assume effective depth,
d= (span/25) = (5.8674/25) = 234 mm
Adopt, d= 240mm
D=270mm
Loads:
Loads per meter horizontal width of stairs are as follows,
Weight of waist slab = $D\,(1+(R/T)^2)^{0.5} \times 25$
= 7.65 kN/m
Weight of steps = $\frac{0.5RT}{T} \times 25 = \frac{R}{2} \times 25$
$= \frac{0.5\times0.15\times0.28}{0.28} \times 25 = 1.875$ kN/m^2
Live load = 3 kN/m^2
Floor finished load = 0.6 kN/m^2
Total load = 13.125 kN/m^2
Factored load $= w_u$ = 1.5 x total load = 19.6875 kN/m^2
Factored B.M:
$M_u = (w_u l^2)/8$
= 84.77 kN-m.
= 84.77x10^6 N-mm.

Minimum depth required:
The minimum depth required to resist bending moment
$M_u = 0.138\, f_{ck} b d^2$
$84.77\times10^6 = 0.138\times30\times1000\times d^2$
d =143.09mm < 240 mm (provided depth)
Hence provided depth is adequate.
Tension Reinforcement:
$M_u = 0.87\, f_y A_{st} d(1-(f_y A_{st})/(f_{ck}.b.d)$
$A_{st} = (0.5 f_{ck}.b.d/f_y)\times(1-(1-(4.6\, R_U/f_{ck}))^{0.5})$
$R_U = (M_u/b.d^2)$
= 1041.26mm^2
Using 12 mm diameter bars
Spacing of bars,
$S = (a_{st}/A_{st}) \times 1000$ = 108.61 mm =100mm
Distribution Reinforcement
A_{st} = 0.12% of gross area
=0.12x1000x270/100 = 324mm^2
Using 8mm bars
Spacing,
S= 3.14x8^2x1000/(4x324) = 155.14 mm spacing
Hence, provide 8mm bars at 150 mm c/c.



Plan view of the semi landing stair case

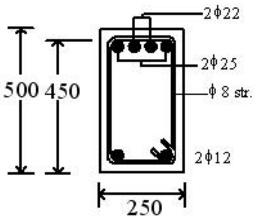
5.4 Estimation and costing
Bar bending schedule of a column:

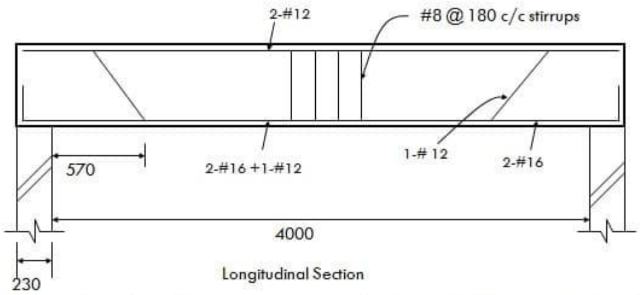


SNO	PARTICULARS	DIA	NOS	LENGTH	WEIGHT PER MT	QUANTITY
	16mm @300 mm c/c	16	3	800	1.58	3.792
2.	16mm @300 mm c/c	16	3	800	1.58	3.792
3.	Column vertical bar	16	4	4.37	2.58	27.61
4.	Links 8Φ@300 mm c/c	8	14	0.59	0.39	3.2214
					Total	34.62

Total quantity =34.62
Rate per unit quantity =RS 60 /kg
Total rate = 2077.2*42
Total amount = 87242.4

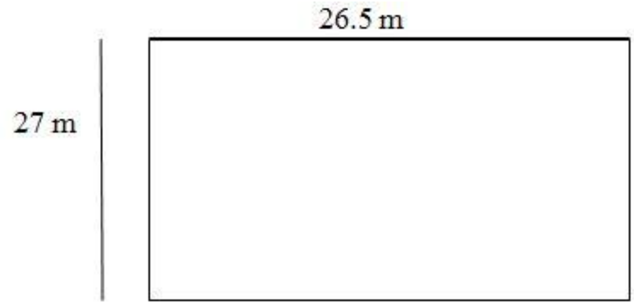
BAR BENDING SCHEDULE FOR BEAMS:





sno	particular	dia	no	length	weight	quantity
1	bottom bar	25mm	2	4.95	3.85	38.115
2	top bar	22mm	2	4.896	2.98	29.18
3	cranked bar	12mm	2	5.116	0.88	9
4	stirrups	8mm	25	1.418	0.395	14
					Total	90.29

Total quantity = 90.29
Rate per unit quantity =Rs 60
Total amount = Rs 2,27,530
Total amount for steels =2,27,530 +87,242.4
= RS 3,14,772.4
Cement motar
1 cubic feet -500 bricks
Amount = 500 *5
= 2500 *80
= Rs 2,00,000
Aggregates –Rs 5 lakhs
Flooring -1 lakh
Sand- 1 ton –Rs 6000
20 tons –rs 6 *20
=Rs 1,20,000
Mason-Rs 1,00,000
Helpers-70,000
Doors and windows- Rs 1,00,000
Furnitures-Rs70,000
Centering labour –Rs1,00,000
Water tanker –Rs 70,000
Plumbing work –Rs1,00,000
White washing /painting-1,00,000
Total amount for one floor =Rs27,24,772.4



V=l*b*d
=27*26.5*0.15
=107.325
Dry volume =107.325*1.54
=165.280
Volume of cement
1/7 *165.280 =23.611 *Rs 300 cft
=5547

Sand =2/7*165.280=47.22 cft =283320

4/7 *165.280 =94.44
=94440
Total =383307 *10
= Rs 38,33070
Total amount =Rs 65,57842.4
Contractor profit =10% =RS 655784.24
Water charges =1.5% =Rs 98367763.6
Total cost of the building =3-4 crores

VI. SUMMARY & CONCLUSION

6.1. SUMMARY

The obtained results of static and dynamic analysis in OMRF & SMRF are compared for different columns under axial, torsion, bending moment and displacement forces. The

results in graph-1 shows that there is equal values obtained of axial forces in static and dynamic analysis of OMRF structure. The results in graph-2 shows that the values are obtained for torsion in static analysis are negative and dynamic analysis values are positive. The results in graph-3 here we can observe that the values for bending moment at dynamic analysis values are high in initially for other columns it decreased gradually as compared to that of static analysis. The results in graph-4 we can observe that the values for displacement in static analysis of OMRF values are more compared to that of dynamic analysis values of same columns. The results in graph-5 shows that the values obtained of axial forces in dynamic analysis of SMRF structure values are high compare to static analysis.
The results in graph-6 shows that the values are obtained for torsion in static analysis are negative and dynamic analysis values are positive with more difference. In the results graph-7, we can observe that the values for bending moment at dynamic analysis values are more as compared to that of static analysis SMRF structure. In the results graph-8, we can observe that the values for displacement in dynamic analysis of SMRF values are gradually increased compared to that of static analysis values of same columns. The static and dynamic analysis of OMRF & SMRF values is observed. Finally it can conclude that the results of static analysis in OMRF & SMRF values are low when comparing to that of dynamic analysis in OMRF & SMRF values. Hence the performance of dynamic analysis SMRF structure is quiet good in resisting the earthquake forces compared to that of the static analysis OMRF & SMRF.

VII. CONCLUSION

The effect of the basement on the seismic response of high-rise buildings and the effect of the lateral forces applied to the superstructure on the member forces in the basement were investigated in this study and the following conclusions could be drawn.

1. Lateral stiffness of a high-rise building structure may be significantly overestimated resulting in larger lateral displacements and shorter natural periods of vibration if the basement of a high rise building is ignored in the analytical model. Especially in the case of the building structures with shear walls, the effect of the basement on the seismic response turned out to be more significant. Therefore, it is necessary to include the effect of basement in the analysis of high rise building structures.
2. Lateral loads affect not only the response of the super structure but also that of the basement structure.

Therefore, seismic loads as well as gravity loads should be considered in the analysis of a high-rise building structure for the design of the basement structure.

3. The story shear forces in the basement may be significantly overestimated if the rigid diaphragm assumption is applied to the basement. Therefore, an efficient analysis method using partial rigid diaphragms is proposed in this study for the analysis of high-rise buildings subjected to lateral forces such as the seismic loads including the effects of basement.

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