Asian Development Bank
National Capital Region Planning Board

Capacity Development of the
National Capital Region Planning Board
Package 2 Component B
TA No. 7055-IND

Volume V-D1: Main Report
DPR for Multi-level Parking Facility in Ghaziabad

WilburSmith

July 2010
Capacity Development of the National Capital Region Planning Board (NCRPB) – Component B (TA No. 7055-IND)

FINAL REPORT
Volume V-D1: DPR for Multi-level Parking Facility at Ghaziabad
Main Report

July 2010
### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Full Form</th>
</tr>
</thead>
<tbody>
<tr>
<td>ADB</td>
<td>Asian Development Bank</td>
</tr>
<tr>
<td>BIS</td>
<td>Bureau of Indian Standard</td>
</tr>
<tr>
<td>BOQ</td>
<td>Bill of Quantities</td>
</tr>
<tr>
<td>CBR</td>
<td>California Bearing Ratio</td>
</tr>
<tr>
<td>CMSA</td>
<td>Cumulative number of Million Standard Axles</td>
</tr>
<tr>
<td>DFR</td>
<td>Draft Final Report</td>
</tr>
<tr>
<td>DL</td>
<td>Deal Load</td>
</tr>
<tr>
<td>DPR</td>
<td>Detailed Project Report</td>
</tr>
<tr>
<td>ECS</td>
<td>Equivalent Car Space</td>
</tr>
<tr>
<td>GDA</td>
<td>Ghaziabad Development Authority</td>
</tr>
<tr>
<td>INR</td>
<td>Indian Rupees</td>
</tr>
<tr>
<td>IRC</td>
<td>Indian Road Congress</td>
</tr>
<tr>
<td>IS</td>
<td>Indian Standard</td>
</tr>
<tr>
<td>KMPH</td>
<td>Kilometer per Hour</td>
</tr>
<tr>
<td>LCV</td>
<td>Light Commercial Vehicle</td>
</tr>
<tr>
<td>LL</td>
<td>Live Load</td>
</tr>
<tr>
<td>MAV</td>
<td>Multi-axle Vehicle</td>
</tr>
<tr>
<td>MORT&amp;H</td>
<td>Ministry of Road Transport and Highways</td>
</tr>
<tr>
<td>NCR</td>
<td>National Capital Region</td>
</tr>
<tr>
<td>NCRPB</td>
<td>National Capital Region Planning Board</td>
</tr>
<tr>
<td>NH</td>
<td>National Highway</td>
</tr>
<tr>
<td>RCC</td>
<td>Reinforced Cement Concrete</td>
</tr>
<tr>
<td>ROW</td>
<td>Right of Way</td>
</tr>
<tr>
<td>SP</td>
<td>Standard Procedure</td>
</tr>
<tr>
<td>TA</td>
<td>Technical Assistance</td>
</tr>
<tr>
<td>UP</td>
<td>Uttar Pradesh</td>
</tr>
<tr>
<td>UPSRTC</td>
<td>Uttar Pradesh State Road Transport Corporation</td>
</tr>
</tbody>
</table>
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Compendium Volumes

Besides this Volume V-D1, the DPR Multi-Level Parking in Ghaziabad has following Volumes appended separately.

Volume V-D2: Financial & Economic Analysis
Volume V-D3: Initial Environmental Examination
Volume V-D4: Short Resettlement Plan
1. INTRODUCTION

A. Background

1. The National Capital Region Planning Board, constituted in 1985 under the provisions of NCRPB Act, 1985, is a statutory body functioning under the Ministry of Urban Development, Government of India. NCRPB has a mandate to systematically develop the National Capital Region (NCR) of India. It is one of the functions of the Board to arrange and oversee the financing of selected development projects in the NCR through Central and State Plan funds and other sources of revenue.

2. On Government of India’s request, Asian Development Bank (ADB) has formulated the technical assistance (TA) to enhance the capacities of National Capital Region Planning Board and its associated implementing agencies. The TA has been designed in three components: Component A relates to improving the business processes in NCRPB; Component B relates to improving the capacity of the implementing agencies in project identification, feasibility studies and preparing detailed engineering design; and Component C relates to urban planning and other activities.

3. ADB has appointed M/s Wilbur Smith Associates to perform consultancy services envisaged under Component B. In the context of this contract, the first deliverable – Inception Report, was submitted in October 2008. The second deliverable –Interim Report comprising Master Plan for sewerage in Hapur, Master Plan for Water Supply for Panipat, Master Plan for Drainage for Hapur, Master Plan for Solid Waste management for Ghaziabad, Traffic and Transport analysis for Ghaziabad, Socio-Economic base line survey result in 3 sample project towns and proceedings of workshop 1 was submitted in January 2009. The four Master Plans as stated above are also made available on NCRPB web site for use of the implementing agencies.

4. The third deliverable Draft Final Report (DFR) comprising Detailed Project Report (DPR) for water supply in Panipat, DPR for sewerage in Hapur, DPR for drainage in Hapur, DPR for drainage in Sonipat, DPR for solid waste management in Ghaziabad, DPR for four selected transport components (Flyover, Road widening, Multi-level Parking and Bus Terminal) in Ghaziabad, and a Report on Capacity Building Activities were submitted.

5. Now, this is the Final Report (FR) and is the fourth and final deliverable. The comments/feedback on Draft Final Report received from ADB, NCRPB and respective implementing agencies were duly incorporated and final DPRs for components of Water Supply, Sewerage, Drainage, Solid Waste Management, and Transport are submitted as part of this Final Report. This is the Detailed Project Report for Transport Component of Multi-level Parking in Ghaziabad.
B. Overview of this ADB TA

6. Objectives. The objective of this TA is to strengthen the capacity at NCRPB, state-level NCR cells, and other implementing agencies in the area of planning for urban infrastructure and to impart necessary skills to conceive, design, develop, appraise and implement good quality infrastructure projects for planned development of NCR. The increased institutional capacity of the NCRPB and the implementing agencies will lead to effective and time scaling-up of urban infrastructure to (i) improve quality of basic urban services in the NCR; (ii) develop counter magnet towns; (iii) reduce in migration into Delhi and orderly development of NCR; and (iv) accelerate economic growth in the NCR.

7. The TA – Capacity Development of the NCRPB, Component B focuses on strengthening the capacities of NCRPB and implementing agencies relating to project feasibility studies and preparation, and detailed engineering design in the implementing agencies. Specifically this component B of the TA will support the project preparation efforts of the implementing agencies by preparing demonstration feasibility studies that include all due diligence documentation required for processing of the project in accordance with best practices, including ADB’s policies and guidelines.

8. Scope of Work. According to the terms of reference of the TA assignment, the following activities are envisaged in component B of the TA:

(i) Conduct technical, institutional, economic and financial feasibility analysis of identified subprojects in the six sample implementing agencies;
(ii) Conduct safeguards due diligence on the subprojects, including environmental assessment report and resettlement plan for all subprojects covered in the sample implementing agencies;
(iii) Prepare environmental assessment framework and resettlement framework; and
(iv) Develop a capacity building and policy reform program for the implementing agencies, including governance strengthening, institutional development and financial management.

9. Besides, this component of the TA will also:

(i) help in assessing the current practices and procedures of project identification and preparation of detailed project reports including technical, financial, economic and social safeguard due diligence;
(ii) support preparation of standard procedure manuals for project identification and preparation of detailed project reports including technical, financial, economic and social safeguard due diligence;
(iii) train the implementing agencies in the preparation of detailed project reports by using the sample subprojects, reports on deficiency of current practices and standard protocol manuals; and
(iv) help in developing a user-friendly web-page where different manuals and guidelines for preparation of DPRs will be made available for the implementing agencies.
C. About the Final Report

10. At Interim Report stage of the TA, the Master Plans for Water Supply in Panipat, Sewerage system in Hapur, Drainage for Hapur and Municipal Solid Waste Management for Ghaziabad were prepared. The Master Plans provided 100 percent coverage of population and the area likely to be in planning horizon year 2031/2041. All works required up to planning horizon year were conceptualized, broadly designed and block cost was estimated. The Master Plans also provided phasing of investment such that under phase 1 works required to cover present spread of city were proposed.

11. At draft final report stage of the TA the Detailed Project Reports (DPRs) were prepared for Phase 1 works as suggested in the Master Plans. For preparation of DPRs, engineering surveys and investigations were conducted and various possible and feasible alternatives evaluated. Finally for the selected options the DPRs prepared with detailed designs, item wise detailed cost estimate, work specifications, implementation process and proposed implementation arrangements. Further, according to ADB procedures these DPRs in addition to technical analysis included institutional, financial and economic feasibility analysis and environmental and social safeguards due diligence – environmental assessment and resettlement plans.

12. The DPR's submitted as part of Draft Final Report was reviewed by the implementing agencies, NCRPB and the ADB. Now this Final Report comprising DPR's modified in light of comments of IA's is being submitted. The draft DPR for water supply in Panipat was reviewed by PHED Haryana. Detailed discussions were held with Superintending Engineer (Urban), Executive Engineer (Urban), Superintending Engineer (Karnal) and Executive Engineer Panipat. The comments made by PHED have been suitably incorporated in this Final Report.

13. These DPRs are proposed to be made available to the ULBs and other implementing agencies of the state governments as model DPRs so that they may replicate the methodology/approach in the future DPRs prepared by them for obtaining finances from the NCRPB.

14. Organization of this Final Report. The Final Report of the TA Component B is organized in following Seven Volumes:

**Volume I**: Detailed Project Report for Water Supply System in Panipat

**Volume II**: Detailed Project Report for Rehabilitation and Augmentation of Sewerage System in Hapur

**Volume III**: Detailed Project Report for Rehabilitation of Major Drains in Hapur

**Volume IV**: Detailed Project Report for Improvement of Solid Waste Management System in Ghaziabad

**Volume V**: Detailed Project Reports for Four Transport Components in Ghaziabad

**Volume VI**: Detailed Project Reports Rehabilitation of Drainage in Sonipat
Volume VII: Capacity Building Activities

D. Structure of Volume V Report

15. The DPRs for all four transport components are compiled in Volume V. This is Volume V is presented four volumes:

(i) Volume V-A: DPR for Mohan Nagar Flyover
(ii) Volume V-B: DPR for Road Widening
(iii) Volume V-C: DPR for Bus Terminal
(iv) Volume V-D: DPR for Multi-level Parking

1. Structure of this Volume V-D Report

16. This DPR for Multi-level Parking Facility in Ghaziabad is compiled in following four sub-volumes (Volumes V-D1 to V-D4) including this Main Report:

Volume V-D1: Main Report:

- **Section 1** Introduction
- **Section 2** presents parking demand analysis
- **Section 3** presents planning and design of the proposed parking facility
- **Section 4** presents cost estimates

Volume V-D2: Financial & Economic Analysis
Volume V-D3: Initial Environmental Examination
Volume V-D4: Short Resettlement Plan
2. PARKING DEMAND ANALYSIS

A. Overview

17. The unprecedented growth of personalized vehicles and the unplanned road infrastructure have made the provision for parking an important aspect of transportation planning. As part of Traffic Study conducted as part of this ADB TA, a parking study was also conducted at important locations in Ghaziabad. The area surrounding the old Bus Stand at Navyug Market is major centre and is CBD of Ghaziabad. This centre is busy with various activities; a number of commercial establishments, markets, government offices (Ghaziabad Nagar Nigam and Ghaziabad Development Authority) and the bus stand are situated here. Since most of these places are frequented by public and busy with floating population, the demand for parking is very high.

17. On-street parking is observed on all the roads surrounding Old Bus Stand and Navyug Market. Many cars and two wheelers are seen parked on either side of the roads. Both angular as well as parallel type of parking was noticed on almost all the stretches of the roads. This has reduced the capacity of the carriageway and endangering pedestrians and motorists alike. The frontage of almost all the roads in this area has been converted into commercial land use without taking into account the demand for parking of the vehicles. There is no planned parking space available.

18. Following sections assess the parking situation in the CBD area, its demand and the supply analysis.

B. Existing Parking Scenario

19. At present, the vicinity of the old bus stand has developed in to a business and commercial hub along with government offices and restaurants. Thus, the demand for the parking has increased leading to parking irregularities. On-street parking is observed on all the roads surrounding Old Bus Stand and Navyug Market which is adjacent to Ghaziabad Development Authority. Many cars and two wheelers are seen parked on either side of the roads. Both angular as well as parallel type of parking can be noticed on almost all the stretches of the roads. Consequently, almost one lane of the carriageway is taken up, in turn creating traffic chaos.

20. Following Photographs show the on-street parking scenario in CBD Area near GDA office and Figure 2-1 show the location of existing parking places.
**Photographs:** On-Street Parking near GDA Office
**Figure 2-1**: On-street and Off-street Parking in the Study Area
C. Parking Demand Analysis

21. The following steps are involved in the parking study for Ghaziabad CBD area:

- Site reconnaissance
- Conduct of Parking and Traffic Surveys
- Estimation of current parking supply and demand
- Future demand forecast
- Development of conceptual parking facility layout

22. The following sections illustrate the approach and methodology, which have been followed to undertake the study. Before the betterment of parking problems, it is necessary to analyze the existing parking characteristics at various locations. Parking surveys are intended to provide all the information needed for assessment of the parking demand and supply for the study area. All the survey formats are included in Annexure I.

23. Site Reconnaissance Survey. In order to understand the study, a site reconnaissance survey is essential. This survey was done for the study area, to capture the road characteristics like available ROW and carriageway width. Land use in the site vicinity was also ascertained. Major establishments and traffic generators were identified. Traffic circulation and accessibility to the site were also assessed. Based on the above surveys, major issues in the area were identified.

24. Parking Surveys. The following surveys were conducted for understanding the parking characteristics, estimation of demand and supply for parking and for projecting the future parking demand.

- On-street parking surveys to study the parking characteristics and demand along the roadside.
- Opinion surveys (willingness to pay surveys) elicited opinion of the users about the facilities available for parking and their willingness to pay the fee for using the facility. The opinion survey also revealed the extent of suppressed parking demand.
- Inventory surveys were conducted to collect the potential of the existing facility in terms of available space, road characteristics, type of parking, land use of the location, etc.

25. Existing parking demand has been estimated through aggregation of on-street and off-street parking surveys. After the reconnaissance survey, the study area layout map was prepared on which the road stretches and off street parking lots within 500m (walk distance) from the proposed locations were identified for parking surveys. The roads for survey were identified based on the intensity of parking on the selected stretches, and also their connectivity to the proposed site.
1. **On Street Survey**

26. On street parking surveys are intended to collect the extent of usage of parking facilities along the roadside. The survey has been conducted by counting the vehicles parked on the road at regular intervals for a particular duration of the day.

27. The locations for the on-street survey have been identified through reconnaissance survey. The survey was conducted to ascertain the characteristics and magnitude of parking and accumulation on the adjoining streets of the proposed parking. The proposed site and about 500 m around proposed parking were surveyed for on-street parking survey. Registration numbers of the parked vehicles were noted down at half an hour interval in major Parking areas. The survey was conducted from 8AM to 10 PM for three weekdays. The data collected include the time, type and registration number of the vehicle. Parking pattern of unregistered vehicles (like cycles, cycle rickshaws, etc.) was also estimated by counting such type of vehicles parked along the selected locations.

28. The data were entered in the format with the codes for each vehicle type. The codes adopted for various vehicle types are given in Table 2-1.

### Table 2-1: Codes adopted for various types

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Vehicle Category</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Big Car</td>
<td>bc</td>
</tr>
<tr>
<td>2</td>
<td>Small Car</td>
<td>sc</td>
</tr>
<tr>
<td>3</td>
<td>Two Wheelers</td>
<td>tw</td>
</tr>
<tr>
<td>4</td>
<td>Van</td>
<td>v</td>
</tr>
<tr>
<td>5</td>
<td>Jeep</td>
<td>j</td>
</tr>
<tr>
<td>6</td>
<td>Bus</td>
<td>b</td>
</tr>
<tr>
<td>7</td>
<td>Trucks</td>
<td>t</td>
</tr>
<tr>
<td>8</td>
<td>MAV</td>
<td>mA</td>
</tr>
<tr>
<td>9</td>
<td>LCV</td>
<td>IC</td>
</tr>
<tr>
<td>10</td>
<td>Auto Rickshaw</td>
<td>aR</td>
</tr>
</tbody>
</table>

29. The data has been analyzed and the results are presented in terms of accumulation graphs and duration diagrams. Different Equivalent Car Spaces (ECS) values were adopted for different vehicle types and are given in Table 2-2. The ECS values were arrived based on the size of various vehicles and compared with that of passenger cars. The duration of vehicles parked was classified into three categories and is given in Table 2-3.
Table 2-2: ECS Values adopted for Various Vehicle Types

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Vehicle Category</th>
<th>ECS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Car</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>Two Wheelers</td>
<td>0.25*</td>
</tr>
<tr>
<td>3</td>
<td>Bus</td>
<td>2.5</td>
</tr>
<tr>
<td>4</td>
<td>Trucks</td>
<td>2.5</td>
</tr>
<tr>
<td>5</td>
<td>LCV</td>
<td>1.75</td>
</tr>
<tr>
<td>6</td>
<td>Auto</td>
<td>0.5</td>
</tr>
<tr>
<td>7</td>
<td>Cycles</td>
<td>0.1</td>
</tr>
<tr>
<td>8</td>
<td>Cycle Rickshaw</td>
<td>0.8</td>
</tr>
<tr>
<td>9</td>
<td>Carts</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Source: Parking Requirements in CMA, 2003, Wilbur Smith Pvt Ltd
*Source: Module 4-Guidelines for Parking Measures-Policy and Options, MOUD and PADECO Co. Ltd

Table 2-3: Classification of Duration of Parking

<table>
<thead>
<tr>
<th>SL. No.</th>
<th>Duration of Parking</th>
<th>Designation of Parking</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt; 1 Hr</td>
<td>Very Short Duration</td>
</tr>
<tr>
<td>2</td>
<td>1-2 Hours</td>
<td>Short Duration</td>
</tr>
<tr>
<td>3</td>
<td>2-5 Hours</td>
<td>Medium Duration</td>
</tr>
<tr>
<td>4</td>
<td>5-10 Hours</td>
<td>Long Duration</td>
</tr>
<tr>
<td>5</td>
<td>&gt; 10 Hours</td>
<td>Long Term Parking</td>
</tr>
</tbody>
</table>

30. Some important terms associated with parking are explained below:

(i) Parking Accumulation- Total number of vehicles parked in an area at a particular time period.
(ii) Parking Duration- Length of time a vehicle spent in a parking space
(iii) Parking Occupancy- Number of spaces occupied as a percent of total available spaces.
(iv) Parking Turnover-It is usually calculated as the number of time a parking space is been used during the day. Since there are no parking space demarcated in the study areas, and observed parking does not follow any parking norms, the parking turnover for the study is calculated per zone per ECS.

31. Outcome of the surveys include estimates of on-street parking volume, duration and accumulation along each designated roadway stretch.
2.  *Willingness to Pay Survey*

32. Opinion surveys were conducted to elicit the opinion of the parkers about the facilities available for parking and about their willingness to pay the fee for using the proposed facility. The survey was done on a random sample basis during peak and off-peak periods. Users of the parking lot were interviewed and responses elicited include problems in existing parking facility, origin, destination, distance traveled, frequency of the visit, purpose of the visit, parking duration, occupancy, opinion about the existing parking rate with respect to existing facility, opinion about the parking fee system and willingness to pay. This survey was conducted on both weekdays and weekends.

33. A total of 522 samples were collected which comprised of 260 samples from Car users and 262 samples from non car users. Outcome of the survey includes identification of influence area of the market, frequency of the visit, purpose of the visit, problems with existing parking facility, occupancy rate opinion about the existing parking rate and about the future parking charge system with improved parking facility.

3.  *Survey Schedule*

34. Parking surveys are conducted in the third and fourth weeks of August 2009. Detailed schedule of all surveys is presented in the following Table 2-4.

<table>
<thead>
<tr>
<th>Type</th>
<th>Schedule</th>
<th>Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Junction Volume Count</td>
<td>17-08-09</td>
<td>Monday</td>
</tr>
<tr>
<td>Junction Volume Count</td>
<td>18-08-09</td>
<td>Tuesday</td>
</tr>
<tr>
<td>Junction Volume Count</td>
<td>19-10-08</td>
<td>Wednesday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>20-08-09</td>
<td>Thursday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>21-08-09</td>
<td>Friday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>24-08-09</td>
<td>Monday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>25-08-09</td>
<td>Tuesday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>26-08-09</td>
<td>Wednesday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>27-08-09</td>
<td>Thursday</td>
</tr>
<tr>
<td>Parking Accumulation Survey</td>
<td>28-08-09</td>
<td>Friday</td>
</tr>
<tr>
<td>Opinion Survey</td>
<td>20-08-09</td>
<td>Thursday</td>
</tr>
<tr>
<td>Opinion Survey</td>
<td>21-08-09</td>
<td>Friday</td>
</tr>
<tr>
<td>Opinion Survey</td>
<td>24-08-09</td>
<td>Monday</td>
</tr>
</tbody>
</table>

*Table 2-4: Schedule of Surveys*
D. Parking Analysis and Findings

1. Roadway Inventory (Carriageway and ROW) and Issues at the proposed site for parking.

35. The roadway details near the site are as follows:

- Dr Ambedkar Marg, the section leading the main entry to the site, is a four-lane divided roadway (2 lanes in each direction) with ~21 meter width.
- The right of entry to the site has no issues because the road width is sufficient enough to accommodate the entry-exit ramps.
- The roads covering the site have the width of about 7 meters on service lane of the Hapur Road on the northern side of the old bus stand and about 10 m on the cross road on the southern side of the old Bus Stand Road respectively.
- All the roads at the vicinity of the site are two way roads.

36. Major issues near the site are as follows:

- An existing off street paid parking lot near Ghaziabad Development Authority next to the old bus stand cannot cater to the parking demand in the area.
- Public tend to park on street (parallel parking) on the service lane adjacent to old bus stand near the GDA causing inconvenience to the traffic which takes almost one lane from the existing 7m width of the road.
- Unauthorized parking and vehicle waiting in front of Ghaziabad Development Authority and Nagar Nigam cause a lot of confusion and create bottleneck for the turning vehicles.
- Unauthorized Parking near Ghaziabad Development Authority and Nagar Nigam uses up the capacity of the roadway, thus affecting the flow of traffic. Parking needs to be prohibited in this area.
- Parking of auto rickshaws and cycle rickshaws is haphazard in front of the bus stand, where the entry and exit are at the same point. This causes queuing and traffic bottlenecks in this area.

2. Parking Supply

37. Two types of on street parking - parallel, and angular are prevailing in the study area. On street parking includes vehicles parked on the street. There may be authorized parking stretches, as well as stretches where parking is prohibited, but still parking is observed. Majority of the on-street and off-street are unauthorized free parking. On street supply is estimated based on the number of authorized parking slots on a given stretch by maintaining the present configuration and availability of adequate right of way. Parking
supply is calculated as per the equivalent car space norms and available width of the right of way. For parallel and perpendicular parking, supply is calculated by dividing the length of the stretch by 7.5 m and 4 m respectively.

38. Total parking supply is calculated for the whole area based on the method mentioned above. Parking Inventory of all the roads surveyed in the study area is given in Annexure II.

39. On-street and off-street parking as shown in Figure 2-1 was used as zoning system for the parking analysis. The parking supply and demand estimations are shown in Table 2-5, Table 2-6, and Table 2-7. The hourly parking accumulations are shown in Figure 2-2 to Figure 2-6.

Table 2-5: Parking Demand for 16 hours –On Street (ECS) Cars and Two wheelers

<table>
<thead>
<tr>
<th>Zones</th>
<th>Day 1 ECS</th>
<th>Day 2 ECS</th>
<th>Day 3 ECS</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>3960</td>
<td>3130</td>
<td>2460</td>
<td>9550</td>
</tr>
<tr>
<td>Zone 2</td>
<td>1260</td>
<td>1250</td>
<td>1220</td>
<td>3730</td>
</tr>
<tr>
<td>Zone 3</td>
<td>840</td>
<td>842</td>
<td>830</td>
<td>2512</td>
</tr>
<tr>
<td>Zone 4</td>
<td>680</td>
<td>675</td>
<td>670</td>
<td>2025</td>
</tr>
<tr>
<td>Ambedkar Road</td>
<td>630</td>
<td>810</td>
<td>740</td>
<td>2180</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7370</strong></td>
<td><strong>6707</strong></td>
<td><strong>5920</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table 2-6: Parking Demand for Peak Hour –On Street (ECS) Cars and Two wheelers

<table>
<thead>
<tr>
<th>Zones</th>
<th>Day 1 Peak hour ECS</th>
<th>Day 2 Peak hour ECS</th>
<th>Day 3 Peak hour ECS</th>
<th>Total Peak hour ECS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>373</td>
<td>300</td>
<td>190</td>
<td>863</td>
</tr>
<tr>
<td>Zone 2</td>
<td>90</td>
<td>95</td>
<td>90</td>
<td>275</td>
</tr>
<tr>
<td>Zone 3</td>
<td>55</td>
<td>50</td>
<td>53</td>
<td>158</td>
</tr>
<tr>
<td>Zone 4</td>
<td>40</td>
<td>47</td>
<td>45</td>
<td>132</td>
</tr>
<tr>
<td>Ambedkar Road</td>
<td>50</td>
<td>65</td>
<td>70</td>
<td>185</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>608</strong></td>
<td><strong>557</strong></td>
<td><strong>448</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table 2-7: Peak hour Parking Demand, Parking Supply and Gap – On Street (ECS) Cars and Two wheelers

<table>
<thead>
<tr>
<th>Zones</th>
<th>Parking Demand (1)</th>
<th>Parking Supply (2)</th>
<th>Gap (1) - (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>373</td>
<td>234</td>
<td>139</td>
</tr>
<tr>
<td>Zone 2</td>
<td>95</td>
<td>44</td>
<td>51</td>
</tr>
<tr>
<td>Zone 3</td>
<td>55</td>
<td>49</td>
<td>6</td>
</tr>
<tr>
<td>Zone 4</td>
<td>47</td>
<td>49</td>
<td>-2</td>
</tr>
<tr>
<td>Ambedkar Road</td>
<td>70</td>
<td>116</td>
<td>-46</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>640</strong></td>
<td><strong>491</strong></td>
<td><strong>149</strong></td>
</tr>
</tbody>
</table>
Figure 2-2: Average Hourly variation of parking Accumulation in zone 1

Figure 2-3: Average Hourly variation of parking Accumulation in zone 2
Figure 2-4: Average Hourly variation of parking Accumulation in zone 3

Figure 2-5: Average Hourly variation of parking Accumulation in zone 4

Figure 2-6: Average Hourly variation of parking Accumulation at Ambedkar Veedhi
3. **Survey Findings**

40. *Trip Purpose.* Analysis on purpose of trip revealed that work trips are more with 87% (Car users) and 83% (Non-car users) followed by shopping trips about 6% (Car users) and 9% (Non-car users). The details of journey purpose are presented in Table 2-8.

<table>
<thead>
<tr>
<th>Trip Purpose</th>
<th>Car Users (%)</th>
<th>Non-Car Users</th>
</tr>
</thead>
<tbody>
<tr>
<td>Work</td>
<td>87%</td>
<td>83%</td>
</tr>
<tr>
<td>Shopping</td>
<td>2%</td>
<td>6%</td>
</tr>
<tr>
<td>Leisure</td>
<td>6%</td>
<td>9%</td>
</tr>
<tr>
<td>Others</td>
<td>5%</td>
<td>2%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100%</strong></td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

41. *Trip Frequency.* Analysis of trip frequency shows that daily trips are more (40% Car users & 22% Non-car users) followed by weekly trips (22% Car users & 35% Non-car users) and occasional trips (23% Car users & 30% Non-car users). Based on the samples collected (about 300 samples) the trip frequency distribution of the survey is presented in Table 2-9.

<table>
<thead>
<tr>
<th>Trip Purpose</th>
<th>Car Users (%)</th>
<th>Non-Car Users</th>
</tr>
</thead>
<tbody>
<tr>
<td>Daily</td>
<td>89%</td>
<td>86%</td>
</tr>
<tr>
<td>Weekly</td>
<td>0%</td>
<td>12%</td>
</tr>
<tr>
<td>Occasionally</td>
<td>11%</td>
<td>0%</td>
</tr>
<tr>
<td>Others</td>
<td>0%</td>
<td>2%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100%</strong></td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

42. *Willingness to Pay.* The survey also included the willingness to pay survey of the likely users. The survey indicates that users are willing to pay a charge in the range of Rs. 10 – 25 for a closed and secured parking.

E. **Parking Demand Forecast**

1. *Parking Demand Forecast*

43. The present Parking demand is projected to 2030. The growth rate considered for projecting the future parking demand is 6% which is taken in with respect to the average vehicular growth in the region. The details are shown in Table 2-10.
Table 2-10: Parking demand is projected to 2030

<table>
<thead>
<tr>
<th>Base year Parking Demand</th>
<th>Projected Parking Demand (No. of vehicles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base year 2010</td>
<td>650</td>
</tr>
<tr>
<td>2010 – 2020</td>
<td>723</td>
</tr>
<tr>
<td>2020 – 2025</td>
<td>805</td>
</tr>
<tr>
<td>2025 – 2030</td>
<td>896</td>
</tr>
</tbody>
</table>

Notes:

(i) Econometric modeling is used to derive the Growth Factor. To obtain the Growth Factor we consider the data related to Population, Per Capita Income (PCI), Net State Domestic Product (NSDP) and Gross Domestic Product (GDP).

(ii) The influence area of the study includes the state of Uttar Pradesh and Delhi.

(iii) An econometric model measures past relationships among various variables and then tries to forecast how changes in some variables will affect the future course of others.

(iv) Formula Recommended by IRC (108 – 1996) is:

\[
\log_e P = A_0 + A_1 \log_e GDP + A_2 \log_e NSDP + A_3 \log_e Population + A_4 \log_e PCI
\]

Where,

- \( P \) = Traffic Volume
- \( A_0 \) = Regression Constant
- \( A_1, A_2, A_3, A_4 \) are the Elasticity Coefficients

(v) The time series data of traffic at the study area and the corresponding data on GDP, NSDP, Population and PCI are tabulated.

(vi) Multiple Regression Analysis is done to arrive at the following equation

\[
\log_e P = A_0 + A_1 \log_e GDP + A_2 \log_e NSDP + A_3 \log_e Population + A_4 \log_e PCI
\]

The values of \( A_1, A_2, A_3, A_4 \) are found

(viii) Growth rate of traffic = (\( A_1 \) * Expected Growth rate of GDP) + (\( A_2 \) * Expected Growth rate of NSDP)

(\( A_3 \) * Expected Growth rate of Population) + (\( A_4 \) * Expected Growth rate of PCI)

The growth of the traffic is projected with the obtained growth factors. The growth rates obtained are

- 6.5 For the period from 2009 to 2020
- 6.0 For the period from 2020 to 2025
- 5.7 For the period from 2025 to 2030

(ix) The reason behind the variation of growth factor periodically is because of the predicted periodic changes in factors considered in the regression equation.

2. Space Requirement

44. As per the parking demand forecasted for the year 2030 at 5% nominal increase taken with parallel to the traffic growth in the region, the current parking demand stands at 640 – 650 PCU’s. Based on the present demand, the future parking demand is projected to be around 900 PCU’s.
45. Considering the scenario to be 80% parking for cars and 20% for two wheelers, approximate space required for car parking would be about 9000 square meters and 540 Square meters for two wheeler parking. Besides additional spaces to be provided for drive ways, columns, off sets, stairs, Lifts etc.

F. Recommendations

46. The parking analysis clearly established the need for constructing a multi-level parking lot in the vicinity of GDA in the CBD area. The best option available is to make use of the site that is currently functioning as the old bus stand. Since the Master Plan for Ghaziabad has already identified a new site for the future bus stand, it has been recommended to shift the existing old bus stand to the new location on Loni Road as identified in the Master Plan. In the event of this, the old bus stand can be used for building the multi-storey parking lot. Total area of the site is 10,040 sq m.

G. Description of the proposed parking site (Old Bus Stand)

1. Access roads to site

47. The main access to the site under consideration (Old Bus Stand) is from Dr. Ambedkar Road. There are other cross roads running perpendicular to Dr. Ambedkar Road which also connect the Old Bus Stand. One of the cross roads is the service lane below the flyover on the NH-24 /Hapur Road. Dr. Ambedkar road is a four lane divided roadway and the cross road is a two lane undivided roadway as shown in Figure 2-7. The photographs presented below also show the existing scenario in the study area.

Figure 2-7: Road network around the proposed Parking Lot
Photographs

**Photo 1**: Entry/Exit Gate of Bus Stand

**Photo 2**: View of Bus Stand

**Photo 3**: Ambedkar Circle near Gate 1

**Photo 4**: Service Lane near Gate 3
48. *Entry and Exist Details.* At present, the Old Bus Stand has three main entry gates - G1, G2 and G3 as shown in Figure 2-8. The impact of traffic is expected to be more from Dr Ambedkar Road since Gate 1 and Gate 2 are positioned on Dr Ambedkar Road. The major problem that will affect G1 and G3 is that the intersections are close to the gates which may affect the traffic on that particular stretch of road. Gate 3 does not have a problem at present, since it only serves for the entry and exit for the two wheeler parking inside the bus stand.

Figure 2-8: Existing Entry and Exist Details at Old Bus Stand
H. Proposed Multistory Parking Facility & Traffic Considerations

49. The proposed multistory parking facility at the old bus stand accommodates for business and commercial center in the ground floor and parking facility for two wheelers; cars and two wheelers on the first floor, and the second and third floors exclusively for car parking only. Depending on the demand, the terrace (fourth level) can also be used for open car parking.

50. Once the parking facility is developed and parking facility is given to accommodate a number of vehicles in the small place, the impact of this parking facility on the adjacent roads needs to be studied. There will change in the existing flow patterns and will affect the existing traffic density and the roadway capacities.

51. To ascertain the impact, turning movement counts (8 hours) were carried out at three intersections of the roads like Ambedkar Road/ GT Road, Dr. Shyam Prasad Mukharjee Road, Maliwara Road/ Dasana Marg - through which the traffic will enter or leave the existing bus terminal. These junctions include:

(i) Bus Stand Junction (Ambedkar Road / Hapur Road (NH 24) )
(ii) Maliwara Junction (Ambedkar Road / Dasna Marg )
(iii) Chowdary Junction (Ambedkar Road / G T Road (NH-91) )

52. From the turning volume counts, the actual PCUs on the stretch of roadways leading to the bus terminal from these junctions are calculated. It is important to note that because of the shifting of the existing bus terminal, no bus traffic will come on these roads in the future, and also, there will be a significant amount of reduction in the other categories of vehicles on the road network in the vicinity. Factoring these in to projecting the future traffic on the roads leading to the proposed parking facility, it can be stated that the new parking lot will not significantly impact the roadway network capacity in the vicinity of GDA.
3. PLANNING & DESIGN OF MULTI-LEVEL PARKING FACILITY

A. General

53. Need and objective of the project has been explained in the Chapter on parking demand of the region. The location selected is the old bus stand next to Ghaziabad Development Authority. A reconnaissance survey was carried out to gather basic information about the site, type of area like commercial or residential, climate etc. from different sources. Primary and secondary data available were also collected for further studies.

B. Surveys & Investigations

54. The following site surveys were carried out for the finalization of the structure:

- Location Survey
- Topographic Survey
- Traffic surveys

55. Due to busy activities in the existing Bus terminal the Consultant was not able to carry out geotechnical investigations at the site. Hence the geotechnical details taken for the proposed Bus Terminal building is considered for the design of Multilevel Car Parking. The lowest value of SBC at a depth of 3m given in the area of Bus Terminal is 225 KN/m2 and this value is considered for design. It is also recommended to take adequate number of confirmatory bore holes during execution. This item is included in the cost estimates.

1. Topographical Survey

56. The basic objective of the topographical survey was to collect the essential ground features of the area using Total Station so as to develop a Digital Terrain Model (DTM), to take care of design requirements. The data collected will result in the final design and is also used for the computation of earthwork and other quantities required.

57. As first step of the field study, satellite imagery maps of the location were collected and examined thoroughly to have first hand information about the area and to decide on the possible improvement options. It also helped out in finalizing the extent of topographical survey.

58. Spot levels were taken along the proposed area at regular intervals to understand the ground variation. The utility services present along the existing area were also plotted. Topographic survey was carried out using Total Station of 5-sec accuracy for detailed mapping and with higher accuracy total station during the traversing (min 3 sec). The
existing features surveyed were directly imported into Computer Aided Software and the details of the same has been plotted and presented for ready reference.

59. In order to prepare the plan of the Multi Level parking building the following technical factors were taken into consideration:

- Land use requirement for various activities
- Planning norms and regulations
- Topographical and geotechnical factors such as ground features and slope, type of soil, ground water level etc.
- Standards for provision of parking requirement
- Traffic growth trend and future demand
- Seismic zone and wind direction
- Safety and security

C. Planning Considerations

60. The site earmarked for the proposed construction of the multi level parking facility is located in a busy area with major very congested roads (NH 24 and Dr. Ambedkar road on two sides and a cross road less hectic on the third side. The site has rectangular shape with an area of 10,040 sq.m. The built-up area comes to 8,569 sq.m leaving the mandatory minimum setback distances specified in the National building code of India. Norms have been followed and safety measures taken in the parking spaces as well as ramps. The building is a four storied framed structure with commercial space in the ground floor and parking facilities in the three floors and roof above. The demand is to park 650 numbers of vehicles in the base year itself which will increase to 900 numbers within the next 20 years. 80% of parking area is provided for cars and 20% for two wheelers. Ramps on slope 1 in 10 are provided for the entry and exit of vehicles to and from different levels. The structure has two lifts and a staircase for the use of customers utilizing the facility, located at the centre of the building. The building shall be covered with walls only in the ground floor. Other floors are provided with 1m high parapets in the outer periphery. Fire fighting system is not proposed as the building is kept open.

61. General Approach. The entry of vehicles to the parking building is proposed through the cross road and exit to NH 24. Unidirectional flow of vehicles is maintained inside the structure. The circulation pattern shall be guided by proper signage system. The movement of vehicles inside the building is channelized through driveways laid down between the parking bays.

62. Separate parking bays are allocated for two and four wheelers. Considering the predominant business surrounding of the area, ground floor is set aside for commercial purpose. Brick walls are proposed along the exterior of the ground floor with partition inside. Typical floor plans are presented in Figure 3-1 and Figure 3-2. Detailed floor plans, sections and elevations are presented in Appendix 1.
<table>
<thead>
<tr>
<th>S. No.</th>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Site area</td>
<td>Sq. m</td>
<td>10,040</td>
</tr>
<tr>
<td>2</td>
<td>Area (Ground Floor)</td>
<td>Sq. m</td>
<td>8,323</td>
</tr>
<tr>
<td>3</td>
<td>Area (1st, 2nd and 3rd Floors)</td>
<td>Sq. m</td>
<td>25,833</td>
</tr>
<tr>
<td>3</td>
<td>Commercial/Retail (Ground Floor)</td>
<td>Sq. m</td>
<td>5,000</td>
</tr>
<tr>
<td>4</td>
<td>Parking (Ground Floor) – 2-wheelers</td>
<td>No. s</td>
<td>117</td>
</tr>
<tr>
<td>5</td>
<td>Parking (1st floor) – 2-wheelers</td>
<td>No. s</td>
<td>213</td>
</tr>
<tr>
<td>6</td>
<td>Parking (1st, 2nd and 3rd Floors) – Cars</td>
<td>No. s</td>
<td>777</td>
</tr>
</tbody>
</table>
Figure 3-1: Ground Floor Plan of the Proposed Multilevel Parking
**Figure 3-2**: Typical Floor Plan of the Proposed Multilevel Parking Facility
D. **Structural System**

63. The proposed building is 80.46m long and 106.5m wide with a plinth area of 8569 sq.m for each floor. The floor height shall be 3.6m. A combination of column beam arrangement is proposed for the building. Large column spacing of 10.8m is adopted along the breadth and 8.4m is given lengthwise to facilitate easy vehicle movement. Considering the larger size of slab panels, grid beam arrangement is proposed for floors and roof. Ordinary beam slab arrangement is adopted for ramps. Mild condition of exposure is considered in design. Isolated, combined and raft foundations are the different types of foundations adopted. The minimum depth of foundation shall generally be 2.5 m below ground.

64. Salient features of the building are:

- Length 80.46m
- Breadth 106.50m
- Column spacing (along the length): varies from 8.4m to 4.96m.
- Column spacing (along the breadth): 10.8m
- Column spacing for ramp (along the length): 5.345m
- Column spacing for ramp (along the breadth): varies from 8.36m to 5.46m.
- Plinth area: 8569 sq.m

65. **Design criterion.**

- Exposure Condition - Mild (as per IS 456 – Table Clause 8.2.2.1 & 5.3.2)
- Grade of Concrete – M30 (as per IS 456 – Table 5 Clause 6.1.2, 8.2.4.1 & 9.1.2)
- Reinforcing Steel - Fe 415 conforming to IS 1786.
- Safe Bearing Capacity of the soil considered – 225 KN/m2
- Depth of foundation – 2.5m below ground

66. **Design codes and standards.**

67. The structural design is carried out as per the latest versions of Indian Standard codes published by Bureau of Indian Standards. Various design codes and standards referred are:

- IS 456 for Plain and Reinforced Concrete.
- IS 875 Part 1,2,3 & 5 for dead load, live load, wind load and combinations
- SP 34 for detailing of reinforcement

68. Ghaziabad being in seismic zone IV, the earthquake resistant design became mandatory. The codes followed are:
– IS 1893 Part I for earthquake resistant design and
– IS 13920 for ductile detailing of reinforced concrete subjected to seismic forces.

69. **Loads considered.**

(i) Self Weight of members
(ii) Wall Load
(iii) Slab Live Load (3kN/m² as per IS 875 Part II)
(iv) Stair/Lift/Ramp Load
(v) Load due to Wind

70. For wind load the four Cases considered are:

– Wind force acting in X direction
– Wind acting in -X direction
– Wind force acting in Z direction
– Wind acting in -Z direction

71. **Wind Load Analysis.** General load combinations considered in the design are: (as per IS 456 – Table 18 Clause 18.2.3.1, 36.4 & B-4.3)

- 1.5 * (DL+WX)
- 1.5 * (DL-WX)
- 1.5 * (DL+WZ)
- 1.5 * (DL-WZ)
- 1.2 * (DL+LL+WX)
- 1.2 * (DL+LL-WX)
- 1.2 * (DL+LL+WZ)
- 1.2 * (DL+LL-WZ)
- 0.9 * DL+ 1.5 * WX
- 0.9 * DL - 1.5 * WX
- 0.9 * DL+ 1.5 * WZ
- 0.9 * DL - 1.5 * WZ

72. **Load due to Earthquake.** The two cases considered are: (i) force acting in X direction, and (ii) force acting in Z direction: Load combinations considered are:

- 1.5 * (DL+LL)
- 1.5 * (DL+EQX)
- 1.5 * (DL-EQX)
- 1.5 * (DL+EQZ)
- 1.5 * (DL-EQZ)
73. Following densities and load values are considered for design:

(i) Density of Reinforced concrete: 24 kN/m³
(ii) Density of brick masonry : 18.85 kN/m³
(iii) Density of earth : 18 kN/m³
(iv) Superimposed Live Load : 4 kN/m²
(v) Floor Finishes : 1 kN/m²

74. Data for wind load design.

(i) Basic wind speed – Ghaziabad 47 m/sec (Appendix A Clause 5.2)
(ii) Wind Intensity – 1.73 kN/m²

75. Criteria for Earthquake Resistant Design of Structures. (IS 1893-2002) Clause 6.3.1.2
Partial safety factors for limit state design of reinforced concrete and prestressed concrete structures.

76. In the limit state design of reinforced concrete structures, the following load combinations are to be accounted for:

(i) 1.5(DL+IL)
(ii) 1.2(DL+IL±EL)
(iii) 1.5(DL±EL)
(iv) 0.9DL±1.5EL

77. Factors Considered for Earth Quake Analysis.

– Ghaziabad is Located in Zone IV
– Zone Factor : 0.24
– Importance Factor : 1.5
– Response Reduction Factor : 3.0
– Rock & Soil Site Factor : 1.0
– Damping Ratio : 0.5
– Suitable increase in SBC is considered as per IS 1893-2002

Ref: [Table1 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils (clause6.3.5.2)]
For Medium soil - Percentage of Permissible Increase is 25% for isolated RCC footing without tie beams, or unreinforced strip foundations.

78. **Clear cover to reinforcement.** The following clear cover to the outer reinforcement shall be adopted:

   – For Foundation : 50 mm.
   – For Beams : 30 mm.
   – For Slabs : 20mm.
   – For columns : 40 mm.

79. The framed system is analyzed as a 3D structure using STAAD Pro 2007. The member forces and moments from the STAAD output are taken for the design. The beams are designed as singly reinforced as well as doubly reinforced depending upon the requirement. The columns are designed as square or rectangular in shape. The slabs supported by beams and columns are designed using the method specified in Annexure D of IS 456:2000 and the grid slab is designed as normal practice. The various structural elements are designed for the worst combination of loads.

**E. Analysis of the Multilevel Parking**

80. Multilevel Parking has a plinth area 8569 sq.m with length of 80.46 and width of 106.5m. Ramp will have a width of 10.69m with varying length. STAAD Pro 2007 is used for the modeling of the structure. For the accuracy of results the whole structure has been split into number of units & modeled separately such as ramp portion, lift & staircase & also for different panels within the structure.

81. To take care of temperature stresses in slab an expansion joint of 20 mm is provided along the width of the structure. Two expansion joints are provided forming a total of three sections of 35.587m, 32.045m & 37.693m.

82. Based on the axial load following types of footings are designed:

   - Raft foundation of 14.5m x 8.5 for lift & staircase portion
   - Isolated footings of 6 different sizes.
   - Combined Footings of 10 different sizes.

83. Details are given in the structural drawings. All footings shall have a minimum depth of
2.5m from ground level based on the bearing capacity of the soil.

84. Columns were designed for biaxial bending considering axial Load & moments in X & Y directions. There are about 10 different types of columns within the structure considering span & load.

85. Slab of structure is designed for traffic load of 3kN/m² (as per IS 875 Part II) with additional 25% is taken as impact load. Therefore overall super imposed load is taken as 4kN/m² considering overall safety of the structure. The slab is designed as grid slab to give more stability & to enhance serviceability. Whereas the slab in the ramp portion is designed as ordinary slab & beam arrangement to facilitate the slope of the ramp.

86. Seismic Analysis. Static Equivalent Method is used for the seismic analysis utilizing the rules of IS: 1893(part 1) – 2002.

87. Methodology. In seismic load generation using a static equivalent approach, encompassed in code IS 1893, the weights in the structure are specified. There are three methods for specifying the weights: self weight, joint weight and member weight. Weights, which could be treated as being lumped at a node, could be assigned using Joint Weight the same has been used during this analysis.

88. The joint loads at all the nodes are obtained from the initial analysis by assuming pinned supports at all the beam column joints. These loads are applied as weight for the seismic analysis.

89. Analysis of this system for all the loads/load combinations is carried out. Please refer following appendices for detailed structural analysis and drawings.

Appendix 2: STAAD Model
Appendix 3: STAAD Input File
Appendix 4: Design of Footing
Appendix 5: Design of Columns
Appendix 6: Design of Beams
Appendix 7: Design of Slab
Appendix 8: Structural Drawings
4. COST ESTIMATES

A. Rate Analysis

90. The unit rates shall be arrived by considering the basic rates, lead distances, man power, machinery, and materials. The unit rate for every individual item is arrived based on Uttar Pradesh Lok Nirman Vibhag (UP Public Works Department), Schedule of Rates for Ghaziabad District 2008 and Central Public Works Department Delhi, Schedule of rates 2007. For items of work with no rates specified in the schedule of rates, market rates are obtained and used.

B. Bill of Quantities & Cost Estimates

91. Total item wise quantities are calculated as per the detailed drawings. Separate heads for all different items of work is included in the BOQ. The major work items considered are:

- Earth work excavation
- Concrete
  - PCC leveling Course
  - Reinforced Cement concrete M30
    - Foundation
    - For walls, columns, beams, slab etc
- Steel
  - Reinforcement
    - Foundation
    - For Walls, columns, beams, slab etc
- Electrical cost
- Miscellaneous Items
  - caution/warning Signs, expansion joints, and etc.,
  - Painting, white washing, finishes and etc.

92. The total based cost of this Multi-level Parking Project works out to be INR 366.8 million. Bill of quantities and detailed quantities and estimates are presented in Appendix 9.
STAAD MODEL
Multi Level Car Parking Lift & Stair Portion

MODEL
Multi Level Car Parking Central Portion with Column Spacing of 10.8 x 8.4
Raft Slab
Multi Level Car Parking Ramp Portion
STAAD INPUT FILE
### STAAD INPUT FILE

**STAAD SPACE**

**START JOB INFORMATION**

**ENGINEER DATE** 09-Dec-09

**END JOB INFORMATION**

**INPUT WIDTH** 79

**UNIT METER KN**

**JOINT COORDINATES**

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FINISH
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ALPHA 1e-005
DAMP 0.05
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393 TO 399 407 500 TO 503 512 TO 518 PRIS YD 0.4 ZD 0.23
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216 TO 265 310 312 313 315 317 324 326 328 330 332 TO 382 427 429 430 432 -
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681 683 685 TO 696 701 703 705 707 709 711 713 715 717 TO 737 739 741 743 -
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1129 1131 1133 1135 PRIS AX 0.455 IX 0.0164 IY 0.0001 IZ 0.0316
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53 WEIGHT -101.018
54 WEIGHT 306.482
55 WEIGHT -101.018
56 WEIGHT 1154.33
57 WEIGHT 1154.33
58 WEIGHT 63.701
59 WEIGHT 873.03
60 WEIGHT 873.03
61 WEIGHT 289.386
62 WEIGHT 289.386
63 WEIGHT 289.386
64 WEIGHT 289.386
65 WEIGHT 11.869
66 WEIGHT 52.549
67 WEIGHT 11.869
68 WEIGHT -97.903
69 WEIGHT 303.566
70 WEIGHT 870.451
71 WEIGHT 870.451
72 WEIGHT 289.829
73 WEIGHT 289.829
74 WEIGHT 10.994
75 WEIGHT 52.316
76 WEIGHT 10.994
77 WEIGHT -101.018
78 WEIGHT 306.482
79 WEIGHT -101.018
80 WEIGHT 1154.33
81 WEIGHT 1154.33
82 WEIGHT 63.701
83 WEIGHT 873.03
84 WEIGHT 873.03
85 WEIGHT 289.386
86 WEIGHT 289.386
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89 WEIGHT 11.869
90 WEIGHT 52.549
91 WEIGHT 11.869
92 WEIGHT -97.903
93 WEIGHT 303.566
94 WEIGHT 870.451
95 WEIGHT 870.451
96 WEIGHT 289.829
97 WEIGHT 289.829
98 WEIGHT 10.994
99 WEIGHT -101.018
100 WEIGHT 306.482
101 WEIGHT -101.018
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103 WEIGHT 1154.33
104 WEIGHT 63.701
151 WEIGHT 873.03
152 WEIGHT 873.03
153 WEIGHT 289.386
154 WEIGHT 289.386
155 WEIGHT 289.386
156 WEIGHT 289.386
157 WEIGHT 11.869
158 WEIGHT 52.549
159 WEIGHT 11.869
160 WEIGHT -97.903
161 WEIGHT 303.566
LOAD 1 LOADTYPE Seismic  TITLE EQX
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LOAD 2 LOADTYPE Seismic  TITLE EQZ
1893 LOAD Z 1
LOAD 3 LOADTYPE Wind  TITLE WIND X
WIND LOAD X 1 TYPE 1 XR 0 32.4 YR 1 15.5 ZR 0 10.8 OPEN
LOAD 4 LOADTYPE Wind  TITLE WIND Z
WIND LOAD Z 1 TYPE 1 XR 0 25.2 YR 1 15.5 OPEN
LOAD 5 LOADTYPE Wind  TITLE WIND -X
WIND LOAD -X -1 TYPE 1 XR 0 32.4 YR 1 16.5 OPEN
LOAD 6 LOADTYPE Wind  TITLE WIND -Z
WIND LOAD -Z -1 TYPE 1 XR 0 25.2 YR 1 16.5 OPEN
LOAD 7 LOADTYPE None  TITLE SW
SELFWEIGHT Y -1
LOAD 8 LOADTYPE None  TITLE WL
MEMBER LOAD
LOAD 9 LOADTYPE None TITLE SLAB DL + FLOOR FINISH
FLOOR LOAD
YRANGE 4.6 4.7 FLOAD -3.5 XRANGE 0 1.4 ZRANGE 7.65 9.25 GY
YRANGE 4.6 4.7 FLOAD -3.5 XRANGE 7 8.4 ZRANGE 7.65 9.25 GY
YRANGE 8.2 8.3 FLOAD -3.5 XRANGE 7 8.4 ZRANGE 7.65 9.25 GY
YRANGE 11.8 11.9 FLOAD -3.5 XRANGE 0 1.4 ZRANGE 7.65 9.25 GY
YRANGE 11.8 11.9 FLOAD -3.5 XRANGE 7 8.4 ZRANGE 7.65 9.25 GY
YRANGE 15.4 15.5 FLOAD -3.5 XRANGE 0 1.4 ZRANGE 7.65 9.25 GY
YRANGE 15.4 15.5 FLOAD -3.5 XRANGE 7 8.4 ZRANGE 7.65 9.25 GY
YRANGE 17.9 18 FLOAD -4 XRANGE 2.35 6.05 ZRANGE 7.65 9.25 GY
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MEMBER LOAD
JOINT LOAD
273 TO 278 FY -0.5
LOAD 11 LOADTYPE None TITLE SLAB LL
FLOOR LOAD
YRANGE 4.6 4.7 FLOAD -3 XRANGE -8.4 16.8 ZRANGE 0 10.8 GY
YRANGE 8.2 8.3 FLOAD -3 XRANGE -8.4 16.8 ZRANGE 0 10.8 GY
YRANGE 11.8 11.9 FLOAD -3 XRANGE -8.4 16.8 ZRANGE 0 10.8 GY
YRANGE 15.4 15.5 FLOAD -3 XRANGE -8.4 16.8 ZRANGE 0 10.8 GY
YRANGE 17.9 18.1 FLOAD -1.5 GY
LOAD COMB 12 1.5 * (DL+LL)
7 1.5 8 1.5 9 1.5 10 1.5 11 1.5
LOAD COMB 13 1.5 * (DL+EQX)
7 1.5 8 1.5 9 1.5 10 1.5 11 1.5
LOAD COMB 14 1.5 * (DL-EQX)
7 1.5 8 1.5 9 1.5 10 1.5 11 1.5
LOAD COMB 15 1.5 * (DL+EQZ)
7 1.5 8 1.5 9 1.5 10 1.5 2 1.5
LOAD COMB 16 1.5 * (DL-EQZ)
7 1.5 8 1.5 9 1.5 10 1.5 2 -1.5
LOAD COMB 17 1.2 * (DL+LL+EQX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 1 1.2
LOAD COMB 18 1.2 * (DL+LL-EQX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 1 -1.2
LOAD COMB 19 1.2 * (DL+LL+EQZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 2 1.2
LOAD COMB 20 1.2 * (DL+LL-EQZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 2 -1.2
LOAD COMB 21 0.9 * (DL + 1.5 * EQX)
7 0.9 8 0.9 9 0.9 10 0.9 1 1.5
LOAD COMB 22 0.9 * (DL - 1.5 * EQX)
7 0.9 8 0.9 9 0.9 10 0.9 1 -1.5
LOAD COMB 23 0.9 * (DL+ 1.5 * EQZ)
7 0.9 8 0.9 9 0.9 10 0.9 2 1.5
LOAD COMB 24 0.9 * (DL - 1.5 * EQZ)
7 0.9 8 0.9 9 0.9 10 0.9 2 -1.5
LOAD COMB 25 1.5 * (DL+WX)
7 1.5 8 1.5 9 1.5 10 1.5 3 1.5
LOAD COMB 26 1.5 * (DL-WX)
7 1.5 8 1.5 9 1.5 10 1.5 5 -1.5
LOAD COMB 27 1.5 * (DL+WZ)
7 1.5 8 1.5 9 1.5 10 1.5 4 1.5
LOAD COMB 28 1.5 * (DL-WZ)
7 1.5 8 1.5 9 1.5 10 1.5 6 -1.5
LOAD COMB 29 1.2 * (DL+LL+WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 3 1.2
LOAD COMB 30 1.2 * (DL+LL-WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 5 -1.2
LOAD COMB 31 1.2 * (DL+LL+WZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 4 1.2
LOAD COMB 32 1.2 * (DL+LL-WZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 6 -1.2
LOAD COMB 33 0.9 * DL+ 1.5 * WX
7 0.9 8 0.9 9 0.9 10 0.9 3 1.5
LOAD COMB 34 0.9 * DL - 1.5 * WX
7 0.9 8 0.9 9 0.9 10 0.9 5 -1.5
LOAD COMB 35 0.9 * DL+ 1.5 * WZ
7 0.9 8 0.9 9 0.9 10 0.9 4 1.5
LOAD COMB 36 0.9 * DL - 1.5 * WZ
7 0.9 8 0.9 9 0.9 10 0.9 6 -1.5
PERFORM ANALYSIS PRINT ALL
FINISH
## STAAD INPUT FILE

**STAAD SPACE**

**START JOB INFORMATION**

**ENGINEER DATE**: 24-Dec-09

**END JOB INFORMATION**

**INPUT WIDTH**: 79

**UNIT METER KN**

### JOINT COORDINATES

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### JOINT COORDINATES (UNIT METER KN)

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<th>Z (m)</th>
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POISSON 0.17
DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
2 5 6 8 9 32 TO 34 52 TO 54 59 61 62 64 66 109 TO 111 116 118 119 121 123 - 166 TO 168 173 175 176 178 180 223 TO 225 230 232 233 235 - 237 PRIS YD 0.3 ZD 0.2
35 184 171; 354 42 185; 355 185 186; 356 186 170; 357 181 185; 358 185 182; 359 183 186; 360 186 184; 361 64 187; 362 69 188; 363 65 189; 364 187 190; 365 190 172; 366 188 191; 367 191 173; 368 189 192; 369 192 174; 370 187 188; 371 188 189; 372 190 191; 373 191 192; 374 118 193; 375 123 194; 376 119 195; 377 119 196; 378 116 197; 379 119 197; 380 197 179; 381 195 198; 382 198 180; 383 193 194; 384 194 195; 385 196 197; 386 197 198; 387 91 199; 388 96 200; 389 92 201; 390 199 202; 391 202 175; 392 200 203; 393 203 176; 394 201 204; 395 204 177; 396 199 200; 397 200 201; 398 202 203; 399 203 204;

DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 2.17185e+007
POISSON 0.17
DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
3 60 117 174 231 PRIS YD 0.5 ZD 0.35
MEMBER PROPERTY AMERICAN
321 322 327 328 332 333 337 338 342 343 PRIS YD 0.3 ZD 0.4
MEMBER PROPERTY AMERICAN
11 12 56 58 113 115 170 172 227 229 PRIS YD 0.3 ZD 0.8
37 45 94 102 151 159 208 216 PRIS YD 0.8 ZD 0.3
CONSTANTS
MATERIAL CONCRETE ALL
SUPPORTS
1 2 5 6 9 10 13 14 27 TO 29 165 166 FIXED
DEFINE WIND LOAD
TYPE 1
INT 1.73 1.73 1.73 1.73 1.73 1.73 1.73 HEIG 1 4.6 8.2 11.8 15.4
EXP 1 JOINT 3 7 11 15 31 33 35 37 46 49 58 60 62 64 73 76 85 87 89 91 100 -
103 112 114 116 118 127 130 163 169 172 175 178
DEFINE 1893 LOAD
ZONE 0.24 RF 3 I 1.5 SS 1 DM 0.05 DT 2.5
JOINT WEIGHT
1 WEIGHT 7.952
2 WEIGHT 7.952
3 WEIGHT 53.408
4 WEIGHT 53.408
5 WEIGHT 7.952
6 WEIGHT 7.952
7 WEIGHT 53.194
8 WEIGHT 53.194
9 13 165 WEIGHT 7.952
10 14 166 WEIGHT 7.952
11 15 163 WEIGHT 47.303
12 16 167 WEIGHT 47.303
13 165 WEIGHT 7.952
14 166 WEIGHT 7.952
15 163 WEIGHT 45.876
16 167 WEIGHT 45.876
17 WEIGHT 66.125
18 WEIGHT 51.662
19 20 164 WEIGHT 48.306
20 164 WEIGHT 59.869
27 WEIGHT 7.952
28 WEIGHT 7.952
29 WEIGHT 7.952
31 WEIGHT 252.101
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33 WEIGHT 376.928
34 WEIGHT 376.928
35 37 169 WEIGHT 291.626
36 38 171 WEIGHT 291.626
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40 WEIGHT 758.125
41 42 170 WEIGHT 612.534
42 170 WEIGHT 358.009
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LOAD 1 LOADTYPE Seismic TITLE EQX
1893 LOAD X 1
LOAD 2 LOADTYPE Seismic TITLE EQZ
1893 LOAD Z 1
LOAD 3 LOADTYPE Wind TITLE WIND X
WIND LOAD X -1 TYPE 1 XR 0 32.4 YR 16.5 OPEN
LOAD 4 LOADTYPE Wind TITLE WIND Z
WIND LOAD Z -1 TYPE 1 XR 0 25.2 YR 3.3 16.5 OPEN
LOAD 5 LOADTYPE Wind TITLE WIND -X
WIND LOAD -X -1 TYPE 1 XR 0 32.4 YR 3.3 16.5 OPEN
LOAD 6 LOADTYPE Wind TITLE WIND -Z
WIND LOAD -Z -1 TYPE 1 XR 0 25.2 YR 3.3 16.5 OPEN
LOAD 7 LOADTYPE None TITLE SW
SELFWEIGHT Y -1
LOAD 8 LOADTYPE None TITLE WALL
MEMBER LOAD
36 TO 41 44 TO 47 67 TO 69 78 82 93 TO 98 101 TO 104 124 TO 126 135 136 -
139 150 TO 155 158 TO 161 181 TO 183 192 193 196 207 TO 212 215 TO 218 238 -
239 TO 240 249 250 253 325 326 330 331 335 336 340 341 UNI GY -5
LOAD 9 LOADTYPE None TITLE SLAB DL
FLOOR LOAD
YRANGE 4.6 4.8 FLOAD -4.8 GY
YRANGE 8.2 8.3 FLOAD -4.8 GY
YRANGE 11.8 11.9 FLOAD -4.8 GY
YRANGE 15.4 15.5 FLOAD -4.8 GY
LOAD 10 LOADTYPE None TITLE FLOOR FINISH
FLOOR LOAD
YRANGE 4.6 4.8 FLOAD -1 GY
YRANGE 8.2 8.3 FLOAD -1 GY
YRANGE 11.8 11.9 FLOAD -1 GY
YRANGE 15.4 15.5 FLOAD -1 GY
LOAD 11 LOADTYPE None TITLE SLAB LL
FLOOR LOAD
YRANGE 4.6 4.8 FLOAD -4 GY
YRANGE 8.2 8.3 FLOAD -4 GY
YRANGE 11.8 11.9 FLOAD -4 GY
YRANGE 15.4 15.5 FLOAD -4 GY
LOAD COMB 12 1.5 * (DL+LL)
7 1.5 8 1.5 9 1.5 10 1.5 11 1.5
LOAD COMB 13 1.5 * (DL+EQX)
7 1.5 8 1.5 9 1.5 10 1.5 11 1.5
LOAD COMB 14 1.5 * (DL+EQX)
7 1.5 8 1.5 9 1.5 10 1.5 1 1.5
LOAD COMB 15 1.5 * (DL+EQX)
7 1.5 8 1.5 9 1.5 10 1.5 2 1.5
LOAD COMB 16 1.5 * (DL+EQZ)
7 1.5 8 1.5 9 1.5 10 1.5 2 -1.5
LOAD COMB 17 1.2 * (DL+LL+EQX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 1 1.2
LOAD COMB 18 1.2 * (DL+LL-EQX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 1 -1.2
LOAD COMB 19 1.2 * (DL+LL+EQZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 2 1.2
LOAD COMB 20 1.2 * (DL+LL-EQZ)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 1 -1.2
LOAD COMB 21 0.9 * DL * 1.5 * EQX
7 0.9 8 0.9 9 0.9 10 0.9 1 1.5
LOAD COMB 22 0.9 * DL - 1.5 * EQX
7 0.9 8 0.9 9 0.9 10 0.9 1 -1.5
LOAD COMB 23 0.9 * DL+ 1.5 * EQZ
7 0.9 8 0.9 9 0.9 10 0.9 2 1.5
LOAD COMB 24 0.9 * DL - 1.5 * EQZ
7 0.9 8 0.9 9 0.9 10 0.9 2 -1.5
LOAD COMB 25 1.5 * (DL+WX)
7 1.5 8 1.5 9 1.5 10 1.5 3 1.5
LOAD COMB 26 1.5 * (DL-WX)
7 1.5 8 1.5 9 1.5 10 1.5 5 -1.5
LOAD COMB 27 1.5 * (DL+WZ)
7 1.5 8 1.5 9 1.5 10 1.5 4 1.5
LOAD COMB 28 1.5 * (DL+WX)
7 1.5 8 1.5 9 1.5 10 1.5 6 -1.5
LOAD COMB 29 1.2 * (DL+LL+WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 3 1.2
LOAD COMB 30 1.2 * (DL+LL+WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 5 -1.2
LOAD COMB 31 1.2 * (DL+LL+WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 4 1.2
LOAD COMB 32 1.2 * (DL+LL+WX)
7 1.2 8 1.2 9 1.2 10 1.2 11 1.2 6 -1.2
LOAD COMB 33 0.9 * DL+ 1.5 * WX
7 0.9 8 0.9 9 0.9 10 0.9 3 1.5
LOAD COMB 34 0.9 * DL - 1.5 * WX
7 0.9 8 0.9 9 0.9 10 0.9 5 -1.5
LOAD COMB 35 0.9 * DL+ 1.5 * WZ
7 0.9 8 0.9 9 0.9 10 0.9 4 1.5
LOAD COMB 36 0.9 * DL - 1.5 * WZ
7 0.9 8 0.9 9 0.9 10 0.9 6 -1.5
PERFORM ANALYSIS PRINT ALL
FINISH
## Design Summary For Multi Level Parking

### Footing

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<tr>
<th>Footing No</th>
<th>Axial Load</th>
<th>Length</th>
<th>Breadth</th>
<th>d</th>
<th>D</th>
<th>L</th>
<th>Reinforcement</th>
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<td>3400</td>
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<td>20 @ 100 c/c</td>
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<tr>
<td>F3</td>
<td>Pu = 3000</td>
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<td>3300</td>
<td>700</td>
<td>1400</td>
<td>200</td>
<td>20 @ 100 c/c</td>
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<td>Pu = 2800</td>
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<td>660</td>
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### Combined Footing

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<td>870</td>
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<td>-</td>
<td>900</td>
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<td>7600</td>
<td>-</td>
<td>900</td>
<td>12 @ 150 c/c</td>
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<tr>
<td>CF10</td>
<td>Pu = 9600</td>
<td>8000</td>
<td>6400</td>
<td>-</td>
<td>900</td>
<td>12 @ 150 c/c</td>
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</tr>
</tbody>
</table>

### Columns

<table>
<thead>
<tr>
<th>Column No</th>
<th>Load Case</th>
<th>Axial Force</th>
<th>Moment X</th>
<th>Moment Y</th>
<th>Size of column</th>
<th>Reinforcement</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>1.5(DL+EQX)</td>
<td>374.77</td>
<td>233.8</td>
<td>140.15</td>
<td>230 x 230</td>
<td>8 of 20</td>
<td>8 @ 200 c/c</td>
</tr>
<tr>
<td>C1a</td>
<td>1.5(DL+EQX)</td>
<td>1307.33</td>
<td>340.03</td>
<td>769.88</td>
<td>400 x 800</td>
<td>12 of 25</td>
<td>8 @ 200 c/c</td>
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<tr>
<td>C2</td>
<td>1.5(DL+EQX)</td>
<td>925</td>
<td>334</td>
<td>228.15</td>
<td>350 x 800</td>
<td>12 of 25</td>
<td>8 @ 200 c/c</td>
</tr>
<tr>
<td>C3</td>
<td>1.2(DL+LL-EQX)</td>
<td>366.32</td>
<td>169.29</td>
<td>353.51</td>
<td>300 x 600</td>
<td>12 of 25</td>
<td>8 @ 200 c/c</td>
</tr>
<tr>
<td>C4</td>
<td>1.5(DL+EQX)</td>
<td>226.3</td>
<td>138.53</td>
<td>192.01</td>
<td>350 x 500</td>
<td>12 of 25</td>
<td>8 @ 200 c/c</td>
</tr>
<tr>
<td>C5</td>
<td>1.2(DL+LL-EQX)</td>
<td>630.8</td>
<td>59.63</td>
<td>224.86</td>
<td>350 x 600</td>
<td>12 of 25</td>
<td>8 @ 200 c/c</td>
</tr>
<tr>
<td>C6</td>
<td>1.5(DL+EQX)</td>
<td>560.34</td>
<td>563.33</td>
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<tr>
<td>C7</td>
<td>1.5(DL+EQX)</td>
<td>4861.814</td>
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<tr>
<td>C8</td>
<td>1.5(DL+EQX)</td>
<td>895</td>
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<td>8 @ 200 c/c</td>
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<tr>
<td>C9</td>
<td>1.5(DL+EQX)</td>
<td>717.13</td>
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<td>8 @ 200 c/c</td>
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<td>C10</td>
<td>1.5(DL+EQX)</td>
<td>3015.88</td>
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<td>550 x 850</td>
<td>22 of 25</td>
<td>8 @ 200 c/c</td>
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### Plinth Beams

<table>
<thead>
<tr>
<th>Beam No</th>
<th>Load Case</th>
<th>Moments</th>
<th>Shear</th>
<th>B</th>
<th>D</th>
<th>Ast Top</th>
<th>Ast Bottom</th>
<th>Shear Rft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Support</td>
<td>Midspan</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>PB1</td>
<td>1.5(DL-EOX)</td>
<td>380</td>
<td>127</td>
<td>152</td>
<td>151</td>
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### SLAB

<table>
<thead>
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<tbody>
<tr>
<td>GF,FF,SF,TF</td>
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</table>

### GF,FF,SF,TF Roof Beams for 8.36 & 5.5

<table>
<thead>
<tr>
<th>Beam No</th>
<th>Load Case</th>
<th>Support</th>
<th>Midspan</th>
<th>Torsion</th>
<th>B</th>
<th>D</th>
<th>Ast Top</th>
<th>Ast Bottom</th>
<th>Shear Rft</th>
<th>Legs</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>1.5(DL+EQX)</td>
<td>1053</td>
<td>331</td>
<td>150</td>
<td>233</td>
<td>218</td>
<td>400</td>
<td>900</td>
<td>12 of 25</td>
<td>5 of 25</td>
</tr>
<tr>
<td>B-2</td>
<td>1.5(DL+EQX)</td>
<td>2065</td>
<td>855</td>
<td>125</td>
<td>908</td>
<td>884</td>
<td>800</td>
<td>900</td>
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<td>8 of 25</td>
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<td>B-2A</td>
<td>1.5(DL-EQX)</td>
<td>1428</td>
<td>746</td>
<td>163</td>
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<td>184</td>
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<td>8 of 25</td>
</tr>
<tr>
<td>B-3</td>
<td>1.5(DL-EQX)</td>
<td>672</td>
<td>303</td>
<td>11</td>
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<td>47</td>
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<td>5 of 25</td>
</tr>
<tr>
<td>B-4</td>
<td>1.5(DL-EQX)</td>
<td>706</td>
<td>314</td>
<td>20</td>
<td>282</td>
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<td>900</td>
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<td>5 of 25</td>
</tr>
<tr>
<td>B-5</td>
<td>1.5(DL-EQZ)</td>
<td>803</td>
<td>692</td>
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<td>60</td>
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<td>2 of 20</td>
</tr>
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<td>B-6</td>
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<td>58</td>
<td>12</td>
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<td>3 of 20</td>
</tr>
<tr>
<td>B-7</td>
<td>1.5(DL+EQX)</td>
<td>196</td>
<td>102</td>
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<td>3 of 20</td>
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### SLAB Ramp

<table>
<thead>
<tr>
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### GF,FF,SF,TF Roof Beams for 8.36 & 5.5

<table>
<thead>
<tr>
<th>Beam No</th>
<th>Load Case</th>
<th>Support</th>
<th>Midspan</th>
<th>Torsion</th>
<th>B</th>
<th>D</th>
<th>Ast Top</th>
<th>Ast Bottom</th>
<th>Shear Rft</th>
<th>Legs</th>
</tr>
</thead>
<tbody>
<tr>
<td>RB1</td>
<td>1.5(DL+EQX)</td>
<td>235</td>
<td>150</td>
<td>0</td>
<td>125</td>
<td>120</td>
<td>300</td>
<td>500</td>
<td>4 of 25</td>
<td>2 of 25+1 of 20</td>
</tr>
<tr>
<td>RB2</td>
<td>1.5(DL-EQX)</td>
<td>325</td>
<td>190</td>
<td>0</td>
<td>70</td>
<td>165</td>
<td>300</td>
<td>400</td>
<td>2 of 25+1 of 20</td>
<td>3 of 25+1 of 20</td>
</tr>
<tr>
<td>RB3</td>
<td>1.5(DL-EQX)</td>
<td>803</td>
<td>360</td>
<td>0</td>
<td>377</td>
<td>270</td>
<td>300</td>
<td>850</td>
<td>8 of 25</td>
<td>3 of 25</td>
</tr>
<tr>
<td>RB4</td>
<td>1.5(DL-EQZ)</td>
<td>283</td>
<td>120</td>
<td>0</td>
<td>90</td>
<td>139</td>
<td>300</td>
<td>600</td>
<td>4 of 25</td>
<td>3 of 25</td>
</tr>
</tbody>
</table>
DESIGN OF FOOTING
### DESIGN OF ISOLATED FOOTING F2

**Design Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum factored axial load coming on footing</td>
<td>3200 kN</td>
</tr>
<tr>
<td>Safe Bearing capacity of the soil</td>
<td>225 kN/m²</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete</td>
<td>30 N/mm²</td>
</tr>
<tr>
<td>Characteristic yield strength of steel</td>
<td>415 N/mm²</td>
</tr>
<tr>
<td>Unit weight of concrete, ( \gamma_c )</td>
<td>24 kN/m³</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition</td>
<td>50 mm</td>
</tr>
<tr>
<td>Diameter of bars</td>
<td>20 mm</td>
</tr>
</tbody>
</table>

**Column Dimensions**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of the column</td>
<td>300</td>
</tr>
<tr>
<td>Depth of the column</td>
<td>800</td>
</tr>
</tbody>
</table>

**Design**

- Maximum axial load coming on footing = 2000.00 kN
- Add 10% toward the self-weight of footing = 200.00 kN
- Total load = 2200.00 kN

SBC of Soil: 225 kN/m² is considered in the design of foundations.

Area of footing required = \[ \frac{2200}{225} \] = 9.778 m²

<table>
<thead>
<tr>
<th>L</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.39 m</td>
<td>2.89 m</td>
</tr>
</tbody>
</table>

Provide footing of size 3.4 m x 2.9 m

- Net Upward Pressure on the foundation = 306.812 kN/m²
- B.M @ Section XX = Mx = 869.30 kNm
- Factored Moment = Mux = 1303.95 kNm
- Equating \( M_{u,lim} \) to Mux = 0.138fckbd² = Mux
  \[ M_{u,lim} = 3312 d^2 \]
  \[ d = 627 \] mm

- B.M @ Section YY = My = 741 kNm
- Factored Moment = Muy = 1111 kNm
- Equating \( M_{u,lim} \) to Muy = 0.138fckbd² = Muy
  \[ M_{u,lim} = 1242 d^2 \]
  \[ d = 946 \] mm

Effective cover to lower layer of steel = 50 mm + 10 mm = 60 mm
Effective cover to upper layer of steel = 60 mm + 20 mm = 80 mm
Overall depth required = 946 mm + 80 mm = 1026 mm

The overall depth may be increased by 30% to limit the shear stress

| Overall depth reqd | 1340 mm |

---

57
Effective depth for short span = 1340 mm - 60 mm = 1280 mm
Effective depth for long span = 1340 mm - 80 mm = 1260 mm

Steel Req'd for Longer Direction

\[
\frac{Muy}{bd^2} = 2.334
\]
\[
\% \text{ of steel} = 0.718 \%
\]

Area of steel required = 2714 mm\(^2\)

Provide 9 bars of 20 mm dia
Spacing of 20 mm dia bars 115 mm c/c

Steel Req'd for Shorter Direction

\[
\frac{Mux}{bd^2} = 0.995
\]
\[
\% \text{ of steel} = 0.287 \%
\]

Area of steel required = 2940 mm\(^2\)

Reinforcement Req'd for central band of 3.19 m = 2183 mm\(^2\)

Provide 9 bars of 20 mm dia
Spacing of 20 mm dia bars 143 mm c/c

Check For Shear

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e at a distance of 1280 mm from the face of the column

Shear force at this critical section X1 X1

\[
V = 14 \text{ kN}
\]
\[
Vu = 21 \text{ kN}
\]

Overall depth of the critical section \(D'\) = 608 mm
Effective depth of the critical section \(d'\) = 548 mm
Breadth of the footing @ tp @this critical section \(b'\) = 3360 mm
Nominal shear stress \(\tau_0\) = 0.01 N/mm\(^2\)

Percentage of steel provided = 0.15%
Permissible punching shear stress \(= 0.25 \times \text{sqrt}(f_{ck})\)

\[
1.37 \text{ N/mm}^2 > 0.01 \text{ N/mm}^2
\]

Provided Section is adequate.
DESIGN OF ISOLATED FOOTING F3

Design Parameters

Maximum factored axial load coming on footing = 3000 kN
Safe Bearing capacity of the soil = 225 kN/ m²
Grade of Concrete M30
Grade of Steel Fe415
Characteristic compressive strength of concrete, fck (N/mm²) = 30
Characteristic yield strength of steel, fy (N/mm²) = 415
Unit weight of concrete, γc (kN/m³) = 24
Partial safety factor for concrete = 1.5
Nominal Cover to exposure condition (mm) = 50
Diameter of bars (mm) = 20

Column Dimensions

Breadth of the column (mm) B = 300
Depth of the column (mm) D = 600

Design

Maximum axial load coming on footing = 2000.00 kN
Add 10% toward the self-weight of footing = 200.00 kN
Total load = 2200.00 kN

SBC of Soil : 225 kN/m² is considered in the design of foundations.

Area of footing required = 2200 / 225 = 9.778 m²
L = 3.28 m
B = 2.98 m

Provide footing of size 3.3 m x 3 m

Projection beyond Column Faces = 1.34 m
Net Upward Pressure on the foundation = 306.812 kN/m²

B.M @ Section XX = Mx = 904.04 kNm
Factored Moment = Mux = 1356.06 kNm
Equating Mu,lim to Mux = 0.138fckbd² = Mux

Mux = 2484 d²
739 mm

B.M @ Section YY = My = 821 kNm
Factored Moment = Muy = 1232 kNm
Equating Mu,lim to Muy = 0.138fckbd² = Muy

Muy = 1242 d²
996 mm

Effective cover to lower layer of steel = 50 mm + 10 mm = 60 mm
Effective cover to upper layer of steel = 60 mm + 20 mm = 80 mm
Overall depth required = 996 mm + 80 mm = 1076 mm

The overall depth may be increased by 30% to limit the shear stress

Overall depth reqd = 1400 mm
Effective depth for short span = 1400 mm - 60 mm = 1340 mm
Effective depth for long span = 1400 mm - 80 mm = 1320 mm

Steel Req'd for Longer Direction

\[
\frac{M_{uy}}{bd^2} = 2.357 \\
\% \text{ of steel} = 0.726 \%
\]

Area of steel required = 2875 mm²

Provide 10 bars of 20 mm dia
Spacing of 20 mm dia bars 109 mm c/c

Steel Req'd for Shorter Direction

\[
\frac{M_{ux}}{bd^2} = 1.259 \\
\% \text{ of steel} = 0.367 \%
\]

Area of steel required = 2954 mm²

Reinforcement Req'd for central band of 3.08 m = 2238 mm²

Provide 10 bars of 20 mm dia
Spacing of 20 mm dia bars 140 mm c/c

Check For Shear

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e at a distance of 1340 mm from the face of the column
Shear force at this critical section X1 X1

\[
V = 0 \text{ kN} \\
V_{u} = 0 \text{ kN}
\]

Factored Shear

Overall depth of the critical section \(D'\) = 700 mm
Effective depth of the critical section \(d'\) = 640 mm
Breadth of the footing @ tp @ this critical section \(b'\) = 3280 mm
Nominal shear stress \(\tau_{n}\) = 0.00 N/mm²

Percentage of steel provided = 0.15 %
Permissible punching shear stress = 0.25 \(\times\) sqrt(fck)

\[
1.37 \text{ N/mm}^2 > 0.00 \text{ N/mm2}
\]

Provided Section is adequate.
DESIGN OF ISOLATED FOOTING F4

Design Parameters

Maximum factored axial load coming on footing = 1200 kN
Safe Bearing capacity of the soil = 225 kN/m²
Grade of Concrete = M30
Grade of Steel = Fe415
Characteristic compressive strength of concrete, fck (N/mm²) = 30
Characteristic yield strength of steel, fy (N/mm²) = 415
Unit weight of concrete, γc (kN/m³) = 24
Partial safety factor for concrete = 1.5
Nominal Cover to exposure condition (mm) = 50
Diameter of bars (mm) = 16

Column Dimensions

| Breadth of the column (mm) B | 350 |
| Depth of the column (mm) D | 500 |

Design

Maximum axial load coming on footing = 800.00 kN
Add 10% toward the self-weight of footing = 80.00 kN
Total load = 880.00 kN

SBC of Soil : 225 kN/m² is considered in the design of foundations.

Area of footing required = 880 / 225 = 3.912 m²
L = 2.05 m
B = 1.90 m

Provide footing of size 2.1 m x 2 m

Projection beyond Column Faces = 0.78 m
Net Upward Pressure on the foundation = 306.749 kN/m²

B.M @ Section XX = Mx = 190.29 kNm
Factored Moment = Mux = 285.44 kNm
Equating Mu,lim to Mux = 0.138fckbd² = Mux
Mu,lim = 2070 d²

B.M @ Section YY = My = 176 kNm
Factored Moment = Muy = 265 kNm
Equating Mu,lim to Muy = 0.138fckbd² = Muy
Mu,lim = 1449 d²

Effective cover to lower layer of steel = 50 mm + 8 mm = 58 mm
Effective cover to upper layer of steel = 58 mm + 16 mm = 74 mm
Overall depth required = 427 mm + 74 mm = 501 mm

The overall depth may be increased by 30% to limit the shear stress

Overall depth reqd = 660 mm
Effective depth for short span = 660 mm - 58 mm = 602 mm
Effective depth for long span = 660 mm - 74 mm = 586 mm

**Steel Req'd for Longer Direction**

\[
\frac{M_u}{bd^2} = 2.202
\]
\[
\% \text{ of steel} = 0.673\%
\]

Area of steel required = 1380 mm\(^2\)

*Provide 7 bars of 16 mm dia*

Spacing of 16 mm dia bars 145 mm c/c

**Steel Req'd for Shorter Direction**

\[
\frac{M_u}{bd^2} = 1.575
\]
\[
\% \text{ of steel} = 0.467\%
\]

Area of steel required = 1405 mm\(^2\)

Reinforcement Req'd for central band of 1.85 m = 1386 mm\(^2\)

*Provide 9 bars of 16 mm dia*

Spacing of 16 mm dia bars 145 mm c/c

**Check For Shear**

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e at a distance of 602 mm from the face of the column

Shear force at this critical section X1 X1

\[
V = 110 \text{ kN}
\]

Factored Shear \(V_u = 166 \text{ kN}\)

Overall depth of the critical section \(D' = 381 \text{ mm}\)

Effective depth of the critical section \(d' = 323 \text{ mm}\)

Breadth of the footing @ tp @this critical section \(b' = 1704 \text{ mm}\)

Nominal shear stress \(\tau_n = 0.30 \text{ N/mm}^2\)

Percentage of steel provided = 0.33 %

Permissible punching shear stress = \(0.25 \times \sqrt{f_{ck}}\)

\[
1.37 \text{ N/mm}^2 > 0.30 \text{ N/mm}^2
\]

*Provided Section is adequate.*
DESIGN OF ISOLATED FOOTING F5

Design Parameters

Maximum factored axial load coming on footing = 2000 kN
Safe Bearing capacity of the soil = 225 kN/m²
Grade of Concrete = M30
Grade of Steel = Fe415
Characteristic compressive strength of concrete, fck (N/mm²) = 30
Characteristic yield strength of steel, fy (N/mm²) = 415
Unit weight of concrete, γc (kN/m³) = 24
Partial safety factor for concrete = 1.5
Nominal Cover to exposure condition (mm) = 50
Diameter of bars (mm) = 16

Column Dimensions

Breadth of the column (mm) B = 350
Depth of the column (mm) D = 600

Design

Maximum axial load coming on footing = 1333.33 kN
Add 10% toward the self-weight of footing = 133.33 kN
Total load = 1466.67 kN
SBC of Soil : 225 kN/m² is considered in the design of foundations.

Area of footing required = 1466.67 / 225 = 6.519 m²
L = 2.68 m
B = 2.43 m

Provide footing of size 2.7 m x 2.5 m

Projection beyond Column Faces = 1.04 m
Net Upward Pressure on the foundation = 306.796 kN/m²

B.M @ Section XX = Mx = 445.42 kNm
Factored Moment = Mux = 668.13 kNm
Equating Mu,lim to Mux = 0.138fckbd² = Mux
Mu,lim = 2484 d²
519 mm

B.M @ Section YY = My = 404 kNm
Factored Moment = Muy = 606 kNm
Equating Mu,lim to Muy = 0.138fckbd² = Muy
Mu,lim = 1449 d²
647 mm

Effective cover to lower layer of steel = 50 mm + 8 mm = 58 mm
Effective cover to upper layer of steel = 58 mm + 16 mm = 74 mm
Overall depth required = 647 mm + 74 mm = 721 mm

The overall depth may be increased by 30% to limit the shear stress

Overall depth reqd = 940 mm
Effective depth for short span = 940 mm - 58 mm = 882 mm
Effective depth for long span = 940 mm - 74 mm = 866 mm

**Steel Req'd for Longer Direction**

\[
\frac{M_{uy}}{bd^2} = 2.308 \\
\% of steel = 0.709 \%
\]

Area of steel required = 2149 mm²

Provide 11 bars of 16 mm dia
Spacing of 16 mm dia bars 100 mm c/c

**Steel Req'd for Shorter Direction**

\[
\frac{M_{ux}}{bd^2} = 1.431 \\
\% of steel = 0.421 \%
\]

Area of steel required = 2229 mm²

Reinforcement Req'd for central band of 2.48 m = 1905 mm²

Provide 12 bars of 16 mm dia
Spacing of 16 mm dia bars 105 mm c/c

**Check For Shear**

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e. at a distance of 882 mm from the face of the column.

Shear force at this critical section X1 X1

\[
V = 131 \text{ kN} \\
V_u = 196 \text{ kN}
\]

Overall depth of the critical section \( D' = 482 \text{ mm} \)
Effective depth of the critical section \( d' = 424 \text{ mm} \)
Breadth of the footing @ tp @this critical section \( b' = 2364 \text{ mm} \)
Nominal shear stress \( \tau = 0.20 \text{ N/mm}^2 \)

Percentage of steel provided = 0.24%
Permissible punching shear stress = \( 0.25 \times \sqrt{f_{ck}} \)

\[
1.37 \text{ N/mm}^2 > 0.20 \text{ N/mm}^2
\]

Provided Section is adequate.
DESIGN OF ISOLATED FOOTING F6

Design Parameters

- Maximum factored axial load coming on footing = 3220 kN
- Safe Bearing capacity of the soil = 225 kN/m²
- Grade of Concrete = M30
- Grade of Steel = Fe415
- Characteristic compressive strength of concrete, \( f_{ck} \) (N/mm²) = 30
- Characteristic yield strength of steel, \( f_y \) (N/mm²) = 415
- Unit weight of concrete, \( \gamma_c \) (kN/m³) = 24
- Partial safety factor for concrete = 1.5
- Nominal Cover to exposure condition (mm) = 50
- Diameter of bars (mm) = 20

Column Dimensions

<table>
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<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
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<td>Depth of the column (mm)</td>
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Design

- Maximum axial load coming on footing = 2146.67 kN
- Add 10% toward the self-weight of footing = 214.67 kN
- Total load = 2361.34 kN

SBC of Soil: 225 kN/m² is considered in the design of foundations.

Area of footing required = \( \frac{2361.34}{225} \) m² = 10.495 m²
L = 3.50 m
B = 3.00 m

Provide footing of size 3.5 m x 3 m

- Projection beyond Column Faces = 1.30 m
- Net Upward Pressure on the foundation = 306.813 kN/m²

B.M @ Section XX = Mx = 906.66 kNm
Factored Moment = Mux = 1359.99 kNm
Equating Mu,lim to Mux = 0.138fckbd² = Mux
\( M_{u,lim} = \frac{3726 d^2}{604} \) mm

B.M @ Section YY = My = 777 kNm
Factored Moment = Muy = 1166 kNm
Equating Mu,lim to Muy = 0.138fckbd² = Muy
\( M_{u,lim} = \frac{1656 d^2}{839} \) mm

Effective cover to lower layer of steel = 50 mm + 10 mm = 60 mm
Effective cover to upper layer of steel = 60 mm + 20 mm = 80 mm
Overall depth required = 839 mm + 80 mm = 919 mm

The overall depth may be increased by 30% to limit the shear stress

Overall depth reqd = 1300 mm
Effective depth for short span = 1300 mm - 60 mm = 1240 mm
Effective depth for long span = 1300 mm - 80 mm = 1220 mm

**Steel Req'd for Longer Direction**

\[
\frac{M_{y}}{b'd^2} = 1.958
\]

\[
\% \text{ of steel} = 0.591 \%
\]

\[
\text{Area of steel required} = 2883 \text{ mm}^2
\]

*Provide 10 bars of 20 mm dia*

*Spacing of 20 mm dia bars 108 mm c/c*

**Steel Req'd for Shorter Direction**

\[
\frac{M_{x}}{b'd^2} = 0.983
\]

\[
\% \text{ of steel} = 0.283 \%
\]

\[
\text{Area of steel required} = 3163 \text{ mm}^2
\]

*Reinforcement Req'd for central band of 3.3 m = 2301 mm$^2$*

*Provide 10 bars of 20 mm dia*

*Spacing of 20 mm dia bars 136 mm c/c*

**Check For Shear**

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e. at a distance of 1240 mm from the face of the column.

Shear force at this critical section X1 X1

\[
V = 64 \text{ kN}
\]

\[
V_{u} = 96 \text{ kN}
\]

Overall depth of the critical section \(D' = 632 \text{ mm}\)

Effective depth of the critical section \(d' = 572 \text{ mm}\)

Breadth of the footing at tp at this critical section \(b' = 3380 \text{ mm}\)

Nominal shear stress \(\tau_{n} = 0.05 \text{ N/mm}^2\)

Percentage of steel provided = 0.16 \%

Permissible punching shear stress = 0.25 x sqrt(fck)

\[
1.37 \text{ N/mm}^2 > 0.05 \text{ N/mm}^2
\]

Provided Section is adequate.
DESIGN OF ISOLATED FOOTING F7

Design Parameters

Maximum factored axial load coming on footing = 9860 kN
Safe Bearing capacity of the soil = 225 kN/m²
Grade of Concrete = M30
Grade of Steel = Fe415
Characteristic compressive strength of concrete , \( f_{ck} \) (N/mm²) = 30
Characteristic yield strength of steel , \( f_y \) (N/mm²) = 415
Unit weight of concrete , \( \gamma_c \) (kN/m³) = 24
Partial safety factor for concrete = 1.5
Nominal Cover to exposure condition (mm) = 50
Diameter of bars (mm) = 25

Column Dimensions

Breadth of the column (mm) B = 800
Depth of the column (mm) D = 800

Design

Maximum axial load coming on footing = 6573.33 kN
Add 10% toward the self-weight of footing = 657.33 kN
Total load = 7230.67 kN
SBC of Soil : 225 kN/m² is considered in the design of foundations.

Area of footing required = \( \frac{7230.67}{225} \) = 32.137 m²
L = 5.67 m
B = 5.67 m

Provide footing of size 5.7 m x 5.7 m

Projection beyond Column Faces = 2.43 m
Net Upward Pressure on the foundation = 306.812 kN/m²

B.M @ Section XX = Mx = 5154.13 kNm
Factored Moment = Mux = 7731.20 kNm
Equating \( Mu_{,lim} \) to Mux = 0.138\( f_{ck}bd^2 \) = Mux
\( Mu_{,lim} = 3312 d^2 \)
\( 1528 \) mm

B.M @ Section YY = My = 5154 kNm
Factored Moment = Muy = 7731 kNm
Equating \( Mu_{,lim} \) to Muy = 0.138\( f_{ck}bd^2 \) = Muy
\( Mu_{,lim} = 3312 d^2 \)
\( 1528 \) mm

Effective cover to lower layer of steel = 50 mm + 12.5 mm = 62.5 mm
Effective cover to upper layer of steel = 62.5 mm + 25 mm = 87.5 mm
Overall depth required = 1528 mm + 87.5 mm = 1615 mm

The overall depth may be increased by 30% to limit the shear stress

Overall depth reqd = 2600 mm
Effective depth for short span = 2600 mm - 62.5 mm = 2537.5 mm
Effective depth for long span = 2600 mm - 87.5 mm = 2512.5 mm

Steel Reqd for Longer Direction

\[
\frac{M_{uy}}{bd^2} = 1.531 \\
\%	ext{ of steel} = 0.453 \%
\]

Area of steel required = \(9096 \text{ mm}^2\)
**Provide 19 bars of 25 mm dia**
**Spacing of 25 mm dia bars 100 mm c/c**

Steel Reqd for Shorter Direction

\[
\frac{M_{ux}}{bd^2} = 1.501 \\
\%	ext{ of steel} = 0.443 \%
\]

Area of steel required = \(8994 \text{ mm}^2\)

**Reinforcement Reqd for central band of 5.47 m = 4691 mm^2**
**Provide 12 bars of 25 mm dia**
**Spacing of 25 mm dia bars 104 mm c/c**

Check For Shear

Critical section X1 X1 is considered at a distance equal to the effective depth from the face of the column, i.e, at a distance of 2537.5 mm from the face of the column

Shear force at this critical section X1 X1

\[
V = -179 \text{ kN} \\
Vu = -269 \text{ kN}
\]

Overall depth of the critical section \(D' = 1245 \text{ mm}\)
Effective depth of the critical section \(d' = 1182 \text{ mm}\)
Breadth of the footing @ tp @this critical section \(b' = 5875 \text{ mm}\)
Nominal shear stress \(\tau_n = -0.04 \text{ N/mm}^2\)

Percentage of steel provided = 0.08 %
Permissible punching shear stress = \(0.25 \times \sqrt{f_{ck}}\)

\[
1.37 \text{ N/mm}^2 > -0.04 \text{ N/mm}^2
\]

**Provided Section is adequate.**
**Combined Footing Analysis and Design**

### Dimensions:

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<td>Left width, b (m)</td>
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<tr>
<td>Distance, Xb (m)</td>
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<td>Right width, a (m)</td>
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<td>Distance, Xa (m)</td>
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<td>Length, L (m)</td>
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<td>Area, (m²)</td>
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### Material Properties:
- Conc comp strength f’c, (Mpa): 30
- Steel comp strength fy, (Mpa): 415
- Allow soil pressure, qa (kPa): 281.25

### Loads:

<table>
<thead>
<tr>
<th>Col. Load</th>
<th>Working loads</th>
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<th>Live</th>
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<td>M2 (kN.m)</td>
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</table>

### Checkings:
- Allowable soil pressure, qa (kPa): 281.25
- Maximum soil pressure, qmax (kPa): 249.134 \( (q_{max} < qa) \) Ok
- Minimum soil pressure, qmin (kPa): 162.55 \( (q_{min} > 0) \) Ok
- Maximum wide beam shear, Vw (kN/m width): 446.275
- Maximum punching shear, Vp (kN/m width): 4437.18
- Wide beam shear strength, Vc1 (kN/m width): 547.723 \( Vw < Vc1, OK \)
- Punching shear strength, Vc2 (kN/m width): 3999.06 \( Vp > Vc2, \) increase depth

### Area of Steel:
- Use area of steel, As (cm²): 24.5 for bottom reinforcement
- As (cm²): 20.241 for top reinforcement

### Details:

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<tr>
<th>x</th>
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<th>M, kN-m</th>
<th>b, m</th>
<th>As, cm²/m</th>
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## Combined Footing Analysis and Design

### Dimensions:

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</thead>
<tbody>
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<td>Length, x (m)</td>
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<td>Distance, Xb (m)</td>
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<td>Eff. depth, d (m)</td>
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### Material Properties:

- Conc comp strength f'c, (Mpa) 30
- Steel comp strength fy, (Mpa) 415
- Allow soil pressure, qa (kPa) 281.25

### Loads:

<table>
<thead>
<tr>
<th>Col. Load</th>
<th>Working loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
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<td>P1 (kN)</td>
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<tr>
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<tr>
<td>M2 (kN.m)</td>
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### Checkings:

- Allowable soil pressure, qa (kPa) 281.25
- Maximum soil pressure, qmax (kPa) 274.953 (qmax < qa) Ok
- Minimum soil pressure, qmin (kPa) 202.173 (qmin > 0) Ok
- Maximum wide beam shear, Vw (kN/m width) 364.76
- Maximum punching shear, Vp (kN/m width) 8200.62
- Wide beam shear strength, Vc1 (kN/m width) 912.871 Vw < Vc1, OK
- Punching shear strength, Vc2 (kN/m width) 3999.06 Vp > Vc2, increase depth

### Area of Steel:

- Use area of steel, As (cm²) 34 for bottom reinforcement
- As (cm²) 33.7349 for top reinforcement

### Details:

<table>
<thead>
<tr>
<th>x</th>
<th>V, kN</th>
<th>M, kN-m</th>
<th>b, m</th>
<th>As, cm²/m</th>
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Combined Footing Analysis and Design

Dimensions:

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<th>Col.2</th>
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Material Properties:

- Conc comp strength f'c, (Mpa) 30
- Steel comp strength fy, (Mpa) 415
- Allow soil pressure, qa (kPa) 281.25

Loads:

<table>
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<tr>
<th>Col. Load</th>
<th>Working loads</th>
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<tbody>
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<td>M2 (kN.m)</td>
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Checkings:

- Allowable soil pressure, qa (kPa) 281.25
- Maximum soil pressure, qmax (kPa) 256.964 (qmax < qa) Ok
- Minimum soil pressure, qmin (kPa) 60.8704 (qmin > 0) Ok
- Maximum wide beam shear, Vw (kN/m width) 295.907
- Maximum punching shear, Vp (kN/m width) 9963.89
- Wide beam shear strength, Vc1 (kN/m width) 1095.45 Vw < Vc1, OK
- Punching shear strength, Vc2 (kN/m width) 3999.06 Vp > Vc2, increase depth

Area of Steel:

- Use area of steel, As (cm²) for bottom reinforcement
  - As (cm²) 34
- As (cm²) 40.4819 for top reinforcement

Details:

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<tr>
<th>x</th>
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<th>b, m</th>
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Combined Footing Analysis and Design

Dimensions:

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<tr>
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Material Properties:

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<tr>
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<tr>
<td>Steel comp strength f_y</td>
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</tr>
<tr>
<td>Allow soil pressure, qa</td>
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Loads:

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<tr>
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<th>Wind</th>
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<tr>
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</table>

Checkings:

| Allowable soil pressure, qa (kPa) | 281.25 |
| Maximum soil pressure, qmax (kPa) | 276    |
| Minimum soil pressure, qmin (kPa) | 108.756|
| Maximum wide beam shear, Vw (kN/m width) | 440.731|
| Maximum punching shear, Vp (kN/m width) | 8842   |
| Wide beam shear strength, Vc1 (kN/m width) | 1004.16|
| Punching shear strength, Vc2 (kN/m width) | 3999.06|

Ww < Vc1, OK

Area of Steel:

<table>
<thead>
<tr>
<th>Use area of steel, As (cm²)</th>
<th>37.5  for bottom reinforcement</th>
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</thead>
<tbody>
<tr>
<td>As (cm²)</td>
<td>37.1084 for top reinforcement</td>
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</tbody>
</table>

Details:

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<tr>
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<th>b, m</th>
<th>As, cm²/m</th>
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## Combined Footing Analysis and Design

### Dimensions:

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<td>Length, L (m)</td>
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</table>

### Material Properties:

- Conc comp strength f'c, (Mpa) | 30 |
- Steel comp strength fy, (Mpa) | 415 |
- Allow soil pressure, qa (kPa) | 281.25 |

### Loads:

<table>
<thead>
<tr>
<th>Col. Load</th>
<th>Working loads</th>
</tr>
</thead>
<tbody>
<tr>
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<td>P1 (kN)</td>
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<td>P2 (kN)</td>
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<tr>
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<tr>
<td>M2 (kN.m)</td>
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</tr>
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</table>

### Checkings:

- Allowable soil pressure, qa (kPa) | 281.25 |
- Maximum soil pressure, qmax (kPa) | 280.014 |
- Minimum soil pressure, qmin (kPa) | 197.455 |

- Maximum wide beam shear, Vw (kN/m width) | 274.462 |
- Maximum punching shear, Vp (kN/m width) | 8158.51 |

- Wide beam shear strength, Vc1 (kN/m width) | 1369.31 |
- Punching shear strength, Vc2 (kN/m width) | 3999.06 |

### Area of Steel:

- Use area of steel, As (cm²) for bottom reinforcement:
  - 51 cm²
- As (cm²) for top reinforcement:
  - 50.6024 cm²

### Details:

<table>
<thead>
<tr>
<th>x</th>
<th>V, kN</th>
<th>M, kN-m</th>
<th>b, m</th>
<th>As, cm²/m</th>
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<tbody>
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<td>0</td>
<td>0</td>
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<tr>
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**Combined Footing Analysis and Design**

**Dimensions:**

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<th>Col.2</th>
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<td>Width, y (m)</td>
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<td>Distance, Xa (m)</td>
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**Material Properties:**

- Conc comp strength f’c, (Mpa) 30
- Steel comp strength fy, (Mpa) 415
- Allow soil pressure, qa (kPa) 281.25

**Loads:**

<table>
<thead>
<tr>
<th>Col. Load</th>
<th>Working loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead</td>
</tr>
<tr>
<td>P1 (kN)</td>
<td>7042.86</td>
</tr>
<tr>
<td>P2 (kN)</td>
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<tr>
<td>M1 (kN.m)</td>
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</tr>
<tr>
<td>M2 (kN.m)</td>
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</table>

**Checkings:**

- Allowable soil pressure, qa (kPa) 281.25
- Maximum soil pressure, qmax (kPa) 277.825 (qmax < qa) Ok
- Minimum soil pressure, qmin (kPa) 208.369 (qmin > 0) Ok
- Maximum wide beam shear, Vw (kN/m width) 505.641
- Maximum punching shear, Vp (kN/m width) 8304.18
- Wide beam shear strength, Vc1 (kN/m width) 1095.45 Vw < Vc1, OK
- Punching shear strength, Vc2 (kN/m width) 3999.06 Vp < Vc2, OK

**Area of Steel:**

- Use area of steel, As (cm²) 40.5 for bottom reinforcement
- As (cm²) 40.4819 for top reinforcement

**Details:**

<table>
<thead>
<tr>
<th>x</th>
<th>V, kN</th>
<th>M, kN·m</th>
<th>b, m</th>
<th>As, cm²/m</th>
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<td>0</td>
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**Combined Footing Analysis and Design**

**Dimensions:**

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<th>Col.1</th>
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<tr>
<td>Width, y (m)</td>
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<tr>
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<td>Distance, Xa (m)</td>
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| Eff. depth, d (m) | 1.15 |

| Area, (m²) | 51.2 |

**Material Properties:**

<p>| | |</p>
<table>
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<tr>
<th></th>
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<tbody>
<tr>
<td>Conc comp strength f'c, (Mpa)</td>
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</tr>
<tr>
<td>Steel comp strength fy, (Mpa)</td>
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</tr>
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<td>Allow soil pressure, qa (kPa)</td>
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**Loads:**

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<tr>
<th>Col. Load</th>
<th>Working loads</th>
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</thead>
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<tr>
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<td>P1 (kN)</td>
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<td>P2 (kN)</td>
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<td>M1 (kN.m)</td>
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<tr>
<td>M2 (kN.m)</td>
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</tbody>
</table>

**Checkings:**

- Allowable soil pressure, qa (kPa) 281.25
- Maximum soil pressure, qmax (kPa) 275.215 (qmax < qa) Ok
- Minimum soil pressure, qmin (kPa) 206.256 (qmin > 0) Ok
- Maximum wide beam shear, Vw (kN/m width) 280.288
- Maximum punching shear, Vp (kN/m width) 8357.77
- Wide beam shear strength, Vc1 (kN/m width) 1049.8 Vw < Vc1, OK
- Punching shear strength, Vc2 (kN/m width) 3999.06 Vp < Vc2, OK

**Area of Steel:**

Use area of steel, As (cm²) 39 for bottom reinforcement

As (cm²) 38.7952 for top reinforcement

**Details:**

<table>
<thead>
<tr>
<th>x</th>
<th>V, kN</th>
<th>M, kN-m</th>
<th>b, m</th>
<th>As, cm²/m</th>
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<td>0.01</td>
<td>6.6</td>
<td>26.988</td>
</tr>
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</table>
DESIGN OF COLUMNS
Rectangular Short Column with Biaxial bending - Bresler method

**COLUMN NO C1**

**Load Case** 1.5*(DL - EQX)

**Grade of Concrete** M30

**Grade of Steel** Fe415

**Characteristic compressive strength of concrete, f_{ck} (N/mm^2)** 30

**Characteristic yield strength of steel, f_y (N/mm^2)** 415

**Unit weight of concrete, γ_c (kN/m^3)** 25

**Partial safety factor for concrete** 1.5

**Exposure condition** Mild

**Nominal Cover to exposure condition (mm)** 40

**Assumed effective cover all around, d' (mm)** 50

**Dimensions of the Column**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
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<tbody>
<tr>
<td>Unsupported length of column, L</td>
<td>3600 mm</td>
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<tr>
<td>Least lateral dimension</td>
<td>230 mm</td>
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<tr>
<td>Breadth of the column B (mm)</td>
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</tr>
<tr>
<td>Depth of the Column D (mm)</td>
<td>230</td>
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<tr>
<td>Effective length of the column, l_{ex} (m)</td>
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<tr>
<td>Effective length of the column, l_{ey} (m)</td>
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**Check for Slenderness ratio, L/D**

<table>
<thead>
<tr>
<th>Slenderness ratio, λ_{ex}</th>
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<th>&lt;12</th>
<th>column is Short</th>
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<tbody>
<tr>
<td>Slenderness ratio, λ_{ey}</td>
<td>10.17</td>
<td>&lt;12</td>
<td>column is Short</td>
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</table>

**Design Factors**

<table>
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<th>Factor</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
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<tr>
<td>Factored moment acting parallel to the larger dimension, M_{ux}</td>
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</tr>
<tr>
<td>Factored moment acting parallel to the shorter dimension, M_{uy}</td>
<td>3 KN-m</td>
</tr>
</tbody>
</table>

1. **Check for accidental eccentricity**

   Equivalent eccentricity of loads is given by
   
   \[
   \frac{M_{ux}}{P_u} = 90.91 \text{ mm} \\
   \frac{M_{uy}}{P_u} = 327.27 \text{ mm} 
   \]

   Both are more than 20 mm minimum

2. **Assume percentage of steel**

   (assuming steel larger than required by P and M_u)

   \[
   \frac{M_u}{f_{ck} \times b \times D^2} = 0.07 \\
   \frac{P_u}{f_{ck} \times b \times D} = 0.17
   \]

   \[
   d'/D = 0.2
   \]
From SP16 chart 44

\[
\frac{P}{f_{ck}} = 0.06 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\) is 0.09

Assumed, \(P = 2.70\) per cent

Area of steel, \(A_s = 1428.30\) mm\(^2\)

Use 6 no. s of 20 mm

Area of steel provided = 1884 mm\(^2\)

3. Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

\[
\frac{M_{x1}}{(f_{ck} \times b \times D^2)} = 0.13 \quad \text{From table}
\]

\[
M_{x1} = 47.45 \text{ KN-m}
\]

About Y-axis

\[
\frac{M_{y1}}{(f_{ck} \times D \times b^2)} = 0.13 \quad \text{From table}
\]

\[
M_{y1} = 47.45 \text{ KN-m}
\]

4. Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_c + 0.75f_{y}A_s
\]

\[
P_z = 1301 \text{ KN}
\]

\[
P/P_z = 0.21
\]

By formula

\[
\alpha^n = 2/3[1+5/2 \times P/P_z]
\]

\[
\alpha^n = 1.02
\]

5. Criteria for biaxial bending

\[
(M_x/M_{x1})^{an} + (M_y/M_{y1})^{an} \leq 1.0
\]

= 0.9000 < or = 1

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

**COLUMN NO C1a**

**Load Case** 1.5*(DL - EQX)

**Grade of Concrete** M30

**Grade of Steel** Fe415

**Characteristic compressive strength of concrete, \( f_{ck} \) (N/mm²)** 30

**Characteristic yield strength of steel, \( f_y \) (N/mm²)** 415

**Unit weight of concrete, \( \gamma_c \) (kN/m³)** 25

**Partial safety factor for concrete** 1.5

**Exposure condition** Mild

**Nominal Cover to exposure condition (mm)** 40

**Assumed effective cover all around, \( d' \) (mm)** 50

**Dimensions of the Column**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsupported length of column, L</td>
<td>3600 mm</td>
</tr>
<tr>
<td>Least lateral dimension</td>
<td>400 mm</td>
</tr>
<tr>
<td>Breadth of the column B (mm)</td>
<td>400</td>
</tr>
<tr>
<td>Depth of the Column D (mm)</td>
<td>700</td>
</tr>
</tbody>
</table>

| Effective length of the column, \( l_{ex} \) (m) | 2.34 |
| Effective length of the column, \( l_{ey} \) (m) | 2.34 |

**Check for Slenderness ratio, L/D**

<table>
<thead>
<tr>
<th>Slenderness ratio, ( \lambda_{ex} )</th>
<th>5.85 &lt;12 column is Short</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slenderness ratio, ( \lambda_{ey} )</td>
<td>3.34 &lt;12 column is Short</td>
</tr>
</tbody>
</table>

**Design Factors**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored load, ( P_u )</td>
<td>1307.33 KN</td>
</tr>
<tr>
<td>Factored moment acting parallel to the larger dimension, ( M_{ux} )</td>
<td>340.03 KN-m</td>
</tr>
<tr>
<td>Factored moment acting parallel to the shorter dimension, ( M_{uy} )</td>
<td>769.88 KN-m</td>
</tr>
</tbody>
</table>

1. **Check for accidental eccentricity**
   - Equivalent eccentricity of loads is given by
     \[
     \frac{M_{ux}}{P_u} = 260.10 \text{ mm} \\
     \frac{M_{uy}}{P_u} = 68.84 \text{ mm}
     \]
   - Both are more than 20mm minimum

2. **Assume percentage of steel**
   - (assuming steel larger than required by P and \( M_x \))
     \[
     \frac{d'}{D} = 0.1 \\
     \frac{M_x}{f_{ck} \times b \times D^2} = 0.06
     \]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.16
\]

From SP16 chart44
\[
\frac{P}{f_{ck}} = 0.05 \quad \text{From table}
\]
Assuming a higher value \(P/f_{ck}\)
\[
\text{Assumed , } P = 0.075 \quad \text{per cent}
\]
Area of steel, \(A_s\)
\[
= 6300.00 \text{ mm}^2
\]
Use 16 no.s of 25 mm
Area of steel provided
\[
= 7850 \text{ mm}^2
\]

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis
\[
d'/D = 0.07
\]
\[
P/f_{ck} \times b \times D^2 = 0.16
\]
\[
P/f_{ck} = 0.075
\]
\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.12 \quad \text{From table}
\]
\[
M_{x1} = 705.60 \text{ KN-m}
\]

About Y-axis
\[
d'/D = 0.13
\]
\[
P/f_{ck} \times b \times D^2 = 0.16
\]
\[
P/f_{ck} = 0.075
\]
\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.12 \quad \text{From table}
\]
\[
M_{y1} = 403.20 \text{ KN-m}
\]

4 Calculate \(\alpha^n\)
\[
P_z = 0.45f_{ck}A_s + 0.75f_{y}A_s
\]
\[
Pz = 6223 \text{ KN}
\]
\[
P/P_z = 0.21
\]

By formula
\[
\alpha^n = 2/3[1+5/2 \times P/P_z]
\]
\[
\alpha^n = 1.02
\]

5 Criteria for biaxial bending
\[
(M_x/M_{x1})^{\text{mm}} + (M_y/M_{y1})^{\text{mm}} < or =1.0
\]
\[
= 0.6903 < or = 1
\]

Hence the column is safe
### Rectangular Short Column with Biaxial bending - Bresler method

<table>
<thead>
<tr>
<th>COLUMN NO</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case</td>
<td>1.5*(DL - EQX)</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete, $f_{ck}$ (N/mm²)</td>
<td>30</td>
</tr>
<tr>
<td>Characteristic yield strength of steel, $f_y$ (N/mm²)</td>
<td>415</td>
</tr>
<tr>
<td>Unit weight of concrete, $\gamma_c$ (kN/m³)</td>
<td>25</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Exposure condition</td>
<td>Mild</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition (mm)</td>
<td>40</td>
</tr>
<tr>
<td>Assumed effective cover all around, $d'$ (mm)</td>
<td>50</td>
</tr>
</tbody>
</table>

#### Dimensions of the Column

- Unsupported length of column, $L = 3750$ mm
- Least lateral dimension = 350 mm
- Breadth of the column $B = 350$ mm
- Depth of the column $D = 800$ mm

- Effective length of the column, $l_{ex}$ (m) = 2.34
- Effective length of the column, $l_{ey}$ (m) = 2.34

#### Check for Slenderness ratio, $L/D$

- Slenderness ratio, $\lambda_{ex} = 7.80$ <12 column is Short
- Slenderness ratio, $\lambda_{ey} = 2.93$ <12 column is Short

#### Design Factors

- Factored load, $P_u = 925$ KN
- Factored moment acting parallel to the larger dimension, $M_{ux} = 334$ KN-m
- Factored moment acting parallel to the shorter dimension, $M_{uy} = 228.13$ KN-m

1. Check for accidental eccentricity
   - Equivalent eccentricity of loads is given by
     $M_{ux}/P_u = 362.01$ mm
     $M_{uy}/P_u = 100.56$ mm
   - Both are more than 20mm minimum

2. Assume percentage of steel
   - (assuming steel larger than required by $P$ and $M_x$)
     $d'/D = 0.1$

   $$\frac{M_x}{f_{ck} \times b \times D^2} = 0.06$$
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.12
\]

From SP16 chart

\[
\frac{P}{f_{ck}} = 0.05 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\)

\[
\text{Assumed } P = 2.25 \text{ per cent}
\]

Area of steel, \(A_s\)

\[
A_s = 5400.00 \text{ mm}^2
\]

Use 12 no.s of 25 mm

Area of steel provided

\[
= 5888 \text{ mm}^2
\]

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis
\[
d'/D = 0.06
\]

\[
\frac{P}{f_{ck} \times b \times D^2} = 0.12
\]

\[
\frac{P}{f_{ck}} = 0.075
\]

\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.12 \quad \text{From table}
\]

\[
M_{x1} = 691.20 \text{ KN-m}
\]

About Y-axis
\[
d'/D = 0.17
\]

\[
\frac{P}{f_{ck} \times D \times b^2} = 0.12
\]

\[
\frac{P}{f_{ck}} = 0.075
\]

\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.12 \quad \text{From table}
\]

\[
M_{y1} = 269.20 \text{ KN-m}
\]

4 Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_s + 0.75f_yA_s
\]

\[
P_z = 5072 \text{ KN}
\]

\[
P/P_z = 0.18
\]

By formula

\[
\alpha^n = 2/3[1 + 5/2 \times P/P_z]
\]

\[
\alpha^n = 0.97
\]

5 Criteria for biaxial bending

\[
(M_x/M_{x1})^{an} + (M_y/M_{y1})^{an} < 1.0
\]

\[
= 0.8613 < 1
\]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO C3
Load Case \(1.2*(DL + LL - EQX)\)
Grade of Concrete M30
Grade of Steel Fe415
Characteristic compressive strength of concrete, \(f_{ck} \text{ (N/mm}^2\) \(= 30\)
Characteristic yield strength of steel, \(f_y \text{ (N/mm}^2\) \(= 415\)
Unit weight of concrete, \(\gamma_c \text{ (kN/m}^3\) \(= 25\)
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition (mm) 40
Assumed effective cover all around, \(d' \text{ (mm) } = 60\)

Dimensions of the Column

Unsupported length of column, \(L = 3600 \text{ mm}\)
Least lateral dimension = 300 \text{ mm}
Breadth of the column \(B \text{ (mm)} = 300\)
Depth of the Column \(D \text{ (mm)} = 600\)

Effective length of the column \(l_{ex}, \text{ (m)} = 2.34\)
Effective length of the column \(l_{ey}, \text{ (m)} = 2.34\)

Check for Slenderness ratio, \(L/D\)

\[\lambda_{ex} = \frac{L}{D} = 7.80 < 12 \text{ column is Short}\]
\[\lambda_{ey} = \frac{L}{D} = 3.90 < 12 \text{ column is Short}\]

Design Factors

Factored load, \(P_u = 366.32 \text{ KN}\)
Factored moment acting parallel to the larger dimension, \(M_{ux} = 162.29 \text{ KN-m}\)
Factored moment acting parallel to the shorter dimension, \(M_{uy} = 353.51 \text{ KN-m}\)

1 Check for accidental eccentricity
   Equivalent eccentricity of loads is given by
   \[\frac{M_{ux}}{P_u} = 443.03 \text{ mm}\]
   \[\frac{M_{uy}}{P_u} = 245.69 \text{ mm}\]
   Both are more than 20mm minimum

2 Assume percentage of steel
   (assuming steel larger than required by \(P\) and \(M_x\))
   \[\frac{d'}{D} = 0.1\]
   \[\frac{M_x}{f_{ck} \times b \times D^2} = 0.05\]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.07
\]

From SP16 chart44

\[
\frac{P}{f_{ck}} = 0.07 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\)

Assumed, \(P = 3.15\) per cent

Area of steel, \(A_s\)

\[
= 5670.00 \, \text{mm}^2
\]

Use 12 no.s of 25 mm

Area of steel provided

\[
= 5888 \, \text{mm}^2
\]

3. Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis

\[
d'/D = 0.10
\]

\[
P/f_{ck} \times b \times D^2 = 0.07
\]

\[
P/f_{ck} = 0.105
\]

\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.125 \quad \text{From table}
\]

\[
M_{x1} = 405.00 \, \text{KN-m}
\]

About Y-axis

\[
d'/D = 0.20
\]

\[
P/f_{ck} \times b \times D^2 = 0.07
\]

\[
P/f_{ck} = 0.105
\]

\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.125 \quad \text{From table}
\]

\[
M_{y1} = 202.50 \, \text{KN-m}
\]

4. Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_c + 0.75f_yA_s
\]

\[
P_z = 4262 \, \text{KN}
\]

\[
P/P_z = 0.09
\]

By formula

\[
\alpha^n = 2/3[1+5/2 \times P/P_z]
\]

\[
\alpha^n = 0.81
\]

5. Criteria for biaxial bending

\[
(M_x/M_{x1})_{cm} + (M_y/M_{y1})_{cm} < or \leq 1.0
\]

\[
= 0.9919 < or = 1
\]

Hence the column is safe
Rectangular Short Column with Biaxial bending -  Bresler method

COLUMNS NO C4
Load Case 1.2*(DL + LL - EQX)
Grade of Concrete M30
Grade of Steel Fe415
Characteristic compressive strength of concrete, fck (N/mm²) 30
Characteristic yield strength of steel, fy (N/mm²) 415
Unit weight of concrete, γc (kN/m³) 25
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition (mm) 40
Assumed effective cover all around, d' (mm) 60

Dimensions of the Column

Unsupported length of column, L = 3600 mm
Least lateral dimension = 350 mm
Breadth of the column B (mm) = 350
Depth of the Column D (mm) = 500

Effective length of the column, lex, (m) = 2.34
Effective length of the column, ley, (m) = 2.34

Check for Slenderness ratio, L/D

Slenderness ratio, λex = 6.69 <12 column is Short
Slenderness ratio, λey = 4.68 <12 column is Short

Design Factors

Factored load, Pu = 226.3 KN
Factored moment acting parallel to the larger dimension, Mux = 138.53 KN-m
Factored moment acting parallel to the shorter dimension, Muy = 192.01 KN-m

1 Check for accidental eccentricity
   Equivalent eccentricity of loads is given by
   \[
   \frac{M_{ux}}{P_u} = 612.15 \text{ mm}
   \]
   \[
   \frac{M_{uy}}{P_u} = 397.70 \text{ mm}
   \]
   Both are more than 20mm minimum

2 Assume percentage of steel
   (assuming steel larger than required by P and Mx)
   \[
   \frac{M_x}{f_{ck} \times b \times D^2} = 0.05
   \]
   \[
   d'/D = 0.1
   \]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.04
\]

From SP16 chart44
\[
\frac{P}{f_{ck}} = 0.07 \quad \text{From table}
\]
Assuming a higher value \(P/f_{ck}\)
\[
\text{Assumed } , P = 0.105 \quad \text{per cent}
\]
Area of steel, \(A_s\)
\[
\text{Area of steel, } A_s = 5512.50 \text{ mm}^2
\]
Use 12 no.s of 25 mm
\[
\text{Area of steel provided } = 5888 \text{ mm}^2
\]

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis
\[
d'/D = 0.12
\]
\[
P/f_{ck} \times b \times D^2 = 0.04
\]
\[
P/f_{ck} = 0.06
\]
\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.13 \quad \text{From table}
\]
\[
M_{x1} = 341.25 \text{ KN-m}
\]

About Y-axis
\[
d'/D = 0.17
\]
\[
P/f_{ck} \times b \times D^2 = 0.04
\]
\[
P/f_{ck} = 0.105
\]
\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.13 \quad \text{From table}
\]
\[
M_{y1} = 238.88 \text{ KN-m}
\]

4 Calculate \(\alpha^n\)
\[
P_z = 0.45f_{ck}A_s + 0.75f_yA_s
\]
\[
P_z = 4195 \text{ KN}
\]
\[
P/P_z = 0.05
\]
By formula
\[
\alpha^n = 2/3[1+5/2 \times P/P_z]
\]
\[
\alpha^n = 0.76
\]

5 Criteria for biaxial bending
\[
(M_x/M_{x1})^{cm} + (M_y/M_{y1})^{cm} < or =1.0
\]
\[
= 0.9799 < or = 1
\]
Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

**COLUMN NO** C5

**Load Case** 1.5*(DL - EQX)

Grade of Concrete **M30**

Grade of Steel **Fe415**

Characteristic compressive strength of concrete, $f_{ck}$ (N/mm$^2$) **30**

Characteristic yield strength of steel, $f_y$ (N/mm$^2$) **415**

Unit weight of concrete, $\gamma_c$ (kN/m$^3$) **25**

Partial safety factor for concrete **1.5**

Exposure condition **Mild**

Nominal Cover to exposure condition (mm) **40**

Assumed effective cover all around, $d'$ (mm) **50**

**Dimensions of the Column**

Unsupported length of column, $L$ = **3600** mm

Least lateral dimension = **350** mm

Breadth of the column, $B$ (mm) = **350**

Depth of the Column, $D$ (mm) = **600**

Effective length of the column, $l_{ex}$ (m) = **2.34**

Effective length of the column, $l_{ey}$ (m) = **2.34**

**Check for Slenderness ratio, L/D**

Slenderness ratio, $\lambda_{ex}$ = **6.69** <12 column is Short

Slenderness ratio, $\lambda_{ey}$ = **3.90** <12 column is Short

**Design Factors**

Factored load, $P_u$ = **830.8** kN

Factored moment acting parallel to the larger dimension, $M_{ux}$ = **59.63** kN-m

Factored moment acting parallel to the shorter dimension, $M_{uy}$ = **224.86** kN-m

1. Check for accidental eccentricity

   Equivalent eccentricity of loads is given by

   $M_{ux}/P_u = 71.77$ mm

   $M_{uy}/P_u = 108.33$ mm

   Both are more than 20mm minimum

2. Assume percentage of steel

   (assuming steel larger than required by P and $M_u$)

   \[
   \frac{d'}{D} = 0.1
   \]

   \[
   \frac{M_u}{f_{ck} \times b \times D^2} = 0.02
   \]
\[ \frac{P_u}{f_{ck} \times b \times D} = 0.13 \]

From SP16 chart 44
\[ \frac{P}{f_{ck}} = 0.05 \quad \text{From table} \]

Assuming a higher value \( P/f_{ck} \)

Assumed, \( P = 2.25 \) per cent

Area of steel, \( A_s = 4725.00 \text{ mm}^2 \)

Use 10 no.s of 25 mm

Area of steel provided = 4906 mm²

3 Find the moment capacities \( M_{x1} \) and \( M_{y1} \)

About X-axis
\[ d'/D = 0.08 \]
\[ \frac{P}{f_{ck} \times b \times D^2} = 0.13 \]
\[ \frac{P}{f_{ck}} = 0.075 \]

\[ M_{x1}/(f_{ck} \times b \times D^2) = 0.12 \quad \text{From table} \]

\( M_{x1} = 453.60 \text{ KN-m} \)

About Y-axis
\[ d'/D = 0.14 \]
\[ \frac{P}{f_{ck} \times b \times D^2} = 0.13 \]
\[ \frac{P}{f_{ck}} = 0.075 \]

\[ M_{y1}/(f_{ck} \times D \times b^2) = 0.12 \quad \text{From table} \]

\( M_{y1} = 264.60 \text{ KN-m} \)

4 Calculate \( \phi^n \)

\[ P_z = 0.45f_{ck}A_s + 0.75f_YA_s \]

\[ Pz = 4362 \text{ KN} \]

\[ P/P_z = 0.19 \]

By formula
\[ \phi^n = 2/3[1+5/2 \times P/Pz] \]
\[ \phi^n = 0.99 \]

5 Criteria for biaxial bending
\[ (M_x/M_{x1})^{cm} + (M_y/M_{y1})^{cm} < \text{or} = 1.0 \]

\[ = 0.4786 < \text{or} = 1 \]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

<table>
<thead>
<tr>
<th>COLUMN NO</th>
<th>C6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case</td>
<td>1.5*(DL - EQX)</td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete, $f_{ck}$ (N/mm²)</td>
<td>30</td>
</tr>
<tr>
<td>Characteristic yield strength of steel, $f_y$ (N/mm²)</td>
<td>415</td>
</tr>
<tr>
<td>Unit weight of concrete, $\gamma_c$ (kN/m³)</td>
<td>25</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Exposure condition</td>
<td>Mild</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition (mm)</td>
<td>40</td>
</tr>
<tr>
<td>Assumed effective cover all around, $d'$ (mm)</td>
<td>50</td>
</tr>
</tbody>
</table>

Dimensions of the Column

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsupported length of column, L</td>
<td>3600 mm</td>
</tr>
<tr>
<td>Least lateral dimension</td>
<td>400 mm</td>
</tr>
<tr>
<td>Breadth of the column B (mm)</td>
<td>400</td>
</tr>
<tr>
<td>Depth of the Column D (mm)</td>
<td>900</td>
</tr>
</tbody>
</table>

Effective length of the column, $l_{ex}$ (m) = 2.34
Effective length of the column, $l_{ey}$ (m) = 2.34

Check for Slenderness ratio, L/D

\[
\lambda_{ex} = \frac{L}{D} = 5.85 < 12 \text{ column is Short}
\]
\[
\lambda_{ey} = \frac{L}{B} = 2.60 < 12 \text{ column is Short}
\]

Design Factors

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored load, $P_u$</td>
<td>560.34 KN</td>
</tr>
<tr>
<td>Factored moment acting parallel to the larger dimension, $M_{ux}$</td>
<td>563.33 KN-m</td>
</tr>
<tr>
<td>Factored moment acting parallel to the shorter dimension, $M_{uy}$</td>
<td>210.34 KN-m</td>
</tr>
</tbody>
</table>

1 Check for accidental eccentricity
Equivalent eccentricity of loads is given by
\[
\frac{M_{ux}}{P_u} = 1005.34 \text{ mm} \\
\frac{M_{uy}}{P_u} = 160.62 \text{ mm}
\]
Both are more than 20mm minimum

2 Assume percentage of steel
(assuming steel larger than required by P and $M_x$)
\[
d'/D = 0.1
\]
\[
\frac{M_x}{f_{ck} \times b \times D^2} = 0.06
\]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.05
\]

From SP16 chart 44

\[
\frac{P}{f_{ck}} = 0.045 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\)

Assumed, \(P\) = 0.0675

Area of steel, \(A_s\) = 7290.00 mm²

Use 16 no.s of 25 mm

Area of steel provided = 7850 mm²

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis
\[d'/D = 0.06\]
\[P/f_{ck} \times b \times D^2 = 0.05\]
\[P/f_{ck}\]

\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.12 \quad \text{From table}
\]

\[M_{x1} = 1166.40 \text{ KN-m}\]

About Y-axis
\[d'/D = 0.13\]
\[P/f_{ck} \times b \times D^2 = 0.05\]
\[P/f_{ck}\]

\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.12 \quad \text{From table}
\]

\[M_{y1} = 518.40 \text{ KN-m}\]

4 Calculate \(\alpha^n\)

\[P_z = 0.45f_{ck}A_s + 0.75f_iA_s\]

\[P_z = 7303 \text{ KN}\]

\[P/P_z = 0.08\]

By formula
\[\alpha^n = 2/3[1+5/2 \times P/P_z]\]

\[\alpha^n = 0.80\]

5 Criteria for biaxial bending

\[
(M_x/M_{x1})^{bn} + (M_y/M_{y1})^{bn} < or = 1.0
\]

\[= 0.8063 < or = 1\]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO  C7
Load Case  1.5*(DL - EQX)
Grade of Concrete  M30
Grade of Steel  Fe415
Characteristic compressive strength of concrete , f_{ck} ( N/mm² )  30
Characteristic yield strength of steel , f_y ( N/mm² )  415
Unit weight of concrete , \( \gamma_c \) ( kN/m³ )  25
Partial safety factor for concrete  1.5
Exposure condition  Mild
Nominal Cover to exposure condition( mm )  40
Assumed effective cover all around , \( d' \) ( mm )  60

Dimensions of the Column

Unsupported length of column, L = 3600 mm
Least lateral dimension = 800 mm
Breadth of the column B (mm) = 800
Depth of the Column D (mm) = 800

Effective length of the column , \( l_{ex} \) , ( m ) = 2.34
Effective length of the column , \( l_{ey} \) , ( m ) = 2.34

Check for Slenderness ratio, L/D

\[
\begin{align*}
\text{Slenderness ratio , } \lambda_{ex} &= 2.93 < 12 \text{ column is Short} \\
\text{Slenderness ratio , } \lambda_{ey} &= 2.93 < 12 \text{ column is Short}
\end{align*}
\]

Design Factors

Factored load, \( P_u \) = 4861.81 KN
Factored moment acting parallel to the larger dimension , \( M_{ux} \) = 1817.69 KN-m
Factored moment acting parallel to the shorter dimension, \( M_{uy} \) = 0.23 KN-m

1 Check for accidental eccentricity
Equivalent eccentricity of loads is given by
\[
\begin{align*}
\frac{M_{ux}}{P_u} &= 373.87 \text{ mm} \\
\frac{M_{uy}}{P_u} &= 18.51 \text{ mm}
\end{align*}
\]
Both are more than 20mm minimum

2 Assume percentage of steel
( assuming steel larger than required by P and \( M_{x} \) )
\[
\begin{align*}
d' / D &= 0.1 \\
\frac{M_{x}}{f_{ck} x b x D^2} &= 0.12
\end{align*}
\]
\[ \frac{P_u}{f_{ck} \times b \times D} = 0.25 \]

From SP16 chart 44

\[ \frac{P}{f_{ck}} = 0.055 \quad \text{From table} \]

Assuming a higher value \( P/f_{ck} \)

Assumed, \( P = 2.48 \) per cent

Area of steel, \( A_s \) = 15840.00 mm²

Use 20 no.s of 32 mm

Area of steel provided = 16077 mm²

3 Find the moment capacities \( M_{x1} \) and \( M_{y1} \)

About X-axis
\[ \frac{d}{D} = 0.08 \]
\[ \frac{P}{f_{ck} \times b \times D^2} = 0.25 \]
\[ \frac{P}{f_{ck}} = 0.0825 \]

\[ M_{x1}/(f_{ck} \times b \times D^2) = 0.13 \quad \text{From table} \]

\( M_{x1} = 1996.80 \) KN-m

About Y-axis
\[ \frac{d}{D} = 0.08 \]
\[ \frac{P}{f_{ck} \times b \times D^2} = 0.25 \]
\[ \frac{P}{f_{ck}} = 0.0825 \]

\[ M_{y1}/(f_{ck} \times D \times b^2) = 0.13 \quad \text{From table} \]

\( M_{y1} = 1996.80 \) KN-m

4 Calculate \( \alpha^n \)
\[ P_z = 0.45f_{ck}A_s + 0.75f_yA_s \]
\[ P_z = 13644 \text{ KN} \]
\[ P/P_z = 0.36 \]

By formula
\[ \alpha^n = 2/3[1+5/2 \times P/P_z] \]
\[ \alpha^n = 1.27 \]

5 Criteria for biaxial bending
\[ (M_x/M_{x1})^{cm} + (M_y/M_{y1})^{cm} < or = 1.0 \]
\[ = 0.9075 < or = 1 \]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

COLUMNS NO C8

Load Case 1.5*(DL - EQX)
Grade of Concrete M30
Grade of Steel Fe415

Characteristic compressive strength of concrete, \( f_{ck} \) (N/mm²) 30
Characteristic yield strength of steel, \( f_y \) (N/mm²) 415
Unit weight of concrete, \( \gamma_c \) (kN/m³) 25
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition (mm) 40
Assumed effective cover all around, \( d' \) (mm) 50

Dimensions of the Column

Unsupported length of column, \( L \) = 3600 mm
Least lateral dimension = 300 mm
Breadth of the column \( B \) (mm) = 300
Depth of the Column \( D \) (mm) = 800

Effective length of the column, \( l_{ex} \) (m) = 2.34
Effective length of the column, \( l_{ey} \) (m) = 2.34

Check for Slenderness ratio, \( L/D \)

\[
\lambda_{ex} = \frac{L}{D} = 7.80 < 12 \quad \text{column is Short}
\]

\[
\lambda_{ey} = \frac{L}{D} = 2.93 < 12 \quad \text{column is Short}
\]

Design Factors

Factored load, \( P_u \) = 895 KN
Factored moment acting parallel to the larger dimension, \( M_{ux} \) = 324 KN-m
Factored moment acting parallel to the shorter dimension, \( M_{uy} \) = 240.15 KN-m

1 Check for accidental eccentricity

Equivalent eccentricity of loads is given by

\[
\frac{M_{ux}}{P_u} = 362.01 \text{ mm}
\]

\[
\frac{M_{uy}}{P_u} = 100.56 \text{ mm}
\]

Both are more than 20 mm minimum

2 Assume percentage of steel

(assuming steel larger than required by \( P \) and \( M_x \))

\[
d'/D = 0.1
\]

\[
\frac{M_x}{f_{ck} \times b \times D^2} = 0.06
\]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.12
\]

From SP16 chart44

\[
\frac{P}{f_{ck}} = 0.05 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\)

\[
\text{Assumed , } P = 0.075 \quad \text{per cent}
\]

Area of steel, \(A_s\)

\[
= 5400.00 \quad \text{mm}^2
\]

Use 12 no.s of 25 mm

Area of steel provided

\[
= 5888 \quad \text{mm}^2
\]

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis

\[
d'/D = 0.06
\]

\[
P/f_{ck} \times b \times D^2 = 0.12
\]

\[
P/f_{ck} = 0.075
\]

\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.12 \quad \text{From table}
\]

\[
M_{x1} = 691.20 \quad \text{KN-m}
\]

About Y-axis

\[
d'/D = 0.17
\]

\[
P/f_{ck} \times D \times b^2 = 0.12
\]

\[
P/f_{ck} = 0.075
\]

\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.12 \quad \text{From table}
\]

\[
M_{y1} = 259.20 \quad \text{KN-m}
\]

4 Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_s + 0.75f_{ck}A_s
\]

\[
P_z = 5072 \quad \text{KN}
\]

\[
P/P_z = 0.18
\]

By formula

\[
\alpha^n = 2/3[1+5/2 \times P/P_z]
\]

\[
\alpha^n = 0.97
\]

5 Criteria for biaxial bending

\[
(M_x/M_{x1})^{cm} + (M_y/M_{y1})^{cm} < 1.0
\]

\[
= 0.8413 < \text{or} = 1
\]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO C9
Load Case 1.5*(DL - EQX)
Grade of Concrete M30
Grade of Steel Fe415
Characteristic compressive strength of concrete , $f_{ck}$ (N/mm$^2$) 30
Characteristic yield strength of steel , $f_y$ (N/mm$^2$) 415
Unit weight of concrete , $\gamma_c$ (kN/m$^3$) 25
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition( mm ) 40
Assumed effective cover all around , d’ ( mm ) 50

Dimensions of the Column

Unsupported length of column, L = 3600 mm
Least lateral dimension = 300 mm
Breadth of the column B (mm) = 300
Depth of the Column D (mm) = 400

Effective length of the column , $l_{ex}$ ( m ) = 2.34
Effective length of the column , $l_{ey}$ ( m ) = 2.34

Check for Slenderness ratio, L/D

Slenderness ratio , $\lambda_{ex}$ = 7.80 <12 column is Short
Slenderness ratio , $\lambda_{ey}$ = 5.85 <12 column is Short

Design Factors

Factored load, $P_u$ = 717.13 KN
Factored moment acting parallel to the larger dimension , $M_{ux}$ = 112.72 KN-m
Factored moment acting parallel to the shorter dimension, $M_{uy}$ = 60.37 KN-m

1 Check for accidental eccentricity
   Equivalent eccentricity of loads is given by
   $M_{ux}/P_u$ = 157.18 mm
   $M_{uy}/P_u$ = 125.50 mm
   Both are more than 20mm minimum

2 Assume percentage of steel
   ( assuming steel larger than required by $P$ and $M_x$)

\[
\frac{d'/D}{f_{ck} \times b \times D^2} = 0.1
\]
\[
\frac{M_x}{f_{ck} \times b \times D^2} = 0.08
\]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.20
\]

From SP16 chart44

\[
\frac{P}{f_{ck}} = 0.07 \quad \text{From table}
\]

Assuming a higher value \(P/f_{ck}\) = 0.105

Assumed, \(P = 3.15\) per cent

Area of steel, \(A_s\) = 3780.00 mm\(^2\)

Use 8 no.s of 25 mm

Area of steel provided = 3925 mm\(^2\)

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis

\[
d'/D = 0.13
\]

\[
P/f_{ck} \times b \times D^2 = 0.20
\]

\[
P/f_{ck} = 0.105
\]

\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.07 \quad \text{From table}
\]

\[
M_{x1} = 100.80\ \text{KN-m}
\]

About Y-axis

\[
d'/D = 0.17
\]

\[
P/f_{ck} \times b \times D^2 = 0.20
\]

\[
P/f_{ck} = 0.105
\]

\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.07 \quad \text{From table}
\]

\[
M_{y1} = 75.60\ \text{KN-m}
\]

4 Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_s + 0.75f_yA_s
\]

\[
P_z = 2842\ \text{KN}
\]

\[
P/P_z = 0.25
\]

By formula

\[
\alpha^n = 2/3[1 + 5/2 \times P/P_z]
\]

\[
\alpha^n = 1.09
\]

5 Criteria for biaxial bending

\[
(M_x/M_{x1})^{0.4} + (M_y/M_{y1})^{0.4} \leq 1.0
\]

\[
= 0.9000 \quad \text{or} \quad = 1
\]

Hence the column is safe
Rectangular Short Column with Biaxial bending - Bresler method

**COLUMN NO** C10
Load Case 1.5*(DL - EQX)
Grade of Concrete M30
Grade of Steel Fe415
Characteristic compressive strength of concrete, $f_{ck}$ (N/mm²) 30
Characteristic yield strength of steel, $f_y$ (N/mm²) 415
Unit weight of concrete, $\gamma_c$ (kN/m³) 25
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition (mm) 40
Assumed effective cover all around, $d'$ (mm) 50

**Dimensions of the Column**

Unsupported length of column, $L$ = 3600 mm
Least lateral dimension = 550 mm
Breadth of the column $B$ (mm) = 550
Depth of the Column $D$ (mm) = 850

Effective length of the column, $l_{ex}$ (m) = 2.34
Effective length of the column, $l_{ey}$ (m) = 2.34

**Check for Slenderness ratio, $L/D$**

Slenderness ratio, $\lambda_{ex}$ = 4.25 <12 column is Short
Slenderness ratio, $\lambda_{ey}$ = 2.75 <12 column is Short

**Design Factors**

Factored load, $P_u$ = 3015.88 KN
Factored moment acting parallel to the larger dimension, $M_{ux}$ = 918.44 KN-m
Factored moment acting parallel to the shorter dimension, $M_{uy}$ = 2 KN-m

1 Check for accidental eccentricity
Equivalent eccentricity of loads is given by

\[
\frac{M_{ux}}{P_{u}} = 304.53 \text{ mm} \\
\frac{M_{uy}}{P_{u}} = 29.84 \text{ mm}
\]
Both are more than 20mm minimum

2 Assume percentage of steel
(assuming steel larger than required by $P$ and $M_u$)

\[
\frac{d'}{D} = 0.1 \\
\frac{M_u}{f_{ck} \times b \times D^2} = 0.08
\]
\[
\frac{P_u}{f_{ck} \times b \times D} = 0.22
\]

From SP16 chart 44
\[
\frac{P}{f_{ck}} = 0.05 \quad \text{From table}
\]
Assuming a higher value \(P/f_{ck}\)
\[
\text{Assumed }, P = 0.075
\]

Area of steel, \(A_s\)
\[
= 10518.75 \text{ mm}^2
\]

Use 22 no.s of 25 mm

Area of steel provided
\[
= 10794 \text{ mm}^2
\]

3 Find the moment capacities \(M_{x1}\) and \(M_{y1}\)

About X-axis
\[
d'/D = 0.06
\]
\[
\frac{P/f_{ck} \times b \times D^2}{f_{ck}} = 0.22
\]
\[
P/f_{ck} = 0.075
\]
\[
M_{x1}/(f_{ck} \times b \times D^2) = 0.125 \quad \text{From table}
\]

\[
M_{x1} = 1490.16 \text{ KN-m}
\]

About Y-axis
\[
d'/D = 0.09
\]
\[
\frac{P/f_{ck} \times b \times D^2}{f_{ck}} = 0.22
\]
\[
P/f_{ck} = 0.075
\]
\[
M_{y1}/(f_{ck} \times D \times b^2) = 0.125 \quad \text{From table}
\]

\[
M_{y1} = 964.22 \text{ KN-m}
\]

4 Calculate \(\alpha^n\)

\[
P_z = 0.45f_{ck}A_s + 0.75f_yA_s
\]
\[
P_z = 9671 \text{ KN}
\]
\[
P/P_z = 0.31
\]

By formula
\[
\alpha^n = \frac{2}{3}[1+5/2 \times P/P_z]
\]
\[
\alpha^n = 1.19
\]

5 Criteria for biaxial bending

\[
(M_x/M_{x1})^{on} + (M_y/M_{y1})^{on} < or = 1.0
\]
\[
= 0.6207 \quad < or = 1
\]

Hence the column is safe
DESIGN OF BEAMS
**Beam PB1 Support**

**Design Parameters**

Load Case 14 \[1.5*(DL - EQX)\]
Grade of Concrete \[M30\]
Grade of Steel \[Fe415\]
Characteristic compressive strength of concrete , \(f_{ck}\) (N/mm²) \[30\]
Characteristic yield strength of steel , \(f_y\) (N/mm²) \[415\]
Unit weight of concrete , \(\gamma_c\) (kN/m³) \[24\]
Partial safety factor for concrete \[1.5\]
Exposure condition \[Mild\]
Nominal Cover to exposure condition( mm ) \[20\]

**Dimensions of the beam**

- C/C Span of the beam , \(l\) (m) \[10.80\]
- Breadth of the beam , \(b\) (mm) \[250\]
- Overall depth of the beam , \(D\) (mm) \[600\]

**Details of reinforcements**

- Diameter of tension reinforcement (mm) \[25\]
- Diameter of compression reinforcement (mm) \[25\]
- Diameter of stirrups (mm) \[8\]

**Effective depth**

- Effective depth , \(d\) (mm) \(= (600-20-8-25/2) = 560\)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment \[0\] kN-m
- Bending Moment \(M_u\) (kN-m) \[380\]
- Equivalent Bending Moment , \(M_e\) (kNm) \[380\]
- Shear force at critical distance , \(V_{ud}\) (kN) \[152\]
- Equivalent Shear (kN) \[152\]

**Singly reinforced or doubly reinforced section ?**

The limiting moment of resistance , \(M_{u,lim}\) is given by

\[
M_{ulim} = 0.362f_{ck} \times \frac{b x u_{max} \times 0.416 x u_{max}}{d}
\]

Where \(b = \) Breadth of the Section
\(x u_{max} = \) Limiting depth of Neutral Axis
\(d = \) Effective depth of the Section

The limiting percentage of steel , \(p_{lim}\) is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

For (M30 and Fe415)

\[
\frac{Pt}{100} \times \frac{A_{st}}{bd} \times \frac{f_{ck}}{2f_y} = 4.598 \times \frac{R}{f_{ck}}
\]

Where \( R = \frac{M_u}{bd^2} \)

\[
Pt_{lim} = 41.61 \times \frac{f_{ck} \times xu_{max}}{f_y \times d}
\]

Where \( f_{ck} = \) Characteristic Compressive strength of concrete

\( f_y = \) Characteristic strength of steel

The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[
Pt_{lim} = 41.61 \times \frac{f_{ck} \times xu_{max}}{f_y \times d}
\]

\[
M_{u,lim} = 0.1389 \times f_{ck} \times b \times d^2
\]

\[
xu_{max} / d = 0.48
\]

\[
M_{u,lim} = (0.1389 \times 30 \times 250 \times 559.5^2 / 1000000) = 326.11 \text{ kNm}
\]

\[
p_{lim} = (41.3 \times 30 / 415 \times 0.48) = 1.433
\]

If \( M_u > M_{u,lim} \), the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If \( M_u < M_{u,lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 380.00 \text{ kNm}
\]

\[
M_{u,lim} = 326.11 \text{ kNm}
\]

\[
\Rightarrow \text{section cannot be designed as singly reinforced. Doubly Reinforced Section Needed}
\]

**Determining \( A_{st} \)**

- Considering a 'balanced section' \((x_u = xu_{max})\)

\[
A_{st} = A_{st,lim} + \Delta A_{st}
\]

where \( A_{st,lim} = p_{lim} / 100 \times (b \times d) \)

\[
A_{st,lim} \approx (1.433 / 100 \times 250 \times 559.5) = 2004 \text{ mm}^2
\]

- Assuming 25 mm bars for compression steel,

\[
d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm}
\]
\[ A_{st} = \frac{M_u}{0.87 f_y d - d'} \]

\[ p_t \rightarrow R_{lim} = \frac{R}{0.87 f_y 1 - \frac{d'}{d}} \]

\[ M_u = 0.87 f_y^{**} A_{st} d (1 - (A_{st} f_y) / b * d * f_{ck}) \]

\[ A_{st} \text{ Reqd} = 2498 \text{ mm}^2 \]

\[ \therefore \text{ No of tension bars required ( # )} \]
\[ (2498 / (\pi / 4 x 25^2)) = 6.00 \]

Actual percentage of steel, \( p_t \) (%)
\[ (6 \times \pi / 4 \times 25^2 / 250 / 560 \times 100) = 2.11 \]

Actual area of steel, \( A_{st} \) (mm\(^2\))
\[ (6 \times \pi / 4 \times 25^2) = 2945 \]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 f_y A_{st}}{f_{sc} - 0.447 f_{ck}} \]

or

\[ p_c = \frac{0.87 f_y p_t}{f_{sc} - 0.447 f_{ck}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,max} \) for various \( d'/d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d'/d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,max} \), for \( d'/d = (40.5 / 559.5) = 0.072 \)

From the above table: by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,max} \), it suffices to establish \( p_c \geq p_{c} \).
where \( p_c^* \) is given by

\[
\frac{0.87 f_y}{f_{sc}} - \frac{0.447 f_{ck}}{p_{t} - p_{t,lim}}
\]

Actual \( p_t \) provided : \( p_t = 2.11 \)
Actual \( p_c \) provided : \( p_c = 0.35 \)

\[
\Rightarrow p_c^* = \left( \frac{0.87 \times 415 \times (2.106 - 1.433)}{354.73 - 0.447 \times 30} \right) \\
\Rightarrow p_c^* = 0.71
\]

**Section is over reinforced**

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting \( l/d \) ratios are specified by the Code (Cl. 23.2.1) as:

\[
\frac{l}{d_{max}} \leq \frac{l}{d_{basic}} F_1 F_2
\]

where
- \( \frac{l}{d_{basic}} \leq 7 \) for cantilever spans
- \( \frac{l}{d_{basic}} \leq 20 \) for simply supported spans
- \( \frac{l}{d_{basic}} \leq 26 \) for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor \( F \)

\[
F = \frac{10}{\text{span in metres}}
\]

The modification factors \( F_1 \) (which varies with \( p_t \) and \( f_{st} \)) and \( F_2 \) (which varies with \( p_c \)) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of \( f_{st} \) as follows:

\[
f_{st} = 0.58 f_y \times \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
\]

\[
\Rightarrow f_{st} = \left( 0.58 \times 415 \times 2292 / 2945 \right) = 187.32 \text{ N/mm}^2
\]

\[
F = 0.93 \hspace{1cm} F_1 = 0.77
\]
F_2 = 0.90

\[ \therefore \quad \frac{l}{d}_{\text{max}} = \left( 26 \times 0.93 \times 0.77 \times 0.9 \right) = 16.69 \]
\[ \left( \frac{l}{d} \right)_{\text{provided}} = 19.30 \]
\[ \Rightarrow \text{Not O.K} \]

**Check for shear**

Shear force at critical distance , \( V_{ud} \) (kN)

The critical section for shear is at a distance of 560 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress , \( \tau_v \)

\[
\left( \frac{152 \times 1000}{250 \times 560} \right) = 1.09 \text{ N/mm}^2
\]

The maximum shear stress is given by : \( Tc_{\text{max}} = 0.62 f_{ck} \)

\[
\Rightarrow \tau_{c,\text{max}} = (0.62 \times \text{Sqrt}(30)) = 3.40 \text{ N/mm}^2
\]
\[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section , \( A_{st} \) is given by 2945 mm²

Percentage of steel , \( p_t \) ( % ) 2.11

The design shear strength of the concrete , \( \tau_c \), is given by :

\[
\tau_c = 0.86 \left( \frac{0.8 f_{ck}}{0.89 p_t} \right) \left( 0.86 \times 250 \times 560 / 1000 \right) = 120 \text{ kN}
\]

- **Design of "vertical" stirrups**

The shear to be resisted by steel , \( V_{us} \) is given by : \( V_{us} = V_u - V_{uc} \)

\[ \Rightarrow \quad V_{us} = (152 - 120) = 32 \text{ kN} \]

Using 8 mm bars and 2

No of legs
Area of stirrups, \( A_s \) (mm\(^2\))

\[
\Rightarrow \text{required spacing } s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 560}{32 \times 1000} \right)
\]

\[
\Rightarrow \text{Spacing, } s_v = 635 \text{ mm}
\]

**Check whether** \( \tau_v > 0.5 \tau_c \)

Nominal shear stress, \( \tau_v \) (N/mm\(^2\))

Design shear stress, \( \tau_c \) (N/mm\(^2\))

\[
\tau_v > 0.5 \tau_c \quad \text{Yes}
\]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):

\[
\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5 t_c
\]

\[
s_v = \frac{2.175 f_y A_{sv}}{b}
\]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[
s_v = 0.75 d \quad 300 \text{ mm}
\]

**Code requirements for maximum spacing.**

i) \( < \left( \frac{2.175 \times 415 \times 101}{250} \right) = 363 \text{ mm} \)

ii) \( \leq \left( \frac{0.75 \times 559.5}{32} \right) = 420 \text{ mm} \)

iii) \( \leq 300 \text{ mm} \)

iv) \( \leq \left( \frac{0.87 \times 415 \times 101 \times 560}{32 \times 1000} \right) = 635 \text{ mm} \)
**Beam PB1 Mid Span**

**Design Parameters**

Load Case 14 \( [1.5 \cdot (DL - EQX)] \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete ( f_{ck} ) (N/mm(^2))</td>
<td>30</td>
</tr>
<tr>
<td>Characteristic yield strength of steel ( f_y ) (N/mm(^2))</td>
<td>415</td>
</tr>
<tr>
<td>Unit weight of concrete ( \gamma_c ) (kN/m(^3))</td>
<td>24</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Exposure condition</td>
<td>Mild</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition (mm)</td>
<td>20</td>
</tr>
</tbody>
</table>

**Dimensions of the beam**

- C/C Span of the beam, \( l \) (m): 4.96
- Breadth of the beam, \( b \) (mm): 250
- Overall depth of the beam, \( D \) (mm): 600

**Details of reinforcements**

- Diameter of tension reinforcement (mm): 25
- Diameter of compression reinforcement (mm): 25
- Diameter of stirrups (mm): 8

**Effective depth**

- Effective depth, \( d \) (mm): \((600 - 20 - 8 - 25/2) = 560\) mm

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>0 kN-m</td>
</tr>
<tr>
<td>Bending Moment ( M_u ) (kN-m)</td>
<td>289</td>
</tr>
<tr>
<td>Equivalent Bending Moment ( M_e ) (kNm)</td>
<td>289</td>
</tr>
<tr>
<td>Shear force at critical distance ( V_{ud} ) (kN)</td>
<td>179</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>179</td>
</tr>
</tbody>
</table>

**Singly reinforced or doubly reinforced section?**

The *limiting moment of resistance*, \( M_{u,\text{lim}} \) is given by

\[
M_{u,\text{lim}} = 0.362 f_{ck} \times \frac{b x u_{\text{max}}}{d} \times 0.416 x u_{\text{max}}
\]

Where
- \( b \) = Breadth of the Section
- \( x u_{\text{max}} \) = Limiting depth of Neutral Axis
- \( d \) = Effective depth of the Section

The limiting percentage of steel, \( p_{\text{lim}} \) is given by

-
The area of steel for a singly reinforced section with width, b and depth, d and ultimate moment, $M_u$ is given by:

$$ Pt_{\text{lim}} = 41.61 \frac{f_{ck}}{f_{y}} \frac{x_{u,\text{max}}}{d} $$

Where $f_{ck}$ = Characteristic Compressive strength of concrete  
$f_{y}$ = Characteristic strength of steel

The area of steel for a singly reinforced section with width, b and depth, d and ultimate moment, $M_u$ is given by:

$$ \frac{Pt}{100} = \frac{Ast}{bd} \frac{f_{ck}}{2f_{y}} = 4.598 \frac{R}{f_{ck}} $$

Where $R = \frac{M_u}{bd^2}$

For ( M30 and Fe415 )

$M_{u,\text{lim}} = 0.1389 \frac{f_{ck} b d^2}{d}$

$x_{u,\text{max}} / d = 0.48$

$\Rightarrow M_{u,\text{lim}} = (0.1389 \times 30 \times 250 \times 559.5^2 / 1000000) = 326.11 \text{ kNm}$

$\Rightarrow p_{u,\text{lim}} = (41.3 \times 30 / 415 \times 0.48) = 1.433$

If $M_u > M_{u,\text{lim}}$, the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If $M_u < M_{u,\text{lim}}$, the section can be designed as singly reinforced.

**Check for the type of section**

$M_u = 289.00 \text{ kNm}$

$M_{u,\text{lim}} = 326.11 \text{ kNm}$

$\Rightarrow$ Section can be designed as singly reinforced.

**Determining $A_{st}$**

- Considering a 'balanced section' ($x_u = x_{u,\text{max}}$)
  $$ A_{st} = A_{st,\text{lim}} + \Delta A_{st} $$

  where $A_{st,\text{lim}} = p_{u,\text{lim}} / 100 (b \times d)$

  $$ \Rightarrow A_{st,\text{lim}} = (1.433 / 100 \times 250 \times 559.5) = 2004 \text{ mm}^2$$

- Assuming 25 mm bars for compression steel,
  $$ d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm} $$
\[ A_{\text{d}} \quad \frac{M_u}{0.87 \, f_y} \quad \frac{M_{u,\text{lim}}}{d \, d'} \]

\[ p_t \quad \frac{R}{100} \quad \frac{R_{\text{lim}}}{0.87 \, f_y} \quad \frac{1}{d} \frac{d'}{d} \]

\[ M_u = 0.87 \, f_y \, A_{\text{st}} \, d \left(1 - (A_{\text{st}} \, f_y) / b \, d' \, f_{\text{ck}}\right) \]

\[ \text{Ast Reqd} = 1725 \quad \text{mm}^2 \]

\[ \therefore \text{No of tension bars required ( \# )} \]

\[ \frac{1725}{(\pi/4 \times 25^2)} = 4.00 \]

Actual percentage of steel, \( p_t \) (\%)

\[ \frac{4 \times \pi / 4 \times 25^2 / 250 / 560 \times 100}{4} = 1.40 \]

Actual area of steel, \( A_{\text{st}} \) (mm²)

\[ \frac{4 \times \pi / 4 \times 25^2}{250 / 560 \times 100} = 1963 \]

**Determining \( A_{\text{sc}} \)**

The compression steel, \( A_{\text{sc}} \), is given by

\[ A_{\text{sc}} = \frac{0.87 \, f_y}{f_{\text{sc}}} \quad \frac{A_{\text{d}}}{0.447 \, f_{\text{ck}}} \]

or

\[ p_c = \frac{0.87 \, f_y}{f_{\text{sc}}} \quad \frac{p_t}{0.447 \, f_{\text{ck}}} \quad \frac{P_{\text{lim}}}{p_t} \]

where \( f_{\text{sc}} \) is the stress in compression steel.

The values of \( f_{\text{sc}} \) (in MPa units) at \( x_u = x_{u,\text{max}} \) for various \( d'/d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d'/d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d'/d = \frac{40.5}{559.5} \) = 0.072

From the above table: by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_{c}^{*} \).
where $p_c^*$ is given by

$$
p_c^* = \frac{0.87 \cdot f_y}{f_{ck}} - \frac{0.447 \cdot f_{ck}}{p_t - p_{t,\text{lim}}}
$$

Actual $p_t$ provided: $p_t = 1.40$
Actual $p_c$ provided: $p_c = 0.35$

$$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (1.404 - 1.433))}{(354.73 - 0.447 \times 30)}$$
$$\Rightarrow p_c^* = -0.03$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} \cdot F_1 \cdot F_2$$

where

- $l/d_{\text{basic}} \geq 7$ for cantilever spans
- $l/d_{\text{basic}} \geq 20$ for simply supported spans
- $l/d_{\text{basic}} \geq 26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$.

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

$$f_{st} = 0.58 \cdot f_y \cdot \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 1806)}{1963} = 221.45 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 0.87$
\( F_2 = 0.90 \)

\[
\therefore \quad \frac{l}{d}_{\text{max}} = \left( \frac{26 \times 1 \times 0.87 \times 0.9}{20.28} \right) = 8.87
\]

\( \left( \frac{l}{d} \right)_{\text{provided}} = \)

\( \Rightarrow \text{Hence O.K.} \)

**Check for shear**

Shear force at critical distance , \( V_{ud} \) ( kN )

The critical section for shear is at a distance of 560 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress , \( \tau_v \)

\[
\left( \frac{179 \times 1000}{(250 \times 560)} \right) = 1.28 \text{ N/mm}^2
\]

The maximum shear stress is given by:

\[
\tau_{c,max} = (0.62 \times \text{sqrt}(30)) = 3.40 \text{ N/mm}^2
\]

\( \Rightarrow \text{Adopted section is adequate} \)

- **Design shear resistance at critical section**

At critical section , \( A_s \) is given by 1963 mm²

Percentage of steel , \( p_t (\%) = 1.40 \)

The design shear strength of the concrete , \( \tau_c \), is given by:

\[
\tau_c = \frac{0.8 f_{ck}}{6} \left( 1 + \frac{5}{1} \right)
\]

where

\[
\frac{0.8 f_{ck}}{6.89 p_t} \quad \text{whichever is greater}
\]

For (M30 and Fe415)

\( \Rightarrow \tau_c = 0.74 \text{ N/mm}^2 \)

\( \Rightarrow V_{uc} = \left( 0.74 \times 250 \times 560 / 1000 \right) = 104 \text{ kN} \)

- **Design of "vertical" stirrups**

The shear to be resisted by steel , \( V_{us} \) is given by:

\[
V_{us} = V_u \cdot V_{uc}
\]

\( \Rightarrow V_{us} = (179 - 104) = 75 \text{ kN} \)

Using 8 mm bars and

No of legs 2
Area of stirrups, $A_{sv}$ (mm$^2$) = 101

$\Rightarrow$ required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 560}{74.88 \times 1000} \right)$

$\Rightarrow$ Spacing, $s_v = 271$ mm

**Check whether** $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm$^2$) = 1.28
Design shear stress, $\tau_c$ (N/mm$^2$) = 0.74

$\tau_v > 0.5 \tau_c \quad Yes$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5 t_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v < 0.75 d$
$s_v = 300$ mm

**Code requirements for maximum spacing:**

i) $s_v < \left( \frac{2.175 \times 415 \times 101}{250} \right) = 363$ mm
ii) $s_v \leq \left( \frac{0.75 \times 559.5}{1000} \right) = 420$ mm
iii) $s_v \leq 300$ mm
iv) $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 560}{74.88 \times 1000} \right) = 271$ mm
**Beam B1 Support**

**Design Parameters**

Load Case 13 \[1.5*(DL + EQX)\]

Grade of Concrete \( \text{M30} \)

Grade of Steel \( \text{Fe}415 \)

Characteristic compressive strength of concrete, \( f_{ck} \ (\text{N/mm}^2) \) \( 30 \)

Characteristic yield strength of steel, \( f_y \ (\text{N/mm}^2) \) \( 415 \)

Unit weight of concrete, \( \gamma_c \ (\text{kN/m}^3) \) \( 24 \)

Partial safety factor for concrete \( 1.5 \)

Exposure condition \( \text{Mild} \)

Nominal Cover to exposure condition( mm ) \( 20 \)

**Dimensions of the beam**

- C/C Span of the beam, \( l \ (\text{m}) \) \( 4.96 \)
- Breadth of the beam, \( b \ (\text{mm}) \) \( 400 \)
- Overall depth of the beam, \( D \ (\text{mm}) \) \( 900 \)

**Details of reinforcements**

- Diameter of tension reinforcement ( mm ) \( 25 \)
- Diameter of compression reinforcement ( mm ) \( 25 \)
- Diameter of stirrups ( mm ) \( 8 \)

**Effective depth**

Effective depth, \( d \ (\text{mm}) \) \( (900-20-8-25/2) = 860 \)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment \( 150 \ \text{kN-m} \)
- Bending Moment \( M_u (\text{kN-m}) \) \( 1053 \)
- Equivalent Bending Moment, \( M_e \ (\text{kNm}) \) \( 1340 \)
- Shear force at critical distance, \( V_ud \ (\text{kN}) \) \( 233 \)
- Equivalent Shear (kN) \( 833 \)

**Singly reinforced or doubly reinforced section**

The limiting moment of resistance, \( M_{ulim} \) is given by

\[
M_{ulim} = 0.362fck \times \frac{bxu_{\text{max}}}{d} \times 0.416xu_{\text{max}}
\]

Where \( b = \) Breadth of the Section

\( xu_{\text{max}} = \) Limiting depth of Neutral Axis

\( d = \) Effective depth of the Section

The limiting percentage of steel, \( p_{ulim} \) is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

For (M30 and Fe415) 
\[
x_{u,\text{max}} / d = 0.48
\]
\[
\Rightarrow M_u,\text{lim} = \left( 0.1389 \times 30 \times 400 \times 859.5^2 / 1000000 \right) = 1,231.33 \text{ kNm}
\]
\[
\Rightarrow p_{t,\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433
\]

If \( M_u > M_u,\text{lim} \), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \( M_u < M_u,\text{lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_d = 1,339.76 \text{ kNm}
\]
\[
M_u,\text{lim} = 1,231.33 \text{ kNm}
\]

\[
\Rightarrow \text{Section cannot be designed as singly reinforced. Doubly Reinforced Section Needed}
\]

**Determining \( A_{st} \)**

- Considering a 'balanced section' \( x_u = x_{u,\text{max}} \)

\[
A_{st} = A_{st,\text{lim}} + \Delta A_{st}
\]

where \( A_{st,\text{lim}} = p_{t,\text{lim}} / 100 \times (b \times d) \)

\[
\Rightarrow A_{st,\text{lim}} \left( 1.433 / 100 \times 400 \times 859.5 \right) = 4927 \text{ mm}^2
\]

- Assuming 25 mm bars for compression steel,

\[
d' \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}
\]
\[\begin{align*}
A_{st} & = \frac{M_u}{0.87 f_y d d'} M_{ulim} \\
\rho_t & = \frac{R}{100} \frac{R_{lim}}{0.87 f_y} \\
Mu & = 0.87 f_y^{*} A{*}\ast d(1-(A{*}\ast f_y)/b*d*fck)
\end{align*}\]

\[\text{Ast Req'd} = 5562 \text{ mm}^2\]

\[\therefore \text{ No of tension bars required ( # ) } \]
\[\left( \frac{5562}{\pi/4 \times 25^2} \right) = 12.00\]

\[\text{Actual percentage of steel , } \rho_t ( \% ) \]
\[\left( \frac{12 \times \pi/4 \times 25^2}{400 \times 860 \times 100} \right) = 1.71\]

\[\text{Actual area of steel , } A_{st} ( \text{ mm}^2 ) \]
\[\left( \frac{12 \times \pi/4 \times 25^2}{400 \times 860 \times 100} \right) = 5890\]

**Determining \( A_{sc} \)**

The compression steel , \( A_{sc} \), is given by

\[\begin{align*}
A_{sc} & = \frac{0.87 f_y A_{st}}{f_{sc} 0.447 f_{ck}} \\
\text{or } \rho_c & = \frac{0.87 f_y \rho_t}{f_{sc} 0.447 f_{ck}}
\end{align*}\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d' / d = \left( \frac{40.5}{859.5} \right) = 0.047 \)
  - From the above table : by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( \rho_c \geq \rho_{c,\text{c}} \)
where $p_c^*$ is given by

\[
p_c^* = \frac{0.87 f_y}{f_{y,c}} \frac{I}{d} \left( \frac{p_t - p_{\text{lim}}}{0.447 f_{ck}} \right)
\]

Actual $p_t$ provided: $p_t = 1.71$
Actual $p_c$ provided: $p_c = 0.14$

⇒ $p_c^* = \frac{0.87 \times 415 \times (1.713 - 1.433)}{(354.98 - 0.447 \times 30)}$
⇒ $p_c^* = 0.30$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

\[
\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2
\]

where $\frac{l}{d_{\text{basic}}}$

- 7 for cantilever spans
- 20 for simply supported spans
- 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

\[
F = \frac{10}{\text{span in metres}}
\]

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

\[
f_{st} = \frac{0.58 f_y}{\frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}}
\]

⇒ $f_{st} = \frac{(0.58 \times 415 \times 5294)}{5890} = 216.31$ N/mm²

$F = 1.00$
$F_1 = 0.79$
\[
F_2 = 0.55
\]

\[
\therefore \frac{l}{d}_{\text{max}} = \frac{26 \times 1 \times 0.79 \times 0.55}{5.77} = 11.30
\]

\[
\frac{l}{d}_{\text{provided}} =
\]

\[\Rightarrow \text{Hence O.K.}\]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)

  \[
  \left( \frac{833 \times 1000}{400 \times 860} \right) = 2.42 \, \text{N/mm}^2
  \]

  The maximum shear stress is given by:

  \[
  T_{c_{\text{max}}} = 0.62 f'ck
  \]

  \[
  \Rightarrow \tau_{c_{\text{max}}} = (0.62 \times \text{sqrt}(30)) = 3.40 \, \text{N/mm}^2
  \]

  \[\Rightarrow \text{Adopted section is adequate}\]

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by

  \[
  5890 \, \text{mm}^2
  \]

  Percentage of steel, \( p_t \) (\%)

  \[
  1.71
  \]

  The design shear strength of the concrete, \( \tau_c \), is given by:

  \[
  \tau_c = \frac{0.85}{6} \cdot 0.8 f'ck - 1 \cdot 1
  \]

  \[
  \text{where } \frac{0.8 f'ck}{6.89 p_t} \text{ whichever is greater } 1
  \]

  For (M30 and Fe415)

  \[
  \Rightarrow \tau_c = 0.80 \, \text{N/mm}^2
  \]

  \[
  \Rightarrow V_{uc} = \left(0.8 \times 400 \times 860 / 1000\right) = 275 \, \text{kN}
  \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:

  \[
  V_{us} = V_u - V_{uc}
  \]

  \[
  \Rightarrow V_{us} = (833 - 275) = 558 \, \text{kN}
  \]

  Using 12 mm bars and

  No of legs 4
Area of stirrups, \( A_{sv} \) \( (\text{mm}^2) \) 452

\[ \Rightarrow \text{required spacing } s_v \leq \left( \frac{0.87 \times 415 \times 452 \times 860}{558.2 \times 1000} \right) \]

\[ \Rightarrow \text{Spacing, } s_v = 251 \text{ mm} \]

*Check whether \( \tau_v > 0.5 \tau_c \)*

Nominal shear stress, \( \tau_v \) \( (\text{N/mm}^2) \) 2.42
Design shear stress, \( \tau_c \) \( (\text{N/mm}^2) \) 0.80

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):

\[ \frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 \cdot f_y} \quad \text{When } s_v = 0.5 \tau_c \]

\[ s_v = \frac{2.175 \cdot f_y \cdot A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v \leq 0.75 \cdot d \quad \text{300 mm} \]

Code requirements for maximum spacing:

i) \( s_v \leq \left( \frac{2.175 \times 415 \times 452}{400} \right) = 1021 \text{ mm} \)
ii) \( s_v \leq \left( \frac{0.75 \times 859.5}{300} \right) = 645 \text{ mm} \)
iii) \( s_v \leq 300 \text{ mm} \)
iv) \( s_v \leq \left( \frac{0.87 \times 415 \times 452 \times 860}{558.2 \times 1000} \right) = 251 \text{ mm} \)
**Beam B1 Mid**

**Design Parameters**

- Load Case 13 \[1.5*(DL + EQX)\]
- Grade of Concrete: **M30**
- Grade of Steel: **Fe415**
- Characteristic compressive strength of concrete, \(f_{ck} ( \text{N/mm}^2)\): 30
- Characteristic yield strength of steel, \(f_y ( \text{N/mm}^2)\): 415
- Unit weight of concrete, \(\gamma_c ( \text{kN/m}^3)\): 24
- Partial safety factor for concrete: 1.5
- Exposure condition: **Mild**
- Nominal Cover to exposure condition: 20

**Dimensions of the beam**

- C/C Span of the beam, \(l (\text{m})\): 4.96
- Breadth of the beam, \(b (\text{mm})\): 400
- Overall depth of the beam, \(D (\text{mm})\): 900

**Details of reinforcements**

- Diameter of tension reinforcement (mm): 25
- Diameter of compression reinforcement (mm): 25
- Diameter of stirrups (mm): 8

**Effective depth**

\[
\text{Effective depth, } d (\text{mm}) = (900-20-8-25/2) = 860
\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment: 156 kN-m
- Bending Moment \(M_u (\text{kN-m})\): 331
- Equivalent Bending Moment, \(M_e (\text{kNm})\): 629
- Shear force at critical distance, \(V_{ud} (\text{kN})\): 218
- Equivalent Shear (kN): 842

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, \(M_{u,\text{lim}}\) is given by

\[
M_{u,\text{lim}} = 0.362f_{ck} \cdot \frac{bx_u_{\text{max}}}{d} \cdot 0.416x_u_{\text{max}}
\]

Where
- \(b = \text{Breadth of the Section}\)
- \(x_u_{\text{max}} = \text{Limiting depth of Neutral Axis}\)
- \(d = \text{Effective depth of the Section}\)

The limiting percentage of steel, \(\rho_{u,\text{lim}}\) is given by
The area of steel for a singly reinforced section with width, \( b \) and depth, \( d \) and ultimate moment, \( M_u \) is given by:

\[
Pt = 41.61 \times \frac{f_{ck}}{f_y} \times \frac{x_{u,\text{max}}}{d} \times Ast
\]

Where \( f_{ck} = \) Characteristic Compressive strength of concrete  
\( f_y = \) Characteristic strength of steel

The area of steel for a singly reinforced section with width, \( b \) and depth, \( d \) and ultimate moment, \( M_u \) is given by:

\[
\frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{f_{ck}}{2f_y} = 4.598 \times \frac{R}{f_{ck}}
\]

Where \( R = \frac{M_u}{bd^2} \)

For (M30 and Fe415):

\[
M_u,\text{lim} = 0.1389 \times f_{ck} \times b \times d^2
\]

\[
x_{u,\text{max}} / d = 0.48
\]

\[
M_u,\text{lim} = \left( 0.1389 \times 30 \times 400 \times 859.5^2 / 1000000 \right) = 1,231.33 \text{ kNm}
\]

\[
p_{\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433
\]

If \( M_u > M_u,\text{lim} \), the section has to be
  i) get increased by depth or width (preferably depth)  
  ii) doubly reinforced

If \( M_u < M_u,\text{lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 629.24 \text{ kNm}
\]

\[
M_u,\text{lim} = 1,231.33 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- **Considering a 'balanced section'** (\( x_u = x_{u,\text{max}} \))
  
  \[
  A_{st} = A_{st,\text{lim}} + \Delta A_{st}
  \]

  where \( A_{st,\text{lim}} = \frac{p_{\text{lim}}}{100} \times (b \times d) \)

  \[
  \Rightarrow A_{st,\text{lim}} = \left( 1.433 / 100 \times 400 \times 859.5 \right) = 4927 \text{ mm}^2
  \]

- **Assuming 25 mm bars for compression steel,**

  \[
  d^* \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}
  \]
\[ A_{st} \frac{M_u}{0.87 \ f_y \ d \ d'} \]

\[ p_t \frac{R}{100} \frac{R_{lim}}{0.87 \ f_y \ d' \ d} \]

\[ M_u = 0.87 \ f_y \ \ast s \ast d (1 - (\ast s \ast f_y) / b * d * f_c k) \]

\[ \text{Ast Reqd} = 2227 \ \text{mm}^2 \]

\[ \therefore \ \text{No of tension bars required ( # )} \]

\[ (2227 / (\pi / 4 x 25^2) = 5.00 \]

\[ \text{Actual percentage of steel , } p_t (\%) \]

\[ (5 \times \pi / 4 \times 25^2 / 400 / 860 \times 100) = 0.71 \]

\[ \text{Actual area of steel , } A_{st} (\text{mm}^2) \]

\[ (5 \times \pi / 4 \times 25^2) = 2454 \]

**Determining \( A_{sc} \)**

The compression steel , \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 \ f_y \ A_{st}}{f_{sc} - 0.447 \ f_c k} \]

or

\[ p_c = \frac{0.87 \ f_y \ p_t}{f_{sc} - 0.447 \ f_c k} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,max} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,max} \), for \( d' / d = (40.5 / 859.5) = 0.047 \)

From the above table : by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,max} \), it suffices to establish \( p_c \geq p_c^* \)
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{fc}} - \frac{0.447 f_{ck}}{p_{t,lim} - p_t}$$

Actual $p_t$ provided : $p_t = 0.71$
Actual $p_c$ provided : $p_c = 0.71$

$\Rightarrow p_c^* = (0.87 \times 415 \times (0.714 - 1.433) / (354.98 - 0.447 \times 30))$

$\Rightarrow p_c^* = -0.76$

Section is not over reinforced

Check for deflection control

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} / \frac{l}{d_{\text{basic}}} = F_1 F_2$$

where

- $l/d_{\text{basic}}$ (7 for cantilever spans)
- $l/d_{\text{basic}}$ (20 for simply supported spans)
- $l/d_{\text{basic}}$ (26 for continuous spans)

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$.

$F = \frac{10}{\text{span in metres}}$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

$$f_{t} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = (0.58 \times 415 \times 2891 / 2454) = 283.49 \text{ N/mm}^2$$

$F = 1.00$

$F_1 = 1.08$
\[ F_2 = 1.16 \]

\[ \therefore \quad \left( \frac{l}{d} \right)_{\text{max}} = \left( 26 \times 1 \times 1.08 \times 1.16 \right) = 32.39 \]
\[ \left( \frac{l}{d} \right)_{\text{provided}} = 5.77 \]
\[ \Rightarrow \quad \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

842

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[
( 842 \times 1000 / (400 \times 860) ) = 2.45 \text{ N/mm}^2
\]

The maximum shear stress is given by:

\[
\frac{0.62 \times \text{sqrt}(30)}{0.85 \times \text{sqrt}(6.89 \times p_t)} = 3.40 \text{ N/mm}^2
\]

\[ \Rightarrow \quad \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by

2454 mm\(^2\)

Percentage of steel, \( p_t \) (%)

0.71

The design shear strength of the concrete, \( \tau_c \), is given by:

\[
\tau_c = \frac{0.85 \times 0.8 \times f_{ck} \times 5 \times 1}{6}
\]

where

\[
\frac{0.8 \times f_{ck}}{6.89 \times p_t} \text{ whichever is greater}
\]

For (M30 and Fe415)

\[ \Rightarrow \quad \tau_c = 0.57 \text{ N/mm}^2 \]

\[ \Rightarrow \quad V_{uc} = (0.57 \times 400 \times 860 / 1000) = 198 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_{u} - V_{uc} \]

\[ \Rightarrow \quad V_{us} = (842 - 198) = 644 \text{ kN} \]

Using 12 mm bars and 4 No of legs
Area of stirrups, \( A_{sv} \) (mm\(^2\)) = 452

\[ \Rightarrow \text{required spacing } s_v \leq \left( \frac{0.87 \times 415 \times 452 \times 860}{644.46 \times 1000} \right) \]

\[ \Rightarrow \text{Spacing, } s_v = 218 \text{ mm} \]

**Check whether \( \tau_v > 0.5 \tau_c \)**

- Nominal shear stress, \( \tau_v \) (N/mm\(^2\)) = 2.45
- Design shear stress, \( \tau_c \) (N/mm\(^2\)) = 0.57

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \) :

\[ \frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5tc \]

\[ s_v = \frac{2.175 f_y A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v \]

- 0.75 \( d \)
- 300 mm

**Code requirements for maximum spacing.**

- i) \( < \left( \frac{2.175 \times 415 \times 452}{400} \right) = 1021 \text{ mm} \)
- ii) \( \leq \left( \frac{0.75 \times 859.5}{400} \right) = 645 \text{ mm} \)
- iii) \( \leq 300 \text{ mm} \)
- iv) \( \leq \left( \frac{0.87 \times 415 \times 452 \times 860}{644.46 \times 1000} \right) = 218 \text{ mm} \)
Beam B2 Support

Design Parameters

Load Case 14  [1.5*(DL - EQX)]
Grade of Concrete  M30
Grade of Steel  Fe415
Characteristic compressive strength of concrete , f_{ck} ( N/mm² )  30
Characteristic yield strength of steel , f_y ( N/mm² )  415
Unit weight of concrete , \gamma_c ( kN/m³ )  24
Partial safety factor for concrete  1.5
Exposure condition  Mild
Nominal Cover to exposure condition( mm )  20

Dimensions of the beam

C/C Span of the beam , l ( m )  10.80
Breadth of the beam , b ( mm )  800
Overall depth of the beam , D ( mm )  900

Details of reinforcements

Diameter of tension reinforcement ( mm )  25
Diameter of compression reinforcement ( mm )  25
Diameter of stirrups ( mm )  8

Effective depth

Effective depth , d ( mm )  ( 900-20-8-25/2 ) = 860

Design Moment, Shear Force

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.
Torsional Moment  125 kN-m
Bending Moment Mu(kN-m)  2065
Equivalent Bending Moment , M_e ( kNm )  2221
Shear force at critical distance , V_ud ( kN )  908
Equivalent Shear (kN)  1158

Singly reinforced or doubly reinforced section?

The limiting moment of resistance , M_{u,lim} is given by

\[ M_{u,lim} = 0.362f_{ck} \frac{b x_u}{d} \frac{x_u}{x_u_{max}} \times 0.416x_u_{max} \]

Where b = Breadth of the Section
x_u_{max} = Limiting of the Section
x_u = Limiting depth of Neutral Axis
\[ d = \text{Effective depth of the Section} \]

The limiting percentage of steel , p_{lim} is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

For (M30 and Fe415)

\[
x_{u,max} / d = 0.48
\]

\[
M_{u,lim} = \frac{0.1389 \times 30 \times 800 \times 859.5^2}{1000000} = 2,462.66 \text{ kNm}
\]

\[
p_{u,lim} = \frac{41.3 \times 30}{415 \times 0.48} = 1.433
\]

If \( M_u > M_{u,lim} \), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \( M_u < M_{u,lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 2,221.25 \text{ kNm}
\]

\[
M_{u,lim} = 2,462.66 \text{ kNm}
\]

\[
\Rightarrow \quad \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a 'balanced section' (\( x_u = x_{u,max} \))
  \[
  A_{st} = A_{st,lim} + \Delta A_{st}
  \]

  where \( A_{st,lim} = p_{u,lim} / 100 \times b \times d \)

  \[
  \Rightarrow \quad A_{st,lim} \left( \frac{1.433}{100} \times 800 \times 859.5 \right) = 9854 \text{ mm}^2
  \]

- Assuming 25 mm bars for compression steel,

  \[
  d' \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}
  \]
\[ M_u = 0.87 f_y \cdot A_{st} \cdot d \left( 1 - \frac{A_{st} f_y}{b' d' f_{ck}} \right) \]

\[ A_{st} = \frac{M_u}{0.87 f_y \cdot d' d} \]

\[ p_t = \frac{R}{100} \cdot \frac{R_{lim}}{0.87 f_y} \cdot d' \]

\[ M_u = 0.87 f_y \cdot A_{st} \cdot d \left( 1 - \frac{A_{st} f_y}{b' d' f_{ck}} \right) \]

\[ A_{st} = \frac{8670}{\pi / 4 \cdot 25^2} = 18.00 \text{ mm}^2 \]

\[ \therefore \text{No of tension bars required ( # )} \]

\[ \frac{8670}{\pi / 4 \cdot 25^2} = 18.00 \]

\[ \text{Actual percentage of steel , } p_t (\% ) \]

\[ \frac{18 \cdot \pi / 4 \cdot 25^2}{800 \cdot 660 x 100} = 1.29 \]

\[ \text{Actual area of steel , } A_{st} (\text{ mm}^2 ) \]

\[ \frac{18 \cdot \pi / 4 \cdot 25^2}{800} = 8836 \]

**Determining** \( A_{sc} \)

The compression steel , \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 f_y}{f_{sc}} \cdot A_{st} \cdot \frac{0.447 f_{ck}}{d'} \]

or

\[ p_c = \frac{0.87 f_y}{f_{sc}} \cdot \frac{p_t}{0.447 f_{ck}} \cdot \frac{p_{t,lim}}{d'} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

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</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>342.4</td>
<td>339.2</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

\[ \text{Assuming } x_u = x_{u,\text{max}} \text{, for } d' / d = \left( \frac{40.5}{859.5} \right) = 0.047 \]

From the above table : by interpolation

**Design Check**

\[ \text{To ensure } x_u \leq x_{u,\text{max}} \text{, it suffices to establish } p_c \geq p_{c,*} \]
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{ck}} - \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}},$$

Actual $p_t$ provided: $p_t = 1.29$
Actual $p_c$ provided: $p_c = 0.14$

$$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (1.285 - 1.433))}{(354.98 - 0.447 \times 30)}$$

$$\Rightarrow p_c^* = -0.16$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} \times F_1 \times F_2$$

where $\frac{l}{d_{\text{basic}}}$
- 7 for cantilever spans
- 20 for simply supported spans
- 26 for continuous spans

For simply supported and continuous spans over 10m, these ratios are multiplied by a factor $F_{10}$

$$\frac{F}{10}$$

span in metres

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$f_{st} = 0.58 f_y \times \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 9037 / 8836)}{246.19} = 246.19 \text{ N/mm}^2$$

$F = 0.93$

$F_1 = 0.85$
$$F_2 = 0.55$$

\[ \therefore \frac{l}{d}_{\text{max}} = \frac{26 \times 0.93 \times 0.85 \times 0.55}{12.57} = 11.21 \]

\[ \frac{l}{d}_{\text{provided}} = 12.57 \]

\[ \Rightarrow \text{Not O.K} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

1158

The critical section for shear is at a distance of 860 mm from the face of the support.

**Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[
\frac{1158 \times 1000}{800 \times 860} = 1.68 \text{ N/mm}^2
\]

The maximum shear stress is given by:

\[ T_{c\text{max}} = 0.62 \cdot f_{ck} \]

\[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \text{Adopted section is adequate} \]

**Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by

8836 mm²

Percentage of steel, \( p_t \) (%)

1.29

The design shear strength of the concrete, \( \tau_c \), is given by:

\[
\tau_c = 0.95 \cdot 0.8 \cdot f_{ck} \cdot 4 \cdot 5 \cdot 1 \div 6
\]

where

\[
\frac{0.8 \cdot f_{ck}}{0.89 \cdot p_t} \quad \text{whichever is greater}
\]

For (M30 and Fe415)

\[ \Rightarrow \tau_c = 0.72 \text{ N/mm}^2 \]

\[ \Rightarrow V_{uc} = (0.72 \times 800 \times 860 / 1000) = 496 \text{ kN} \]

**Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow V_{us} = (1158 - 496) = 662 \text{ kN} \]

Using 12 mm bars and

No of legs 4
Area of stirrups, $A_{sv}$ (mm$^2$) = 452

⇒ required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 452 \times 860}{662.45 \times 1000} \right)$

⇒ Spacing, $s_v = 212$ mm

\textit{Check whether $\tau_v > 0.5 \tau_c$}

Nominal shear stress, $\tau_v$ (N/mm$^2$) = 1.68
Design shear stress, $\tau_c$ (N/mm$^2$) = 0.72

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

\[
\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 \cdot f_y} \quad \text{When} \ s_v = 0.5 \tau_c
\]

\[
s_v = \frac{2.175 \cdot f_y \cdot A_{sv}}{b}
\]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v = 0.75 \cdot d \quad 300 \text{ mm} \]

Code requirements for maximum spacing:

\begin{itemize}
  \item[i)] < \( \left( \frac{2.175 \times 415 \times 452}{800} \right) = 510 \text{ mm} \)
  \item[ii)] \( \leq \ ( 0.75 \times 859.5 ) \quad = 645 \text{ mm} \)
  \item[iii)] \( \leq \ 300 \text{ mm} \quad 300 \text{ mm} \)
  \item[iv)] \( \leq \ ( \frac{0.87 \times 415 \times 452 \times 860}{662.45 \times 1000} ) = 212 \text{ mm} \)
\end{itemize}
**Beam B2 Mid**

**Design Parameters**

Load Case 14  \[1.5*(DL - EQX)\]
Grade of Concrete \text{M30}
Grade of Steel \text{Fe415}
Characteristic compressive strength of concrete , \(f_{ck}\) ( N/mm\(^2\) ) 30
Characteristic yield strength of steel , \(f_y\) ( N/mm\(^2\) ) 415
Unit weight of concrete , \(\gamma_c\) ( kN/m\(^3\) ) 24
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition( mm ) 20

**Dimensions of the beam**

\[
\begin{align*}
\text{C/C Span of the beam , } l, \text{ ( m )} & \quad 10.80 \\
\text{Breadth of the beam , } b \text{ ( mm )} & \quad 800 \\
\text{Overall depth of the beam , } D \text{ ( mm )} & \quad 900
\end{align*}
\]

**Details of reinforcements**

\[
\begin{align*}
\text{Diameter of tension reinforcement ( mm )} & \quad 25 \\
\text{Diameter of compression reinforcement ( mm )} & \quad 25 \\
\text{Diameter of stirrups ( mm )} & \quad 8
\end{align*}
\]

**Effective depth**

\[
\text{Effective depth , } d \text{ ( mm )} \quad ( 900-20-8-25/2 ) = 860
\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Moment or Shear</th>
<th>Value (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>125</td>
</tr>
<tr>
<td>Bending Moment Mu</td>
<td>855</td>
</tr>
<tr>
<td>Equivalent Bending Moment , (M_e)</td>
<td>1011</td>
</tr>
<tr>
<td>Shear force at critical distance , (V_{ud}) ( kN )</td>
<td>884</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>1134</td>
</tr>
</tbody>
</table>

**Singly reinforced or doubly reinforced section ?**

The limiting moment of resistance , \(M_{u,\text{lim}}\) is given by

\[
M_{u,\text{lim}} = 0.362f_{ck} \cdot \frac{b x_u}{d} \cdot 0.416 x_u
\]

Where \(b\) = Breadth of the Section
\(x_u\) = Limiting depth of Neutral Axis
\(d\) = Effective depth of the Section

The limiting percentage of steel , \(\rho_{u,\text{lim}}\) is given by
The area of steel for a singly reinforced section with width, b and depth, d and ultimate moment, $M_u$ is given by:

\[ x_u,\text{max} / d = 0.48 \]

\[ \Rightarrow \quad M_{u,\text{lim}} = \left( 0.1389 \times 30 \times 800 \times 859.5^2 / 1000000 \right) = 2,462.66 \text{ kNm} \]

\[ \Rightarrow \quad p_{t,\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433 \]

If $M_u > M_{u,\text{lim}}$, the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If $M_u < M_{u,\text{lim}}$, the section can be designed as singly reinforced.

**Check for the type of section**

\[ M_u = 1,011.25 \text{ kNm} \]
\[ M_{u,\text{lim}} = 2,462.66 \text{ kNm} \]

\[ \Rightarrow \quad \text{Section can be designed as singly reinforced.} \]

**Determining $A_{st}$**

- Considering a 'balanced section' ($x_u = x_{u,\text{max}}$)
  \[ A_{st} = A_{st,\text{lim}} + \Delta A_{st} \]
  where $A_{st,\text{lim}} = p_{t,\text{lim}} / 100 \times b \times d$

\[ \Rightarrow \quad A_{st,\text{lim}} \left( 1.433 / 100 \times 800 \times 859.5 \right) = 9854 \text{ mm}^2 \]

- Assuming 25 mm bars for compression steel,

\[ d^* \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm} \]
\[ A_{st} = \frac{M_u}{0.87 f_y d - d'} \frac{M_{u,lim}}{d'} \]

\[ p_t \frac{R}{100} = \frac{R_{lim}}{0.87 f_y \frac{1}{d'}} \]

\[ M_u = 0.87 f_y^{Ast}(1-(Ast*f_y)/b*d*fck) \]

\[ Ast \text{ Req'd} = 3506 \text{ mm}^2 \]

\[ \therefore \text{ No of tension bars required ( # ) } \]

\[ \frac{3506}{( \frac{\pi}{4} \times 25^2 )} = 8.00 \]

\[ \text{Actual percentage of steel, } p_t (\% ) \]

\[ \left( \frac{8 \times \pi}{4 \times 25^2} / 800 / 860 \times 100 \right) = 0.57 \]

\[ \text{Actual area of steel, } A_{st} (\text{mm}^2) \]

\[ \left( 8 \times \pi / 4 \times 25^2 \right) = 3927 \]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 f_y}{f_{sc}} \frac{A_{st}}{0.447 f_{ck}} \]

or

\[ p_c = \frac{0.87 f_y}{f_{sc}} \frac{p_t}{0.447 f_{ck}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
</tr>
<tr>
<td>Fe250</td>
<td>217.5</td>
</tr>
<tr>
<td></td>
<td>217.5</td>
</tr>
<tr>
<td></td>
<td>217.5</td>
</tr>
<tr>
<td></td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
</tr>
<tr>
<td></td>
<td>351.9</td>
</tr>
<tr>
<td></td>
<td>342.4</td>
</tr>
<tr>
<td></td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
</tr>
<tr>
<td></td>
<td>411.3</td>
</tr>
<tr>
<td></td>
<td>395.1</td>
</tr>
<tr>
<td></td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d' / d = \frac{40.5}{859.5} \) = 0.047

  From the above table: by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_c^* \).
where $p_{c}^{*}$ is given by

$$p_{c}^{*} = \frac{0.87 f_{y}}{f_{ck}} \left( \frac{0.571 - 1.433}{354.98 - 0.447 \times 30} \right)$$

Actual $p_{t}$ provided: $p_{t} = 0.57$
Actual $p_{c}$ provided: $p_{c} = 0.79$

$$\Rightarrow p_{c}^{*} = \left( 0.87 \times 415 \times (0.571 - 1.433) / (354.98 - 0.447 \times 30) \right)$$

$$\Rightarrow p_{c}^{*} = -0.91$$

Section is not over reinforced

Check for deflection control

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_{1} F_{2}$$

where

- $\frac{l}{d_{\text{basic}}}$ for cantilever spans
- $\frac{l}{d_{\text{basic}}}$ for simply supported spans
- $\frac{l}{d_{\text{basic}}}$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_{1}$ (which varies with $p_{t}$ and $f_{st}$) and $F_{2}$ (which varies with $p_{c}$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$f_{t} = 0.58 f_{y} \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \left( 0.58 \times 415 \times 4945 / 3927 \right) = 303.12 \text{ N/mm}^{2}$$

$F = 0.93$

$F_{1} = 1.18$
\[ F_2 = 1.19 \]

\[ \therefore \quad (l/d)_{\text{max}} = (26 \times 0.93 \times 1.18 \times 1.19) = 33.53 \]

\[ (l/d)_{\text{provided}} = 12.57 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

1134

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)

  \[
  (1134 \times 1000 / (800 \times 860)) = 1.65 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:

  \[ T_{c,\text{max}} = 0.62 f_{ck} \]

  \[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

  \[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by

  3927 mm²

  Percentage of steel, \( p_t (\%) \)

  0.57

  The design shear strength of the concrete, \( \tau_c \), is given by:

  \[
  \tau_c = \frac{0.85 \times 0.8 \times f_{ck}}{6} - \frac{1}{5} \]

  where

  \[
  \frac{0.8 f_{ck}}{6.89 p_t} \text{ whichever is greater} - \frac{1}{6}
  \]

  For (M30 and Fe415)

  \[ \Rightarrow \tau_c = 0.52 \text{ N/mm}^2 \]

  \[ \Rightarrow V_{uc} = (0.52 \times 800 \times 860 / 1000) = 361 \text{ kN} \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by: \( V_{us} = V_u - V_{uc} \)

  \[ \Rightarrow V_{us} = (1134 - 361) = 773 \text{ kN} \]

  Using 12 mm bars and 4 legs
Area of stirrups, \( A_{sv} \) (mm\(^2\)) = 452

\[ \Rightarrow \text{required spacing } s_v \leq \frac{0.87 \times 415 \times 452 \times 860}{773.14 \times 1000} \]

\[ \Rightarrow \text{Spacing }, s_v = 182 \text{ mm} \]

**Check whether \( \tau_v > 0.5 \tau_c \)**

Nominal shear stress, \( \tau_v \) (N/mm\(^2\)) = 1.65

Design shear stress, \( \tau_c \) (N/mm\(^2\)) = 0.52

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):

\[ \frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 \cdot f_y} \text{ When } s_v = 0.5 \tau_c \]

\[ s_v = \frac{2.175 \cdot f_y \cdot A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v = 0.75 \cdot d \]

\[ s_v = 300 \text{ mm} \]

**Code requirements for maximum spacing..**

i) \( < \frac{2.175 \times 415 \times 452}{800} \) = 510 mm

ii) \( \leq \frac{0.75 \times 859.5}{\text{mm}} \) = 645 mm

iii) \( \leq 300 \text{ mm} \)

iv) \( \leq \frac{0.87 \times 415 \times 452 \times 860}{773.14 \times 1000} \) = 182 mm
**Beam B2A Support**

**Design Parameters**

Load Case 15 \[1.5 \cdot (DL + EQZ)\]

- **Grade of Concrete**: M30
- **Grade of Steel**: Fe415
- **Characteristic compressive strength of concrete**, \( f_{ck} \text{ (N/mm}^2 \text{) } 30\)
- **Characteristic yield strength of steel**, \( f_y \text{ (N/mm}^2 \text{) } 415\)
- **Unit weight of concrete**, \( \gamma_c \text{ (kN/m}^3 \text{) } 24\)
- **Partial safety factor for concrete**: 1.5
- **Exposure condition**: Mild
- **Nominal Cover to exposure condition**: 20 mm

**Dimensions of the beam**

- **C/C Span of the beam**, \( l \text{ (m) } 4.96\)
- **Breadth of the beam**, \( b \text{ (mm) } 550\)
- **Overall depth of the beam**, \( D \text{ (mm) } 900\)

**Details of reinforcements**

- **Diameter of tension reinforcement**: 25 mm
- **Diameter of compression reinforcement**: 25 mm
- **Diameter of stirrups**: 8 mm

**Effective depth**

**Effective depth**, \( d \text{ (mm) } (900-20-8-25/2) = 860\)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- **Torsional Moment**: 163 kN-m
- **Bending Moment**, \( M_u \text{ (kNm) } 1428\)
- **Equivalent Bending Moment**, \( M_e \text{ (kNm) } 1681\)
- **Shear force at critical distance**, \( V_{ud} \text{ (kN) } 199\)
- **Equivalent Shear**: 673 kN

**Singly reinforced or doubly reinforced section?**

The $limiting$ $moment$ $of$ $resistance$, \( M_{u,lim} \) is given by

\[
M_{ulim} = 0.362 f_{ck} \cdot \frac{b x_{u,\text{max}}}{d} \cdot 0.416 x_{u,\text{max}}
\]

Where: 
- \( b \) = Breadth of the Section
- \( x_{u,\text{max}} \) = Limiting depth of Neutral Axis
- \( d \) = Effective depth of the Section

The limiting percentage of steel, \( p_{u,lim} \) is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[
M_u,\text{lim} = \left( \frac{0.1389 \times 30 \times 550 \times x_{u,\max}^2}{1000000} \right) = 1693.08 \text{ kNm}
\]

\[
pt,\text{lim} = \left( \frac{41.3 \times 30}{415 \times 0.48} \right) = 1.433
\]

If \( M_u > M_u,\text{lim} \), the section has to be:

- i) get increased by depth or width (preferably depth)
- ii) doubly reinforced

If \( M_u < M_u,\text{lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 1680.78 \text{ kNm}
\]

\[
M_u,\text{lim} = 1693.08 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a ‘balanced section’ (\( x_c = x_{u,\max} \))
  \[
  A_{st} = A_{st,\text{lim}} + \Delta A_{st}
  \]
  where \( A_{st,\text{lim}} = pt,\text{lim} / 100 (b \times d) \)

\[
\Rightarrow A_{st,\text{lim}} = \left( \frac{1.433}{100 \times 550 \times 859.5} \right) = 6774 \text{ mm}^2
\]

- Assuming 25 mm bars for compression steel,

\[
d' \approx \left( 20 \text{ mm clear cover + 8 mm stirrup + 25 / 2 } \right) = 40.5 \text{ mm}
\]
\[
A_{st} = \frac{M_u}{0.87 f_y} \frac{M_{u,\text{lim}}}{d} \quad d'
\]

\[
\frac{p_t}{100} = \frac{R}{R_{\text{lim}}} = \frac{0.87 f_y}{1} \frac{1}{d/d'}
\]

\[
M_u = 0.87 f_y^{\ast} d (1 - (f_y^{\ast} f_y) b^d f_{ck})
\]

\[
A_{\text{st Reqd}} = 6749 \text{ mm}^2
\]

\[
\therefore \text{ No of tension bars required ( \# )}
\]

\[
\left( \frac{6749}{\pi / 4 \times 25^2} \right) = 14.00
\]

Actual percentage of steel, \( p_t \) (\%)

\[
\left( \frac{14 \times \pi / 4 x 25^2}{550 / 860 \times 100} \right) = 1.45
\]

Actual area of steel, \( A_{st} \) (\text{mm}^2)

\[
\left( 14 \times \pi / 4 x 25^2 \right) = 6872
\]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by

\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \frac{A_{st}}{0.447 f_{ck}}
\]

or

\[
p_c = \frac{0.87 f_y}{f_{sc}} \frac{p_t}{p_{t,\text{lim}}} \frac{0.447 f_{ck}}{d'/d}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,\text{max}} \) for various \( d'/d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d'/d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d'/d = (40.5 / 859.5) = 0.047 \)

From the above table: by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_{c^*} \)
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{y_{cr}} - 0.447 f_{ck}} \quad p_{t,lim}$$

Actual $p_t$ provided : $p_t = 1.45$
Actual $p_c$ provided : $p_c = 0.10$

$$\Rightarrow p_c^* = \left( \frac{0.87 \times 415 \times (1.454 - 1.433)}{(354.98 - 0.447 \times 30)} \right)$$

$$\Rightarrow p_c^* = 0.02$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l / d$ ratios are specified by the Code (Cl. 23.2.1) as :

$$\frac{l}{d_{max}} = \frac{l}{d_{basic}} F_1 F_2$$

where $\frac{l}{d_{basic}}$ 7 for cantilever spans
20 for simply supported spans
26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows :

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \left( 0.58 \times 415 \times 6733 / 6872 \right) = 235.82 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 0.82$
F_2 = 0.44

\[ \therefore \quad \left( \frac{l}{d} \right)_{\text{max}} = \left( \frac{26 \times 1 \times 0.82 \times 0.44}{1} \right) = 9.36 \]
\[ \left( \frac{l}{d} \right)_{\text{provided}} = 5.77 \]
\[ \Rightarrow \quad \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \ (kN) \) 673.18182

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ \left( \frac{673.181818181818 \times 1000}{550 \times 860} \right) = 1.42 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ T_{c \text{ max}} = 0.62 f'_{ck} \]

\[ \Rightarrow \quad \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \quad \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by 6872 mm^2

Percentage of steel, \( p_t (\%) \) 1.45

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \tau_c = \frac{0.85 \times 0.8 f_{ck} - 1.5}{6} \]

where \[ \frac{0.8 f_{ck}}{0.89 p_t} \] whichever is greater 1

For (M30 and Fe415)

\[ \Rightarrow \quad \tau_c = 0.75 \text{ N/mm}^2 \]

\[ \Rightarrow \quad V_{uc} = (0.75 \times 550 \times 860 / 1000) = 356 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow \quad V_{us} = (673 - 356) = 317 \text{ kN} \]

Using 12 mm bars and

No of legs 2
Area of stirrups, $A_{sv}$ (mm$^2$) 226

⇒ required spacing $s_v \leq \frac{0.87 \times 415 \times 226 \times 860}{(316.79 \times 1000)}$

⇒ Spacing, $s_v = 222$ mm

**Check whether** $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm$^2$) 1.42
Design shear stress, $\tau_c$ (N/mm$^2$) 0.75

$\tau_v > 0.5 \tau_c$  Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \text{ When } s_v = 0.5\tau_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v \leq 0.75 d$
$s_v \leq 300$ mm

Code requirements for maximum spacing:

i) $\leq (2.175 \times 415 \times 226 / 550) = 371$ mm
ii) $\leq (0.75 \times 859.5) = 645$ mm
iii) $\leq 300$ mm
iv) $\leq (0.87 \times 415 \times 226 \times 860 / (316.79 \times 1000)) = 222$ mm
**Beam B2A Mid**

**Design Parameters**

Load Case 15 \([1.5 \times (DL + EQZ)]\)

- Grade of Concrete: M30
- Grade of Steel: Fe415
- Characteristic compressive strength of concrete, \(f_{ck} \text{ (N/mm}^2\): 30
- Characteristic yield strength of steel, \(f_y \text{ (N/mm}^2\): 415
- Unit weight of concrete, \(\gamma_c \text{ (kN/m}^3\): 24
- Partial safety factor for concrete: 1.5
- Exposure condition: Mild
- Nominal Cover to exposure condition (mm): 20

**Dimensions of the beam**

- C/C Span of the beam, \(l \text{ (m)}\): 4.96
- Breadth of the beam, \(b \text{ (mm)}\): 550
- Overall depth of the beam, \(D \text{ (mm)}\): 900

**Details of reinforcements**

- Diameter of tension reinforcement (mm): 25
- Diameter of compression reinforcement (mm): 25
- Diameter of stirrups (mm): 8

**Effective depth**

\[
\text{Effective depth, } d \text{ (mm)} = (900-20-8-25/2) = 860
\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment: 163 kN-m
- Bending Moment, \(M_u \text{ (kN-m)}\): 746
- Equivalent Bending Moment, \(M_e \text{ (kNm)}\): 999
- Shear force at critical distance, \(V_{ud} \text{ (kN)}\): 184
- Equivalent Shear (kN): 658

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, \(M_{ulim}\) is given by

\[
M_{ulim} = 0.362f_{ck} \times \frac{bxu_{\text{max}}}{d} \times 0.416xu_{\text{max}}
\]

Where
- \(b\) = Breadth of the Section
- \(xu_{\text{max}}\) = Limiting depth of Neutral Axis
- \(d\) = Effective depth of the Section

The limiting percentage of steel, \(\rho_{ulim}\) is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[
Pt_{\text{lim}} = 41.61 \cdot \frac{f_{ck} \cdot xu_{\text{max}}}{fy} \cdot \frac{xu}{d}
\]

Where \( f_{ck} \) = Characteristic Compressive strength of concrete
\( fy \) = Characteristic strength of steel

The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[
Pt = \frac{Ast \cdot fck}{bd} \cdot \frac{A_{st}}{2fy} = 4.598 \cdot \frac{R}{f_{ck}}
\]

Where \( R = \frac{Mu}{bd^2} \)

For ( M30 and Fe415 )

\[
M_{u,\text{lim}} = 0.1389 \cdot \frac{fck \cdot bd^2}{bd^2} = 0.48
\]

\[
\Rightarrow M_{u,\text{lim}} = \left( 0.1389 \times 30 \times 550 \times 859.5^2 / 1000000 \right) = 1693.08 \text{ kNm}
\]

\[
\Rightarrow p_{t,\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433
\]

If \( M_u > M_{u,\text{lim}} \), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \( M_u < M_{u,\text{lim}} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 998.78 \text{ kNm}
\]

\[
M_{u,\text{lim}} = 1693.08 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a 'balanced section' (\( xu = xu_{\text{max}} \))

\[
A_{st} = A_{st,\lim} + \Delta A_{st}
\]

where \( A_{st,\lim} = p_{t,\text{lim}} / 100 \cdot (b \times d) \)

\[
\Rightarrow A_{st,\lim} = \left( 1.433 / 100 \times 550 \times 859.5 \right) = 6774 \text{ mm}^2
\]

- Assuming 25 mm bars for compression steel,

\[
d^* \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}
\]
\[ A_{st} = \frac{M_u - M_{u,\text{lim}}}{0.87 f_y \frac{d}{d'}} \]

\[ p_t = \frac{R}{100} \frac{R_{\text{lim}}}{0.87 f_y \frac{1}{d'}} \]

\[ M_u = 0.87 f_y^{**} A_{st} d (1 - (A_{st} f_y / b * d' f_{ck}) \]

Ast Reqd = 3597 mm²

\[ \therefore \text{ No of tension bars required ( # )} \]
\[ \frac{3597}{(\Pi / 4 \times 25^2)} = 8.00 \]

Actual percentage of steel, \( p_t \) ( % )
\[ \frac{8 \times \Pi / 4 \times 25^2}{550/860 \times 100} = 0.83 \]

Actual area of steel, \( A_{st} \) ( mm² )
\[ \frac{8 \times \Pi / 4 \times 25^2}{550/860 \times 100} = 3927 \]

Determining \( A_{sc} \)

The compression steel, \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 f_y A_{st}}{f_{sc} - 0.447 f_{ck}} \]

or

\[ p_c = \frac{0.87 f_y p_t}{f_{sc} - 0.447 f_{ck}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
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<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d' / d = \frac{40.5}{859.5} \) = 0.047
  - From the above table : by interpolation

Design Check

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_{c,*} \)
where $p_c^*$ is given by

$$ p_c^* = \frac{0.87 f_y}{f_{ck}} - \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}} $$

Actual $p_t$ provided : $p_t = 0.83$
Actual $p_c$ provided : $p_c = 0.62$

$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (0.831 - 1.433))}{(354.98 - 0.447 \times 30)}$
$\Rightarrow p_c^* = -0.64$

Section is not over reinforced

Check for deflection control

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$ \frac{l}{d_{\text{max}}} > \frac{l}{d_{\text{basic}}} F_1 F_2 $$

where

- $l/d_{\text{basic}}$ 7 for cantilever spans
- $l/d_{\text{basic}}$ 20 for simply supported spans
- $l/d_{\text{basic}}$ 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$ F = \frac{10}{\text{span in metres}} $$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$ f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}} $$

$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 4426 / 3927)}{271.31} \text{ N/mm}^2$

$F = 1.00$
$F_1 = 1.02$
\[ F_2 = 1.11 \]

\[ \therefore \quad \frac{l}{d} \max = \left( \frac{26 \times 1 \times 1.02 \times 1.11}{5.77} \right) = 29.37 \]

\[ \frac{l}{d} \text{ provided} = 5.77 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

\[ 658.18182 \]

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ \left( \frac{658.1818181818 \times 1000}{550 \times 860} \right) = 1.39 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ Tc_{\max} = 0.62 f'ck \]

\[ \Rightarrow \quad \tau_{c,\max} = \left( 0.62 \times \sqrt{30} \right) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{sd} \) is given by

\[ 3927 \text{ mm}^2 \]

Percentage of steel, \( p_t \) (%)

\[ 0.83 \]

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \begin{aligned} \tau_c &= \frac{0.85 \times 0.8 f'ck}{6} - 1 \\ \text{where} \quad \frac{0.8 f'ck}{6.89 p_t} \text{ whichever is greater} \end{aligned} \]

For (M30 and Fe415)

\[ \Rightarrow \quad \tau_c = 0.61 \text{ N/mm}^2 \]

\[ \Rightarrow \quad V_{uc} = \left( 0.61 \times 550 \times 860 / 1000 \right) = 288 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow \quad V_{us} = \left( 658 - 288 \right) = 370 \text{ kN} \]

Using 12 mm bars and

No of legs 2
Area of stirrups, \( A_{sv} \) (mm\(^2\)) = 226

\[ \Rightarrow \text{required spacing } s_v \leq \left( \frac{0.87 \times 415 \times 226 \times 860}{(369.72 \times 1000)} \right) \]

\[ \Rightarrow \text{Spacing, } s_v = 190 \text{ mm} \]

**Check whether** \( \tau_v > 0.5 \tau_c \)

Nominal shear stress, \( \tau_v \) (N/mm\(^2\)) = 1.39

Design shear stress, \( \tau_c \) (N/mm\(^2\)) = 0.61

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):

\[ \frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5t_c \]

\[ s_v = \frac{2.175 f_y A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v \]

- 0.75 \( d \)
- 300 mm

**Code requirements for maximum spacing**:

i) \( < \left( \frac{2.175 \times 415 \times 226}{550} \right) = 371 \text{ mm} \)

ii) \( \leq \left( \frac{0.75 \times 859.5}{300} \right) = 645 \text{ mm} \)

iii) \( \leq 300 \text{ mm} \)

iv) \( \leq \left( \frac{0.87 \times 415 \times 226 \times 860}{(369.72 \times 1000)} \right) = 190 \text{ mm} \)
**Beam B3 Support**

**Design Parameters**

Load Case 16 \([1.5 \cdot (DL - EQZ)]\)

Grade of Concrete \(M30\)

Grade of Steel \(Fe415\)

Characteristic compressive strength of concrete , \(f_{ck} \ (N/mm^2)\) 30

Characteristic yield strength of steel , \(f_y \ (N/mm^2)\) 415

Unit weight of concrete , \(\gamma_c \ (kN/m^3)\) 24

Partial safety factor for concrete 1.5

Exposure condition Mild

Nominal Cover to exposure condition \((mm)\) 20

**Dimensions of the beam**

- C/C Span of the beam , \(l \ (m)\) 8.36
- Breadth of the beam , \(b \ (mm)\) 300
- Overall depth of the beam , \(D \ (mm)\) 900

**Details of reinforcements**

- Diameter of tension reinforcement \((mm)\) 25
- Diameter of compression reinforcement \((mm)\) 25
- Diameter of stirrups \((mm)\) 8

**Effective depth**

Effective depth , \(d \ (mm)\) \(= (900-20-8-25/2) = 860\)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment \(11 \ kN-m\)
- Bending Moment \(Mu( kN-m)\) 672
- Equivalent Bending Moment , \(M_e \ (kNm)\) 698
- Shear force at critical distance , \(V_{ud} \ (kN)\) 60
- Equivalent Shear \((kN)\) 119

**Singly reinforced or doubly reinforced section ?**

The limiting moment of resistance , \(M_{u,lim}\) is given by

\[ M_{ulim} = 0.362f_{ck} \cdot \frac{bx_{u,\max}}{d} \cdot 0.416x_{u,\max} \]

Where \(b = \) Breadth of the Section
\(x_{u,\max} = \) Limiting depth of Neutral Axis
\(d = \) Effective depth of the Section

The limiting percentage of steel , \(p_{u,lim}\) is given by
The area of steel for a singly reinforced section with width , b and depth , d and ultimate moment , M_u is given by :

\[\frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{fck}{2fy} = \frac{R}{fck}\]

Where \( R = \frac{M_u}{bd^2} \)

For ( M30 and Fe415 )

\[x_{u,\text{max}} / d = 0.48\]

\[M_u,\text{lim} = (0.1389 \times 30 \times 300 \times 859.5^2 / 1000000) = 923.50 \text{ kNm}\]

\[p_u,\text{lim} = (41.3 \times 30 / 415 \times 0.48) = 1.433\]

If \( M_u > M_u,\text{lim} \), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \( M_u < M_u,\text{lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[M_u = 697.88 \text{ kNm}\]
\[M_u,\text{lim} = 923.50 \text{ kNm}\]

\( \Rightarrow \) Section can be designed as singly reinforced.

**Determining \( A_{st} \)**

- Considering a ‘balanced section’ (\( x_u = x_{u,\text{max}} \))
  \[A_{st} = A_{st,\text{lim}} + \Delta A_{st}\]
  where \( A_{st,\text{lim}} = p_u,\text{lim} / 100 (b \times d)\)
  \( \Rightarrow A_{st,\text{lim}} = (1.433 / 100 \times 300 \times 859.5) = 3695 \text{ mm}^2\)

- Assuming 25 mm bars for compression steel,
  \[d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm}\]
\[
\begin{align*}
A_{ak} &= \frac{M_u}{0.87 f_y d} \frac{M_{u,\text{lim}}}{d'} \\
p_t &= \frac{R}{100} \frac{R_{\text{lim}}}{0.87 f_y} \frac{1}{d'}
\end{align*}
\]

\[
M_u = 0.87 f_y \cdot A_{st} \cdot d \cdot (1 - (A_{st} f_y) / b \cdot d \cdot f_{ck})
\]

\[
A_{st} \text{ Reqd} = 2616 \, \text{mm}^2
\]

\[
\therefore \quad \text{No of tension bars required (\#)} = \frac{2616}{\frac{\pi}{4} \times 25^2} = 6.00
\]

Actual percentage of steel, \(p_t(\%)\)
\[
\left(\frac{6 \times \pi}{4} \times 25^2 / 300 / 860 \times 100\right) = 1.14
\]

Actual area of steel, \(A_{st}(\text{mm}^2)\)
\[
\left(\frac{6 \times \pi}{4} \times 25^2\right) = 2945
\]

**Determining \(A_{ak}\)**

The compression steel, \(A_{sc}\), is given by
\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \frac{A_{ak}}{0.447 f_{ck}}
\]
or
\[
p_c = \frac{0.87 f_y}{f_{sc}} \frac{p_t}{0.447 f_{ck}} \frac{P_{\text{lim}}}{p_c}
\]

where \(f_{sc}\) is the stress in compression steel.

The values of \(f_{sc}\) (in MPa units) at \(x_u = x_{u,\text{max}}\) for various \(d' / d\) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>(\frac{d'}{d})</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
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</tr>
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<td>355.1</td>
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<td></td>
</tr>
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<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \(x_u = x_{u,\text{max}}\), for \(d' / d = \frac{40.5}{859.5}\) = 0.047
  From the above table: by interpolation

**Design Check**

- To ensure \(x_u \leq x_{u,\text{max}}\), it suffices to establish \(p_c \geq p_c^*\).
where \( p_c \) is given by

\[
\frac{p_c}{f_{ec}} = \frac{0.87 f_y}{0.447 f_{ck}} \quad p_t \quad p_{t,\text{lim}}
\]

Actual \( p_t \) provided : \( p_t = 1.14 \)
Actual \( p_c \) provided : \( p_c = 0.38 \)

\[
\Rightarrow p_c^* = \left( 0.87 \times 415 \times (1.142 - 1.433) / (354.98 - 0.447 \times 30) \right)
\]

\[
\Rightarrow p_c^* = -0.31
\]

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting \( l/d \) ratios are specified by the Code (Cl. 23.2.1) as:

\[
\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2
\]

where \( \frac{l}{d_{\text{basic}}} \) is 7 for cantilever spans, 20 for simply supported spans, and 26 for continuous spans.

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor \( F \) for each span in metres.

The modification factors \( F_1 \) (which varies with \( p_t \) and \( f_{st} \)) and \( F_2 \) (which varies with \( p_c \)) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of \( f_{st} \) as follows:

\[
f_{st} = 0.58 f_y \times \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
\]

\[
\Rightarrow f_{st} = (0.58 \times 415 \times 2932 / 2945) = 239.63 \text{ N/mm}^2
\]

\( F = 1.00 \)
\( F_1 = 0.93 \)
\[ F_2 = 0.93 \]

\[ \therefore \quad (l/d)_{\text{max}} = \left(26 \times 1 \times 0.93 \times 0.93\right) = 22.37 \]

\[ (l/d)_{\text{provided}} = 9.73 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

\[ V_{ud} = 118.66667 \]

The critical section for shear is at a distance of 860 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ \left(118.666666666667 \times 1000 / (300 \times 860)\right) = 0.46 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ Tc_{\text{max}} = 0.62 fc'k \]

\[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \text{Sqrt}(30)) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by

\[ A_{st} = 2945 \text{ mm}^2 \]

Percentage of steel, \( p_t \) (\%)

\[ p_t = 1.14 \]

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \tau_c = \frac{0.85 \times 0.8 f_{ck} - 1}{6} \]

where \( \frac{0.8 f_{ck}}{\sqrt{2}} \) whichever is greater

For (M30 and Fe415)

\[ \Rightarrow \tau_c = 0.69 \text{ N/mm}^2 \]

\[ \Rightarrow V_{uc} = (0.69 \times 300 \times 860 / 1000) = 178 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow V_{us} = (119 - 178) = -59 \text{ kN} \]

Using 8 mm bars and

No of legs 2
Area of stirrups, $A_{sv}$ (mm²) = 101

⇒ required spacing $s_v \leq \frac{0.87 \times 415 \times 101 \times 860}{-59.19 \times 1000}$

⇒ Spacing, $s_v = -527$ mm

**Check whether $\tau_v \geq 0.5 \tau_c$**

Nominal shear stress, $\tau_v$ (N/mm²) = 0.46
Design shear stress, $\tau_c$ (N/mm²) = 0.69

$\tau_v \geq 0.5 \tau_c$  Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \text{ When } s_v = 0.5 \tau_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$$s_v \leq 0.75 d \leq 300 \text{ mm}$$

**Code requirements for maximum spacing.**

i) $< \frac{2.175 \times 415 \times 101}{300} = 302$ mm
ii) $\leq \frac{0.75 \times 859.5}{300} = 645$ mm
iii) $\leq 300$ mm
iv) $\leq \frac{0.87 \times 415 \times 101 \times 860}{-59.19 \times 1000} = -527$ mm
**Beam B3 Mid**

**Design Parameters**

Load Case 16 \[1.5*(DL - EQZ)]

Grade of Concrete \[M30\]

Grade of Steel \[Fe415\]

Characteristic compressive strength of concrete , \(f_{ck}\) ( N/mm\(^2\) ) \[30\]

Characteristic yield strength of steel , \(f_y\) ( N/mm\(^2\) ) \[415\]

Unit weight of concrete , \(\gamma_c\) ( kN/m\(^3\) ) \[24\]

Partial safety factor for concrete \[1.5\]

Exposure condition \[Mild\]

Nominal Cover to exposure condition( mm ) \[20\]

**Dimensions of the beam**

- C/C Span of the beam , \(l\) ( m ) \[8.36\]
- Breadth of the beam , \(b\) ( mm ) \[300\]
- Overall depth of the beam , \(D\) ( mm ) \[900\]

**Details of reinforcements**

- Diameter of tension reinforcement ( mm ) \[20\]
- Diameter of compression reinforcement ( mm ) \[20\]
- Diameter of stirrups ( mm ) \[8\]

**Effective depth**

\[
\text{Effective depth} , d \text{ ( mm )} = (900-20-8-20/2) = 862 \]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Moment</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>11 kN-m</td>
</tr>
<tr>
<td>Bending Moment Mu(kNm)</td>
<td>303</td>
</tr>
<tr>
<td>Equivalent Bending Moment</td>
<td>329</td>
</tr>
<tr>
<td>Shear force at critical distance</td>
<td>47</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>106</td>
</tr>
</tbody>
</table>

**Singly reinforced or doubly reinforced section ?**

The limiting moment of resistance , \(M_{ul,lim}\) is given by

\[
M_{ul,lim} = 0.362f_{ck} \times \frac{bxu_{max}}{d} \times 0.416xu_{max}
\]

Where \(b\) = Breadth of the Section

\(xu_{max}\) = Limiting depth of Neutral Axis

\(d\) = Effective depth of the Section

The limiting percentage of steel , \(p_{ul,lim}\) is given by
The area of steel for a singly reinforced section with width, $b$, and depth, $d$, and ultimate moment, $M_u$, is given by:

For ($M30$ and $Fe415$)

$$x_{u,\text{max}} / d = 0.48$$

$$\Rightarrow M_{u,\text{lim}} = \left( 0.1389 \times 30 \times 300 \times 862^2 / 1000000 \right) = 928.88 \text{ kNm}$$

$$\Rightarrow \rho_{\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433$$

If $M_u > M_{u,\text{lim}}$, the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If $M_u < M_{u,\text{lim}}$, the section can be designed as singly reinforced.

**Check for the type of section**

$$M_u = 328.88 \text{ kNm}$$

$$M_{u,\text{lim}} = 928.88 \text{ kNm}$$

$$\Rightarrow \text{Section can be designed as singly reinforced.}$$

**Determining $A_{st}$**

- Considering a ‘balancing section’ ($x_u = x_{u,\text{max}}$)

$$A_{st} = A_{st,\text{lim}} + \Delta A_{st}$$

where $A_{st,\text{lim}} = \rho_{\text{lim}} / 100 \left( b \times d \right)$

$$\Rightarrow A_{st,\text{lim}} \left( 1.433 / 100 \times 300 \times 862 \right) = 3706 \text{ mm}^2$$

- Assuming 20 mm bars for compression steel,

$$d' \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 20 / 2 \right) = 38 \text{ mm}$$
\[ A_{\text{st}} = \frac{M_u}{0.87 f_y} \left( \frac{d}{d'} \right) \]

\[ p_t = \frac{R}{100} \frac{R_{\text{lim}}}{0.87 f_y} \left( \frac{d}{d'} \right) \]

\[ M_u = 0.87 f_y A_{\text{st}} d' (1 - (A_{\text{st}} f_y)/b d' f_{\text{ck}}) \]

**Ast Reqd** = 1124 mm²

\[ \therefore \text{No of tension bars required ( # )} \]

\[ \frac{1124}{(\pi / 4 \times 20^2)} = 4.00 \]

Actual percentage of steel, \( p_t (\% ) \)

\[ \frac{4 \times \pi / 4 \times 20^2}{300 / 862 \times 100} \]

Actual area of steel, \( A_{\text{st}} (\text{mm}^2) \)

\[ \frac{4 \times \pi / 4 \times 20^2}{1257} \]

*Determining \( A_{\text{sc}} \)*

The compression steel, \( A_{\text{sc}} \), is given by

\[ A_{\text{sc}} = \frac{0.87 f_y A_{\text{st}}}{f_{\text{sc}} 0.447 f_{\text{ck}}} \]

or

\[ p_c = \frac{0.87 f_y p_t}{f_{\text{sc}} 0.447 f_{\text{ck}}} \]

where \( f_{\text{sc}} \) is the stress in compression steel.

The values of \( f_{\text{sc}} \) (in MPa units) at \( x_u = x_u,_{\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_u,_{\text{max}} \), for \( d' / d = (38 / 862) = 0.044 \)
  
  From the above table: by interpolation

*Design Check*

- To ensure \( x_u \leq x_u,_{\text{max}} \), it suffices to establish \( p_c \geq p_c^* \)
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{y.e}} \frac{0.447 f_{ck}}{0.447 f_{ck}} p_t - p_{t,\text{lim}}$$

Actual $p_t$ provided : $p_t = 0.49$
Actual $p_c$ provided : $p_c = 0.85$

$$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (0.486 - 1.433))}{(354.8 - 0.447 \times 30)}$$

$$\Rightarrow p_c^* = -1.00$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans up to 10 m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2$$

where $\frac{l}{d_{\text{basic}}} = 7$ for cantilever spans
$\frac{l}{d_{\text{basic}}} = 20$ for simply supported spans
$\frac{l}{d_{\text{basic}}} = 26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

**The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.**

$$f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 1689)}{1257} = 323.54 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 1.23$
\[ F_2 = 1.21 \]

\[ \therefore \quad \frac{l}{d}_{\text{max}} = \frac{26 \times 1 \times 1.23 \times 1.21}{9.70} = 38.53 \]

\[ \therefore \quad \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 862 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ \frac{105.666666666667 \times 1000}{300 \times 862} = 0.41 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ \tau_{c,\text{max}} = 0.62 f_{ck} \sqrt{3} \]

\[ \Rightarrow \quad \tau_{c,\text{max}} = (0.62 \times \sqrt{3}) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \quad \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by

Percentage of steel, \( p_t (\%) \)

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \tau_c = \frac{0.85 - 0.8 f_{ck}}{6} \]

where \( \frac{0.8 f_{ck}}{6.89 p_t} \) whichever is greater

For (\( M30 \) and \( Fe415 \))

\[ \Rightarrow \quad \tau_c = 0.49 \text{ N/mm}^2 \]

\[ \Rightarrow \quad V_{uc} = \frac{0.49 \times 300 \times 862}{1000} = 127 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow \quad V_{us} = (106 - 127) = -21 \text{ kN} \]

Using 8 mm bars and

No of legs \( = 2 \)
Area of stirrups, $A_{sv}$ (mm$^2$) 101

Required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 862}{-21.27 \times 1000} \right)$

Spacing, $s_v = -1471$ mm

Check whether $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm$^2$) 0.41
Design shear stress, $\tau_c$ (N/mm$^2$) 0.49

$\tau_v > 0.5 \tau_c$ Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y}$$

When $s_v = 0.5 \tau_c$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$$s_v \leq 0.75 d$$

$$s_v \leq 300 \text{ mm}$$

Code requirements for maximum spacing:

i) $< \left( \frac{2.175 \times 415 \times 101}{300} \right) = 302$ mm
ii) $\leq \left( 0.75 \times 862 \right) = 647$ mm
iii) $\leq 300 \text{ mm}$ $300 \text{ mm}$
iv) $\leq \left( \frac{0.87 \times 415 \times 101 \times 862}{-21.27 \times 1000} \right) = -1471$ mm
**Beam B4 Support**

**Design Parameters**

Load Case 15 \([1.5^{*}(DL + EQZ)]\)

- Grade of Concrete \(M30\)
- Grade of Steel \(Fe415\)
- Characteristic compressive strength of concrete \(f_{ck} (N/mm^2)\) 30
- Characteristic yield strength of steel \(f_y (N/mm^2)\) 415
- Unit weight of concrete \(\gamma_c (kN/m^3)\) 24
- Partial safety factor for concrete 1.5
- Exposure condition Mild
- Nominal Cover to exposure condition (mm) 20

**Dimensions of the beam**

| C/C Span of the beam , \(l\) (m) | 1.45 |
| Breadth of the beam , \(b\) (mm) | 350 |
| Overall depth of the beam , \(D\) (mm) | 900 |

**Details of reinforcements**

| Diameter of tension reinforcement (mm) | 25 |
| Diameter of compression reinforcement (mm) | 25 |
| Diameter of stirrups (mm) | 8 |

**Effective depth**

| Effective depth , \(d\) (mm) | \((900-20-8-25/2) = 860\) |

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

| Torsional Moment | 20 kN-m |
| Bending Moment \(Mu(kN-m)\) | 706 |
| Equivalent Bending Moment , \(M_e (kNm)\) | 753 |
| Shear force at critical distance , \(V_{ud} (kN)\) | 282 |
| Equivalent Shear (kN) | 389 |

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, \(M_{u,lim}\) is given by

\[ M_{u,lim} = 0.362fck * \frac{bxu_{\text{max}}}{d} * 0.416xu_{\text{max}} \]

Where \(b\) = Breadth of the Section  
\(xu_{\text{max}}\) = Limiting depth of Neutral Axis  
\(d\) = Effective depth of the Section

The limiting percentage of steel, \(p_{u,lim}\) is given by
The area of steel for a singly reinforced section with
width , b and depth , d and ultimate moment , M_u is given by :

\[
Pt \times \frac{Ast}{bd} \times \frac{fck}{2fy} = 4.598 \frac{R}{fck}
\]

Where \( R = \frac{Mu}{bd^2} \)

For ( M30 and Fe415 )

\[
M_{u,lim} = 0.1389 \frac{fck b d^2}{} = 923.50 \text{ kNm}
\]

\[
pt,lim = (41.3 \times 30 / 415 \times 0.48) = 1.433
\]

If \( M_u > M_{u,lim} \), the section has to be
i) get increased by depth or width ( preferably depth )
ii) doubly reinforced

If \( M_u < M_{u,lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 753.06 \text{ kNm}
\]

\[
M_{u,lim} = 923.50 \text{ kNm}
\]

\( \Rightarrow \) Section can be designed as singly reinforced.

**Determining \( A_{st} \)**

- Considering a ' balanced section ' ( \( x_u = x_{u,max} \) )
  \[
  A_{st} = A_{st,lim} + \Delta A_{st}
  \]
  where \( A_{st,lim} = pt,lim / 100 (b \times d) \)
  \[
  \Rightarrow A_{st,lim} = (1.433 / 100 \times 300 \times 859.5) = 3695 \text{ mm}^2
  \]

- Assuming 25 mm bars for compression steel,
  \[
  d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm}
  \]
\[
A_{st} = \frac{M_u}{0.87 f_y d d'}
\]
\[
p_t = \frac{R_t}{100} \frac{R_{lim}}{0.87 f_y \frac{1}{d} d'}
\]

\[
M_u = 0.87 f_y \ast d (1 - (\ast f_y / b d fck))
\]

\[
Ast \text{ Req } d = 2868 \text{ mm}^2
\]

\[
\therefore \text{ No of tension bars required ( # )} = \frac{2868}{(\pi / 4 \times 25^2)} = 6.00
\]

\[
\text{Actual percentage of steel , } p_t (\%) = \frac{(6 \times \pi / 4 \times 25^2 / 300 / 860 \times 100)}{1.14}
\]

\[
\text{Actual area of steel , } A_{st} (\text{mm}^2) = \frac{(6 \times \pi / 4 \times 25^2)}{2945}
\]

**Determining A_{sc}**

The compression steel , \( A_{sc} \), is given by

\[
A_{sc} = \frac{0.87 f_y A_{st}}{f_{sc} 0.447 f_{ck}}
\]

or

\[
p_c = \frac{0.87 f_y p_t}{f_{sc} 0.447 f_{ck}}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u, max} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u, max} \), for \( d' / d = \frac{40.5}{859.5} \) = 0.047
  
  From the above table : by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u, max} \), it suffices to establish \( p_c \geq p_c^* \).
where $p_{c}^*$ is given by

$$
p_{c}^* = \frac{0.87 f_{y}}{f_{sc}} \frac{0.447 f_{ck}}{p_{t} - p_{t,lim}}
$$

Actual $p_t$ provided : $p_t = 1.14$

Actual $p_c$ provided : $p_c = 0.38$

$$
\Rightarrow p_{c}^* = \left( 0.87 \times 415 \times ( 1.142 - 1.433 ) \right) / ( 354.98 - 0.447 \times 30 ) = -0.31
$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l / d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$
\frac{l}{d_{max}} = \frac{l}{d_{basic}} \times F_1 \times F_2
$$

where

- $\frac{l}{d_{basic}} = 7$ for cantilever spans
- $\frac{l}{d_{basic}} = 20$ for simply supported spans
- $\frac{l}{d_{basic}} = 26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$:

$$
F = \frac{10}{\text{span in metres}}
$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

![Image](https://via.placeholder.com/150)

**The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.**

$$
f_{st} = 0.58 f_{y} \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
$$

$$
\Rightarrow f_{st} = \left( 0.58 \times 415 \times 3119 / 2945 \right) = 254.88 \text{ N/mm}^2
$$

$F = 1.00$

$F_1 = 0.89$
\[ F_2 = 0.93 \]
\[ \therefore \quad \frac{(l/d)_{\text{max}}}{(l/d)_{\text{provided}}} = \left( \frac{26 \times 1 \times 0.89 \times 0.93}{1.69} \right) = 21.40 \]

Check for shear

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 860 mm from the face of the support.

• Check for adequacy of section

Nominal shear stress, \( \tau_v \)

\[
( 388.666666666667 \times 1000 / (300 \times 860) ) = 1.51 \text{ N/mm}^2
\]

The maximum shear stress is given by:

\[
T_{c\text{ max}} = 0.62 f_{ck}
\]

\[
\Rightarrow \quad \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2
\]

⇒ Adopted section is adequate

• Design shear resistance at critical section

At critical section, \( A_{st} \) is given by 2945 mm²

Percentage of steel, \( p_t \) (%)

1.14

The design shear strength of the concrete, \( \tau_c \), is given by:

\[
\tau_c = \frac{0.95 \cdot 0.8 \cdot f_{ck} \cdot 10^5}{6} \cdot 1
\]

where \( \frac{0.8 f_{ck}}{0.89 p_t} \) whichever is greater

For (M30 and Fe415)

\[
\Rightarrow \quad \tau_c = 0.69 \text{ N/mm}^2
\]

\[
\Rightarrow \quad V_{uc} = (0.69 \times 300 \times 860 / 1000) = 178 \text{ kN}
\]

• Design of "vertical" stirrups

The shear to be resisted by steel, \( V_{us} \) is given by:

\[
V_{us} = V_u - V_{uc}
\]

\[
\Rightarrow \quad V_{us} = (389 - 178) = 211 \text{ kN}
\]

Using 8 mm bars and

No of legs 2

162
Area of stirrups, $A_{sv}$ (mm²) = 101

⇒ required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 860}{210.81 \times 1000} \right)$

⇒ Spacing, $s_v$ = 148 mm

*Check whether $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm²) = 1.51
Design shear stress, $\tau_c$ (N/mm²) = 0.69

$\tau_v > 0.5 \tau_c \quad \text{Yes}$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When} \ s_v = 0.5tc$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$$s_v \quad 0.75 d \quad 300 \text{ mm}$$

Code requirements for maximum spacing:

i) $s_v < \left( \frac{2.175 \times 415 \times 101}{300} \right) = 302 \text{ mm}$

ii) $s_v \leq \left( \frac{0.75 \times 859.5}{300} \right) = 645 \text{ mm}$

iii) $s_v \leq 300 \text{ mm} = 300 \text{ mm}$

iv) $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 860}{210.81 \times 1000} \right) = 148 \text{ mm}$
**Beam B4 Mid**

**Design Parameters**

- Load Case 15 \[1.5\ast (DL + EQZ)\]
- Grade of Concrete \textbf{M30}\par
- Grade of Steel \textbf{Fe415}\par
- Characteristic compressive strength of concrete \(f_{ck} \text{ (N/mm}^2\text{)}\) \textbf{30}\par
- Characteristic yield strength of steel \(f_y \text{ (N/mm}^2\text{)}\) \textbf{415}\par
- Unit weight of concrete \(\gamma_c \text{ (kN/m}^3\text{)}\) \textbf{24}\par
- Partial safety factor for concrete \textbf{1.5}\par
- Exposure condition \textbf{Mild}\par
- Nominal Cover to exposure condition (mm) \textbf{20}\par

**Dimensions of the beam**

- C/C Span of the beam, \(l\) (m) \textbf{1.45}\par
- Breadth of the beam, \(b\) (mm) \textbf{300}\par
- Overall depth of the beam, \(D\) (mm) \textbf{900}\par

**Details of reinforcements**

- Diameter of tension reinforcement (mm) \textbf{20}\par
- Diameter of compression reinforcement (mm) \textbf{20}\par
- Diameter of stirrups (mm) \textbf{8}\par

**Effective depth**

- Effective depth, \(d\) (mm) \((900-20-8-20/2) = \textbf{862}\)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment \textbf{20} kN-m\par
- Bending Moment Mu(kN-m) \textbf{314}\par
- Equivalent Bending Moment, \(M_e\) (kNm) \textbf{361}\par
- Shear force at critical distance, \(V_{ud}\) (kN) \textbf{259}\par
- Equivalent Shear (kN) \textbf{366}\par

**Singly reinforced or doubly reinforced section?**

The \textit{limiting moment of resistance}, \(M_{u,lm}\) is given by

\[
M_{u,lm} = 0.362 f_{ck} \times \frac{bx u_{\text{max}}}{d} \times 0.416 x u_{\text{max}}
\]

Where \(b = \) Breadth of the Section \par
\(x u_{\text{max}} = \) Limiting depth of Neutral Axis \par
\(d = \) Effective depth of the Section

The limiting percentage of steel, \(p_{\text{lim}}\) is given by
The area of steel for a singly reinforced section with width, b and depth, d and ultimate moment, $M_u$ is given by:

For (M30 and Fe415)

\[
\frac{x_{u,max}}{d} = 0.48
\]

\[
M_u,lim = \left( 0.1389 \times 30 \times 300 \times 862^2 / 1000000 \right) = 928.88 \text{ kNm}
\]

\[
\frac{p_{t,lim}}{bd^2} = 1.433
\]

If $M_u > M_{u,lim}$, the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If $M_u < M_{u,lim}$, the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 361.06 \text{ kNm}
\]

\[
M_{u,lim} = 928.88 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining $A_{st}$**

- Considering a 'balanced section' ($x_u = x_{u,max}$)
  
  \[
  A_{st} = A_{st,lim} + \Delta A_{st}
  \]
  
  where $A_{st,lim} = \frac{p_{t,lim}}{100} \times (b \times d)$

\[
\Rightarrow A_{st,lim} = \left( 1.433 / 100 \times 300 \times 862 \right) = 3706 \text{ mm}^2
\]

- Assuming 20 mm bars for compression steel,

\[
d' = \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 20 / 2 \right) = 38 \text{ mm}
\]
\[
M_u = 0.87 f_y A_{st} d (1 - (A_{st} f_y) / b d f_{ck})
\]

\[
A_{st} = \frac{M_u}{0.87 f_y d d'}
\]

\[
p_t = \frac{R}{100} \frac{R_{lim}}{0.87 f_y} 1 \frac{d'}{d}
\]

\[
M_u = 0.87 f_y A_{st} d (1 - (A_{st} f_y) / b d f_{ck})
\]

\[
A_{st} \text{ Reqd} = 1243 \text{ mm}^2
\]

∴ No of tension bars required ( # )

\[
\frac{1243}{(\pi / 4 \times 20^2)} = 5.00
\]

Actual percentage of steel, \( p_t \) ( % )

\[
\frac{4 \times \pi / 4 \times 20^2}{300 / 862 \times 100} = 0.49
\]

Actual area of steel, \( A_{st} \) ( mm² )

\[
\frac{4 \times \pi / 4 \times 20^2}{1571}
\]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by

\[
A_{sc} = \frac{0.87 f_y A_{st}}{f_{sc} 0.447 f_{ck}}
\]

or

\[
p_c = \frac{0.87 f_y p_t}{f_{sc} 0.447 f_{ck}}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>Fe250</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
</tr>
</tbody>
</table>

Assuming \( x_u = x_{u,\text{max}} \), for \( d' / d = (38 / 862) = 0.044 \)

From the above table : by interpolation

**Design Check**

To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_{c,\ast} \)
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 \cdot f_y}{f_{ck}} - \frac{0.447 \cdot f_{ck}}{p_t - p_{t,\text{lim}}}$$

Actual $p_t$ provided: $p_t = 0.49$
Actual $p_c$ provided: $p_c = 0.85$

$$\Rightarrow p_c^* = (0.87 \times 415 \times (0.486 - 1.433) / (354.8 - 0.447 \times 30))$$
$$\Rightarrow p_c^* = -1.00$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l / d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2$$

where

- $\frac{l}{d_{\text{basic}}} = 7$ for cantilever spans
- $\frac{l}{d_{\text{basic}}} = 20$ for simply supported spans
- $\frac{l}{d_{\text{basic}}} = 26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \left(0.58 \times 415 \times 1797 / 1257\right) = 344.26 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 1.13$
\[ F_2 = 1.21 \]

\[ \therefore \quad ( \frac{l}{d} )_{\text{max}} = ( 26 \times 1 \times 1.13 \times 1.21 ) = 35.62 \]

\[ ( \frac{l}{d} )_{\text{provided}} = 1.68 \]

\[ \Rightarrow \quad \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 862 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)

  \[
  ( \frac{365.666666666667 \times 1000}{300 \times 862} ) = 1.41 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:

  \[
  T_{c, \text{max}} = 0.62 f_{ck}
  \]

  \[ \Rightarrow \quad \tau_{c, \text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

  \[ \Rightarrow \quad \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by 1257 mm²

  Percentage of steel, \( p_t \) (%) 0.49

  The design shear strength of the concrete, \( \tau_c \), is given by:

  \[
  \tau_c = \frac{0.95 - 0.8 f_{ck}}{6} \times 1 \quad 5 \quad 1
  \]

  where

  \[
  \frac{0.8 f_{ck}}{6.89 p_t} \quad \text{whichever is greater} \quad 1
  \]

  For (M30 and Fe415)

  \[ \Rightarrow \quad \tau_c = 0.49 \text{ N/mm}^2 \]

  \[ \Rightarrow \quad V_{uc} = (0.49 \times 300 \times 862 / 1000) = 127 \text{ kN} \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:

  \[ V_{us} = V_u - V_{uc} \]

  \[ \Rightarrow \quad V_{us} = (366 - 127) = 239 \text{ kN} \]

  Using 8 mm bars and

  No of legs 2
Area of stirrups, $A_{sv}$ (mm$^2$) 101

⇒ required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 101 \times 862}{238.73 \times 1000} \right)$

⇒ Spacing, $s_v = 131$ mm

*Check whether $\tau_v > 0.5 \tau_c$*

| Nominal shear stress, $\tau_v$ (N/mm$^2$) | 1.41 |
| Design shear stress, $\tau_c$ (N/mm$^2$) | 0.49 |

$\tau_v > 0.5 \tau_c$ *Yes*

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$
A_{sv} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5tc
$$

$$
sv = \frac{2.175 \; fyA_{sv}}{b}
$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$$
s_v = 0.75 \; d
$$

$$
s_v = 300 \; \text{mm}
$$

*Code requirements for maximum spacing.*

1) $< \left( \frac{2.175 \times 415 \times 101}{300} \right) = 302 \; \text{mm}$
2) $\leq \left( 0.75 \times 862 \right) = 647 \; \text{mm}$
3) $\leq 300 \; \text{mm}$
4) $\leq \left( 0.87 \times 415 \times 101 \times 862 \right) \left/ \left( 238.73 \times 1000 \right) \right. = 131 \; \text{mm}$
**Beam B6 Support**

**Design Parameters**

Load Case 14 \([1.5*(DL - EQX)]\)
Grade of Concrete \(M30\)
Grade of Steel \(Fe415\)
Characteristic compressive strength of concrete , \(f_{ck} \ (N/mm^2)\) \(30\)
Characteristic yield strength of steel , \(f_y \ (N/mm^2)\) \(415\)
Unit weight of concrete , \(\gamma_c \ (kN/m^3)\) \(24\)
Partial safety factor for concrete \(1.5\)
Exposure condition \(Mild\)
Nominal Cover to exposure condition\((mm)\) \(20\)

**Dimensions of the beam**

\[
\begin{align*}
&\text{C/C Span of the beam , } l , \ (m) \quad 1.40 \\
&\text{Breadth of the beam , } b \ (mm) \quad 250 \\
&\text{Overall depth of the beam , } D \ (mm) \quad 550
\end{align*}
\]

**Details of reinforcements**

\[
\begin{align*}
&\text{Diameter of tension reinforcement} \ (mm) \quad 20 \\
&\text{Diameter of compression reinforcement} \ (mm) \quad 20 \\
&\text{Diameter of stirrups} \ (mm) \quad 8
\end{align*}
\]

**Effective depth**

\[
\text{Effective depth} \ , \ d \ (mm) \quad (550-20-8-20/2) = 512
\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.

\[
\begin{align*}
&\text{Torsional Moment} \quad 10 \ kN-m \\
&\text{Bending Moment} \ Mu(kN-m) \quad 245 \\
&\text{Equivalent Bending Moment} \ , \ M_e \ (kNm) \quad 264 \\
&\text{Shear force at critical distance} \ , \ V_{ud} \ (kN) \quad 55 \\
&\text{Equivalent Shear} \ (kN) \quad 119
\end{align*}
\]

**Singly reinforced or doubly reinforced section ?**

The *limiting moment of resistance*, \(M_{u,lim}\) is given by

\[M_{u,lim} = 0.362fck \ * \ \frac{bxu_{max}}{d} \ * \ 0.416xu_{max}\]

Where \(b = \text{Breadth of the Section}\)
\(xu_{max} = \text{Limiting depth of Neutral Axis}\)
\(d = \text{Effective depth of the Section}\)

The limiting percentage of steel , \(p_{u,lim}\) is given by
The area of steel for a singly reinforced section with width, \(b\) and depth, \(d\) and ultimate moment, \(M_u\) is given by:

\[
Pt_{\text{lim}} = 41.61 \times f_{ck} \times xu_{\text{max}} \times d / fy \\
Where \ f_{ck} = \text{Characteristic Compressive strength of concrete} \\
fy = \text{Characteristic strength of steel}
\]

For \((M30 \text{ and Fe415})\)

\[
M_{u,\text{lim}} \approx 0.1389 \times f_{ck} b d^2/100 \\
xu_{\text{max}} / d = 0.48
\]

\[
\Rightarrow M_{u,\text{lim}} = (0.1389 \times 30 \times 250 \times 512^2 / 1000000) = 273.09 \text{ kNm}
\]

\[
\Rightarrow p_{\text{lim}} = (41.3 \times 30 / 415 \times 0.48) = 1.433
\]

If \(M_u > M_{u,\text{lim}}\), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \(M_u < M_{u,\text{lim}}\), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 263.82 \text{ kNm} \\
M_{u,\text{lim}} = 273.09 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \(A_{st}\)**

- Considering a ‘balanced section’ (\(x_u = xu_{\text{max}}\))

\[
A_{st} = A_{st,\lim} + \Delta A_{st}
\]

where \(A_{st,\lim} = p_{\text{lim}} / 100 \times b \times d\)

\[
\Rightarrow A_{st,\lim} = (1.433 / 100 \times 250 \times 512) = 1834 \text{ mm}^2
\]

- Assuming 20 mm bars for compression steel,

\[
d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 20 / 2) = 38 \text{ mm}
\]
\[ M_u = 0.87 f_y^* \text{AST} d(1-(\text{AST} f_y)/b d f_{ck}) \]

\[ \text{AST Req'd} = 1763 \text{ mm}^2 \]

\[ \therefore \text{No of tension bars required ( # )} \]
\[ (1763 / (\pi / 4 x 20^2) = 6.00 \]

Actual percentage of steel, \( p_t \) ( % )
\[ (6 x \pi / 4 x 20^2 / 250 / 512 x 100) = 1.47 \]

Actual area of steel, \( A_{st} \) ( mm² )
\[ (6 x \pi / 4 x 20^2) = 1885 \]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by
\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \cdot \frac{A_{st}}{0.447 f_{ck}}
\]

or
\[
\psi_c = \frac{0.87 f_y}{f_{sc}} \cdot \frac{p_t}{p_{t,lim}} \cdot \frac{0.447 f_{ck}}{f_{ck}}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,\max} \) for various \( d'/d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d'/d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

• Assuming \( x_u = x_{u,\max} \), for \( d'/d = (38/512) = 0.074 \)

From the above table : by interpolation

**Design Check**

• To ensure \( x_u \leq x_{u,\max} \), it suffices to establish \( \psi_c \geq \psi_c^* \)
where $p_c^*$ is given by

$$
p_c^* = \frac{0.87 f_y}{f_{sc}^*} \frac{0.447 f_{ck}}{p_t - p_{t,lim}}
$$

Actual $p_t$ provided: $p_t = 1.47$
Actual $p_c$ provided: $p_c = 0.25$

$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (1.473 - 1.433))}{(354.61 \times 0.447 \times 30)}$

$\Rightarrow p_c^* = 0.04$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} \times F_1 \times F_2$$

where $\frac{l}{d_{\text{basic}}}$

- 7 for cantilever spans
- 20 for simply supported spans
- 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_s$ as follows:

$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 1780 / 1885)}{227.32} \approx 227.32 \text{ N/mm}^2$$

$F = 1.00$

$F_1 = 0.83$
\[ F_2 = 0.75 \]

\[ (l/d)_{\text{max}} = \frac{(26 \times 1 \times 0.83 \times 0.75)}{1} = 16.36 \]

\[ (l/d)_{\text{provided}} = 2.73 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

119

The critical section for shear is at a distance of 512 mm from the face of the support.

- **Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ (119 \times 1000) / (250 \times 512) = 0.93 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ T_{c,\text{max}} = 0.62 f'_{ck} \]

\[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by 1885 mm²

Percentage of steel, \( p_t \) (%)

1.47

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \tau_c = \frac{0.85 \times 0.8 f_{ck} \times 1 \times 5}{6} \]

where \( \frac{0.8 f_{ck}}{6.89 p_t} \) whichever is greater

For (M30 and Fe415)

\[ \Rightarrow \tau_c = \frac{0.76}{0.25} = 0.76 \text{ N/mm}^2 \]

\[ \Rightarrow V_{uc} = (0.76 \times 250 \times 512 / 1000) = 97 \text{ kN} \]

- **Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow V_{us} = (119 - 97) = 22 \text{ kN} \]

Using 8 mm bars and

No of legs 2
Area of stirrups, $A_{sv}$ (mm²) = 101

⇒ required spacing $s_v \leq \frac{0.87 \times 415 \times 101 \times 512}{22.05 \times 1000}$

⇒ Spacing, $s_v = 843$ mm

Check whether $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm²) = 0.93
Design shear stress, $\tau_c$ (N/mm²) = 0.76

$\tau_v > 0.5 \tau_c \quad \text{Yes}$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$A_{sv} = \frac{0.4}{b_{sv}} \frac{0.87 f_y}{\tau_v} \quad \text{When} \quad s_v = 0.5 \tau_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v = 0.75 d$

300 mm

Code requirements for maximum spacing:

i) $< \frac{2.175 \times 415 \times 101}{250} = 363$ mm

ii) $\leq \frac{0.75 \times 512}{b} = 384$ mm

iii) $\leq 300$ mm

iv) $\leq \frac{0.87 \times 415 \times 101 \times 512}{22.05 \times 1000} = 843$ mm

175
**Beam B6 Mid**

*Design Parameters*

Load Case 14  
[1.5*(DL - EQX)]

Grade of Concrete  
M30

Grade of Steel  
Fe415

Characteristic compressive strength of concrete , $f_{ck}$ ( N/mm$^2$ )  
30

Characteristic yield strength of steel , $f_y$ ( N/mm$^2$ )  
415

Unit weight of concrete , $\gamma_c$ ( kN/m$^3$ )  
24

Partial safety factor for concrete  
1.5

Exposure condition  
Mild

Nominal Cover to exposure condition( mm )  
20

*Dimensions of the beam*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C/C Span of the beam , $l$ ( m )</td>
<td>1.40</td>
</tr>
<tr>
<td>Breadth of the beam , $b$ ( mm )</td>
<td>250</td>
</tr>
<tr>
<td>Overall depth of the beam , $D$ ( mm )</td>
<td>550</td>
</tr>
</tbody>
</table>

*Details of reinforcements*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of tension reinforcement ( mm )</td>
<td>20</td>
</tr>
<tr>
<td>Diameter of compression reinforcement ( mm )</td>
<td>20</td>
</tr>
<tr>
<td>Diameter of stirrups ( mm )</td>
<td>8</td>
</tr>
</tbody>
</table>

*Effective depth*

Effective depth , $d$ ( mm ) 
( 550-20-8-20/2 ) = 512

*Design Moment, Shear Force*

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Moment</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>10 kN-m</td>
</tr>
<tr>
<td>Bending Moment $Mu$ (kN-m)</td>
<td>118</td>
</tr>
<tr>
<td>Equivalent Bending Moment , $M_e$ ( kNm )</td>
<td>137</td>
</tr>
<tr>
<td>Shear force at critical distance , $V_{ud}$ ( kN )</td>
<td>55</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>119</td>
</tr>
</tbody>
</table>

*Singly reinforced or doubly reinforced section ?*

The limiting moment of resistance , $M_{u,lim}$ is given by

$$M_{u,lim} = 0.362f_{ck} \times \frac{bx_u}{d} \times \frac{0.416x_u}{x_u}$$

Where $b =$ Breadth of the Section

$x_u$ = Limiting depth of Neutral Axis

$d =$ Effective depth of the Section

The limiting percentage of steel , $\rho_{u,lim}$ is given by
The area of steel for a singly reinforced section with width , b and depth , d and ultimate moment , \( M_u \) is given by :

\[
\frac{Pt}{100} \times \frac{Ast \times fck}{bd \times 2fy} = 4.598 \frac{R}{fck}
\]

Where \( R = \frac{Mu}{bd^2} \)

For ( M30 and Fe415 )

\[
M_u,lim = 0.1389 \times fck \times b \times d^2
\]

\[
x_{u,\text{max}} / d = 0.48
\]

\[
M_u,lim = (0.1389 \times 30 \times 250 \times 512^2 / 1000000) = 273.09 \text{ kNm}
\]

\[
p_{u,\text{lim}} = (41.3 \times 30 / 415 \times 0.48) = 1.433
\]

If \( M_u > M_u,lim \) , the section has to be

i) get increased by depth or width ( preferably depth )

ii) doubly reinforced

If \( M_u < M_u,lim \) , the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 136.82 \text{ kNm}
\]

\[
M_u,lim = 273.09 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a ' balanced section ' ( \( x_u = x_{u,\text{max}} \) )

\[
A_{st} = A_{st,lim} + \Delta A_{st}
\]

where \( A_{st,lim} = p_{u,\text{lim}} / 100 \times b \times d \)

\[
\Rightarrow A_{st,lim} = (1.433 / 100 \times 250 \times 512) = 1834 \text{ mm}^2
\]

- Assuming 20 mm bars for compression steel,

\[
d^* \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 20 / 2) = 38 \text{ mm}
\]
(166x622)\[
\begin{align*}
A_{\text{tr}} &= \frac{M_0}{0.87 f_y d} \frac{M_{0,\text{lim}}}{d'} \\
p_t &= \frac{R}{100} \frac{R_{\text{lim}}}{0.87 f_y} \frac{d'}{d}
\end{align*}
\]

\[Mu = 0.87 f_y^{**} \ast d (1-(\ast d f_{\text{ck}})) \]

\[\text{Ast Reqd} = 811 \text{ mm}^2\]

\[\therefore \text{No of tension bars required ( # )} \]

\[\left( \frac{811}{\pi/4 \times 20^2} \right) = 3.00\]

Actual percentage of steel , \( p_t \) ( % )

\[\left( \frac{3 \times \pi/4 \times 20^2}{250 / 512 \times 100} \right) = 0.74\]

Actual area of steel , \( A_{\text{tr}} \) ( mm\(^2\) )

\[\left( \frac{3 \times \pi/4 \times 20^2}{250 / 512} \right) = 942\]

**Determining \( A_{\text{sc}} \)**

The compression steel , \( A_{\text{sc}} \) , is given by

\[A_{\text{sc}} = \frac{0.87 f_y}{f_{\text{sc}}} \frac{A_{\text{tr}}}{0.447 f_{\text{ck}}} \]

or

\[p_c = \frac{0.87 f_y}{f_{\text{sc}}} \frac{p_t}{0.447 f_{\text{ck}}} \]

where \( f_{\text{sc}} \) is the stress in compression steel.

The values of \( f_{\text{sc}} \) ( in MPa units ) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

* Assuming \( x_u = x_{u,\text{max}} \) , for \( d' / d = \left( \frac{38}{512} \right) = 0.074 \)

From the above table : by interpolation

**Design Check**

* To ensure \( x_u \leq x_{u,\text{max}} \) , it suffices to establish \( p_c \geq p_{c}^{*} \).
where $p_c$ is given by

$$pc = \frac{0.87 f_y}{f_{rc}} \frac{pt}{0.447 f_{ck}}$$

Actual $p_t$ provided: $p_t = 0.74$
Actual $p_c$ provided: $p_c = 0.74$

$$\Rightarrow \quad pc^* = \frac{0.87 \times 415 \times (0.736 - 1.433)}{(354.61 - 0.447 \times 30)}$$

$$\Rightarrow \quad pc^* = -0.74$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2$$

where

- $d_{\text{basic}} = 7$ for cantilever spans
- $d_{\text{basic}} = 20$ for simply supported spans
- $d_{\text{basic}} = 26$ for continuous spans

For simply supported and continuous spans over 10m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

$$f_t = 0.58 \times \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow \quad f_{st} = \frac{(0.58 \times 415 \times 1038)}{942} = 265.12 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 1.13$
Check for shear

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 512 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  \[
  (119 \times 1000) / (250 \times 512) = 0.93 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  \[
  T_{c,\text{max}} = 0.62 f_{ck}
  \]

  \[
  \Rightarrow \tau_{c,\text{max}} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2
  \]

  \( \Rightarrow \text{Adopted section is adequate} \)

- **Design shear resistance at critical section**

  At critical section, \( A_s \) is given by 942 mm

  Percentage of steel, \( p_t \) (%)

  0.74

  The design shear strength of the concrete, \( \tau_c \), is given by:
  \[
  \tau_c = \frac{0.85 \times 0.8 f_{ck}}{6}, 1 - \frac{1}{5}
  \]

  where \( \frac{0.8 f_{ck}}{6.89 p_t} \) whichever is greater 1

  For (M30 and Fe415)

  \[
  \Rightarrow \tau_c = 0.58 \text{ N/mm}^2
  \]

  \[
  \Rightarrow V_{uc} = (0.58 \times 250 \times 512 / 1000) = 74 \text{ kN}
  \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  \[
  V_{us} = V_u - V_{uc}
  \]

  \[
  \Rightarrow V_{us} = (119 - 74) = 45 \text{ kN}
  \]

  Using 8 mm bars and 2 mm bars and
Area of stirrups, $A_{sv}$ (mm$^2$)  

\[ \Rightarrow \text{required spacing } sv \leq \left( \frac{0.87 \times 415 \times 101 \times 512}{44.54 \times 1000} \right) \]

\[ \Rightarrow \text{Spacing, } s_v = 417 \text{ mm} \]

**Check whether $\tau_v > 0.5 \tau_c$**

- Nominal shear stress, $\tau_v$ (N/mm$^2$) 0.93
- Design shear stress, $\tau_c$ (N/mm$^2$) 0.58

$\tau_v > 0.5 \tau_c \quad \text{Yes}$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

\[
\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } sv = 0.5tc
\]

\[
sv = \frac{2.175 f_y A_{sv}}{b}
\]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[
s_v = 0.75 \text{ d}
\]

\[
s_v = 300 \text{ mm}
\]

**Code requirements for maximum spacing.**

i) $< \left( \frac{2.175 \times 415 \times 101}{250} \right) = 363 \text{ mm}$

ii) $\leq (0.75 \times 512) = 384 \text{ mm}$

iii) $\leq 300 \text{ mm}$

iv) $\leq \left( \frac{0.87 \times 415 \times 101 \times 512}{44.54 \times 1000} \right) = 417 \text{ mm}$
**Beam RB1 Support**

**Design Parameters**

Load Case 14 \[1.5*(DL - EQX)\]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete, ( f_{ck} ) ( N/mm(^2))</td>
<td>30</td>
</tr>
<tr>
<td>Characteristic yield strength of steel, ( f_y ) ( N/mm(^2))</td>
<td>415</td>
</tr>
<tr>
<td>Unit weight of concrete, ( \gamma_c ) ( kN/m(^3))</td>
<td>24</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Exposure condition</td>
<td>Mild</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition (mm)</td>
<td>20</td>
</tr>
</tbody>
</table>

**Dimensions of the beam**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C/C Span of the beam, ( l ) (m)</td>
<td>5.35</td>
</tr>
<tr>
<td>Breadth of the beam, ( b ) (mm)</td>
<td>300</td>
</tr>
<tr>
<td>Overall depth of the beam, ( D ) (mm)</td>
<td>500</td>
</tr>
</tbody>
</table>

**Details of reinforcements**

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of tension reinforcement</td>
<td>25</td>
</tr>
<tr>
<td>Diameter of compression reinforcement</td>
<td>25</td>
</tr>
<tr>
<td>Diameter of stirrups</td>
<td>8</td>
</tr>
</tbody>
</table>

**Effective depth**

Effective depth, \( d \) (mm) = (500-20-8-25/2) = 460

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Moment/shear</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>0 kN-m</td>
</tr>
<tr>
<td>Bending Moment Mu(kN-m)</td>
<td>235</td>
</tr>
<tr>
<td>Equivalent Bending Moment, ( M_e ) (kNm)</td>
<td>235</td>
</tr>
<tr>
<td>Shear force at critical distance, ( V_{ud} ) (kN)</td>
<td>125</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>125</td>
</tr>
</tbody>
</table>

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, \( M_{u,\text{lim}} \) is given by

\[
M_{\text{ulim}} = 0.362 f_{ck} \cdot \frac{b x_u \cdot \max}{d} \cdot 0.416 x_u \cdot \max
\]

Where \( b \) = Breadth of the Section

\( x_u \cdot \max = \) Limiting depth of Neutral Axis

\( d = \) Effective depth of the Section

The limiting percentage of steel, \( p_{\text{ulim}} \) is given by
The area of steel for a singly reinforced section with width, $b$, and depth, $d$, and ultimate moment, $M_u$, is given by:

For (M30 and Fe415)

$$x_u,\text{max} / d = 0.48$$

$$M_{u,\text{lim}} = \frac{0.1389 \times 300 \times 30 \times (459.5)^2}{1000000} = 263.95 \text{ kNm}$$

$$p_{t,\text{lim}} = \frac{41.3 \times 30}{415 \times 0.48} = 1.433$$

If $M_u > M_{u,\text{lim}}$, the section has to be:

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If $M_u < M_{u,\text{lim}}$, the section can be designed as singly reinforced.

**Check for the type of section**

$$M_u = 235.00 \text{ kNm}$$

$$M_{u,\text{lim}} = 263.95 \text{ kNm}$$

$$\Rightarrow \text{Section can be designed as singly reinforced.}$$

**Determining A_{st}**

- Considering a 'balanced section' ($x_u = x_u,\text{max}$)

$$A_{st} = A_{st,\text{lim}} + \Delta A_{st}$$

where $A_{st,\text{lim}} = p_{t,\text{lim}} / 100 (b \times d)$

$$\Rightarrow A_{st,\text{lim}} = (1.433 / 100 \times 300 \times 459.5) = 1975 \text{ mm}^2$$

- Assuming 25 mm bars for compression steel,

$$d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm}$$
\[ M_u = 0.87 \cdot f_y \cdot \frac{A_{st} \cdot d - d'}{d'} \]
\[ p_t = \frac{R}{100} \cdot \frac{R_{lim}}{0.87 \cdot f_y \cdot \frac{d'}{d}} \]

\[ M_u = 0.87 \cdot f_y \cdot A_{st} \cdot d(1-\frac{A_{st} \cdot f_y}{b \cdot d \cdot f_{ck}}) \]

\[ A_{st} \text{ Reqd} = 1710 \text{ mm}^2 \]

\[ \therefore \text{ No of tension bars required ( # ) } \]
\[ \left( \frac{1710}{\frac{\pi}{4} \times 25^2} \right) = 4.00 \]

Actual percentage of steel, \( p_t \) ( % )
\[ \left( \frac{4 \times \pi}{4} \times 25^2 / 300 / 460 \times 100 \right) = 1.42 \]

Actual area of steel, \( A_{st} \) ( mm\(^2\) )
\[ \left( \frac{4 \times \pi}{4} \times 25^2 \right) = 1963 \]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by
\[ A_{sc} = \frac{0.87 \cdot f_y \cdot A_{st}}{f_{sc} - 0.447 \cdot f_{ck}} \]

or
\[ p_c = \frac{0.87 \cdot f_y \cdot p_t}{f_{sc} - 0.447 \cdot f_{ck}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_u_{max} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fe250</strong></td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td><strong>Fe415</strong></td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td><strong>Fe500</strong></td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_u_{max} \), for \( d' / d = \left( \frac{40.5}{459.5} \right) = 0.088 \)
  - From the above table : by interpolation

**Design Check**

- To ensure \( x_u \leq x_u_{max} \), it suffices to establish \( p_c \geq p_c^* \)
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{ec}^*} \frac{0.447 f_{ck}}{p_t^* - p_{t,lim}}$$

Actual $p_t$ provided : $p_t = 1.42$
Actual $p_c$ provided : $p_c = 0.36$

$$\Rightarrow p_c^* = \frac{(0.87 \times 415 \times (1.424 - 1.433))}{(353.38 - 0.447 \times 30)}$$

$$\Rightarrow p_c^* = 0.01$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{max}} > \frac{l}{d_{basic}} F_1 F_2$$

where $\frac{l}{d_{basic}}$

7 for cantilever spans
20 for simply supported spans
26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

**The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.**

$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 1784 / 1963)}{218.71} = 218.71 \text{ N/mm}^2$$

$F = 1.00$
$F_1 = 0.87$
Check for shear

Shear force at critical distance, $V_{ud}$ (kN)

The critical section for shear is at a distance of 460 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, $\tau_v$
  
  \[
  \left( \frac{125 \times 1000}{300 \times 460} \right) = 0.91 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  
  \[
  \tau_{c,max} = 0.62 f_{ck} \sqrt{30} = 3.40 \text{ N/mm}^2
  \]

  ⇒ **Adopted section is adequate**

- **Design shear resistance at critical section**

  At critical section, $A_{st}$ is given by 1963 mm²

  Percentage of steel, $p_t$ (%) 1.42

  The design shear strength of the concrete, $\tau_c$, is given by:

  
  \[
  \tau_c = 0.85 \frac{0.8 f_{ck}}{6.89 p_t} \left( \frac{1}{5} - 1 \right) 6
  \]

  where

  \[
  \frac{0.8 f_{ck}}{6.89 p_t} \text{ whichever is greater}
  \]

  For (M30 and Fe415)

  \[
  \tau_c = 0.75 \text{ N/mm}^2
  \]

  \[
  V_{uc} = (0.75 \times 300 \times 460 / 1000) = 103 \text{ kN}
  \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, $V_{us}$ is given by:

  \[
  V_{us} = V_u - V_{uc}
  \]

  ⇒ $V_{us} = (125 - 103) = 22 \text{ kN}$

  Using 12 mm bars and

  No of legs 2
Area of stirrups, \( A_{sv} \) (mm\(^2\)) = 226

\[ \Rightarrow \text{required spacing} \, s_v \leq \left( \frac{0.87 \times 415 \times 226 \times 460}{(21.84 \times 1000)} \right) \]

\[ \Rightarrow \text{Spacing}, \, s_v = 1718 \, \text{mm} \]

**Check whether** \( \tau_v > 0.5 \tau_c \)

<table>
<thead>
<tr>
<th>Nominal shear stress, ( \tau_v ) (N/mm(^2))</th>
<th>0.91</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design shear stress, ( \tau_c ) (N/mm(^2))</td>
<td>0.75</td>
</tr>
</tbody>
</table>

\( \tau_v > 0.5 \tau_c \) \[\text{Yes}\]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):

\[
\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 \, f_{yv}} \quad \text{When} \, s_v = 0.5\tau_c
\]

\[
S_v = \frac{2.175 \, f_{yv} A_{sv}}{b}
\]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\( s_v \)

- 0.75 \( d \)
- 300 mm

**Code requirements for maximum spacing**

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>i) ( &lt; ) ((2.175 \times 415 \times 226 / 300)) =</td>
<td>681 mm</td>
</tr>
<tr>
<td>ii) ( \leq ) ((0.75 \times 459.5))</td>
<td>345 mm</td>
</tr>
<tr>
<td>iii) ( \leq ) 300 mm</td>
<td>300 mm</td>
</tr>
<tr>
<td>iv) ( \leq ) ((0.87 \times 415 \times 226 \times 460 / (21.84 \times 1000)))</td>
<td>1718 mm</td>
</tr>
</tbody>
</table>
**Beam RB1 Mid**

**Design Parameters**

Load Case 14 \[ 1.5*(DL - EQX) \]
Grade of Concrete \[ M30 \]
Grade of Steel \[ Fe415 \]
Characteristic compressive strength of concrete , f_{ck} (N/mm\(^2\)) \[ 30 \]
Characteristic yield strength of steel , f_y (N/mm\(^2\)) \[ 415 \]
Unit weight of concrete , \( \gamma_c \) (kN/m\(^3\)) \[ 24 \]
Partial safety factor for concrete \[ 1.5 \]
Exposure condition \[ Mild \]
Nominal Cover to exposure condition (mm) \[ 20 \]

**Dimensions of the beam**

- C/C Span of the beam , l , (m) \[ 5.35 \]
- Breadth of the beam , b (mm) \[ 300 \]
- Overall depth of the beam , D (mm) \[ 500 \]

**Details of reinforcements**

- Diameter of tension reinforcement (mm) \[ 25 \]
- Diameter of compression reinforcement (mm) \[ 25 \]
- Diameter of stirrups (mm) \[ 8 \]

**Effective depth**

Effective depth , d (mm) \[ (500-20-8-25/2) = 460 \]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment \[ 0 \] kN-m
- Bending Moment Mu(kN-m) \[ 150 \]
- Equivalent Bending Moment , M_e (kNm) \[ 150 \]
- Shear force at critical distance , V_{ud} (kN) \[ 120 \]
- Equivalent Shear (kN) \[ 120 \]

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance , M_{ulim} is given by

\[ M_{ulim} = 0.362f_{ck} * \frac{b x_u}{d} \max * 0.416x_u \max \]

Where \( b \) = Breadth of the Section
\( x_u \max = \) Limiting depth of Neutral Axis
\( d = \) Effective depth of the Section

The limiting percentage of steel , P_{ulim} is given by
\[ Pt_{\text{lim}} = 41.61 \times \frac{f_{ck} \times x_{u,\text{max}}}{f_y} \]

Where \( f_{ck} = \) Characteristic Compressive strength of concrete
\( f_y = \) Characteristic strength of steel

The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[ \frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{fck}{2fy} = 4.598 \times \frac{R}{fck} \]

Where \( R = \frac{Mu}{bd^2} \)

For (M30 and Fe415) \( M_{u,\text{lim}} = 0.1389 \times f_{ck} b d^2 \)

\( x_{u,\text{max}} / d = 0.48 \)

\[ \Rightarrow M_{u,\text{lim}} = (0.1389 \times 30 \times 300 \times 459.5^2 / 1000000) = 263.95 \text{ kNm} \]

\[ \Rightarrow p_{\text{lim}} = (41.3 \times 30 / 415 \times 0.48) = 1.433 \]

If \( M_u > M_{u,\text{lim}} \), the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If \( M_u < M_{u,\text{lim}} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[ M_u = 150.00 \text{ kNm} \]
\[ M_{u,\text{lim}} = 263.95 \text{ kNm} \]

\[ \Rightarrow \text{Section can be designed as singly reinforced.} \]

**Determining \( A_{st} \)**

- Considering a 'balanced section' \( (x_c = x_{u,\text{max}}) \)
  \[ A_{st} = A_{st,\text{lim}} + \Delta A_{st} \]
  where \( A_{st,\text{lim}} = p_{\text{lim}} / 100 (b \times d) \)

\[ \Rightarrow A_{st,\text{lim}} = (1.433 / 100 \times 300 \times 459.5) = 1975 \text{ mm}^2 \]

- Assuming 25 mm bars for compression steel,

\[ d^* \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm} \]
\[
A_{st} = \frac{M_u}{0.87 f_y} \frac{M_{ulim}}{d \cdot d'}
\]

\[
\rho_{t} = \frac{R}{100} \frac{R_{lim}}{0.87 f_y} \frac{d'}{d}
\]

\[
\mu = 0.87 f_y^2 \ast d (1-(\ast f_y/b \ast fck))
\]

\[
\ast Req = 1006 \text{ mm}^2
\]

\[
\therefore \text{ No of tension bars required ( # ) ( 1006 / ( \pi / 4 \times 25^2 ) = 3.00}
\]

Actual percentage of steel , \( \rho_t \) ( % )
\[
( 3 \times \pi / 4 \times 25^2 / 300 / 460 \times 100 ) = 1.07
\]

Actual area of steel , \( A_{st} \) ( mm\(^2\) )
\[
( 3 \times \pi / 4 \times 25^2 ) = 1473
\]

**Determining \( A_{sc} \)**

The compression steel , \( A_{sc} \), is given by
\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \frac{A_{st}}{0.447 f_{ck}}
\]
or
\[
\rho_c = \frac{0.87 f_y}{f_{sc}} \frac{\rho_{t}}{0.447 f_{ck}}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,max} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

\* Assuming \( x_u = x_{u,max} \), for \( d' / d = (40.5 / 459.5) = 0.088 \)

From the above table : by interpolation

**Design Check**

\* To ensure \( x_u \leq x_{u,max} \), it suffices to establish \( \rho_c \leq \rho_{c*} \).
where $p_c^*$ is given by

$$
p_c^* = \frac{0.87 f_y}{f_{ck}} \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}} \frac{p_t}{p_{t,\text{lim}}}
$$

Actual $p_t$ provided: $p_t = 1.07$
Actual $p_c$ provided: $p_c = 0.71$

$$
\Rightarrow p_c^* = \frac{0.87 \times 415 \times (1.068 - 1.433)}{(353.38 - 0.447 \times 30)}
\Rightarrow p_c^* = -0.39
$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans up to 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$
\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2
$$

where

- $\frac{l}{d_{\text{basic}}}$ = 7 for cantilever spans
- $\frac{l}{d_{\text{basic}}}$ = 20 for simply supported spans
- $\frac{l}{d_{\text{basic}}}$ = 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$F = \frac{10}{\text{span in metres}}$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

$$
f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
$$

$$
\Rightarrow f_{st} = \frac{(0.58 \times 415 \times 1222)}{1473} = 199.78 \text{ N/mm}^2
$$

$F = 1.00$

$F_1 = 1.10$
\[
F_2 = 1.15
\]
\[
( l / d )_{\text{max}} = \left( \frac{26 \times 1 \times 1.1 \times 1.15}{26 \times 1 \times 1.1 \times 1.15} \right) = 33.00
\]
\[
( l / d )_{\text{provided}} = 11.63
\]
\[
\Rightarrow \text{Hence O.K.}
\]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN) 120

The critical section for shear is at a distance of 460 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  \[
  \left( \frac{120 \times 1000}{300 \times 460} \right) = 0.87 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  \[
  \tau_{c,max} = 0.62 f_{c,k} \]
  \[
  \Rightarrow \tau_{c,max} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2
  \]

  \[
  \Rightarrow \text{Adopted section is adequate}
  \]

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by 1473 \text{ mm}^2

  Percentage of steel, \( p_t \) (\%)

  The design shear strength of the concrete, \( \tau_c \), is given by:
  \[
  \tau_c = \frac{0.85 \times 0.9 \times f_{c,k} - 1}{6} \left( 5 - 1 \right)
  \]
  where \( \frac{0.8f_{ck}}{6.89p_t} \) whichever is greater 1

  For (M30 and Fe415)
  \[
  \Rightarrow \tau_c = 0.67 \text{ N/mm}^2
  \]

  \[
  \Rightarrow V_{uc} = \left( 0.67 \times 300 \times 460 / 1000 \right) = 93 \text{ kN}
  \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  \[
  V_{us} = V_u - V_{uc}
  \]
  \[
  \Rightarrow V_{us} = (120 - 93) = 27 \text{ kN}
  \]

  Using 12 mm bars and
  No of legs 4
Area of stirrups, \( A_{sv} \) (mm²) \( = 452 \)

\[ \Rightarrow \text{required spacing} \ s_v \leq \ (0.87 \times 415 \times 452 \times 460 / (27.29 \times 1000)) \]

\[ \Rightarrow \text{Spacing,} \ s_v = 2750 \text{ mm} \]

**Check whether** \( \tau_v > 0.5 \tau_c \)

Nominal shear stress, \( \tau_v \) (N/mm²) \( = 0.87 \)

Design shear stress, \( \tau_c \) (N/mm²) \( = 0.67 \)

\[ \tau_v > 0.5 \tau_c \quad \text{Yes} \]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds \( 0.5 \tau_c \):\

\[ \frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 \ f_y} \quad \text{When} \ s_v = 0.5 \tau_c \]

\[ s_v = \frac{2.175 \ f_y A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[ s_v = 0.75 \ d \]

\[ s_v = 300 \text{ mm} \]

**Code requirements for maximum spacing.**

\[
\begin{align*}
\text{i)} & \quad < \ (2.175 \times 415 \times 452 / 300) = 1361 \text{ mm} \\
\text{ii)} & \quad \leq \ (0.75 \times 459.5) = 345 \text{ mm} \\
\text{iii)} & \quad \leq \ 300 \text{ mm} = 300 \text{ mm} \\
\text{iv)} & \quad \leq \ (0.87 \times 415 \times 452 \times 460 / (27.29 \times 1000)) = 2750 \text{ mm}
\end{align*}
\]
**Beam RB2 Support**

*Design Parameters*

- **Load Case 14** \([1.5^*(DL - EQX)]\)
- **Grade of Concrete** M30
- **Grade of Steel** Fe415
- **Characteristic compressive strength of concrete** \(f_{ck} (\text{N/mm}^2)\) 30
- **Characteristic yield strength of steel** \(f_y (\text{N/mm}^2)\) 415
- **Unit weight of concrete** \(\gamma_c (\text{kN/m}^3)\) 24
- **Partial safety factor for concrete** 1.5
- **Exposure condition** Mild
- **Nominal Cover to exposure condition** (mm) 20

*Dimensions of the beam*

- C/C Span of the beam, \(l, (\text{m})\) 5.36
- Breadth of the beam, \(b (\text{mm})\) 300
- Overall depth of the beam, \(D (\text{mm})\) 400

*Details of reinforcements*

- Diameter of tension reinforcement (mm) 25
- Diameter of compression reinforcement (mm) 25
- Diameter of stirrups (mm) 8

*Effective depth*

- Effective depth, \(d (\text{mm})\) \((650-20-8-25/2) = 610\)

*Design Moment, Shear Force*

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.

- **Torsional Moment** 16 kN-m
- **Bending Moment** \(M_u (\text{kN-m})\) 325
- **Equivalent Bending Moment** , \(M_e (\text{kNm})\) 355
- **Shear force at critical distance** , \(V_{ud} (\text{kN})\) 70
- **Equivalent Shear** (kN) 155

*Singly reinforced or doubly reinforced section?*

The *limiting moment of resistance*, \(M_{u,lim}\) is given by

\[
M_{u,lim} = 0.362f_{ck} \times \frac{b x_u}{d} \times 0.416 x_u
\]

Where
- \(b\) = Breadth of the Section
- \(x_u\) = Limiting depth of Neutral Axis
- \(d\) = Effective depth of the Section

The limiting percentage of steel , \(p_{u,lim}\) is given by
The area of steel for a singly reinforced section with width , b and depth , d and ultimate moment , \( M_u \) is given by :

\[
Pt_{\text{lim}} = 41.61 \times \frac{f_{ck}}{f_y} \times \frac{x_{u,max}}{d} \times \frac{f_{ck}}{2f_y} = 4.598 \frac{R}{f_{ck}}
\]

Where \( R = \frac{M_u}{bd^2} \)

For ( M30 and Fe415 )

\[
M_{u,\text{lim}} = 0.1389 \frac{f_{ck} b d^2}{2}
\]

\[
x_{u,max} / d = 0.48
\]

\[
\Rightarrow M_{u,\text{lim}} = \left( 0.1389 \times 30 \times 300 \times 609.5^2 / 1000000 \right) = 464.40 \text{ kNm}
\]

\[
\Rightarrow p_{\text{lim}} = \left( 41.3 \times 30 / 415 \times 0.48 \right) = 1.433
\]

If \( M_u > M_{u,\text{lim}} \), the section has to be

i) get increased by depth or width ( preferably depth )

ii) doubly reinforced

If \( M_u < M_{u,\text{lim}} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 354.80 \text{ kNm}
\]

\[
M_{u,\text{lim}} = 464.40 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a ' balanced section ' ( \( x_i = x_{u,max} \) )
  \[
  A_{st} = A_{st,\text{lim}} + \Delta A_{st}
  \]
  where \( A_{st,\text{lim}} = p_{\text{lim}} / 100 \times ( b \times d ) \)

  \[
  \Rightarrow A_{st,\text{lim}} = \left( 1.433 / 100 \times 300 \times 609.5 \right) = 2620 \text{ mm}^2
  \]

- Assuming 25 mm bars for compression steel,

  \[
  d' \approx \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}
  \]
\[
A_{st} = \frac{M_u}{0.87 \ f_y \ d} \ (d' \ d) \\
\frac{p_t}{100} = \frac{R}{0.87 \ f_y \ 1} \\

Mu = 0.87 f_y \ ^* Ast \ d (1 - (Ast \ f_y) / b * d * fck) \\

Ast \ Reqd = 1880 \ mm^2 \\

\therefore \ \text{No of tension bars required ( # )} \\
\frac{1880}{(\ Pi / 4 \times 25^2)} = 4.00 \\

\text{Actual percentage of steel, } p_t (\ %) \\
\frac{(4 \times \ Pi / 4 \times 25^2)}{300 / 610 \times 100} = 1.07 \\

\text{Actual area of steel, } A_{st} (\ mm^2) \\
\frac{(4 \times \ Pi / 4 \times 25^2)}{1963} \\

\text{Determining } A_{sc} \\

\text{The compression steel, } A_{sc}, \text{ is given by} \\
A_{sc} = \frac{0.87 f_y \ A_{st}}{f_{sc} \ 0.447 f_{ck}} \\
\text{or} \\
\frac{p_t}{100} = \frac{0.87 f_y \ p_t}{f_{sc} \ 0.447 f_{ck}} \\

\text{where } f_{sc} \text{ is the stress in compression steel.} \\

\text{The values of } f_{sc} \text{ (in MPa units) at } x_u = x_{u,max} \text{ for various } d' / d \text{ ratios and different grades of compression steel are given in the table below.} \\

\begin{array}{|c|c|c|c|c|}
\hline
\text{Grade of steel} & \text{d' / d} & 0.05 & 0.10 & 0.15 & 0.20 \\
\hline
Fe250 & 217.5 & 217.5 & 217.5 & 217.5 \\
Fe415 & 355.1 & 351.9 & 342.4 & 329.2 \\
Fe500 & 423.9 & 411.3 & 395.1 & 370.3 \\
\hline
\end{array}

\text{• Assuming } x_u = x_{u,max}, \text{ for } d' / d = \frac{40.5}{609.5} = 0.066 \\
\text{From the above table : by interpolation} \\

\text{Design Check} \\

\text{• To ensure } x_u \leq x_{u,max}, \text{ it suffices to establish } p_c \geq p_c^*
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{sc}} \left( \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}} \right)$$

Actual $p_t$ provided : $p_t = 1.07$
Actual $p_c$ provided : $p_c = 0.54$

⇒ $p_c^* = \left( 0.87 \times 415 \times (1.074 - 1.433) / (355.03 - 0.447 \times 30) \right)$

⇒ $p_c^* = -0.38$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2$$

where
- $\frac{l}{d_{\text{basic}}}$ = 7 for cantilever spans
- $\frac{l}{d_{\text{basic}}}$ = 20 for simply supported spans
- $\frac{l}{d_{\text{basic}}}$ = 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_s$ as follows:

**The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.**

$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

⇒ $f_{st} = \left( 0.58 \times 415 \times 2087 / 1963 \right) = 255.82 \text{ N/mm}^2$

$F = 1.00$
$F_1 = 0.91$
\[ F_2 = 1.06 \]

\[
\therefore \quad \frac{L}{d}_{\text{max}} = \frac{26 \times 1 \times 0.91 \times 1.06}{13.72} = 25.16
\]

\Rightarrow \text{Hence O.K.}

**Check for shear**

Shear force at critical distance, \( V_{ul} \) (kN) = 155.33333

The critical section for shear is at a distance of 610 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  \[
  \frac{(155.333333333333 \times 1000)}{(300 \times 610)} = 0.85 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  \[
  \tau_{c,\text{max}} = 0.62 f_{ck} \sqrt{30} = 3.40 \text{ N/mm}^2
  \]

  \Rightarrow \text{Adopted section is adequate}

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by 1963 mm²

  Percentage of steel, \( p_t \) (\%) = 1.07

  The design shear strength of the concrete, \( \tau_c \), is given by:
  \[
  \tau_c = \frac{0.85 \times 0.8 f_{ck} \times 1.5 \times 1}{6} \leq 0.89 p_t \text{ whichever is greater}
  \]

  For (M30 and Fe415)
  \[
  \Rightarrow \tau_c = 0.67 \text{ N/mm}^2
  \]

  \Rightarrow \( V_{uc} = \frac{(0.67 \times 300 \times 610)}{1000} = 123 \text{ kN} \)

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  \[
  V_{us} = V_u - V_{uc}
  \]

  \Rightarrow \( V_{us} = 155 - 123 = 32 \text{ kN} \)

  Using 12 mm bars and
  No of legs 2
Area of stirrups, $A_{sv}$ (mm$^2$) = 226

⇒ required spacing $s_v \leq \frac{0.87 \times 415 \times 226 \times 610}{32.12 \times 1000}$

⇒ Spacing, $s_v = 1550$ mm

Check whether $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm$^2$) = 0.85
Design shear stress, $\tau_c$ (N/mm$^2$) = 0.67

$\tau_v > 0.5 \tau_c$ Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

$$
A_{sv} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5 t_c
$$

$$
s_v = \frac{2.175 f_y A_{sv}}{b}
$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v \leq 0.75 d$

300 mm

Code requirements for maximum spacing:

i) $< (2.175 \times 415 \times 226 / 300) = 681$ mm

ii) $\leq (0.75 \times 609.5) = 457$ mm

iii) $\leq 300$ mm

300 mm

iv) $\leq (0.87 \times 415 \times 226 \times 610 / (32.12 \times 1000)) = 1550$ mm
**Beam RB2 Mid**

**Design Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case 14 [1.5*(DL - EQX)]</td>
<td></td>
</tr>
<tr>
<td>Grade of Concrete</td>
<td>M30</td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>Fe415</td>
</tr>
<tr>
<td>Characteristic compressive strength of concrete , $f_{ck}$ (N/mm$^2$)</td>
<td>30</td>
</tr>
<tr>
<td>Characteristic yield strength of steel , $f_y$ (N/mm$^2$)</td>
<td>415</td>
</tr>
<tr>
<td>Unit weight of concrete , $\gamma_c$ (kN/m$^3$)</td>
<td>24</td>
</tr>
<tr>
<td>Partial safety factor for concrete</td>
<td>1.5</td>
</tr>
<tr>
<td>Exposure condition</td>
<td>Mild</td>
</tr>
<tr>
<td>Nominal Cover to exposure condition (mm)</td>
<td>20</td>
</tr>
</tbody>
</table>

**Dimensions of the beam**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C/C Span of the beam , l , (m)</td>
<td>5.36</td>
</tr>
<tr>
<td>Breadth of the beam , b (mm)</td>
<td>300</td>
</tr>
<tr>
<td>Overall depth of the beam , D (mm)</td>
<td>400</td>
</tr>
</tbody>
</table>

**Details of reinforcements**

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of tension reinforcement (mm)</td>
<td>25</td>
</tr>
<tr>
<td>Diameter of compression reinforcement (mm)</td>
<td>25</td>
</tr>
<tr>
<td>Diameter of stirrups (mm)</td>
<td>8</td>
</tr>
</tbody>
</table>

**Effective depth**

Effective depth , $d$ (mm) \( (650-20-8-25/2) = 610 \)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

- Torsional Moment: 16 kN-m
- Bending Moment $M_u$(kN-m): 190
- Equivalent Bending Moment, $M_e$ (kNm): 220
- Shear force at critical distance, $V_{ud}$ (kN): 165
- Equivalent Shear (kN): 250

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, $M_{u,lim}$ is given by

$$M_{u,lim} = 0.362f_{ck} \times \frac{b x_u_{max}}{d} \times 0.416x_u_{max}$$

Where $b$ = Breadth of the Section
$x_u_{max}$ = Limiting depth of Neutral Axis
$d$ = Effective depth of the Section

The limiting percentage of steel, $\rho_{u,lim}$ is given by
The area of steel for a singly reinforced section with width, \( b \) and depth, \( d \) and ultimate moment, \( M_u \) is given by:

For \( (M30 \text{ and Fe415}) \)

\[
\frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{f_{ck}}{2fy} = \frac{4.598}{f_{ck}} \frac{R}{bd^2}
\]

Where \( R = \frac{M_u}{f_{ck}bd^2} \)

For \( (M30 \text{ and Fe415}) \)

\( M_{ul,lim} = 0.1389 \times f_{ck} b d^2 \)

\( x_{u,max} / d = 0.48 \)

\( M_u,lim = \frac{0.1389 \times 30 \times 300 \times 609.5^2}{1000000} = 464.40 \text{ kNm} \)

\( p_{t,lim} = \frac{41.3 \times 30}{415 \times 0.48} = 1.433 \)

If \( M_u > M_{ul,lim} \), the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If \( M_u < M_{ul,lim} \), the section can be designed as singly reinforced.

\textbf{Check for the type of section}

\[
\begin{align*}
M_u &= 219.80 \text{ kNm} \\
M_{ul,lim} &= 464.40 \text{ kNm}
\end{align*}
\]

\( \Rightarrow \) \textit{Section can be designed as singly reinforced.}

\textbf{Determining } A_{st}

- Considering a 'balanced section' \( (x_u = x_{u,max}) \)

\( A_{st} = A_{st,lim} + \Delta A_{st} \)

where \( A_{st,lim} = p_{t,lim} / 100 (b \times d) \)

\( \Rightarrow A_{st,lim} = \frac{1.433}{100 \times 300 \times 609.5} = 2620 \text{ mm}^2 \)

- Assuming 25 mm bars for compression steel,

\( d^* \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm} \)
\[ A_{st} = \frac{M_u}{0.87 \, f_y \, d \, d'} - \frac{M_{u,\text{lim}}}{0.87 \, f_y} \]

\[ p_t = \frac{R}{100} \, \frac{R_{\text{lim}}}{0.87 \, f_y \, d'} \]

\[ \text{Mu} = 0.87 \, f_y \, A_{st} \, (1-(A_{st} \cdot f_y)/b \cdot d \cdot f_{ck}) \]

\[ \text{Ast Reqd} = 1088 \, \text{mm}^2 \]

\[ \therefore \text{No of tension bars required ( # )} \]

\[ (1088 / (\pi / 4 \times 25^2)) = 3.00 \]

Actual percentage of steel, \( p_t \) ( % )

\[ (3 \times \pi / 4 \times 25^2 / 300 / 610 \times 100) = 0.81 \]

Actual area of steel, \( A_{st} \) ( mm² )

\[ (3 \times \pi / 4 \times 25^2) = 1473 \]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by

\[ A_{sc} = \frac{0.87 \, f_y \, A_{st}}{f_{sc} - 0.447 \, f_{ck}} \]

or

\[ p_c = \frac{0.87 \, f_y \, p_t}{f_{sc} - 0.447 \, f_{ck}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_u = x_{u,\text{max}} \) for various \( d'/d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d'/d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
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<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \( x_u = x_{u,\text{max}} \), for \( d'/d = (40.5/609.5) = 0.066 \)
  From the above table: by interpolation

**Design Check**

- To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_c^{\ast} \).
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{ck}} - \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}}. $$

Actual $p_t$ provided: $p_t = 0.81$

Actual $p_c$ provided: $p_c = 0.81$

$$\Rightarrow p_c^* = \frac{0.87 \times 415 \times (0.805 - 1.433)}{(355.03 - 0.447 \times 30)} = -0.66$$

**Section is not over reinforced**

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d}_{\text{max}} = \frac{l}{d}_{\text{basic}} \times F_1 \times F_2$$

where

- $l/d_{\text{basic}} = 7$ for cantilever spans
- $l/d_{\text{basic}} = 20$ for simply supported spans
- $l/d_{\text{basic}} = 26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$F = \frac{10}{\text{span in metres}}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow f_{st} = \frac{0.58 \times 415 \times 1430}{1473} = 233.69 \text{ N/mm}^2$$

$F = 1.00$

$F_1 = 1.19$
\[ F_2 = 1.19 \]

\[ \therefore \frac{L}{d}_{\text{max}} = \frac{26 \times 1 \times 1.19 \times 1.19}{13.72} = 36.82 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \ (\text{kN}) \) = 250.3333

The critical section for shear is at a distance of 610 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  \[
  \left( \frac{250.3333 \times 1000}{300 \times 610} \right) = 1.37 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  \[
  T_c \text{ max} = 0.62 f_{ck}
  \]
  \[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \text{sqrt}(30)) = 3.40 \text{ N/mm}^2 \]
  \[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

  At critical section, \( A_s \) is given by 1473 mm²
  Percentage of steel, \( p_t \ (% \text{ }) \) = 0.81

  The design shear strength of the concrete, \( \tau_c \), is given by:
  \[
  \tau_c = \frac{0.85 \times 0.9 \times f_{ck} \times 4 \times 5 - 1}{6}
  \]
  where
  \[
  \frac{0.8f_{ck}}{8.89p_t} \text{ whichever is greater}
  \]

  For (M30 and Fe415)
  \[
  \Rightarrow \tau_c = 0.60 \text{ N/mm}^2
  \]
  \[ \Rightarrow V_{uc} = \left( 0.6 \times 300 \times 610 \times 1000 \right) = 110 \text{ kN} \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  \[
  V_{us} = V_u - V_{uc}
  \]
  \[ \Rightarrow V_{us} = (250 - 110) = 140 \text{ kN} \]

  Using 12 mm bars and
  No of legs 2
Area of stirrups, $A_{sv}$ (mm²) = 226

⇒ required spacing $s_v \leq \frac{(0.87 \times 415 \times 226 \times 610)}{(140.11 \times 1000)}$

⇒ Spacing, $s_v = 355$ mm

**Check whether $\tau_v > 0.5 \tau_c$**

Nominal shear stress, $\tau_v$ (N/mm²) = 1.37

Design shear stress, $\tau_c$ (N/mm²) = 0.60

$\tau_v > 0.5 \tau_c \quad \text{Yes}$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5 t_c$$

$$s_v = \frac{2.175 f_v A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v \quad 0.75 d$

$s_v \quad 300$ mm

**Code requirements for maximum spacing..**

i) $< \frac{(2.175 \times 415 \times 226)}{300} = 681$ mm

ii) $\leq \frac{(0.75 \times 609.5)}{300} = 457$ mm

iii) $\leq 300$ mm

iv) $\leq \frac{(0.87 \times 415 \times 226 \times 610)}{(140.11 \times 1000)} = 355$ mm
**Beam RB3 Support**

**Design Parameters**

Load Case 16 \[1.5*(DL - EQZ)]
Grade of Concrete \[M30\]
Grade of Steel \[Fe415\]
Characteristic compressive strength of concrete , \(f_{ck}\) (N/mm\(^2\)) \[30\]
Characteristic yield strength of steel , \(f_y\) (N/mm\(^2\)) \[415\]
Unit weight of concrete , \(\gamma_c\) (kN/m\(^3\)) \[24\]
Partial safety factor for concrete \[1.5\]
Exposure condition \[Mild\]
Nominal Cover to exposure condition (mm) \[20\]

**Dimensions of the beam**

- C/C Span of the beam , \(l\) (m) \[10.80\]
- Breadth of the beam , \(b\) (mm) \[300\]
- Overall depth of the beam , \(D\) (mm) \[850\]

**Details of reinforcements**

- Diameter of tension reinforcement (mm) \[25\]
- Diameter of compression reinforcement (mm) \[25\]
- Diameter of stirrups (mm) \[8\]

**Effective depth**

\[\text{Effective depth , } d\ (\text{mm}) = (850-20-8-25/2) = 810\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.
- Torsional Moment \[6\] kN-m
- Bending Moment \(M_u\) (kNm) \[803\]
- Equivalent Bending Moment , \(M_e\) (kNm) \[817\]
- Shear force at critical distance , \(V_{ud}\) (kN) \[377\]
- Equivalent Shear (kN) \[409\]

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance , \(M_{u,\text{lim}}\) is given by
\[
M_{u,\text{lim}} = 0.362 f_{ck} \times \frac{bx_u \times \text{max}}{d} \times 0.416 x_u \times \text{max}
\]
Where \(b\) = Breadth of the Section
\(x_u\text{max}\) = Limiting depth of Neutral Axis
\(d\) = Effective depth of the Section

The limiting percentage of steel , \(p_{u,\text{lim}}\) is given by
The area of steel for a singly reinforced section with
width , b and depth , d and ultimate moment , Mₜ is given by :

\[ \frac{Pt}{100} \times \frac{Ast \times fck}{bd \times 2fy} = 4.598 \frac{R}{fck} \]

Where \( R = \frac{Mu}{bd^2} \)

For ( M30 and Fe415 )

\[ M_{u,lim} = 0.1389 \times fck \times b \times d^2 \]

\[ \frac{xu,max}{d} = 0.48 \]

\[ \Rightarrow M_{u,lim} = \frac{(0.1389 \times 30 \times 300 \times 809.5^2 \times 100000)}{1000000} = 819.18 \text{ kNm} \]

\[ \Rightarrow p_{u,lim} = \frac{(41.3 \times 30 \times 415 \times 0.48)}{100} = 1.433 \]

If \( M_u > M_{u,lim} \), the section has to be
i) get increased by depth or width ( preferably depth )
ii) doubly reinforced

If \( M_u < M_{u,lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[ M_u = 816.53 \text{ kNm} \]
\[ M_{u,lim} = 819.18 \text{ kNm} \]

\[ \Rightarrow \text{Section can be designed as singly reinforced.} \]

**Determining Aₜ**

- Considering a 'balanced section' ( \( xu = xu,max \) )
  \[ A_{st} = A_{st,lim} + \Delta A_{st} \]
  where \( A_{st,lim} = p_{u,lim} / 100 \times b \times d \)

\[ \Rightarrow A_{st,lim} = \frac{(1.433 \times 100 \times 300 \times 809.5)}{2} = 3480 \text{ mm}^2 \]

- Assuming 25 mm bars for compression steel,

\[ d' \approx \frac{(20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2)}{2} = 40.5 \text{ mm} \]
\[
A_{st} = \frac{M_u}{0.87 f_y d} \frac{M_{u,\text{lim}}}{d'}
\]
\[
p_t = \frac{R}{100} \frac{R_{\text{lim}}}{0.87 f_y} \frac{1}{d'}
\]

\[
M_u = 0.87 f_y A_{st} (1 - (A_{st} f_y) / b d f_{ck})
\]

No of tension bars required (\#)

\[
\left( \frac{3486}{\frac{\pi}{4} \times 25^2} \right) = 8.00
\]

Actual percentage of steel, \(p_t\) (\%)

\[
\left( \frac{8 \times \frac{\pi}{4} \times 25^2}{300 \times 810 \times 100} \right) = 1.62
\]

Actual area of steel, \(A_{st}\) (mm\(^2\))

\[
\left( \frac{8 \times \frac{\pi}{4} \times 25^2}{300 \times 810 \times 100} \right) = 3927
\]

**Determining \(A_{sc}\)**

The compression steel, \(A_{sc}\), is given by

\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \frac{A_{st}}{0.447 f_{ck}}
\]

or

\[
p_c = \frac{0.87 f_y}{f_{sc}} \frac{p_t}{0.447 f_{ck}}
\]

where \(f_{sc}\) is the stress in compression steel.

The values of \(f_{sc}\) (in MPa units) at \(x_u = x_{u,\text{max}}\) for various \(d' / d\) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>(\frac{d'}{d})</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td></td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
<td></td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
<td></td>
</tr>
</tbody>
</table>

- Assuming \(x_u = x_{u,\text{max}}\), for \(d' / d = (40.5 / 809.5) = 0.050\)

From the above table: by interpolation

**Design Check**

- To ensure \(x_u \leq x_{u,\text{max}}\), it suffices to establish \(p_c \geq p_c^*\)
where \( p_c^* \) is given by

\[
\frac{0.87 f_y}{f_{ec}} \left( \frac{0.447 f_{ck}}{p_{t,\text{lim}}} \right) p_t = p_c
\]

Actual \( p_t \) provided : \( p_t = 1.62 \)
Actual \( p_c \) provided : \( p_c = 0.20 \)

\[
\Rightarrow p_c^* = \frac{0.87 \times 415 \times (1.617 - 1.433)}{(355.1 - 0.447 \times 30)} = 0.19
\]

\textit{Section is not over reinforced}

\textbf{Check for deflection control}

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting \( l/d \) ratios are specified by the Code (Cl. 23.2.1) as:

\[
\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} F_1 F_2 \quad \text{7 for cantilever spans}
\]

where \( \frac{l}{d_{\text{basic}}} \) for simply supported spans
\( 20 \) for simply supported spans
\( 26 \) for continuous spans
For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor \( F \)

\[
F = \frac{10}{\text{span in metres}}
\]

The modification factors \( F_1 \) (which varies with \( p_t \) and \( f_{st} \)) and \( F_2 \) (which varies with \( p_c \)) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of \( f_{st} \) as follows:

\[
f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
\]

\[
\Rightarrow f_{st} = \frac{0.58 \times 415 \times 3471}{3927} = 212.73 \text{ N/mm}^2
\]

\( F = 0.93 \)
\( F_1 = 0.83 \)
\[ F_2 = 0.68 \]
\[
\therefore \quad \frac{(l/d)_{\text{max}}}{(l/d)_{\text{provided}}} = \frac{(26 \times 0.93 \times 0.83 \times 0.68)}{13.34} = \frac{13.48}{13.34} \Rightarrow \text{Hence O.K.}
\]

**Check for shear**

Shear force at critical distance, \( V_{ud} (\text{kN}) \) 409

The critical section for shear is at a distance of 810 mm from the face of the support.

*Check for adequacy of section*

Nominal shear stress, \( \tau_v \)
\[
\frac{(409 \times 1000)}{(300 \times 810)} = 1.68 \text{ N/mm}^2
\]

The maximum shear stress is given by:
\[
\tau_{c,max} = 0.62 f_{ck}
\]
\[
\Rightarrow \quad (0.62 \times \text{sqrt}(30)) = 3.40 \text{ N/mm}^2
\]

\Rightarrow Adopted section is adequate

*Design shear resistance at critical section*

At critical section, \( A_{st} \) is given by 3927 mm²

Percentage of steel, \( p_t \) (%) 1.62

The design shear strength of the concrete, \( \tau_c \), is given by:
\[
\tau_c = \frac{0.85 \times 0.8 f_{ck} - 1}{6} \]
\[
\text{where} \quad \frac{0.8 f_{ck}}{0.89 p_t} \text{ whichever is greater} \]
\[
\text{For (M30 and Fe415)}
\]
\[
\Rightarrow \quad \tau_c = 0.78 \text{ N/mm}^2
\]
\[
\Rightarrow \quad V_{uc} = \frac{(0.78 \times 300 \times 810)}{1000} = 190 \text{ kN}
\]

*Design of "vertical" stirrups*

The shear to be resisted by steel, \( V_{us} \) is given by:
\[
V_{us} = V_u \cdot V_{uc}
\]
\[
\Rightarrow \quad V_{us} = (409 - 190) = 219 \text{ kN}
\]

Using 12 mm bars and 2 legs
Area of stirrups, \( A_{sv} \) (mm\(^2\))  

\[
\Rightarrow \text{required spacing } s_v \leq \frac{0.87 \times 415 \times 226 \times 810}{(218.81 \times 1000)} \\
\Rightarrow \text{Spacing, } s_v = 302 \text{ mm}
\]

**Check whether** \( \tau_v > 0.5 \tau_c \)

Nominal shear stress, \( \tau_v \) (N/mm\(^2\)) 1.68  
Design shear stress, \( \tau_c \) (N/mm\(^2\)) 0.78

\( \tau_v > 0.5 \tau_c \)  
Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \( \tau_v \) exceeds 0.5 \( \tau_c \):  

\[
\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When} \quad s_v = 0.5 t_c
\]

\[
s_v = \frac{2.175 f_y A_{sv}}{b}
\]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[
s_v \quad 0.75 d \quad 300 \text{ mm}
\]

**Code requirements for maximum spacing.**

i) \( \leq \frac{(2.175 \times 415 \times 226)}{300} = 681 \text{ mm} \)

ii) \( \leq (0.75 \times 809.5) = 607 \text{ mm} \)

iii) \( \leq 300 \text{ mm} = 300 \text{ mm} \)

iv) \( \leq \frac{(0.87 \times 415 \times 226 \times 810)}{(218.81 \times 1000)} = 302 \text{ mm} \)
**Beam RB3 Mid**

**Design Parameters**

Load Case 16 \[1.5*(DL - EQZ)\]
Grade of Concrete \[\text{M30}\]
Grade of Steel \[\text{Fe415}\]
Characteristic compressive strength of concrete, \(f_{ck}\) (N/mm\(^2\)) \[30\]
Characteristic yield strength of steel, \(f_y\) (N/mm\(^2\)) \[415\]
Unit weight of concrete, \(\gamma_c\) (kN/m\(^3\)) \[24\]
Partial safety factor for concrete \[1.5\]
Exposure condition \[Mild\]
Nominal Cover to exposure condition (mm) \[20\]

**Dimensions of the beam**

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C/C Span of the beam, (l) (m)</td>
<td>10.80</td>
</tr>
<tr>
<td>Breadth of the beam, (b) (mm)</td>
<td>300</td>
</tr>
<tr>
<td>Overall depth of the beam, (D) (mm)</td>
<td>850</td>
</tr>
</tbody>
</table>

**Details of reinforcements**

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>of tension reinforcement (mm)</td>
<td>25</td>
</tr>
<tr>
<td>of compression reinforcement (mm)</td>
<td>25</td>
</tr>
<tr>
<td>of stirrups (mm)</td>
<td>8</td>
</tr>
</tbody>
</table>

**Effective depth**

Effective depth, \(d\) (mm) \[810\] = \((850-20-8-25/2)\)

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

<table>
<thead>
<tr>
<th>Moment or Shear</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional Moment</td>
<td>6 (kN-m)</td>
</tr>
<tr>
<td>Bending Moment, (M_u) (kN-m)</td>
<td>360</td>
</tr>
<tr>
<td>Equivalent Bending Moment, (M_e) (kNm)</td>
<td>374</td>
</tr>
<tr>
<td>Shear force at critical distance, (V_{ud}) (kN)</td>
<td>270</td>
</tr>
<tr>
<td>Equivalent Shear (kN)</td>
<td>302</td>
</tr>
</tbody>
</table>

**Singly reinforced or doubly reinforced section?**

The limiting moment of resistance, \(M_{u,\text{lim}}\) is given by

\[
M_{u,\text{lim}} = 0.362f_{ck} \times \frac{bx_{u,\text{max}}}{d} \times 0.416x_{u,\text{max}}
\]

Where \(b\) = Breadth of the Section
\(x_{u,\text{max}}\) = Limiting depth of Neutral Axis
\(d\) = Effective depth of the Section

The limiting percentage of steel, \(p_{u,\text{lim}}\) is given by
The area of steel for a singly reinforced section with width, b and depth, d and ultimate moment, $M_u$ is given by:

For (M30 and Fe415)

$$x_{u,max} / d = 0.48$$

$$\Rightarrow \quad M_{u,lim} = \left( 0.1389 \times 30 \times 300 \times 809.5^2 / 1000000 \right) = 819.18 \text{ kNm}$$

$$\Rightarrow \quad p_{u,lim} = (41.3 \times 30 / 415 \times 0.48) = 1.433$$

If $M_u > M_{u,lim}$, the section has to be
i) get increased by depth or width (preferably depth)
ii) doubly reinforced

If $M_u < M_{u,lim}$, the section can be designed as singly reinforced.

**Check for the type of section**

$$M_u = 373.53 \text{ kNm}$$

$$M_{u,lim} = 819.18 \text{ kNm}$$

$$\Rightarrow \quad \text{Section can be designed as singly reinforced.}$$

**Determining $A_{st}$**

- Considering a 'balanced section' ($x_u = x_{u,max}$)
  $$A_{st} = A_{st,lim} + \Delta A_{st}$$

  where $A_{st,lim} = p_{u,lim} / 100 \times b \times d$

  $$\Rightarrow \quad A_{st,lim} = (1.433 / 100 \times 300 \times 809.5) = 3480 \text{ mm}^2$$

- Assuming 25 mm bars for compression steel,

  $$d' = \left( 20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2 \right) = 40.5 \text{ mm}$$
\[
A_{st} = \frac{M_u}{0.87 f_y d d'} \quad \frac{M_{u,\text{lim}}}{0.87 f_y d d'}
\]

\[
p_t \quad \frac{R}{100} \quad \frac{R_{\text{lim}}}{0.87 f_y} \quad \frac{1}{d'}
\]

\[
M_u = 0.87 f_y \ast d (1 - (Ast / b) \ast f_{ck})
\]

\[
Ast \quad \text{Reqd} = 1388 \quad \text{mm}^2
\]

\[
\therefore \quad \text{No of tension bars required ( # )} = \frac{1388}{\left( \frac{\pi}{4} \times 25^2 \right)} = 3.00
\]

Actual percentage of steel, \( p_t \) (%):
\[
\left( \frac{3 \times \pi}{4} \times 25^2 \times 2 / 300 / 810 \times 100 \right) = 0.61
\]

Actual area of steel, \( A_{st} \) (mm\(^2\)):
\[
\left( \frac{3 \times \pi}{4} \times 25^2 \right) = 1473
\]

**Determining \( A_{sc} \)**

The compression steel, \( A_{sc} \), is given by
\[
A_{sc} = \frac{0.87 f_y}{f_{sc}} \cdot \frac{A_{st}}{0.447 f_{ck}}
\]

or
\[
p_c = \frac{0.87 f_y}{f_{sc}} \cdot \frac{p_t}{0.447 f_{ck}} \cdot \frac{R_{\text{lim}}}{R_{\text{lim}}}
\]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) (in MPa units) at \( x_c = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( \frac{d'}{d} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>Fe250</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td>355.1</td>
</tr>
<tr>
<td>Fe500</td>
<td>423.9</td>
</tr>
</tbody>
</table>

- Assuming \( x_c = x_{u,\text{max}} \), for \( d' / d = \left( \frac{40.5}{809.5} \right) = 0.050 \)
  From the above table: by interpolation

**Design Check**

- To ensure \( x_c \leq x_{u,\text{max}} \), it suffices to establish \( p_c \geq p_{c^*} \).
where $p_c^*$ is given by

$$p_c^* = \frac{0.87 f_y}{f_{ck}} \cdot \frac{0.447 f_{ck}}{p_t - p_{t,\text{lim}}},$$

Actual $p_t$ provided : $p_t = 0.61$
Actual $p_c$ provided : $p_c = 0.81$

$$\Rightarrow \quad p_c^* = \frac{0.87 \times 415 \times (0.606 - 1.433)}{(355.1 - 0.447 \times 30)}$$
$$\Rightarrow \quad p_c^* = -0.87$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} \cdot F_1 \cdot F_2$$

where

- $l$ basic = 7 for cantilever spans
- $l$ basic = 20 for simply supported spans
- $l$ basic = 26 for continuous spans

For simply supported and continuous spans over 10m, these ratios are multiplied by a factor $F$:

$$F = \frac{\text{span in metres}}{10}$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig. 4 and Fig. 5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$f_{st} = 0.58 f_y \cdot \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\Rightarrow \quad f_{st} = \frac{(0.58 \times 415 \times 1875)}{1473} = 306.48 \text{ N/mm}^2$$

$F = 0.93$
$F_1 = 1.11$
\[ F_2 = 1.19 \]

\[ \therefore \quad \frac{l}{d} \max = (26 \times 0.93 \times 1.11 \times 1.19) = 31.96 \]

\[ \frac{l}{d} \text{ provided} = 13.34 \]

\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

302

The critical section for shear is at a distance of 810 mm from the face of the support.

**Check for adequacy of section**

Nominal shear stress, \( \tau_v \)

\[ \frac{(302 \times 1000)}{(300 \times 810)} = 1.24 \text{ N/mm}^2 \]

The maximum shear stress is given by:

\[ T_{c\ max} = 0.62 f_{ck} \]

\[ \Rightarrow \quad \tau_{c,\ max} = (0.62 \times \sqrt{30}) = 3.40 \text{ N/mm}^2 \]

\[ \Rightarrow \text{Adopted section is adequate} \]

**Design shear resistance at critical section**

At critical section, \( A_{st} \) is given by

1473 mm²

Percentage of steel, \( p_t \) (%)

0.61

The design shear strength of the concrete, \( \tau_c \), is given by:

\[ \frac{0.85 - 0.8 f_{ck} - 1}{5} - 1 \]

where

\[ \frac{0.8 f_{ck}}{6.89 p_t} \text{ whichever is greater} \]

For (M30 and Fe415)

\[ \Rightarrow \quad \tau_c = 0.54 \text{ N/mm}^2 \]

\[ \Rightarrow \quad V_{uc} = \frac{(0.54 \times 300 \times 810 / 1000)}{62} = 131 \text{ kN} \]

**Design of "vertical" stirrups**

The shear to be resisted by steel, \( V_{us} \) is given by:

\[ V_{us} = V_u - V_{uc} \]

\[ \Rightarrow V_{us} = (302 - 131) = 171 \text{ kN} \]

Using 12 mm bars and

No of legs 2
Area of stirrups, \( A_{sv} \) (mm\(^2\)) = 226

\[ \Rightarrow \text{required spacing } s_v \leq \left( \frac{0.87 \times 415 \times 226 \times 810}{171.38 \times 1000} \right) \]

\[ \Rightarrow \text{Spacing, } s_v = 386 \text{ mm} \]

*Check whether* \(\tau_v > 0.5 \tau_c\)

Nominal shear stress, \(\tau_v\) (N/mm\(^2\)) = 1.24

Design shear stress, \(\tau_c\) (N/mm\(^2\)) = 0.54

\[\tau_v > 0.5 \tau_c \quad \text{Yes}\]

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress \(\tau_v\) exceeds 0.5 \(\tau_c\):

\[\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \quad \text{When } s_v = 0.5tc\]

\[s_v = \frac{2.175 f_y A_{sv}}{b} \]

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

\[s_v \leq 0.75 \phi d \leq 300 \text{ mm}\]

Code requirements for maximum spacing:

i) \[< \left( \frac{2.175 \times 415 \times 226}{300} \right) = 681 \text{ mm}\]

ii) \[\leq \left( \frac{0.75 \times 809.5 }{300} \right) = 607 \text{ mm}\]

iii) \[\leq 300 \text{ mm} \leq 300 \text{ mm}\]

iv) \[\leq \left( \frac{0.87 \times 415 \times 226 \times 810}{171.38 \times 1000} \right) = 386 \text{ mm}\]
Beam RB4 Support

Design Parameters

Load Case 16 \[1.5*(DL - EQZ)\]
Grade of Concrete \(M30\)
Grade of Steel \(Fe415\)
Characteristic compressive strength of concrete \(f_{ch} \text{ (N/mm}^2\text{)}\) 30
Characteristic yield strength of steel \(f_y \text{ (N/mm}^2\text{)}\) 415
Unit weight of concrete \(\gamma_c \text{ (kN/m}^3\text{)}\) 24
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition (mm) 20

Dimensions of the beam

C/C Span of the beam, \(l \text{ (m)}\) 5.50
Breadth of the beam, \(b \text{ (mm)}\) 300
Overall depth of the beam, \(D \text{ (mm)}\) 600

Details of reinforcements

Diameter of tension reinforcement (mm) 25
Diameter of compression reinforcement (mm) 25
Diameter of stirrups (mm) 8

Effective depth

Effective depth, \(d \text{ (mm)}\) \((600-20-8-25/2) = 560\)

Design Moment, Shear Force

The moments and shears given below are taken from the STAAD.Pro 2004 output file. The partial factors of safety are already incorporated into the analysis.

Torsional Moment 16 kN-m
Bending Moment \(Mu(kN-m)\) 283
Equivalent Bending Moment , \(M_e \text{ (kNm)}\) 311
Shear force at critical distance , \(V_{ud} \text{ (kN)}\) 90
Equivalent Shear (kN) 175

Singly reinforced or doubly reinforced section ?

The limiting moment of resistance , \(M_{u,lim}\) is given by

\[
M_{u,lim} = 0.362f_{ch} \cdot \frac{bx_u}{d} * 0.416x_u\]

Where \(b = \text{Breadth of the Section}\)
\(x_u\) = Limiting depth of Neutral Axis
\(d = \text{Effective depth of the Section}\)

The limiting percentage of steel , \(p_{u,lim}\) is given by
The area of steel for a singly reinforced section with width \( b \) and depth \( d \) and ultimate moment \( M_u \) is given by:

\[
\frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{fck}{2fy} = 4.598 \times \frac{R}{fck}
\]

Where \( R = \frac{Mu}{bd^2} \)

For ( M30 and Fe415 )

\[
M_u,\text{lim} = 0.1389 \times fck \times b \times d^2
\]

\[
x_u,\text{max} / d = 0.48
\]

\[
\Rightarrow M_u,\text{lim} = (0.1389 \times 30 \times 300 \times 559.5^2) / 1000000 = 391.33 \text{ kNm}
\]

\[
p_u,\text{lim} = (41.3 \times 30 / 415 \times 0.48) = 1.433
\]

If \( M_u > M_u,\text{lim} \), the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If \( M_u < M_u,\text{lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 311.24 \text{ kNm}
\]

\[
M_u,\text{lim} = 391.33 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a 'balanced section' (\( x_u = x_{u,\text{max}} \))

\[
A_{st} = A_{st,\text{lim}} + \Delta A_{st}
\]

where \( A_{st,\text{lim}} = p_u,\text{lim} / 100 \times b \times d \)

\[
\Rightarrow A_{st,\text{lim}} = (1.433 / 100 \times 300 \times 559.5) = 2405 \text{ mm}^2
\]

- Assuming 25 mm bars for compression steel,

\[
d' \approx (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 25 / 2) = 40.5 \text{ mm}
\]
\[ A_{st} \quad \frac{M_u}{0.87 \, f_y} \frac{M_{u,\text{lim}}}{d' \, d} \]

\[ p_t \quad \frac{R}{100} \quad \frac{R_{\text{lim}}}{0.87 \, f_y} \quad 1 \quad \frac{d' \, d}{d} \]

\[ M_u = 0.87 \, f_y \, \text{Ast} \, d \left(1 - \frac{\text{Ast} \, f_y}{b \, d \, f_{ck}}\right) \]

\[ \text{Ast Req'd} = 1811 \quad \text{mm}^2 \]

∴ No of tension bars required ( # )

\[ \frac{1811}{(\frac{\pi}{4} \times 25^2)} = 4.00 \]

Actual percentage of steel , \( p_t \) ( % )

\[ \frac{4 \times \pi \times 4 \times 25^2}{300} \times \frac{560 \times 100}{560} = 1.17 \]

Actual area of steel , \( A_{st} \) ( mm\(^2\) )

\[ \frac{4 \times \pi \times 4 \times 25^2}{560} = 1963 \]

**Determining \( A_{sc} \)**

The compression steel , \( A_{sc} \), is given by

\[ A_{sc} \quad \frac{0.87 \, f_y}{f_{sc}} \quad \frac{A_{st}}{0.447 \, f_{ck}} \]

or

\[ p_{ce} \quad \frac{0.87 \, f_y}{f_{sc}} \quad \frac{p_t}{0.447 \, f_{ck}} \quad P_{\text{lim}} \]

where \( f_{sc} \) is the stress in compression steel.

The values of \( f_{sc} \) ( in MPa units ) at \( x_u = x_{u,\text{max}} \) for various \( d' / d \) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>( d' / d )</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
<td>217.5</td>
</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

• Assuming \( x_u = x_{u,\text{max}} \), for \( d' / d = \frac{40.5}{559.5} = 0.072 \)

From the above table : by interpolation

**Design Check**

• To ensure \( x_u \leq x_{u,\text{max}} \), it suffices to establish \( p_{ce} \geq p_{ce}^* \).
where $p_c^*$ is given by

$$
p_c^* = \frac{0.87 f_y}{f_{cy}} - \frac{0.447 f_{ck}}{p_{t,lim}}
$$

Actual $p_t$ provided : $p_t = 1.17$
Actual $p_c$ provided : $p_c = 0.29$

$$
\Rightarrow p_c^* = (0.87 \times 415 \times (1.17 - 1.433)) / (354.73 - 0.447 \times 30)
$$
$$
\Rightarrow p_c^* = -0.28
$$

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting $l/d$ ratios are specified by the Code (Cl. 23.2.1) as:

$$
\frac{l}{d_{max}} = \frac{l}{d_{basic}} F_1 F_2
$$

where $l/d_{basic} = 7$ for cantilever spans

$20$ for simply supported spans

$26$ for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor $F$

$$
F = \frac{10}{\text{span in metres}}
$$

The modification factors $F_1$ (which varies with $p_t$ and $f_{st}$) and $F_2$ (which varies with $p_c$) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of $f_{st}$ as follows:

$$
f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
$$

$$
\Rightarrow f_{st} = (0.58 \times 415 \times 1978 / 1963) = 242.47\ N/mm^2
$$

$F = 1.00$

$F_1 = 0.91$
Check for shear

Shear force at critical distance, \( V_{ud} \) (kN) = 175.33333

The critical section for shear is at a distance of 560 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  
  \[
  (175.333333333333 x 1000 / (300 x 560)) = 1.04 \text{ N/mm}^2
  \]

  The maximum shear stress is given by:
  
  \[
  Tc_{\text{max}} = 0.62 f'ck
  \]
  
  \[
  \Rightarrow \tau_{c,max} = (0.62 x \sqrt{30}) = 3.40 \text{ N/mm}^2
  \]

  ⇒ *Adopted section is adequate*

- **Design shear resistance at critical section**

  At critical section, \( A_{sl} \) is given by 1963 mm²

  Percentage of steel, \( p_t \) (\%) = 1.17

  The design shear strength of the concrete, \( \tau_c \), is given by:
  
  \[
  \tau_c = \frac{0.85 \cdot 0.8 f'ck \cdot 5}{6} - 1
  \]

  where
  
  \[
  \frac{0.8 f'ck}{6.89 p_t} \quad \text{whichever is greater}
  \]

  For (M30 and Fe415)
  
  \[
  \Rightarrow \tau_c = 0.70 \text{ N/mm}^2
  \]

  \[
  \Rightarrow V_{uc} = (0.7 x 300 x 560 / 1000) = 117 \text{ kN}
  \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  
  \[
  V_{us} = V_u - V_{uc}
  \]

  \[
  \Rightarrow V_{us} = (175 - 117) = 59 \text{ kN}
  \]

  Using 12 mm bars and No of legs 2
Area of stirrups, $A_{sv}$ (mm²) = 226

⇒ required spacing $s_v \leq \left( \frac{0.87 \times 415 \times 226 \times 560}{58.52 \times 1000} \right)$

⇒ Spacing, $s_v = 781$ mm

**Check whether $\tau_v > 0.5 \tau_c$**

Nominal shear stress, $\tau_v$ (N/mm²) = 1.04
Design shear stress, $\tau_c$ (N/mm²) = 0.70

$\tau_v > 0.5 \tau_c \quad \text{Yes}$

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds 0.5 $\tau_c$:

$$\frac{A_{sv}}{b_{sv}} = 0.4 \frac{0.87 f_y}{0.87 f_y} \quad \text{When } s_v = 0.5 \tau_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$s_v = 0.75 d \quad 300$ mm

**Code requirements for maximum spacing.**

i) $< \left( \frac{2.175 \times 415 \times 226}{300} \right) = 681$ mm
ii) $\leq \left( \frac{0.75 \times 559.5}{1} \right) = 420$ mm
iii) $\leq 300$ mm
iv) $\leq \left( \frac{0.87 \times 415 \times 226 \times 560}{58.52 \times 1000} \right) = 781$ mm
**Beam RB4 Mid**

**Design Parameters**

Load Case 16 \[1.5*(DL - EQZ)\]
Grade of Concrete \(M30\)
Grade of Steel \(Fe415\)
Characteristic compressive strength of concrete , \(f_{ck} (N/mm^2)\) 30
Characteristic yield strength of steel , \(f_y (N/mm^2)\) 415
Unit weight of concrete , \(\gamma_c (kN/m^3)\) 24
Partial safety factor for concrete 1.5
Exposure condition Mild
Nominal Cover to exposure condition( mm ) 20

**Dimensions of the beam**

C/C Span of the beam , \(l\) , ( m ) 5.50
Breadth of the beam , \(b\) ( mm ) 300
Overall depth of the beam , \(D\) ( mm ) 600

**Details of reinforcements**

Diameter of tension reinforcement ( mm ) 20
Diameter of compression reinforcement ( mm ) 20
Diameter of stirrups ( mm ) 8

**Effective depth**

Effective depth , \(d\) ( mm ) \[(600-20-8-20/2) = 562\]

**Design Moment, Shear Force**

The moments and shears given below are taken from the STAAD.Pro 2004 output file.
The partial factors of safety are already incorporated into the analysis.

Torsional Moment 0 kN-m
Bending Moment \(M_u(kN-m)\) 120
Equivalent Bending Moment , \(M_e (kN-m)\) 120
Shear force at critical distance , \(V_{ud} (kN)\) 139
Equivalent Shear (kN) 139

**Singly reinforced or doubly reinforced section ?**

The limiting moment of resistance , \(M_{ul,m}\) is given by

\[M_{ul,m} = 0.362f_{ck} \times \frac{bxu_{max}}{d} \times 0.416xu_{max}\]

Where \(b\) = Breadth of the Section
\(xu_{max}\) = Limiting depth of Neutral Axis
\(d\) = Effective depth of the Section

The limiting percentage of steel , \(p_{ul,m}\) is given by
The area of steel for a singly reinforced section with width, \( b \) and depth, \( d \) and ultimate moment, \( M_u \) is given by:

\[
\frac{Pt}{100} \times \frac{Ast}{bd} \times \frac{fck}{2fy} = 4.598 \times \frac{R}{f_{ck}}
\]

Where \( R = \frac{M_u}{bd^2} \)

For (M30 and Fe415)

\[
M_{u,lim} = \frac{0.1389 \times fck \times b \times d^2}{1000000}
\]

\[
x_{u,max} = \frac{0.48}{d}
\]

\[
M_u = 120.00 \text{ kNm}
\]

\[
M_{u,lim} = 394.84 \text{ kNm}
\]

\[
p_{u,lim} = \frac{41.3 \times 30}{415 \times 0.48} = 1.433
\]

If \( M_u > M_{u,lim} \), the section has to be

i) get increased by depth or width (preferably depth)

ii) doubly reinforced

If \( M_u < M_{u,lim} \), the section can be designed as singly reinforced.

**Check for the type of section**

\[
M_u = 120.00 \text{ kNm}
\]

\[
M_{u,lim} = 394.84 \text{ kNm}
\]

\[
\Rightarrow \text{Section can be designed as singly reinforced.}
\]

**Determining \( A_{st} \)**

- Considering a ‘balanced section’ (\( x_u = x_{u,max} \))
  \[
  A_{st} = A_{st,lim} + \Delta A_{st}
  \]
  where \( A_{st,lim} = p_{u,lim} / 100 \times (b \times d) \)

\[
A_{st,lim} = 1.433 / 100 \times 300 \times 562 = 2416 \text{ mm}^2
\]

- Assuming 20 mm bars for compression steel,

\[
d' = (20 \text{ mm clear cover} + 8 \text{ mm stirrup} + 20 / 2) = 38 \text{ mm}
\]
\[
A_{st} = \frac{\frac{M_u}{M_{u,lim}}}{0.87f_y d \frac{d-d'}{d}}
\]

\[
\frac{p_t}{100} = \frac{R}{R_{lim}} \frac{0.87f_y 1}{d} \frac{d'}{d}
\]

\[
M_u = 0.87f_y^*Ast^*d(1-(Ast^*fy)b*d*fck)
\]

\[
\text{Ast Reqd} = 623 \text{ mm}^2
\]

\[
\therefore \text{ No of tension bars required ( # )} \]

\[
\left( \frac{623}{\left( \frac{\pi}{4} \times 20^2 \right)} \right) = 2.00
\]

\[
\text{Actual percentage of steel , } p_t (\%) \]

\[
\left( \frac{2 \times \frac{\pi}{4} \times 20^2}{300 \times 562} \times 100 \right) = 0.37
\]

\[
\text{Actual area of steel , } A_{st} (\text{ mm}^2) \]

\[
\left( \frac{2 \times \frac{\pi}{4} \times 20^2}{300} \right) = 628
\]

**Determining A_{sc}**

The compression steel , A_{sc} , is given by

\[
A_{sc} = \frac{0.87f_y A_{st}}{f_{sc}-0.447f_{ck}}
\]

or

\[
p_c = \frac{0.87f_y p_t}{f_{sc}-0.447f_{ck}}
\]

where \(f_{sc}\) is the stress in compression steel.

The values of \(f_{sc}\) ( in MPa units ) at \(x_u = x_{u,\text{max}}\) for various \(d' / d\) ratios and different grades of compression steel are given in the table below.

<table>
<thead>
<tr>
<th>Grade of steel</th>
<th>(\frac{d'}{d})</th>
<th>0.05</th>
<th>0.10</th>
<th>0.15</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe250</td>
<td></td>
<td>217.5</td>
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</tr>
<tr>
<td>Fe415</td>
<td></td>
<td>355.1</td>
<td>351.9</td>
<td>342.4</td>
<td>329.2</td>
</tr>
<tr>
<td>Fe500</td>
<td></td>
<td>423.9</td>
<td>411.3</td>
<td>395.1</td>
<td>370.3</td>
</tr>
</tbody>
</table>

- Assuming \(x_u = x_{u,\text{max}}\) , for \(d' / d = \left( \frac{38}{562} \right) = 0.068\)

From the above table : by interpolation

**Design Check**

- To ensure \(x_u \leq x_{u,\text{max}}\) , it suffices to establish \(p_c \geq p_c^*\)
where \( p_c^* \) is given by
\[
\frac{0.87 f_y}{f_{ck}} - \frac{0.447 f_{ck}}{0.447 f_{ck}} = p_c^* - p_{t,\text{lim}}
\]

Actual \( p_t \) provided : \( p_t = 0.37 \)
Actual \( p_c \) provided : \( p_c = 0.93 \)

\[
\Rightarrow p_c^* = \frac{0.87 \times 415 \times (0.373 - 1.433)}{(354.98 - 0.447 \times 30)}
\]
\[
\Rightarrow p_c^* = -1.12
\]

Section is not over reinforced

**Check for deflection control**

For prismatic beams of rectangular sections and slabs of uniform thicknesses and spans upto 10m, the limiting \( l/d \) ratios are specified by the Code (Cl. 23.2.1) as:

\[
\frac{l}{d_{\text{max}}} = \frac{l}{d_{\text{basic}}} \cdot F_1 \cdot F_2
\]

where \( \frac{l}{d_{\text{basic}}} \):
- 7 for cantilever spans
- 20 for simply supported spans
- 26 for continuous spans

For simply supported and continuous spans over 10 m, these ratios are multiplied by a factor \( F \)

\[
F = \frac{10}{\text{span in metres}}
\]

The modification factors \( F_1 \) (which varies with \( p_t \) and \( f_{st} \)) and \( F_2 \) (which varies with \( p_c \)) are as given in Fig.4 and Fig.5 of the code.

Code permits an approximate calculation of \( f_{st} \) as follows:

The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

\[
f_{st} = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}
\]

\[
\Rightarrow f_{st} = \frac{0.58 \times 415 \times 963}{628} = 369.08 \text{ N/mm}^2
\]

\[
F = 1.00
\]
\[
F_1 = 1.26
\]
\[ F_2 = 1.24 \]

\[ \therefore \ (l/d)_{\text{max}} = (26 \times 1 \times 1.26 \times 1.24) = 40.32 \]
\[ (l/d)_{\text{provided}} = 9.79 \]
\[ \Rightarrow \text{Hence O.K.} \]

**Check for shear**

Shear force at critical distance, \( V_{ud} \) (kN)

The critical section for shear is at a distance of 562 mm from the face of the support.

- **Check for adequacy of section**

  Nominal shear stress, \( \tau_v \)
  \[ \frac{139 \times 1000}{300 \times 562} = 0.82 \text{ N/mm}^2 \]

  The maximum shear stress is given by:
  \[ T_{c,\text{max}} = 0.62f_{ck} \]
  \[ \Rightarrow \tau_{c,\text{max}} = (0.62 \times \text{Sqrt}(30)) = 3.40 \text{ N/mm}^2 \]

  \[ \Rightarrow \text{Adopted section is adequate} \]

- **Design shear resistance at critical section**

  At critical section, \( A_{st} \) is given by 628 mm²
  Percentage of steel, \( p_t \) (%)
  0.37

  The design shear strength of the concrete, \( \tau_c \), is given by:
  \[ \frac{0.8f_{ck}}{6} = \frac{1}{\text{ whichever is greater}} \]
  For (M30 and Fe415)
  \[ \Rightarrow \tau_c = 0.44 \text{ N/mm}^2 \]
  \[ \Rightarrow V_{uc} = (0.44 \times 300 \times 562 / 1000) = 74 \text{ kN} \]

- **Design of "vertical" stirrups**

  The shear to be resisted by steel, \( V_{us} \) is given by:
  \[ V_{us} = V_u - V_{uc} \]
  \[ \Rightarrow V_{us} = (139 - 74) = 65 \text{ kN} \]

  Using 8 mm bars and
  \[ \text{No of legs} = 2 \]
Area of stirrups, $A_{sv}$ (mm$^2$) 101

⇒ required spacing $s_v \leq \frac{0.87 \times 415 \times 101 \times 562}{65.03 \times 1000}$

⇒ Spacing, $s_v = 314$ mm

Check whether $\tau_v > 0.5 \tau_c$

Nominal shear stress, $\tau_v$ (N/mm$^2$) 0.82
Design shear stress, $\tau_c$ (N/mm$^2$) 0.44

$\tau_v > 0.5 \tau_c$ Yes

The Code (Cl. 26.5.1.6) specifies a minimum shear reinforcement to be provided in the form of stirrups in all beams where the calculated nominal shear stress $\tau_v$ exceeds $0.5 \tau_c$:

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87 f_y} \ \text{When} \ s_v = 0.5 \tau_c$$

$$s_v = \frac{2.175 f_y A_{sv}}{b}$$

The maximum spacing of stirrups should also comply with the requirements mentioned above. For normal "vertical" stirrups, the requirement is

$$s_v \leq 0.75 d \ \text{or} \ 300 \ mm$$

Code requirements for maximum spacing:

i) $< \frac{2.175 \times 415 \times 101}{300} = 302 \ mm$

ii) $\leq \frac{0.75 \times 562}{300} = 422 \ mm$

iii) $\leq 300 \ mm$

iv) $\leq \frac{0.87 \times 415 \times 101 \times 562}{65.03 \times 1000} = 314 \ mm$
DESIGN OF SLAB
Design of Grid Slab

Grade of concrete = 30 KN/m²
Grade of steel = 415 N/mm²
Unit weight of concrete = 24 kN/m³
Live Load = 3 kN/m²
Cover = 20 mm
Roof Finish Load = 1 kN/m²  
\[ L_y = 8.36 \text{ m} \]
\[ L_x = 4.96 \text{ m} \]

Aspect Ratio \[ r = \frac{L_y}{L_x} = 1.69 \]  
\[ r = \frac{4}{2} \]

Grid Spacing

\[ X - \text{Dir} = 1.240 \text{ m} \]
\[ Y - \text{Dir} = 1.400 \text{ m} \]

No of Beams in X - Direction = 3 Nos
No of Beams in Y - Direction = 5 Nos

Thickness of the Slab \[ D_f = 120 \text{ mm} \]
Thickness of the Web \[ b_w = 250 \text{ mm} \]
Depth of the Web \[ D = 430 \text{ mm} \]

Design of the Section:

Self weight of Slab = 2.88 kN/m²
Total Load of Slab = 119.42 kN
Weight of Rib = 1.86 kN/m
Total weight of Beams (x-direction) = 45.86 kN
Total weight of Beams (y-direction) = 39.71 kN
Total weight of Floor Finish = 41.4656 kN
Total Live Load = 124.3968 kN

Total Load = 370.9 kN/m²

Load per m² \[ q = 8.9 \text{ kN/m}^2 \]
Approximate Method (Moments)

If q1 & q2 are the moments shared in the x & y directions

\[ q_1 = q \left( \frac{b_y^4}{a_x^4 + b_y^4} \right) \]

\[ q_1 = 8.0 \text{ kN/m}^2 \]

\[ q_2 = q \left( \frac{a_x^4}{a_x^4 + b_y^4} \right) \]

\[ q_1 = 1.0 \]

Moments in x & y directions at centre of grid for 2 m width is taken as:

\[ M_x = \left( \frac{q_1 b_x a_x^2}{8} \right) \]

\[ M_x = 30.5 \text{ kN.m} \]

\[ M_y = \left( \frac{q_2 a_x b_x^2}{8} \right) \]

\[ M_y = 12.1 \text{ kN.m} \]

\[ Q_x = 24.6 \text{ kN} \]

\[ Q_y = 5.8 \text{ kN} \]

Design of Reinforcement

Max working moment \[ M_w = 18.3 \text{ kN.m/m} \]

Moment resisted by central rib in x-direction over 1.24 m width \[ = 22.7 \text{ kN.m} \]

Ultimate moment \[ M_u = 34.0 \text{ kN.m} \]

Moment capacity of flange section

\[ M_{uf} = 0.36 f_{ck} b_x D_f (d - 0.42 D_f) \]

\[ M_{uf} = 513.6 \text{ kN.m} \]

\[ M_u < M_{uf} \text{ N.A falls within the Flange} \]
\[ M_u = 0.87 f_y A_{st} d \left[ 1 - \frac{A_{st} f_y}{b d f_{ck}} \right] \]

Ast reqd = 300 mm²

20 mm = 1 Nos

Ast Provided = 314 mm²

Max ultimate shear = 25 kN

\[ \tau_v = \frac{V_u}{b d} \]

\[ \tau_v = 0.3 \text{ N/mm}^2 \]

Assuming 2 bars to be bent up near support

Ast at supports = 628 mm²

\[ \frac{100 A_{st}}{bd} = 0.68 \]

\[ \tau_c = 0.56 \text{ N/mm}^2 \]

Nominal Shear reinforcement is Required

No of stirrup legs = 2

\[ S_v = \frac{A_{sv} 0.87 f_y}{0.4 b} \]

\[ S_v = 363.0 \text{ mm} \]

Provide 8 mm dia 2 legged stirrups at 360 mm c/c at support & the spacing can be gradually increased at centre.
Design of Slab for Lift Machine room

Grade of concrete = 30 KN/m²  
Grade of steel = 415 N/mm²  
Live Load = 3 kN/m²  
Cover = 20 mm  

L_y = 1.85 m  
L_x = 1.6 m  

Breadth of slab = 1000 mm  
L_y/L_x = 1.16

Design as a Two way Slab

Depth of the Slab

Depth = 70 mm  
Effective Depth = 95 mm  
Overall Depth = 115 mm

Effective Span = 1.83

Loads

Self weight = 1.875 kN/m²  
Live Load = 2 kN/m²  
Floor Finish = 0.6 kN/m²  
Service Load = 4.475 kN/m²  
Design Load = 6.71 kN/m²

Design moments in the x and y directions

\[ \alpha_x^+ = 0.069 \]  
\[ \alpha_y^+ = 0.056 \]  
\[ \alpha_x^- \]  
\[ \alpha_y^- \]  

INTERPOLATION

<table>
<thead>
<tr>
<th>1.2</th>
<th>1.16</th>
<th>1.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.072</td>
<td><strong>0.069</strong></td>
<td>0.079</td>
</tr>
</tbody>
</table>

\[ M_x = 1.55 \text{ KN-m} \]  
\[ M_y = 1.26 \text{ KN-m} \]  
\[ V_{ux} = 6.14 \text{ KN} \]  

Check for depth

\[ d = 19.35 \text{ mm} \quad d = 110 \text{ mm} \]

Total depth = 39.35 mm  
Reinforcement (short and long span)
\[
\begin{array}{lcccc}
\text{Shorter span} & \text{Longer span} \\
\text{a} & = & 4.99 & 4.99 \\
\text{b} & = & 36105.00 & 36105.00 \\
\text{c} & = & 1549679.93 & 1258851.51 \\
\text{b}^2-4ac & = & 1272611364 & 1278421564 \\
2a & = & 9.99 & 9.99 \\
\text{SQ} & = & 35673.68 & 35755.02 \\
\text{Ast}_1 & = & 43.18 \text{ mm}^2 & 35.04 \text{ mm}^2 \\
\text{Ast}_2 & = & -7185.73627 \text{ mm}^2 & -7193.87945 \\
\end{array}
\]

Reinforcement in Shorter Direction
spacing of 10 mm = 1818 mm c/c
spacing of 12 mm = 2619 mm c/c
Provide 12 mm bars at 150 mm c/c

Reinforcement in Longer Direction
spacing of 10 mm = 2241 mm c/c
spacing of 12 mm = 3228 mm c/c
Provide 12 mm bars at 150 mm c/c

Check for shear stress

Considering the short span & unit width of slab
\[
\zeta_v = \frac{V_u}{bd} = 0.0614 \text{ N/mm}^2
\]
\[
P_t = 0.043
\]

INTERPOLATION

\[
\begin{array}{cccc}
0.5 & 0.04 & 0.75 & \text{Table19} \\
0.48 & 0.334 & 0.56 \\
\end{array}
\]

\[
\zeta_c = 0.334 \text{ N/mm}^2
\]

\[\zeta_c > \zeta_v\] Shear reinforcement is not reqd

Check for deflection Control
Modification Factor From IS 456
\[
K = 1.2
\]
Design of Slab for Ramp

Grade of concrete = 30 KN/m²
Grade of steel = 415 N/mm²
Live Load = 4 kN/m²
Cover = 20 mm

\[ L_y = 5.345 \text{ m} \]
\[ L_x = 3.6 \text{ m} \]

Breadth of slab = 1000 mm

\[ L_y/L_x = 1.48 \]

Design as a Two way Slab

Depth of the Slab

Depth = 112.5 mm
Effective Depth = 112.5 mm
Overall Depth = 137.5 mm

Effective Span = 3.83

Loads

Self weight = 3.4375 kN/m²
Live Load = 4 kN/m²
Floor Finish = 0.6 kN/m²
Service Load = 8.0375 kN/m²
Design Load = 12.06 kN/m²

Design moments in the x and y directions

\[ \alpha_x^+ = 0.092 \]
\[ \alpha_y^+ = 0.056 \]
\[ \alpha_x^- \]
\[ \alpha_y^- \]

INTERPOLATION

\[ \begin{array}{c|c|c|c}
   \lambda & 1.2 & 1.48 & 1.3 \\
   \hline
   M_x & 16.26 & KN-m & \\
   M_y & 9.90 & KN-m & \\
   V_u & 23.09 & KN & \\
\end{array} \]

Check for depth
\[ d = 62.67 \text{ mm} \]
Total depth = 82.67 mm

Reinforcement (shorter and longer span)

\[ \begin{array}{c|c|c}
   \lambda & 0.072 & 0.092 & 0.079 \\
   \hline
   SQ & 43338.54 & mm² & \\
   A_{st1} & 360.19 & mm² & \\
   A_{st2} & -9037.40009 & mm² & \\
\end{array} \]

Reinforcement in Shorter Direction

Spacing of 10 mm = 218 mm c/c
Spacing of 12 mm = 314 mm c/c
Provide 12 mm bars at 100 mm c/c

Reinforcement in Longer Direction

Spacing of 10 mm = 363 mm c/c
Spacing of 12 mm = 524 mm c/c
Provide 12 mm bars at 190 mm c/c

Check for shear stress

\[ \zeta = \frac{V_u}{b d} \]
\[
\begin{align*}
Pt &= 0.1776 \text{ N/mm}^2 \\
\ &= 0.277
\end{align*}
\]

**INTERPOLATION**

\[
\begin{array}{c|c|c|c}
0.5 & 0.28 & 0.75 & \text{Table19} \\
0.48 & 0.409 & 0.56 & \\
\end{array}
\]

\[
\zeta_c = 0.409 \text{ N/mm}^2
\]

\[\zeta_c > \zeta_v\]

Shear reinforcement is not reqd

Check for deflection Control

Modification Factor From IS 456

\[
K = 1.2
\]
BILL OF QUANTITIES
<table>
<thead>
<tr>
<th>Sl. No</th>
<th>SOR Ref No</th>
<th>Description of Work</th>
<th>Unit</th>
<th>Quantity</th>
<th>Rate (Rs)</th>
<th>Amount (Rs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.01</td>
<td>251a</td>
<td>Excavation in foundation in ordinary soil (loam, clay or sand) including lift upto 1.5m and lead upto 30m and including filling watering and ramming of excavated earth into the trenches or into the space between the building and the sides of the foundation trenches or into the plinth and removal and disposal of surplus earth as directed by engineer in charge upto a distance of 30m from the foundation trenches</td>
<td>cum</td>
<td>4,569.00</td>
<td>38.00</td>
<td>173,622.00</td>
</tr>
<tr>
<td>1.01a</td>
<td>254a</td>
<td>Extra for every additional 30m lead or part of 30m or for every additional 1.5m lift or part of 1.50m</td>
<td>cum</td>
<td>3,040.00</td>
<td>43.00</td>
<td>130,720.00</td>
</tr>
<tr>
<td>1.01b</td>
<td>254a</td>
<td>Extra for every additional 30m lead or part of 30m or for every additional 1.5m lift or part of 1.50m</td>
<td>cum</td>
<td>183.00</td>
<td>48.00</td>
<td>8,784.00</td>
</tr>
<tr>
<td>1.02</td>
<td>255a</td>
<td>Sand filling in plinth including supply of necessary quantity of sand from a distance not exceeding 8 km from the site of work and including watering, dressing etc labour and T&amp;P etc required for the proper completion of the work, saplings of girth upto 30cm measured at a height of 1m above ground level and removal of rubbish upto a distance of 50m outside the periphery of area cleared</td>
<td>cum</td>
<td>4,560.00</td>
<td>220.00</td>
<td>1,003,200.00</td>
</tr>
<tr>
<td>1.03</td>
<td>2.27</td>
<td>Supplying and filling in plinth with Jamuna sand under floors including, watering, ramming consolidating and dressing complete.</td>
<td>cum</td>
<td>852.27</td>
<td>331.65</td>
<td>282,656.00</td>
</tr>
<tr>
<td>1.04</td>
<td>281</td>
<td>Cement concrete with 40mm gauge approved stone ballast, coarse sand&amp; cement in the proportion of 8:4:1 including supply of all materials, labour, tools &amp; plants etc. required for proper completion of the work.</td>
<td>cum</td>
<td>469.00</td>
<td>2,500.00</td>
<td>1,172,500.00</td>
</tr>
<tr>
<td>1.05</td>
<td>5.33</td>
<td>Providing and laying in position machine batched, machine mixed and machine vibrated design mix cement concrete of specified grade for reinforced cement concrete work including pumping of concrete to site of laying but excluding the cost of centering, shuttering, finishing and reinforcement. including Admixtures in recommended proportions as per IS 9103 to accelerate, retard setting of concrete, improve workability without impairing strength and durability as per direction of Engineer-in-charge. M-25 grade reinforced cement concrete by using 410kg. of cement per cum of concrete. All work up to floor V level.</td>
<td>cum</td>
<td>21,148.00</td>
<td>4,147.40</td>
<td>87,709,215.00</td>
</tr>
<tr>
<td>1.05a</td>
<td>5.34.1</td>
<td>Add or deduct for providing richer or leaner mixes respectively at all floor levels. Providing M-30 grade concrete by using 420kg of cement per cum of concrete instead of M-25 grade B.M.C/ R.M.C..</td>
<td>cum</td>
<td>21,148.00</td>
<td>54.55</td>
<td>1,153,623.00</td>
</tr>
<tr>
<td>1.06</td>
<td>504</td>
<td>M.S ( Tor steel or Plain ) in plain work such as RCC or R.B work including bending for proper shape and including supply of steel and its wastage, bends hooks and authorised overlapping shall be measured and including cost of binding wire.</td>
<td>MT</td>
<td>3,984.61</td>
<td>49,000.00</td>
<td>195,246,106.00</td>
</tr>
<tr>
<td>1.07</td>
<td></td>
<td>M-150 Brick work in 1:6 one cement and six fine sand mortar including necessary cutting and moulding of brick as required of one brick thick including supply of all materials labour tools and plant etc required for proper completion of the work.</td>
<td>cum</td>
<td>321.00</td>
<td>1,900.00</td>
<td>609,900.00</td>
</tr>
<tr>
<td>1.07a</td>
<td>303</td>
<td>In Foundation</td>
<td>cum</td>
<td>321.00</td>
<td>185.00</td>
<td>59,385.00</td>
</tr>
<tr>
<td>1.07b</td>
<td>310</td>
<td>Extra for Superstructure</td>
<td>cum</td>
<td>321.00</td>
<td>185.00</td>
<td>59,385.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Total Carried Forward</td>
<td></td>
<td></td>
<td></td>
<td>287,549,711.00</td>
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<tr>
<td>Sl. No</td>
<td>SOR Ref No</td>
<td>Description of Work</td>
<td>Unit</td>
<td>Quantity</td>
<td>Rate (Rs)</td>
<td>Amount (Rs)</td>
</tr>
<tr>
<td>-------</td>
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</tr>
<tr>
<td>1.08</td>
<td>13.7.2 CPWD</td>
<td>12 mm cement plaster finished with a floating coat of neat cement of mix :1:4 (1 cement: 4 fine sand)</td>
<td>sqm</td>
<td>1,604.00</td>
<td>97.90</td>
<td>157,032.00</td>
</tr>
<tr>
<td>1.09</td>
<td>13.8.2 CPWD</td>
<td>15 mm cement plaster on rough side of single or half brick wall finished with a floating coat of neat cement of mix : 1:4 (1 cement: 4 fine sand)</td>
<td>sqm</td>
<td>1,604.00</td>
<td>110.70</td>
<td>177,563.00</td>
</tr>
<tr>
<td>1.10</td>
<td>13.16 CPWD</td>
<td>Plastering with CM 1:3 mix (one cement and three sand) 6mm thick including cost, conveyance, labour charges etc. complete as per standard specification- for ceiling</td>
<td>sqm</td>
<td>32,134.00</td>
<td>62.15</td>
<td>1,997,128.00</td>
</tr>
<tr>
<td>1.11</td>
<td>13.48.1 CPWD</td>
<td>Finishing walls with Deluxe Multi surface paint system for interiors and exteriors using Primer as per manufacturers specifications :Two or more coats applied @ 1.25 ltr/10 sqm. over and including one coat of Special primer applied @ 0.75 ltr / 10 sqm.</td>
<td>sqm</td>
<td>68,256.00</td>
<td>62.25</td>
<td>4,248,936.00</td>
</tr>
<tr>
<td>1.12</td>
<td>13.37.1 CPWD</td>
<td>White washing with whitening to give an even shade - new work (three or more coats). for ceiling including cost of materials and labour charges etc. complete as per standard specification</td>
<td>sqm</td>
<td>32,134.00</td>
<td>6.75</td>
<td>216,905.00</td>
</tr>
<tr>
<td>1.13</td>
<td>11.9.5 CPWD</td>
<td>40 mm thick marble chips flooring rubbed and polished to granolithic finish, under layer 34 mm thick cement concrete 1:2:4 (1 cement : 2 coarse sand : 4 graded stone aggregate 12.5mm nominal size) and top layer 6mm thick with white, black, chocolate, grey, yellow or green marble chips of sizes from 1mm to 4mm nominal size laid in cement marble powder mix 3:1 (3 cement : 1 marble powder) by weight in proportion of 4:7 (4 cement marble powder mix : 7 marble chips) by volume including cement slurry etc. complete : Light shade pigment with ordinary cement.</td>
<td>sqm</td>
<td>8,566.00</td>
<td>313.35</td>
<td>2,684,156.00</td>
</tr>
<tr>
<td>1.14</td>
<td>16.64 CPWD</td>
<td>Providing and laying 75mm thick compacted bed of dry brick aggregate of 40mm thick nominal size including spreading, well ramming, consolidating and grouting with jamuna sand including finishing smooth etc. complete as per direction of Engineer-in-charge.</td>
<td>sqm</td>
<td>8,208.00</td>
<td>63.15</td>
<td>518,335.00</td>
</tr>
<tr>
<td>1.15</td>
<td>12.15 CPWD</td>
<td>Painting top of roofs with bitumen of approved quality at 17kg per 10 sqm impregnated with a coat of coarse sand at 60 cudm per 10sqm including cleaning the slab surface with brushes and finally with a piece of cloth lightly soaked in kerosene oil complete : With residual type petroleum bitumen of penetration 80/100</td>
<td>sqm</td>
<td>8,480.00</td>
<td>63.00</td>
<td>534,240.00</td>
</tr>
<tr>
<td>1.16</td>
<td>5.9.13 CPWD</td>
<td>Centering and shuttering including strutting, propping etc. and removal of form for :Vertical and horizontal fins individually or forming box louvers band, facias and eaves boards.</td>
<td>sqm</td>
<td>4,333.00</td>
<td>285.55</td>
<td>1,237,288.00</td>
</tr>
<tr>
<td>1.17</td>
<td>5.8 CPWD</td>
<td>Reinforced cement concrete work in vertical and horizontal fins individually or forming box louvers, facias and eaves boards up to floor five level excluding the cost of centering, shuttering, finishing and reinforcement with 1:1½:3 (1 cement : 1½ coarse sand : 3 graded stone aggregate 20mm nominal size).</td>
<td>cum</td>
<td>432.00</td>
<td>3,928.75</td>
<td>1,697,220.00</td>
</tr>
<tr>
<td>1.18</td>
<td>Confirmatry Boreholes</td>
<td>Each</td>
<td>40.00</td>
<td>22,500.00</td>
<td>900,000.00</td>
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<tr>
<td><strong>Total Carried Forward</strong></td>
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<td></td>
<td></td>
<td><strong>301,918,514.00</strong></td>
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<td>SOR Ref No</td>
<td>Description of Work</td>
<td>Unit</td>
<td>Quantity</td>
<td>Rate (Rs)</td>
<td>Amount (Rs)</td>
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<td>Total Brought Forward</td>
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<td></td>
<td></td>
<td>301,918,514.00</td>
</tr>
<tr>
<td>1.19</td>
<td>5.9.1</td>
<td>Centering and shuttering including strutting, propping etc. and removal of form for: Foundations, footings, bases of columns, etc. for mass concrete.</td>
<td>sqm</td>
<td>4,063.00</td>
<td>119.25</td>
<td>484,513.00</td>
</tr>
<tr>
<td>1.20</td>
<td>5.9.6</td>
<td>Centering and shuttering including strutting, propping etc. And removal of form for: Columns, Pillars, Piers, Abutments, Posts and Struts.</td>
<td>sqm</td>
<td>5,714.00</td>
<td>238.40</td>
<td>1,362,218.00</td>
</tr>
<tr>
<td>1.21</td>
<td>5.9.3</td>
<td>Centering and shuttering including strutting, propping etc. and removal of form for: Suspended floors, roofs, landings, balconies and access platform.</td>
<td>sqm</td>
<td>32,367.00</td>
<td>187.35</td>
<td>6,063,957.00</td>
</tr>
<tr>
<td>1.22</td>
<td>5.9.5</td>
<td>Centering and shuttering including strutting, propping etc. and removal of form for: Lintels, beams, plinth beams, girders, bressumers and cantilevers.</td>
<td>sqm</td>
<td>52,948.00</td>
<td>162.65</td>
<td>8,611,992.00</td>
</tr>
<tr>
<td>1.23</td>
<td>5.9.7</td>
<td>Centering and shuttering including strutting, propping etc. and removal of form for: Lintels, beams, plinth beams, girders, bressumers and cantilevers.</td>
<td>sqm</td>
<td>1,394.00</td>
<td>204.00</td>
<td>284,376.00</td>
</tr>
<tr>
<td>1.24</td>
<td>5.27</td>
<td>Providing and filling in position bitumen mix filler of Proportion 80 kg. of hot bitumen, 1 kg. of cement and 0.25 cubic metre of coarse sand for expansion joints.</td>
<td>cm depth / cm width / 100m</td>
<td>15.43</td>
<td>98.85</td>
<td>1,525.00</td>
</tr>
<tr>
<td>1.25</td>
<td>5.29.1.1</td>
<td>Providing and fixing sheet covering over expansion joints with iron screws as per design to match the colour / shade of wall Non-asbestos fibre cement board 6 mm thick as per IS: 14862. 150mm wide.</td>
<td>m</td>
<td>642.92</td>
<td>69.90</td>
<td>44,940.00</td>
</tr>
<tr>
<td>1.26</td>
<td>16.59.2</td>
<td>Cautionary /warning sign boards of equilateral triangular shape having each side of 900mm with support length of 3650mm.</td>
<td>sqm</td>
<td>14.04</td>
<td>2,616.55</td>
<td>36,736.00</td>
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Electrical Lighting and Related Works
LS
25,504,702.00

Quality Control Charge @ 1%
3,443,135.00

Advertisement Charges
500,000.00

Contingencies @ 0.5%
1,721,567.00

VAT added @ 2.8%
9,640,777.00

Service Tax @ 2.08%
7,161,720.00

Total Rs
366,780,672.00

Total in Crores
36.68
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For lift room upper roof

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Grid 1'-2& G'-K
Exit ramp (from I- Floor ,
Grid 1-1'& B'-G
Entry ramp (To II- Floor ,
Grid 1-2 & G-L)
Exit ramp (from II- Floor ,
Grid 1-1' & A-G)
Entry ramp (To III- Floor ,
Grid 1-1'& K-G'
Exit ramp (from III- Floor ,
Grid 1-1'& B'-E'
Staircase Slab
Landing Slab

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1.05a  5.34.1 CPWD  Add or deduct for providing richer or leaner mixes

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1.06  504 HYSD Steel

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| 2 | 106.05 | 3.60 | 763.96 |

| First Floor | | |
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| 2 | 106.05 | 1.00 | 212.10 |

| Second Floor | | |
| 2 | 80.37 | 1.00 | 160.73 |
| 2 | 106.05 | 1.00 | 212.10 |

| Third Floor | | |
| 2 | 80.37 | 1.00 | 160.73 |
| 2 | 106.05 | 1.00 | 212.10 |

| Parapet wall on terrace | | |
| 2 | 80.37 | 1.50 | 241.10 |
| 2 | 106.05 | 1.50 | 318.15 |

| Total Providing With Distemper on interior walls | | |
| Ground Floor | | |
| 2 | 80.37 | 2.80 | 450.04 |
| 2 | 106.05 | 2.80 | 593.88 |
| 4 | 32.83 | 1.00 | 131.30 |
| 4 | 42.63 | 1.00 | 170.50 |

| First Floor | | |
| 2 | 80.37 | 1.00 | 160.73 |
| 2 | 106.05 | 1.00 | 212.10 |
| 2 | 72.99 | 1.00 | 145.97 |
| 2 | 93.79 | 1.00 | 187.58 |
| 4 | 32.68 | 1.00 | 130.72 |
| 4 | 32.24 | 1.00 | 128.96 |

| Second Floor | | |
| 2 | 80.37 | 1.00 | 160.73 |
| 2 | 106.05 | 1.00 | 212.10 |
| 2 | 72.99 | 1.00 | 145.97 |
| 2 | 93.79 | 1.00 | 187.58 |

| Third Floor | | |
| 2 | 80.37 | 1.00 | 160.73 |
| 2 | 106.05 | 1.00 | 212.10 |
| 2 | 32.68 | 1.00 | 65.36 |

| Parapet wall on terrace | | |
| 2 | 80.37 | 1.50 | 241.10 |
| 2 | 106.05 | 1.50 | 318.15 |

<p>| For beams &amp; Columns | | |
| Columns | | |
| Column G.F | | |
| 7 | 0.92 | 2.80 | 18.03 |
| 2 | 2.40 | 2.80 | 13.44 |
| 17 | 2.30 | 2.80 | 109.48 |
| 5 | 1.80 | 2.80 | 25.20 |
| 13 | 1.70 | 2.80 | 61.88 |
| 13 | 1.90 | 2.80 | 69.16 |</p>
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### ARCHITECTURAL & STRUCTURAL WORKS

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#### Description:
- **Stair case Slab**: 18 - 170 x 100 = 30,600
- **Total**: 65,236.00

#### Remarks:
- Deduction for plinth beams
- Chequers of approved pattern on floors
- Total
- **Total**: 821.00

#### Painting with bitumen on terrace:
- **Total**: 8,480.00

#### Shuttering of parapet wall:
- **Total**: 4,333.00
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