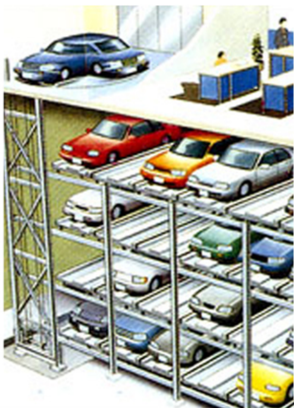
Asian Development Bank

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
ASSOCIATES

July 2010

NCR Planning Board
Asian Development Bank

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

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

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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:** Capacity Building Activities

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Volume VII: Detailed Project Reports Rehabilitation of Drainage in Sonipat

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.

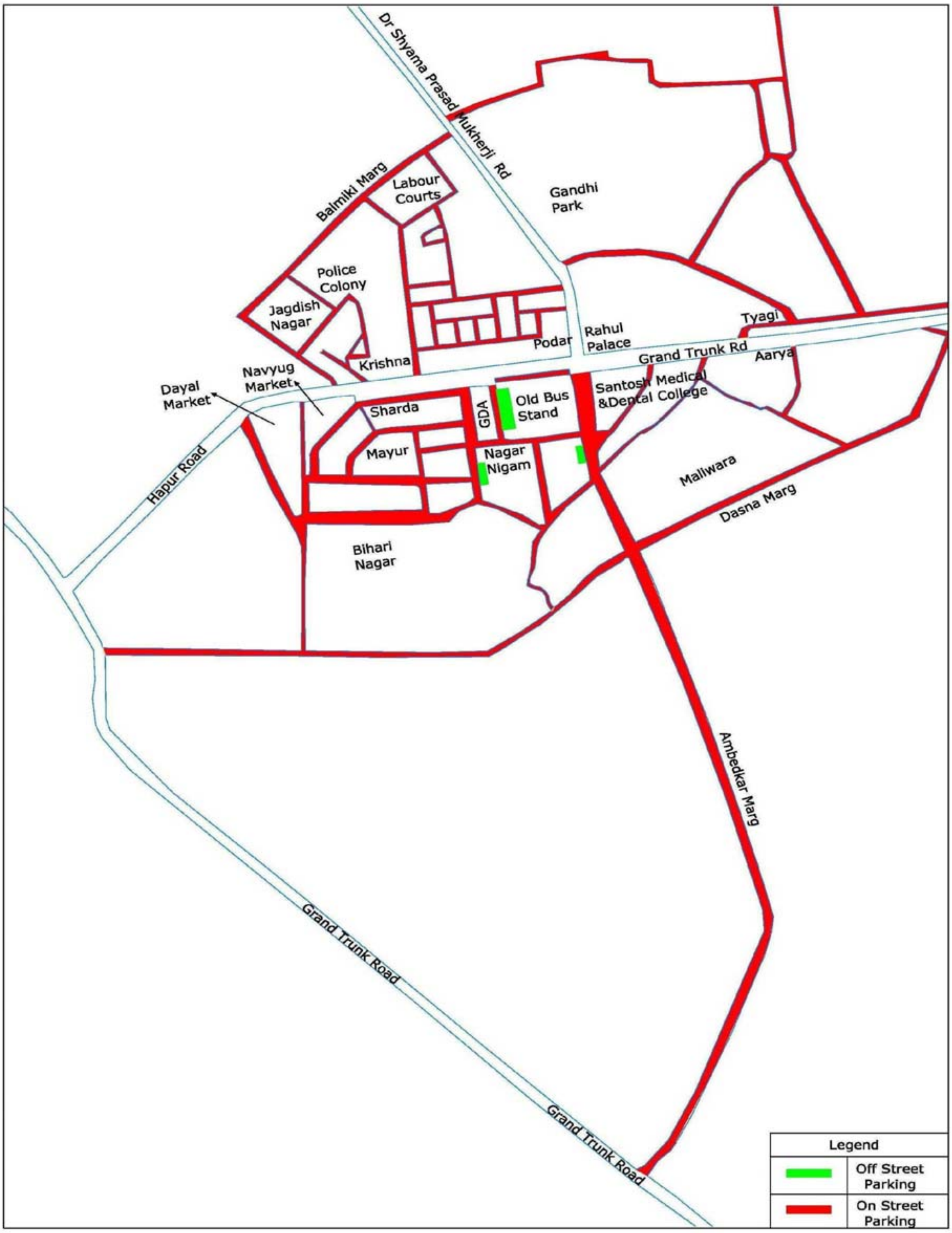
B. Existing Parking Scenario

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



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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

S. No.	Vehicle Category	Code
1	Big Car	bc
2	Small Car	sc
3	Two Wheelers	tw
4	Van	v
5	Jeep	j
6	Bus	b
7	Trucks	t
8	MAV	mAV
9	LCV	lCV
10	Auto Rickshaw	aR

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

S. No.	Vehicle Category	ECS
1	Car	1.0
2	Two Wheelers	0.25*
3	Bus	2.5
4	Trucks	2.5
5	LCV	1.75
6	Auto	0.5
7	Cycles	0.1
8	Cycle Rickshaw	0.8
9	Carts	3.2

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

SL. No.	Duration of Parking	Designation of Parking
1	< 1 Hr	Very Short Duration
2	1-2 Hours	Short Duration
3	2-5 Hours	Medium Duration
4	5-10 Hours	Long Duration
5	> 10 Hours	Long Term Parking

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 2-4: Schedule of Surveys

Type	**Schedule**	
	Date	**Day**
Junction Volume Count	17-08-09	Monday
Junction Volume Count	18-08-09	Tuesday
Junction Volume Count	19-10-08	Wednesday
Type	Schedule	
	Date	Day
Parking Accumulation Survey	20-08-09	Thursday
Parking Accumulation Survey	21-08-09	Friday
Parking Accumulation Survey	24-08-09	Monday
Parking Accumulation Survey	25-08-09	Tuesday
Parking Accumulation Survey	26-08-09	Wednesday
Parking Accumulation Survey	27-08-09	Thursday
Parking Accumulation Survey	28-08-09	Friday
Type	Schedule	
	Date	Day
Opinion Survey	20-08-09	Thursday
Opinion Survey	21-08-09	Friday
Opinion Survey	24-08-09	Monday

D. Parking Analysis and Findings

1. Roadway Inventory (Carriage way 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

Zones	Day 1 ECS	Day 2 ECS	Day 3 ECS	Total
Zone 1	3960	3130	2460	9550
Zone 2	1260	1250	1220	3730
Zone 3	840	842	830	2512
Zone 4	680	675	670	2025
Ambedkar Road	630	810	740	2180
Total	**7370**	**6707**	**5920**	

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

Zones	Day 1 Peak hour ECS	Day 2 Peak hour ECS	Day 3 Peak hour ECS	Total Peak hour ECS
Zone 1	373	300	190	863
Zone 2	90	95	90	275
Zone 3	55	50	53	158
Zone 4	40	47	45	132
Ambedkar Road	50	65	70	185
Total	**608**	**557**	**448**	

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

Zones	Parking Demand (1)	Parking Supply (2)	Gap (1) - (2)
Zone 1	373	234	139
Zone 2	95	44	51
Zone 3	55	49	6
Zone 4	47	49	-2
Ambedkar Road	70	116	-46
Total	**640**	**491**	**149**

Figure 2-2: Average Hourly variation of parking Accumulation in zone 1

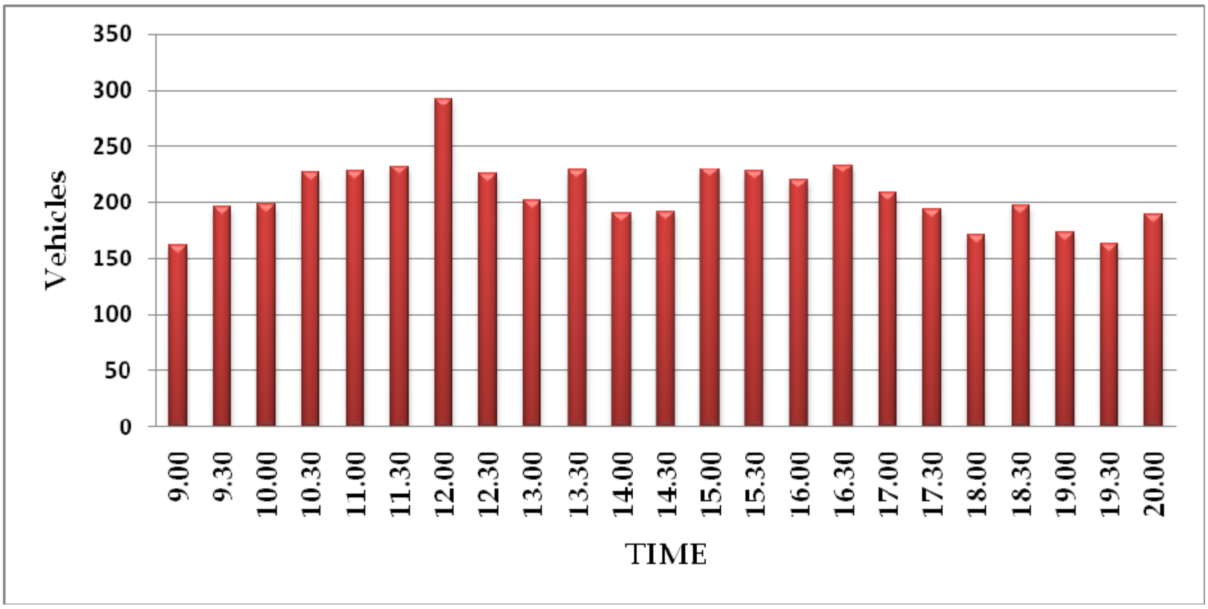


Figure 2-3: Average Hourly variation of parking Accumulation in zone 2

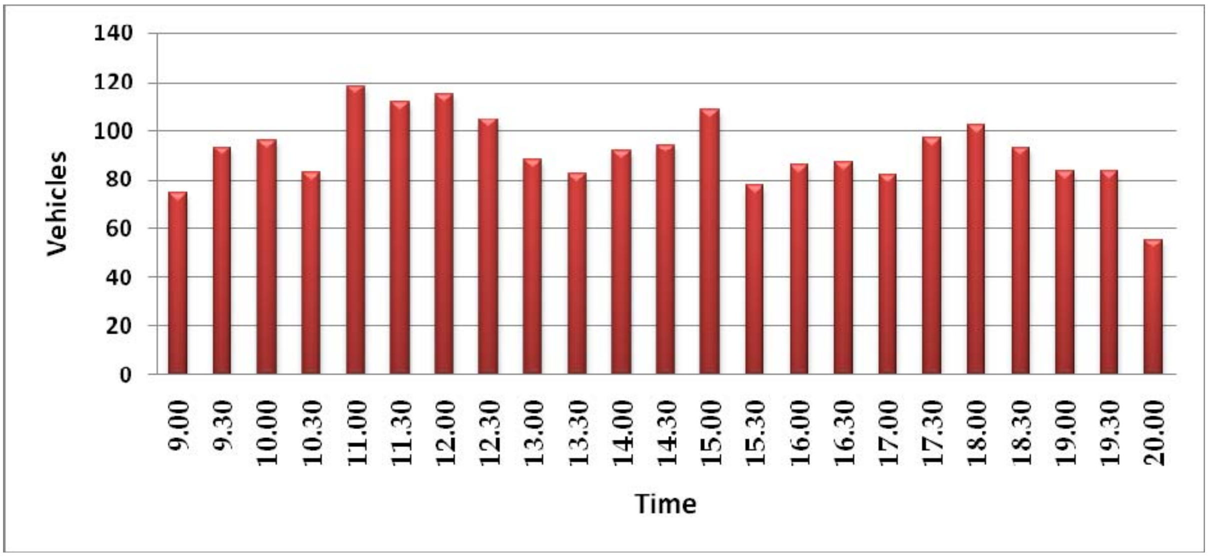


Figure 2-4: Average Hourly variation of parking Accumulation in zone 3

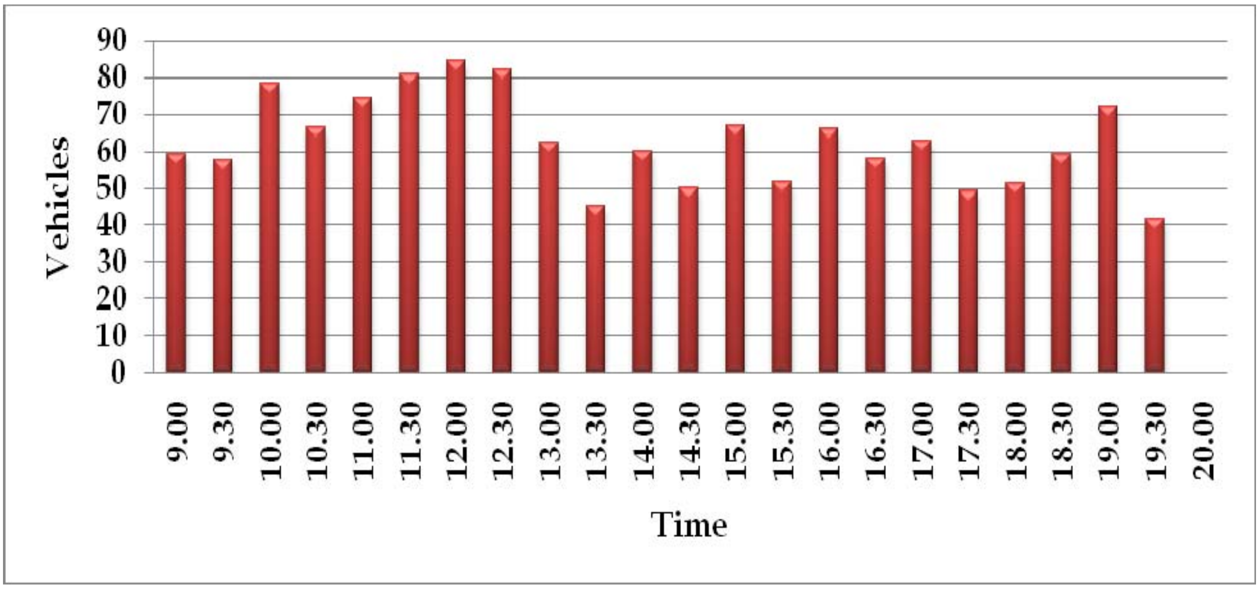


Figure 2-5: Average Hourly variation of parking Accumulation in zone 4

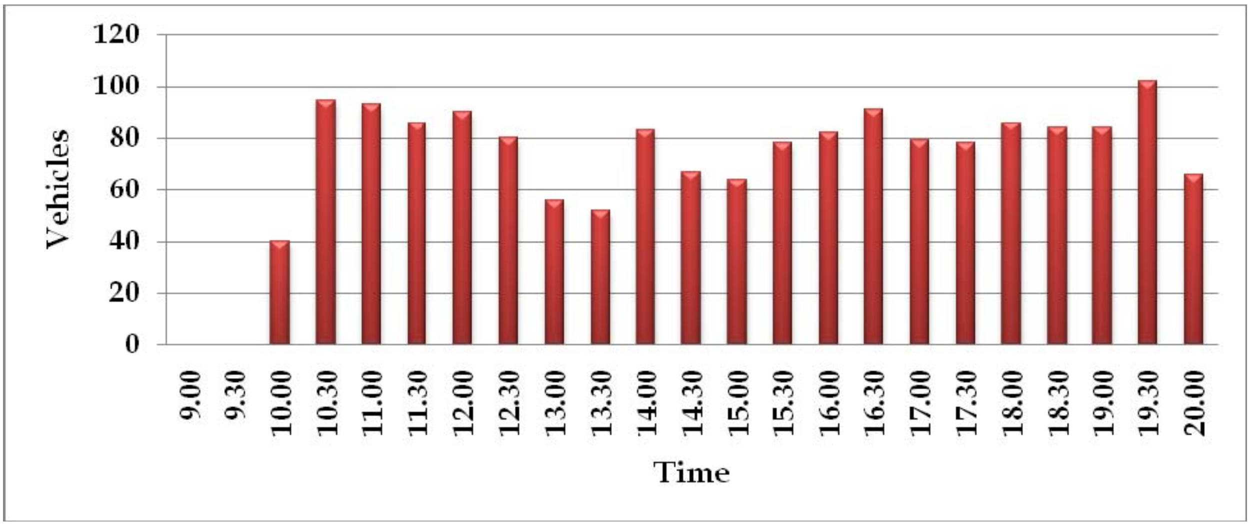
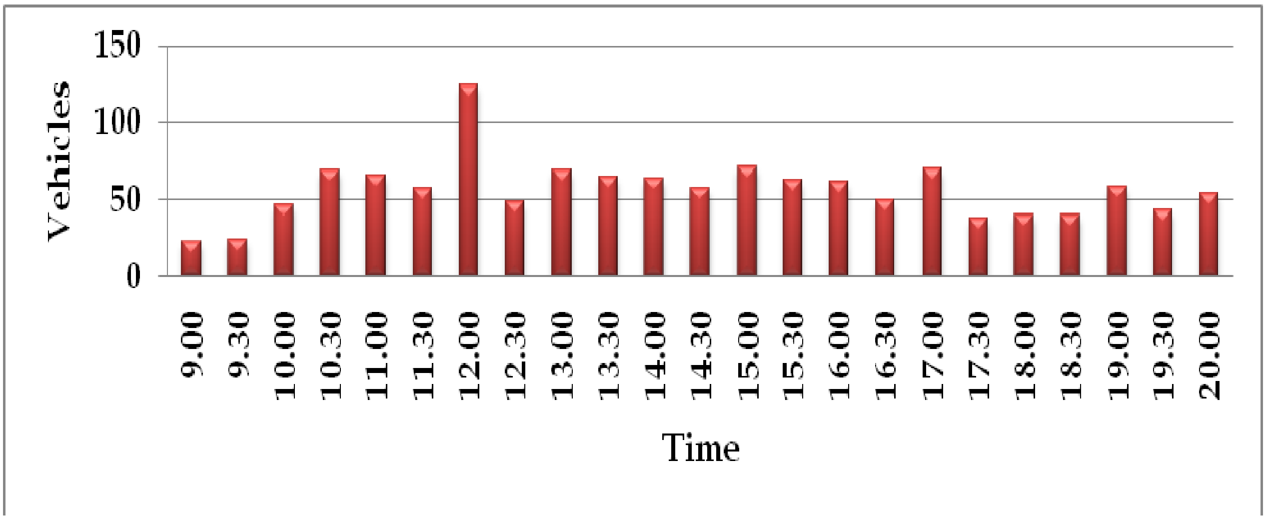


Figure 2-6: Average Hourly variation of parking Accumulation at Ambedkar Veedhi



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 2-8: Details of Trip Purpose

Trip Purpose	**Car Users (%)**	**Non-Car Users**
Work	87%	83%
Shopping	2%	6%
Leisure	6%	9%
Others	5%	2%
Total	100%	100%

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 2-9: Details of Trip Frequency

Trip Purpose	**Car Users (%)**	**Non-Car Users**
Daily	89%	86%
Weekly	0%	12%
Occasionally	11%	0%
Others	0%	2%
Total	100%	100%

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

Base year Parking Demand	Projected Parking Demand (No. of vehicles)
Base year 2010	650
2010 – 2020	723
2020 – 2025	805
2025 – 2030	896

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:

(v) logeP = Ao + A1 logeGDP + A2 logeNSDP +A3 logePopulation + A4 loge PCI

Where,

P = Traffic Volume

A0 = Regression Constant

A1, A2, A3, A4 are the Elasticity Coefficients

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

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

logeP = Ao + A1 logeGDP + A2 logeNSDP +A3 logePopulation + A4 loge PCI

The values of A1, A2, A3, A4 are found

(viii) Growth rate of traffic = (A1 * Expected Growth rate of GDP) + (A2 * Expected Growth rate of NSDP)

(A3 * Expected Growth rate of Population) + (A4 * 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

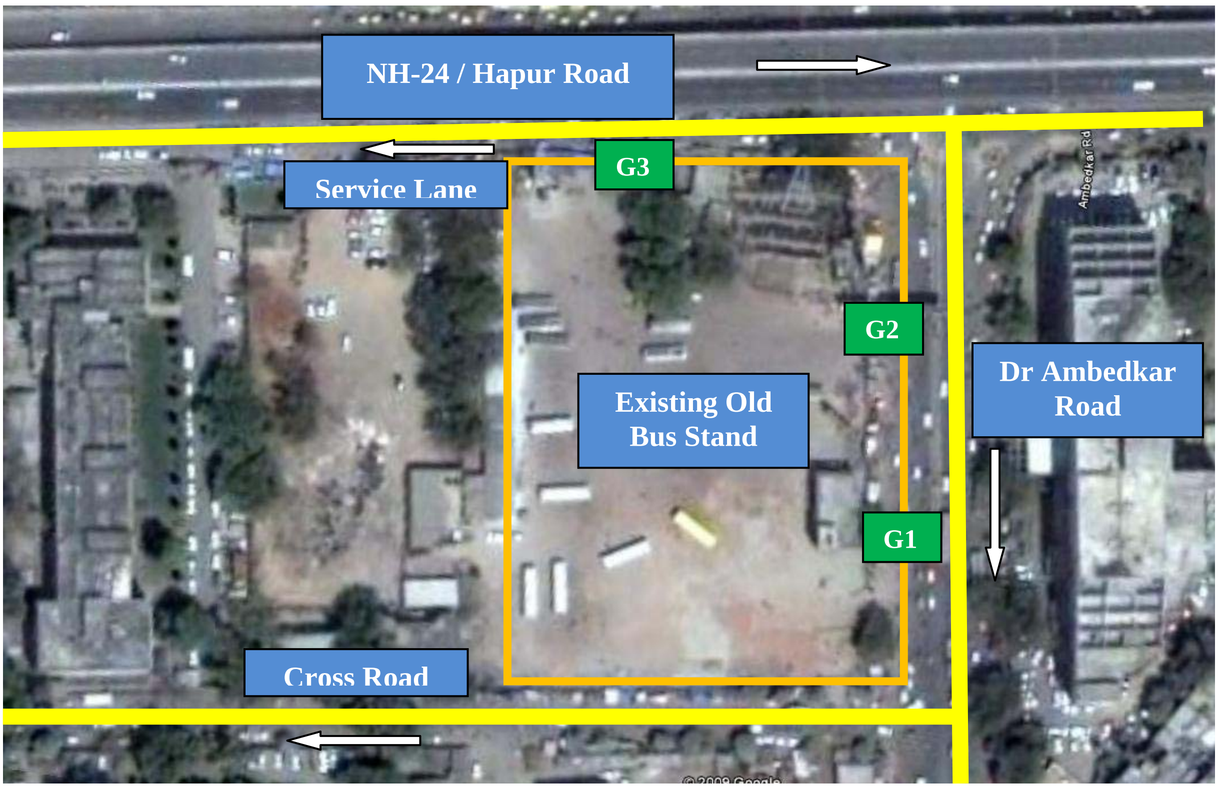
46. The parking analysis clearly established the need for constructing a multi-level parking lot in the vicinity of GDA in the CBD area. The best option available is to make use of the site that is currently functioning as the old bus stand. Since the Master Plan for Ghaziabad has already identified 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

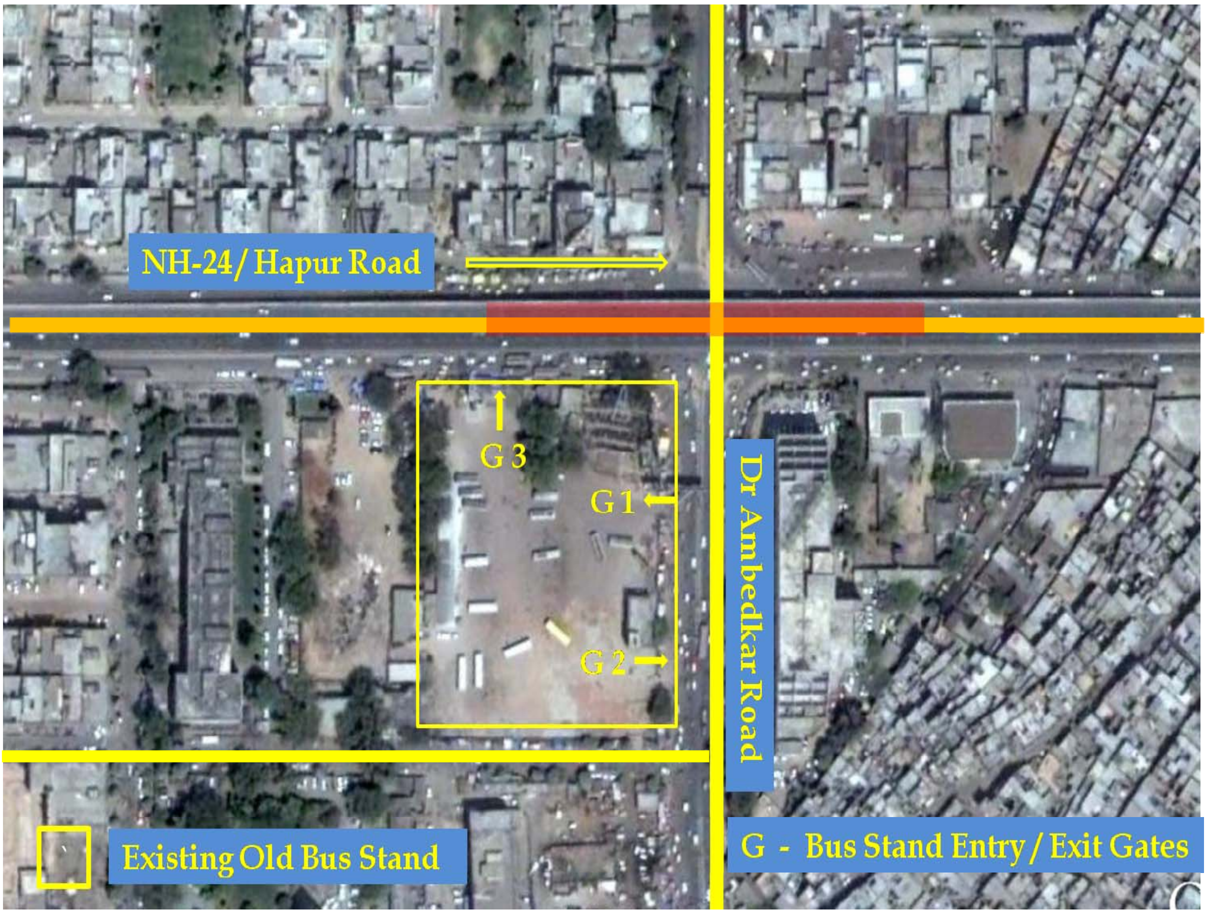
Photo 2: View of Bus Stand

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 3-1: Particulars of Proposed Multi-level Parking Facility

S. No.	Parameter	Unit	Value
1	Site area	Sq. m	10,040
2	Area (Ground Floor)	Sq. m	8,323
3	Area (1st, 2nd and 3rd Floors)	Sq. m	25,833
3	Commercial/Retail (Ground Floor)	Sq. m	5,000
4	Parking (Ground Floor) – 2-wheelers	No. s	117
5	Parking (1st floor) – 2-wheelers	No. s	213
6	Parking (1st, 2nd and 3rd Floors) – Cars	No.s	777

Figure 3-1: Ground Floor Plan of the Proposed Multilevel Parking

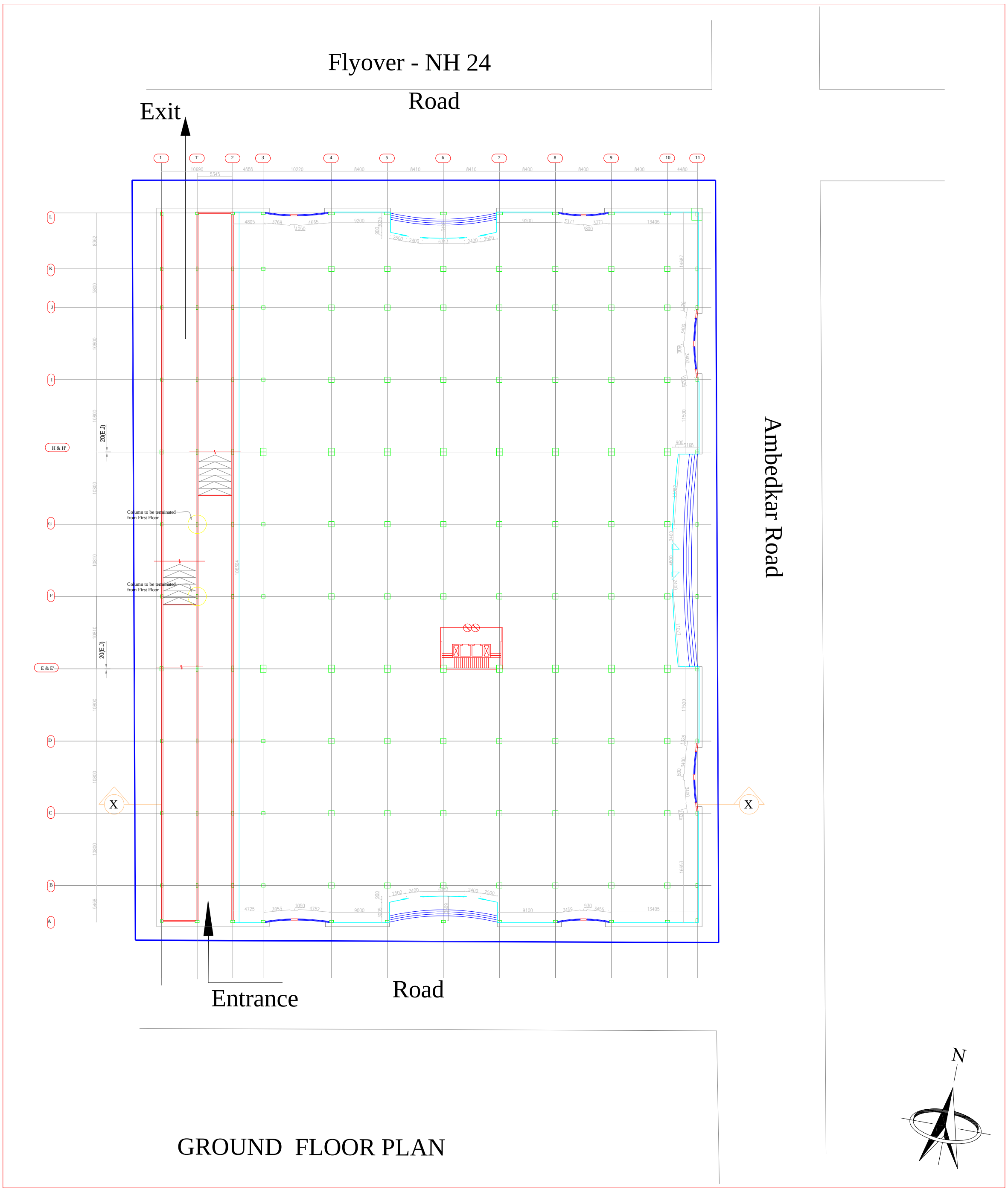
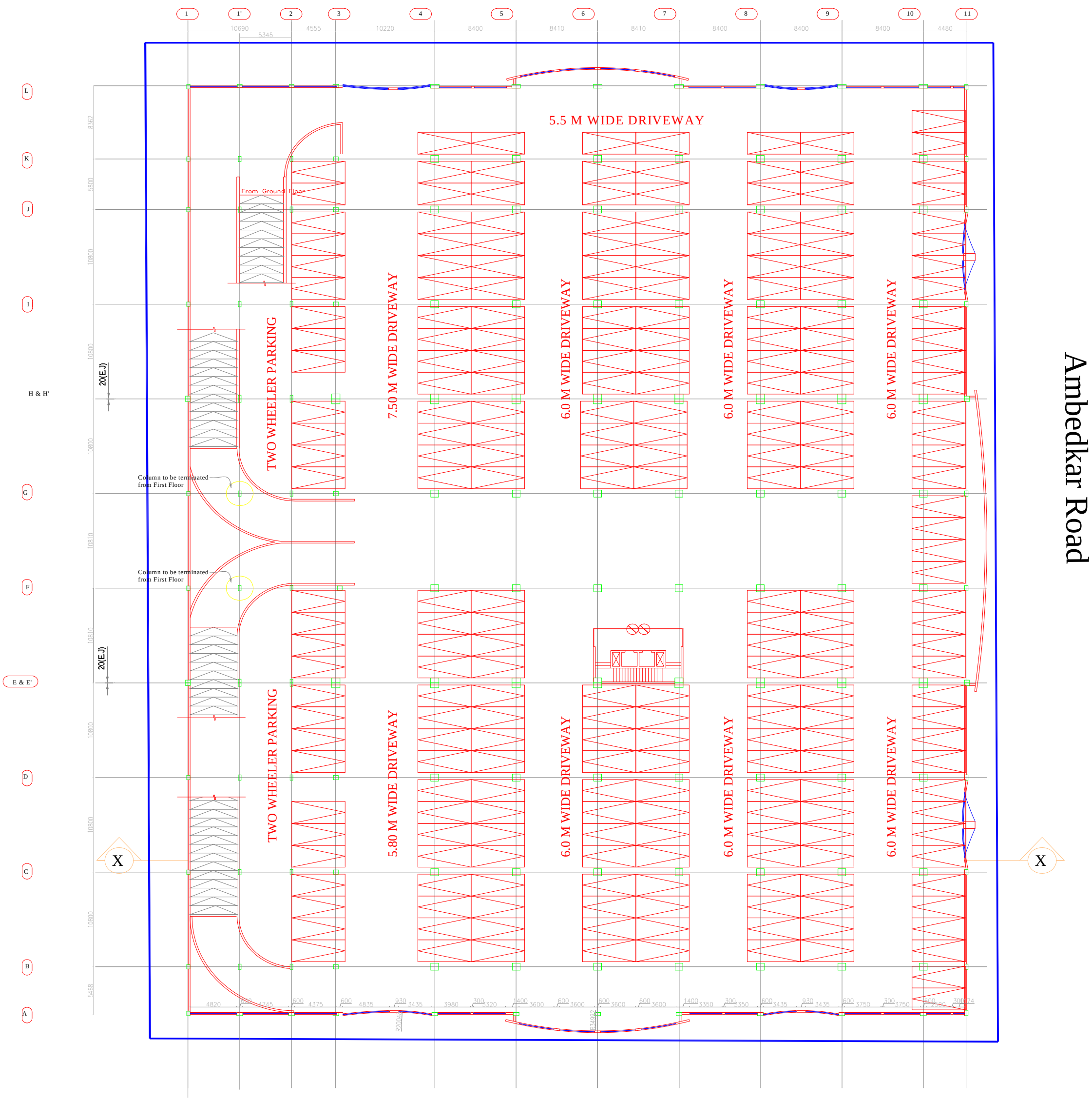


Figure 3-2: Typical Floor Plan of the Proposed Multilevel Parking Facility



Typical Floor Plan

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)

- 1.2 * (DL+LL+EQX)
- 1.2 * (DL+LL-EQX)
- 1.2 * (DL+LL+EQZ)
- 1.2 * (DL+LL-EQZ)
- 0.9 * DL+ 1.5 * EQX
- 0.9 * DL - 1.5 * EQX
- 0.9 * DL+ 1.5 * EQZ
- 0.9 * DL - 1.5 * 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
(iv) 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

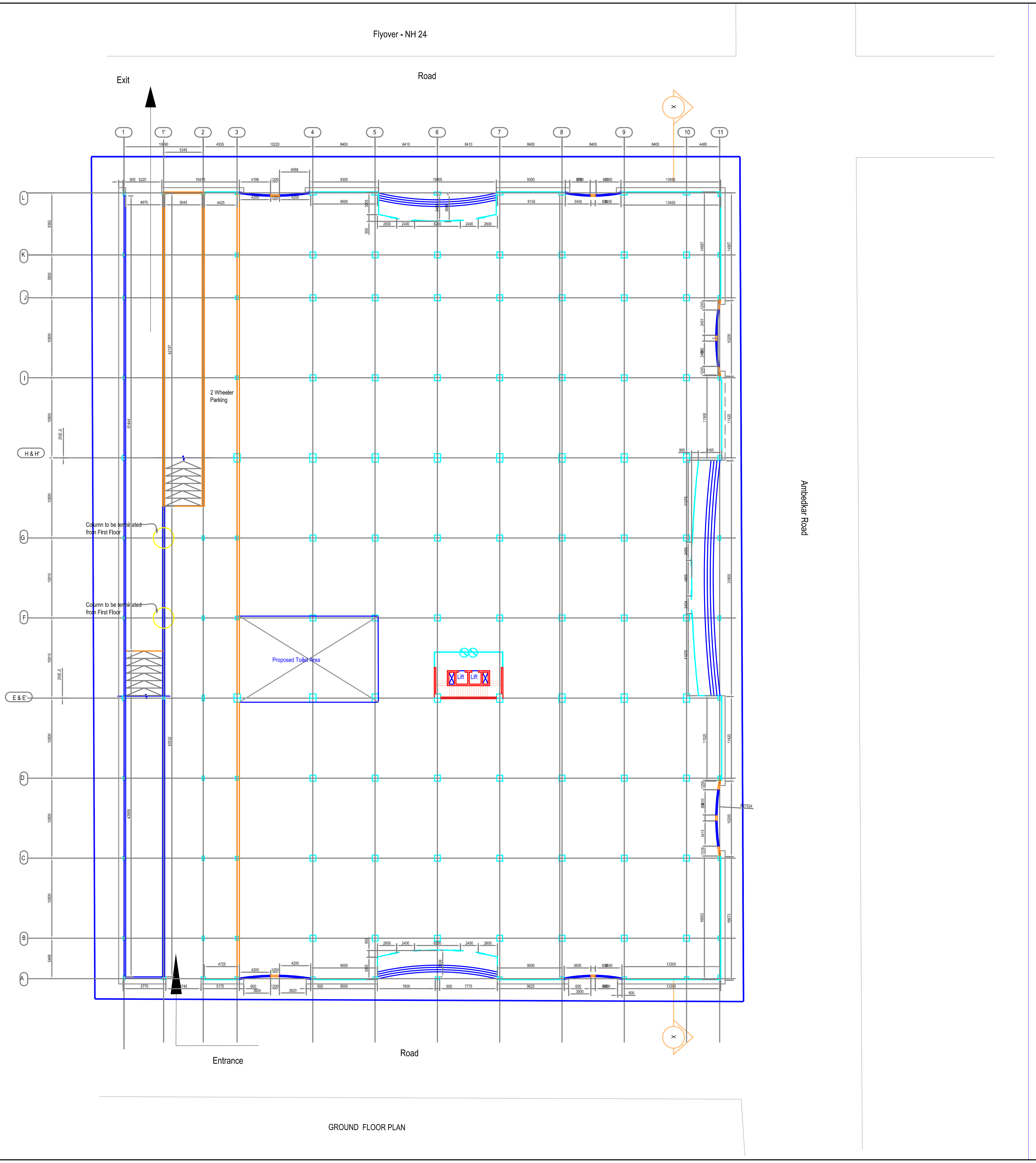
91. Total item wise quantities are calculated as per the detailed drawings. Separate heads for all different items of work is included in the BOQ. The major work items considered are:

- 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**.

Appendix 1



Capacity Development of the NCRPB: Component B (ADB TA-7055)

(MULTI LEVEL PARKING)
GROUND FLOOR PLAN

NOTE:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS MENTIONED OTHERWISE.

Client:

Asian Development Bank
National Capital Region Planning Board

Consultant:

Wilbur Smith Associates

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Design: Yunus	Approved: AN		
Date: April 2010			
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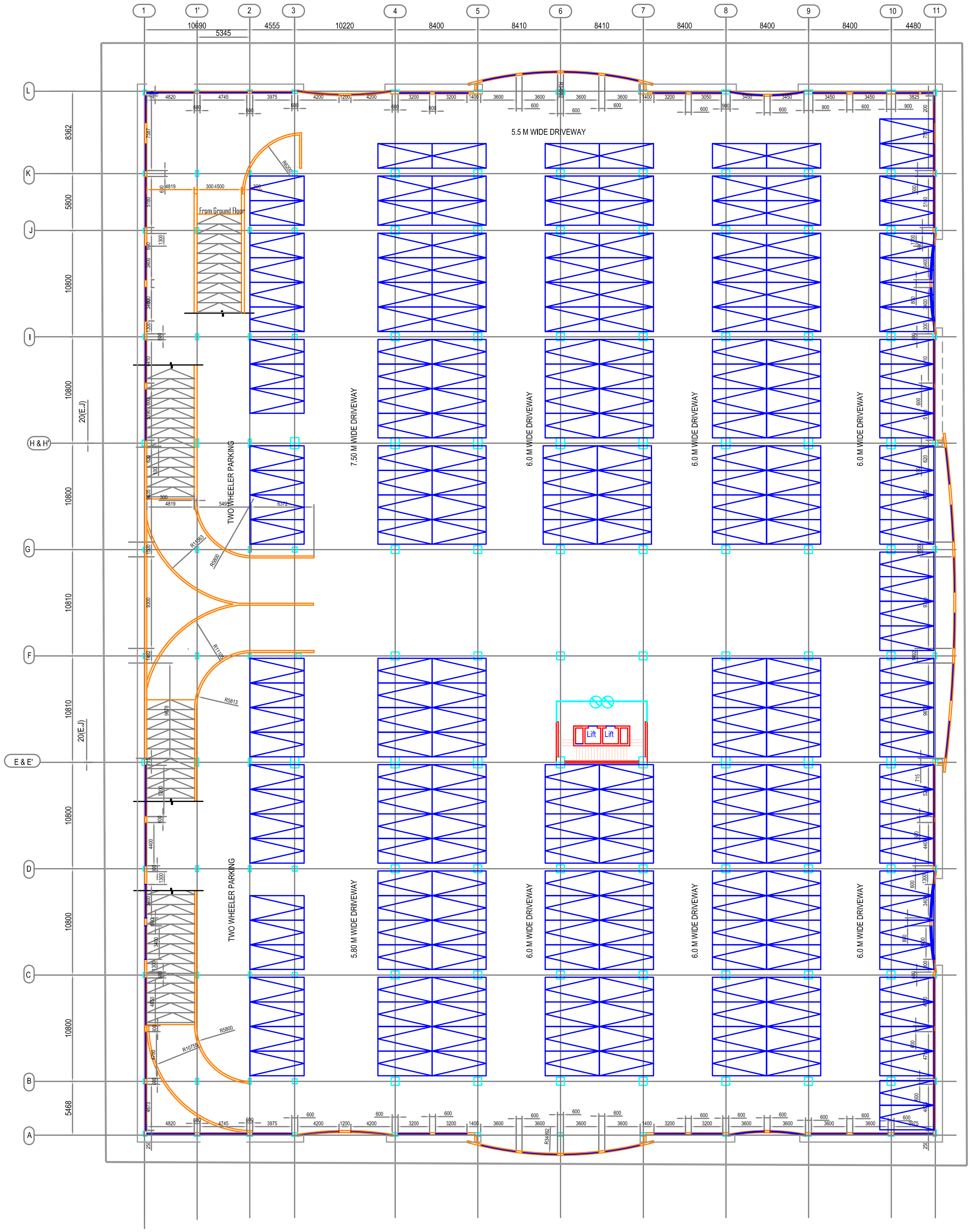

FIRST FLOOR PLAN

Capacity Development of the NCRPB: Component B (ADB TA-7055)

(MULTI LEVEL PARKING)
FRIST FLOOR PLAN

NOTE:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS MENTIONED OTHERWISE.

Client:
Asian Development Bank
National Capital Region Planning Board

Consultant:
Wilbur Smith Associates

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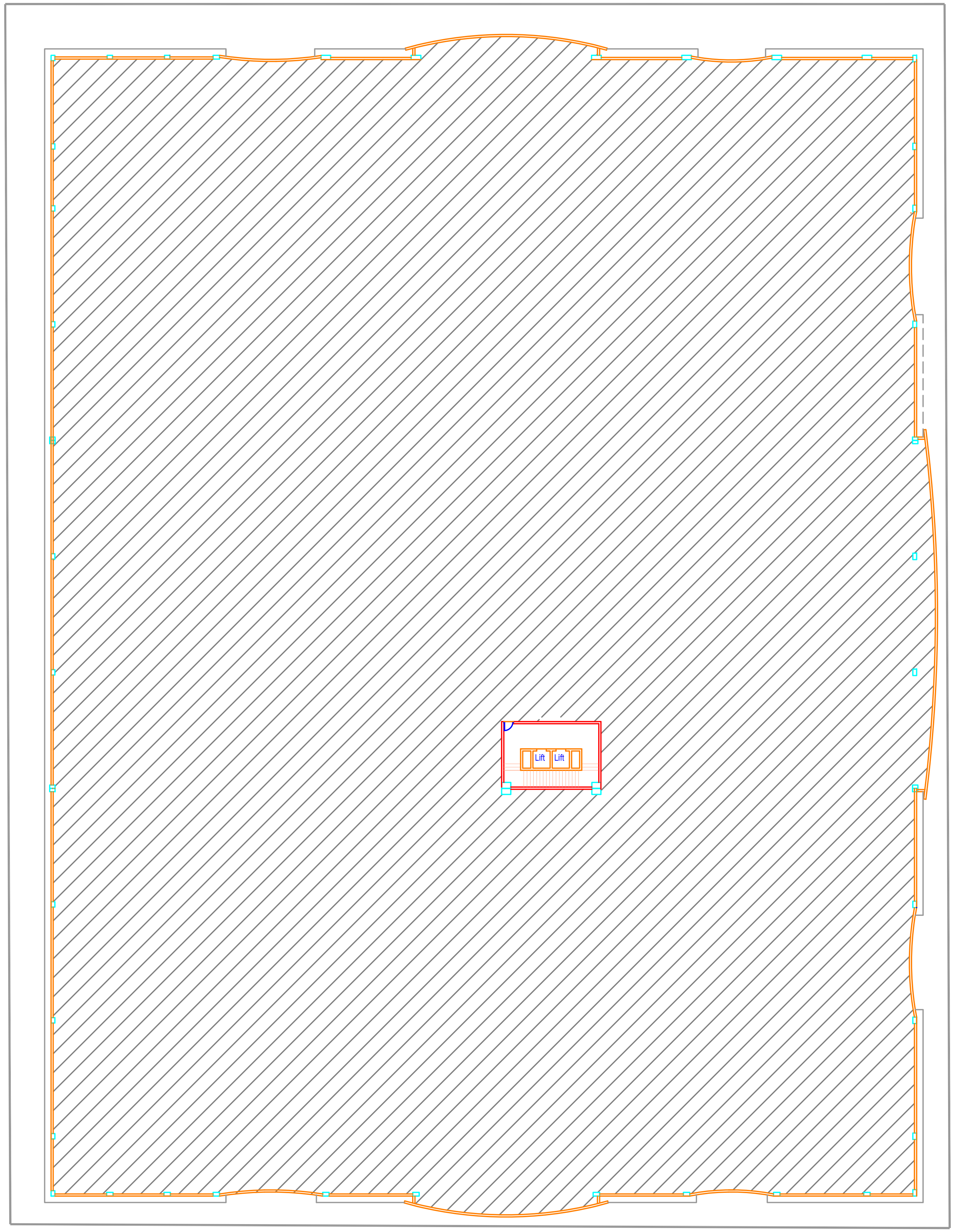

TYPICAL FLOOR PLAN

Capacity Development of the NCRPB: Component B (ADB TA-7055)

(MULTI LEVEL PARKING)
TYPICAL FLOOR PLAN

NOTE:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS MENTIONED OTHERWISE.

Client:
Asian Development Bank
National Capital Region Planning Board

Consultant:
Wilbur Smith Associates

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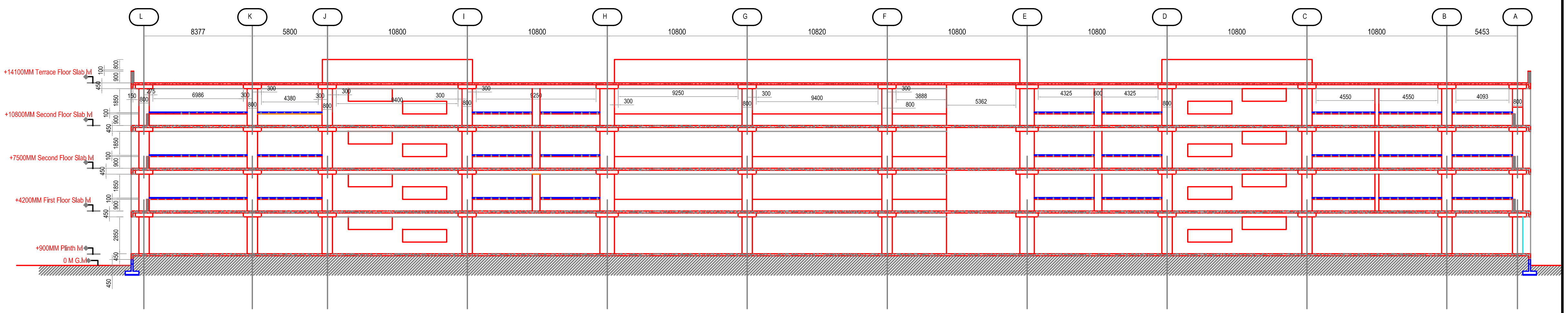
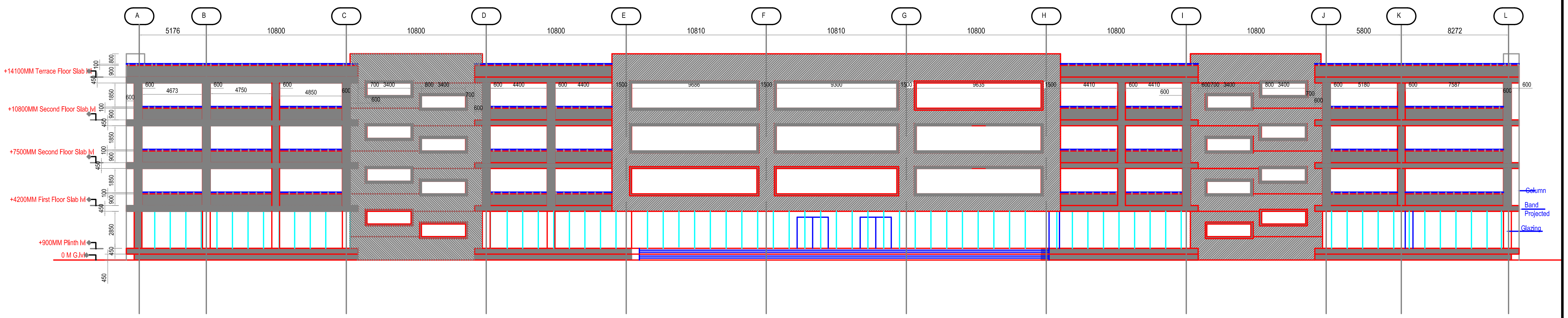
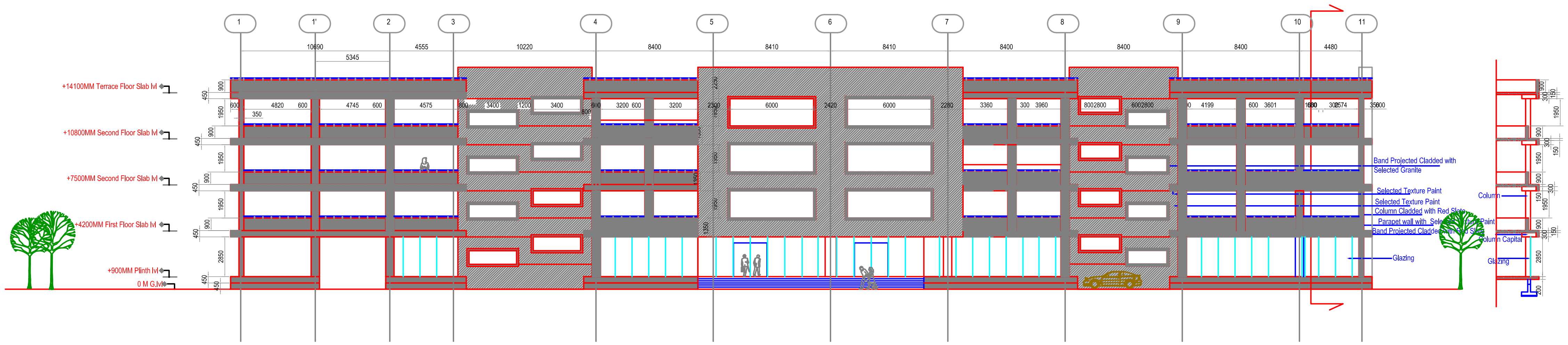


SECTION @ XX



EAST SIDE ELEVATION



SOUTH SIDE ELEVATION

Capacity Development of the NCRPB: Component B (ADB TA-7055)

(MULTI LEVEL PARKING)
SECTIONS & ELEVATION

NOTE:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS MENTIONED OTHERWISE.

Client:

Asian Development Bank
National Capital Region Planning Board

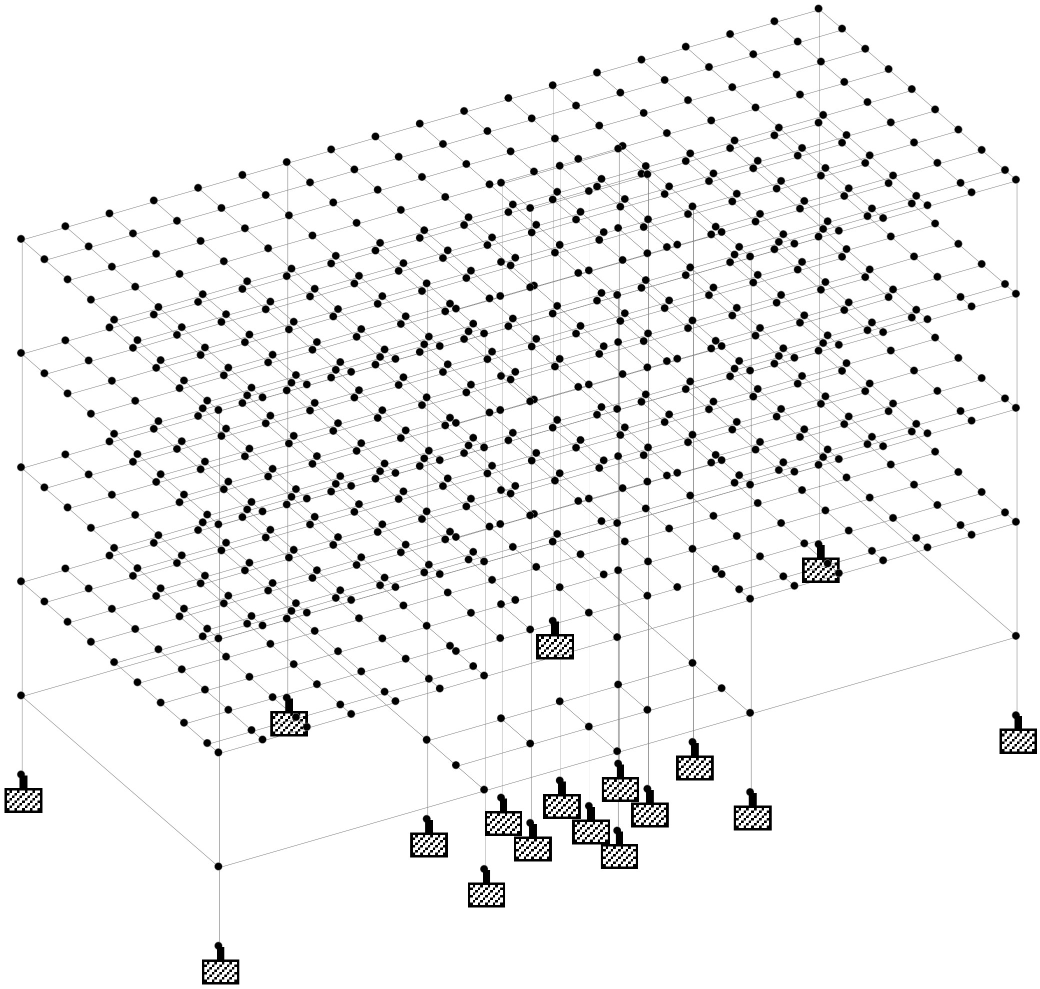
Consultant:

Wilbur Smith Associates

Drawn: Shan	Checked: APK	
Design: Yunus	Approved: AN	Sheet No.
Date: April 2010		
Scale: As shown		
Drawing No: ST-002 -04		

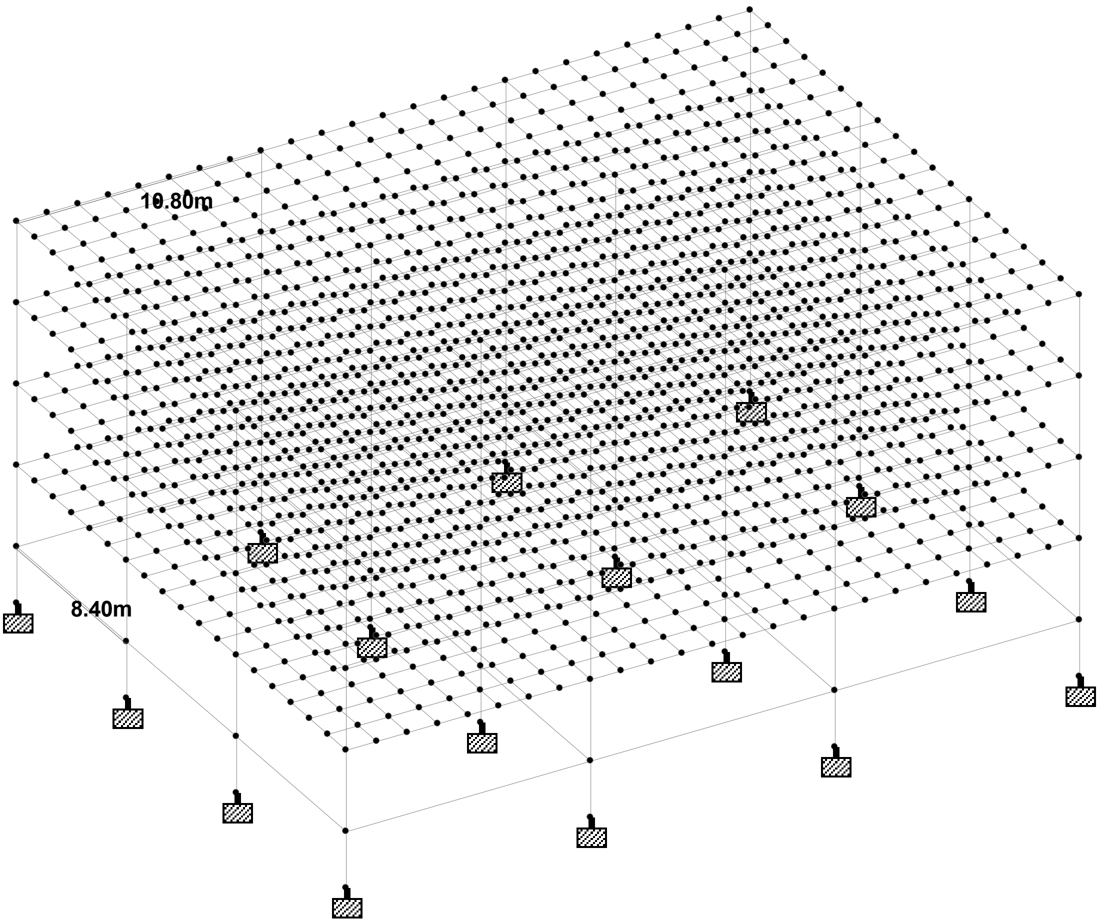
STAAD MODEL

Multi Level Car Parking Lift & Stair Portion

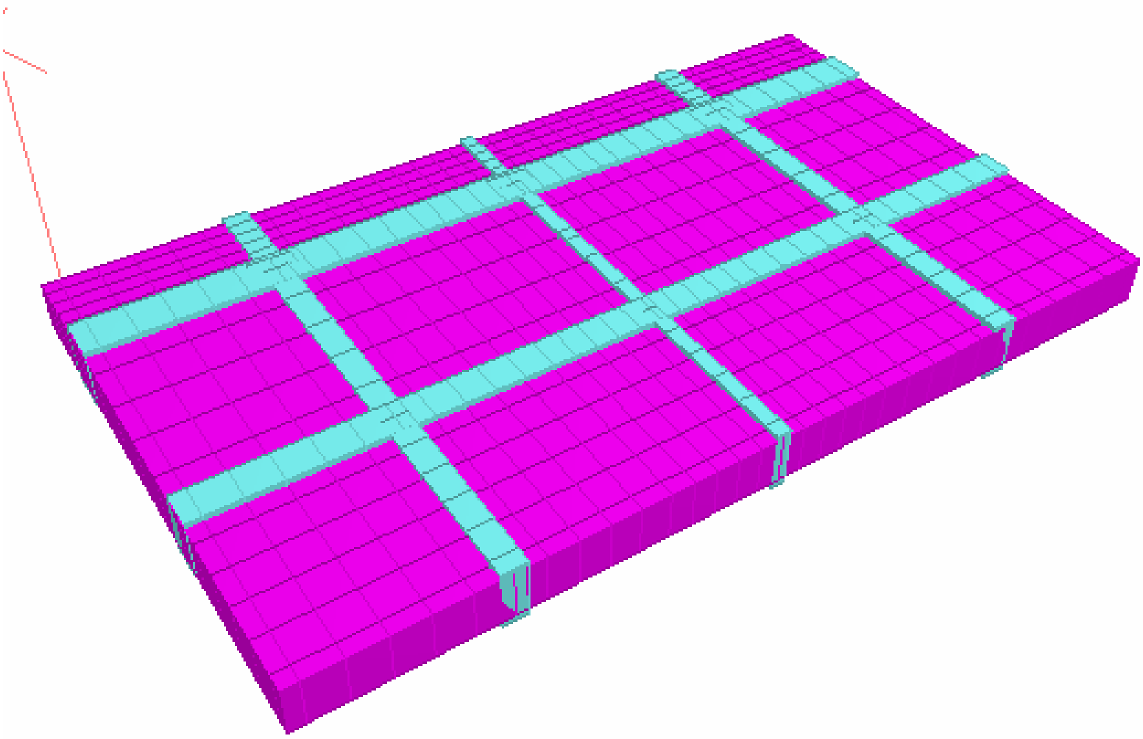


MODEL

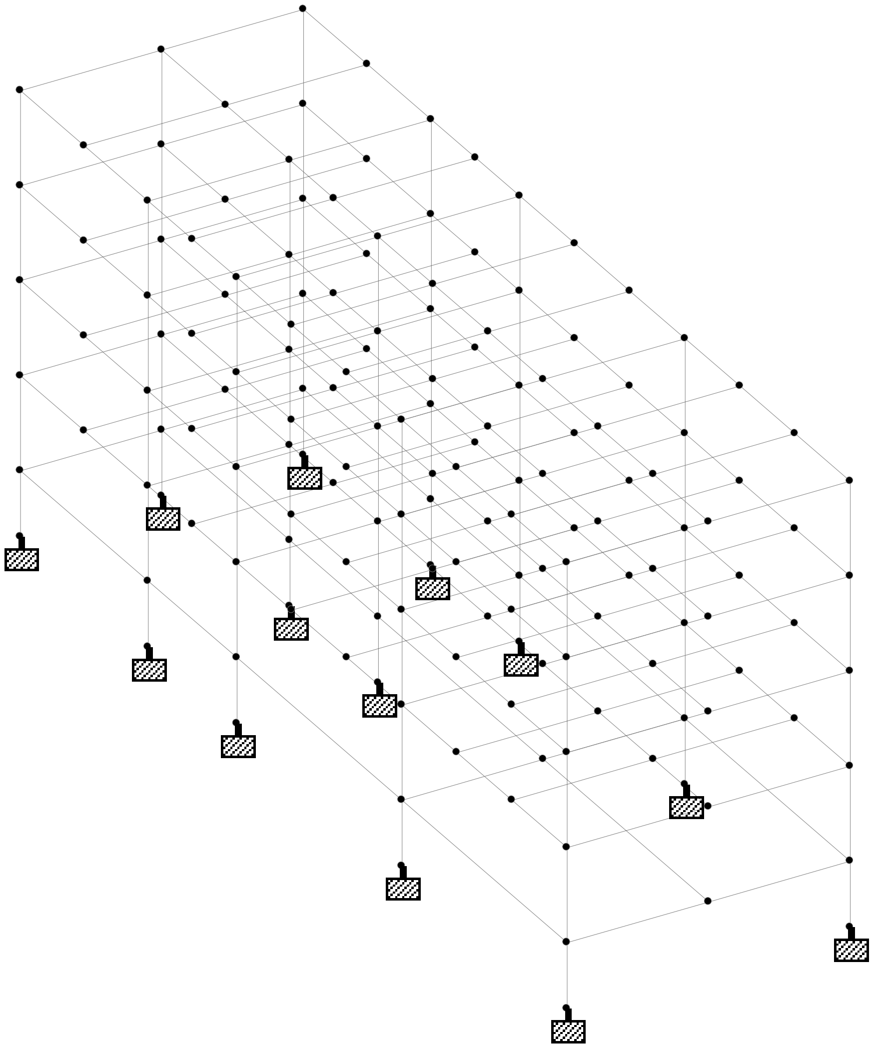
Multi Level Car Parking Central Portion with Column Spacing of 10.8 x 8.4



Raft Slab



Multi Level Car Parking Ramp Portion



STAAD INPUT FILE

STAAD INPUT FILE

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E 2.17185e+007
POISSON 0.17
DENSITY 24
ALPHA 1e-005
DAMP 0.05
ISOTROPIC MATERIAL1
E 2.5e+007
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MEMBER PROPERTY AMERICAN
11 TO 14 18 TO 21 25 TO 28 60 TO 62 64 TO 66 68 TO 70 72 TO 74 982 TO 984 -
986 TO 988 990 TO 992 994 TO 996 1904 TO 1906 1908 TO 1910 1912 TO 1914 1916 -
1917 TO 1918 2826 TO 2828 2830 TO 2832 2834 TO 2836 2838 TO 2839 -
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1 TO 3 22 TO 24 29 TO 40 PRIS YD 0.3 ZD 0.6
118 120 122 124 126 128 130 132 134 136 138 140 142 144 146 148 150 152 154 -
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232 234 236 238 240 242 288 TO 602 1040 1042 1044 1046 1048 1050 1052 1054 -
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2996 2998 3000 3002 3004 3006 3008 3054 TO 3367 -
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603 TO 962 1525 TO 1884 2447 TO 2806 3369 TO 3727 -
3728 PRIS YD 0.55 ZD 1.4 YB 0.43 ZB 0.3
41 TO 46 75 TO 87 93 TO 106 117 119 121 123 125 127 129 131 133 135 137 139 -
141 143 145 147 149 151 153 155 157 159 161 163 165 167 169 171 173 175 177 -
179 181 183 185 187 189 191 193 195 197 199 201 203 205 207 209 211 213 215 -
217 219 221 223 225 227 229 231 233 235 237 239 241 963 TO 968 997 TO 1009 -
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3007 PRIS AX 0.477 IX 0.01945 IY 0.0001 IZ 0.0425
47 TO 58 88 TO 92 107 TO 116 243 TO 287 969 TO 980 1010 TO 1014 1029 TO 1038 -
1165 TO 1209 1891 TO 1902 1932 TO 1936 1951 TO 1960 2087 TO 2131 -
2813 TO 2824 2854 TO 2858 2873 TO 2882 3009 TO 3052 -
3053 PRIS AX 0.48 IX 0.0194 IY 0.0001 IZ 0.043
MEMBER PROPERTY
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2837 PRIS YD 0.45 ZD 1.1
SUPPORTS
1 TO 4 9 TO 12 17 TO 20 25 TO 28 FIXED
DEFINE WIND LOAD
TYPE 1
INT 1.73 1.73 1.73 1.73 1.73 HEIG  3.3  6.6  9.9  13.2  16.5
EXP 1 JOINT 45 TO 48 127 TO 147 520 TO 523 602 TO 622 995 TO 998 1077 TO 1097 -
1470 TO 1473 1552 TO 1572
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DEFINE 1893 LOAD
ZONE 0.24 RF 3 I 1.5 SS 1 DM 0.05 DT 2.5
JOINT WEIGHT
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6 WEIGHT 403.837
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8 WEIGHT 283.489
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10 WEIGHT 28.8
11 WEIGHT 28.8
12 WEIGHT 28.8
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14 WEIGHT 256.987
15 WEIGHT 256.987
16 WEIGHT 257.098
17 WEIGHT 28.8
18 WEIGHT 28.8
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21 WEIGHT 257.093
22 WEIGHT 256.985
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24 WEIGHT 257.098
25 WEIGHT 28.8
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28 WEIGHT 28.8
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31 WEIGHT 233.597
32 WEIGHT 198.579
33 WEIGHT 631.573
34 WEIGHT 1196.26
35 WEIGHT 1196.33
36 WEIGHT 614.433
37 WEIGHT 1233.85
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39 WEIGHT 2438
40 WEIGHT 1196.95
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42 WEIGHT 2438.91
43 WEIGHT 2438.97
44 WEIGHT 1197.35
45 WEIGHT 652.866
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CONSTANTS
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72 TO 74 118 120 122 124 126 128 130 132 134 136 138 140 142 144 146 148 -
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MATERIAL CONCRETE MEMB 4 TO 7 41 TO 59 63 67 71 75 TO 117 119 121 123 125 -
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2037 2039 2041 2043 2045 2047 2049 2051 2053 2055 2057 2059 2061 2063 2065 -
2067 2069 2071 2073 2075 2077 2079 2081 2083 2085 2087 TO 2131 2807 TO 2825 -
2829 2833 2837 2841 TO 2883 2885 2887 2889 2891 2893 2895 2897 2899 2901 -
2903 2905 2907 2909 2911 2913 2915 2917 2919 2921 2923 2925 2927 2929 2931 -
2933 2935 2937 2939 2941 2943 2945 2947 2949 2951 2953 2955 2957 2959 2961 -
2963 2965 2967 2969 2971 2973 2975
MATERIAL CONCRETE MEMB 2977 2979 2981 2983 2985 2987 2989 2991 2993 2995 2997 -
2999 3001 3003 3005 3007 3009 TO 3053
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 3.3 17.7 OPEN
LOAD 4 LOADTYPE Wind  TITLE WIND Z
WIND LOAD Z 1 TYPE 1 YR 3.3 17.7 ZR 0 25.2 OPEN
LOAD 5 LOADTYPE Wind  TITLE WIND -X
WIND LOAD -X -1 TYPE 1 XR 0 32.4 YR 3.3 17.7 OPEN
LOAD 6 LOADTYPE Wind  TITLE WIND -Z
WIND LOAD -Z -1 TYPE 1 YR 3.3 17.7 ZR 0 25.2 OPEN
LOAD 7 LOADTYPE None  TITLE SW
SELFWEIGHT Y -1
LOAD 8 LOADTYPE Dead  TITLE WALL LOAD
MEMBER LOAD
1 TO 3 29 TO 40 UNI GY -10.5
LOAD 9 LOADTYPE Dead  TITLE FLOOR FINISH
FLOOR LOAD
YRANGE 6.9 7 FLOAD -1 GY
YRANGE 10.5 10.6 FLOAD -1 GY
YRANGE 14.1 14.2 FLOAD -1 GY
YRANGE 17.7 17.8 FLOAD -1 GY
LOAD 10 LOADTYPE Dead  TITLE PARAPET LOAD
MEMBER LOAD
44 TO 47 51 55 88 TO 92 107 TO 116 201 203 205 207 209 211 213 215 217 219 -
221 223 225 227 229 231 233 235 237 239 241 966 TO 969 973 977 1010 TO 1014 -
1029 TO 1038 1123 1125 1127 1129 1131 1133 1135 1137 1139 1141 1143 1145 -
1147 1149 1151 1153 1155 1157 1159 1161 1163 1888 TO 1891 1895 1899 1932 -
1933 TO 1936 1951 TO 1960 2045 2047 2049 2051 2053 2055 2057 2059 2061 2063 -
2065 2067 2069 2071 2073 2075 2077 2079 2081 2083 2085 2810 TO 2813 2817 -
2821 2854 TO 2858 2873 TO 2882 2967 2969 2971 2973 2975 2977 2979 2981 2983 -
2985 2987 2989 2991 2993 2995 2997 2999 3001 3003 3005 3007 UNI GY -2.85
LOAD 11 LOADTYPE Roof Live REDUCIBLE TITLE LL ON SLAB WITH DISPERSION
FLOOR LOAD
YRANGE 6.9 7 FLOAD -4 GY
YRANGE 10.5 10.6 FLOAD -4 GY
YRANGE 14.1 14.2 FLOAD -4 GY
YRANGE 17.7 17.8 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 1 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+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 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-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
PRINT MEMBER FORCES ALL
FINISH
```

STAAD INPUT FILE

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STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 08-Jan-10
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 -1.5 0; 2 8.4 -1.5 0; 3 0 1 0; 4 8.4 1 0; 5 0 -1.5 10.8; 6 8.4 -1.5 10.8;
7 0 1 10.8; 8 8.4 1 10.8; 9 0 1 9.25; 10 8.4 1 9.25; 11 2.35 1 9.25;
12 4.2 1 9.25; 13 6.05 1 9.25; 14 2.35 1 7.65; 15 4.2 1 7.65; 16 6.05 1 7.65;
17 0 1 7.65; 18 8.4 1 7.65; 19 4.2 1 10.8; 20 4.2 -1.5 10.8; 21 4.2 -1.5 9.25;
22 4.2 -1.5 7.65; 23 2.35 -1.5 9.25; 24 2.35 -1.5 7.65; 25 6.05 -1.5 9.25;
26 6.05 -1.5 7.65; 27 0 -1.5 7.65; 28 8.4 -1.5 7.65; 29 0 4.6 0; 30 8.4 4.6 0;
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39 4.2 4.6 7.65; 40 6.05 4.6 7.65; 41 0 4.6 7.65; 42 8.4 4.6 7.65;
43 4.2 4.6 10.8; 44 1.4 4.6 0; 45 2.8 4.6 0; 46 4.2 4.6 0; 47 5.6 4.6 0;
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58 8.4 4.6 1.275; 59 8.4 4.6 2.55; 60 8.4 4.6 3.825; 61 8.4 4.6 5.1;
62 8.4 4.6 6.375; 63 1.4 4.6 1.275; 64 1.4 4.6 2.55; 65 1.4 4.6 3.825;
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78 5.6 4.6 1.275; 79 5.6 4.6 2.55; 80 5.6 4.6 3.825; 81 5.6 4.6 5.1;
82 5.6 4.6 6.375; 83 7 4.6 1.275; 84 7 4.6 2.55; 85 7 4.6 3.825; 86 7 4.6 5.1;
87 7 4.6 6.375; 88 1.4 4.6 9.25; 89 7 4.6 9.25; 90 0 8.2 0; 91 8.4 8.2 0;
92 0 8.2 10.8; 93 8.4 8.2 10.8; 94 0 8.2 9.25; 95 8.4 8.2 9.25;
96 2.35 8.2 9.25; 97 4.2 8.2 9.25; 98 6.05 8.2 9.25; 99 2.35 8.2 7.65;
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E 2.17185e+007
POISSON 0.17
DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL
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27 TO 33 60 61 64 TO 68 177 178 181 TO 185 294 295 298 TO 302 411 412 415 -
416 TO 419 504 TO 509 PRIS YD 0.23 ZD 0.23
2 3 57 69 174 186 291 303 408 420 536 TO 540 568 TO 572 PRIS YD 0.8 ZD 0.8
34 35 58 63 175 180 292 297 409 414 PRIS YD 0.4 ZD 0.7
1 4 7 TO 18 20 TO 26 520 522 541 547 549 558 PRIS YD 0.6 ZD 0.3
42 TO 48 56 149 TO 152 159 TO 165 173 266 TO 269 276 TO 282 290 383 TO 386 -
393 TO 399 407 500 TO 503 512 TO 518 PRIS YD 0.4 ZD 0.23
76 78 79 81 83 90 92 94 96 98 TO 148 193 195 196 198 200 207 209 211 213 215 -
216 TO 265 310 312 313 315 317 324 326 328 330 332 TO 382 427 429 430 432 -
434 441 443 445 447 449 TO 499 574 576 578 580 582 584 586 588 590 592 594 -
596 598 600 602 604 606 608 610 612 614 616 618 620 622 624 626 628 630 632 -
634 636 638 640 642 644 646 648 650 652 665 TO 668 670 672 674 676 678 680 -
682 684 697 TO 700 702 704 706 708 710 712 714 716 738 740 742 744 746 748 -
750 752 754 756 758 760 762 764 766 768 770 772 774 776 TO 1076 1098 1100 -
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1132 1134 1136 TO 1436 PRIS YD 0.55 ZD 1.4 YB 0.43 ZB 0.3
36 TO 41 49 TO 55 70 TO 75 77 80 82 84 TO 89 91 93 95 97 153 TO 158 -
166 TO 172 187 TO 192 194 197 199 201 TO 206 208 210 212 214 270 TO 275 283 -
284 TO 289 304 TO 309 311 314 316 318 TO 323 325 327 329 331 387 TO 392 400 -
401 TO 406 421 TO 426 428 431 433 435 TO 440 442 444 446 448 523 TO 530 542 -
543 TO 545 550 TO 557 559 TO 562 573 575 577 579 581 583 585 587 589 591 593 -
595 597 599 601 603 605 607 609 611 613 615 617 619 621 623 625 627 629 631 -
633 635 637 639 641 643 645 647 649 651 653 TO 664 669 671 673 675 677 679 -
681 683 685 TO 696 701 703 705 707 709 711 713 715 717 TO 737 739 741 743 -
745 747 749 751 753 755 757 759 761 763 765 767 769 771 773 775 1077 TO 1097 -
1099 1101 1103 1105 1107 1109 1111 1113 1115 1117 1119 1121 1123 1125 1127 -
1129 1131 1133 1135 PRIS AX 0.435 IX 0.0164 IY 0.0001 IZ 0.0316
MEMBER PROPERTY AMERICAN
5 6 59 62 176 179 293 296 410 413 531 TO 535 563 TO 567 PRIS YD 0.55 ZD 0.85
CONSTANTS
MATERIAL CONCRETE ALL
SUPPORTS
1 2 5 6 20 TO 28 279 281 291 293 FIXED
DEFINE WIND LOAD
TYPE 1
INT 1.73 1.73 1.73 1.73 1.73 HEIG  3.3  6.6  9.9  13.2  16.5
EXP 1 JOINT 292 294 TO 322 343 TO 354
DEFINE 1893 LOAD
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JOINT WEIGHT
1 WEIGHT 28.274
2 WEIGHT 28.274
3 WEIGHT 234.033
4 WEIGHT 234.033
5 WEIGHT 20.653
6 WEIGHT 20.653
7 WEIGHT 174.457
8 WEIGHT 174.457
11 WEIGHT 67.366
12 WEIGHT 72.7
13 WEIGHT 67.366
14 WEIGHT 46.1
15 WEIGHT 17.39
16 WEIGHT 46.1
17 WEIGHT 192.94
18 WEIGHT 192.94
19 WEIGHT 132.958
20 WEIGHT 2.337
21 WEIGHT 2.337
22 WEIGHT 2.337
23 WEIGHT 2.337
24 WEIGHT 2.337
25 WEIGHT 2.337
26 WEIGHT 2.337
27 WEIGHT 9.277
28 WEIGHT 9.277
29 WEIGHT 868.762
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31 WEIGHT 291.863
32 WEIGHT 291.863
35 WEIGHT 9.493
36 WEIGHT 52.208
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38 WEIGHT -108.533
39 WEIGHT 312.17
40 WEIGHT -108.533
41 WEIGHT 1170.56
42 WEIGHT 1170.56
43 WEIGHT 52.42
90 WEIGHT 870.451
91 WEIGHT 870.451
92 WEIGHT 289.829
93 WEIGHT 289.829
96 WEIGHT 10.994
97 WEIGHT 52.316
98 WEIGHT 10.994
99 WEIGHT -101.018
100 WEIGHT 306.482
101 WEIGHT -101.018
102 WEIGHT 1154.33
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104 WEIGHT 63.701
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157 WEIGHT 11.869
158 WEIGHT 52.549
159 WEIGHT 11.869
160 WEIGHT -97.903
161 WEIGHT 303.566
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165 WEIGHT 67.356
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214 WEIGHT 256.043
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218 WEIGHT 3.105
219 WEIGHT 50.038
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221 WEIGHT -129.184
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224 WEIGHT 1220.78
225 WEIGHT 1220.78
226 WEIGHT 21.839
273 WEIGHT 13.364
274 WEIGHT 23.328
275 WEIGHT 13.364
276 WEIGHT 11.069
277 WEIGHT 21.517
278 WEIGHT 11.069
279 WEIGHT 28.274
280 WEIGHT 130.101
281 WEIGHT 20.653
282 WEIGHT 111.323
283 WEIGHT 582.127
284 WEIGHT 570.514
285 WEIGHT 583.651
286 WEIGHT 573.691
287 WEIGHT 585.201
288 WEIGHT 575.567
289 WEIGHT 534.975
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292 WEIGHT 130.101
293 WEIGHT 20.653
294 WEIGHT 111.323
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296 WEIGHT 570.514
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298 WEIGHT 573.691
299 WEIGHT 585.201
300 WEIGHT 575.567
301 WEIGHT 534.975
302 WEIGHT 529.936
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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 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
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4 7 TO 10 12 13 15 17 18 20 25 UNI GY -15
43 44 46 48 160 161 163 165 277 278 280 282 394 395 397 399 UNI GY -11
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 0 1.4  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
LOAD 10 LOADTYPE None  TITLE STAIR LOAD
MEMBER LOAD
37 40 41 55 56 154 157 158 172 173 271 274 275 289 290 388 391 392 406 407 -
677 679 681 683 709 711 713 715 UNI GY -7.5
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 1 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+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 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-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)
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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
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STAAD INPUT FILE

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START JOB INFORMATION
ENGINEER DATE 24-Dec-09
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
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6 10.69 -1.5 8.36; 7 0 1 8.36; 8 10.69 1 8.36; 9 0 -1.5 14.16;
10 10.69 -1.5 14.16; 11 0 1 14.16; 12 10.69 1 14.16; 13 0 -1.5 24.96;
14 10.69 -1.5 24.96; 15 0 1 24.96; 16 10.69 1 24.96; 17 5.345 1 0;
18 5.345 1 8.36; 19 5.345 1 14.16; 20 5.345 1 24.96; 27 5.345 -1.5 0;
28 5.345 -1.5 8.36; 29 5.345 -1.5 14.16; 31 0 4.6 0; 32 10.69 4.6 0;
33 0 4.6 8.36; 34 10.69 4.6 8.36; 35 0 4.6 14.16; 36 10.69 4.6 14.16;
37 0 4.6 24.96; 38 10.69 4.6 24.96; 39 5.345 4.6 0; 40 5.345 4.6 8.36;
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58 0 8.2 0; 59 10.69 8.2 0; 60 0 8.2 8.36; 61 10.69 8.2 8.36; 62 0 8.2 14.16;
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166 10.69 -1.5 35.76; 167 10.69 1 35.76; 169 0 4.6 35.76; 170 5.345 4.6 35.76;
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E 2.17185e+007
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 0.6
1 10 13 TO 16 19 22 TO 24 265 TO 267 320 323 345 TO 347 PRIS YD 0.6 ZD 0.23
36 38 TO 41 44 46 47 67 TO 75 78 79 82 87 88 93 95 TO 98 101 103 104 -
124 TO 132 135 136 139 144 145 150 152 TO 155 158 160 161 181 TO 189 192 -
193 196 201 202 207 209 TO 212 215 217 218 238 TO 246 249 250 253 258 259 -
268 TO 319 325 326 330 331 335 336 340 341 348 TO 399 PRIS YD 0.5 ZD 0.2
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
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DEFINE 1893 LOAD
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JOINT WEIGHT
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2 WEIGHT 7.952
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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
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1893 LOAD Z 1
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WIND LOAD Z 1 TYPE 1 XR 0 25.2 YR 3.3 16.5 OPEN
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WIND LOAD -X -1 TYPE 1 XR 0 32.4 YR 3.3 16.5 OPEN
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WIND LOAD -Z -1 TYPE 1 XR 0 25.2 YR 3.3 16.5 OPEN
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SELFWEIGHT Y -1
LOAD 8 LOADTYPE None  TITLE WALL
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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
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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
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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
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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 1 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+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 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-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
```

Footing

Design Summary For Multi Level Parking

Footing No	Axial Load	Length	Breadth	d	D	L	Reinforcement	
F2	Pu = 3200	2900	3400	600	1340	200	20 @ 100 c/c	20 @ 100 c/c
F3	Pu = 3000	3000	3300	700	1400	200	20 @ 100 c/c	20 @ 100 c/c
F4	Pu = 1200	2000	2100	300	660	200	16 @ 120 c/c	16 @ 120 c/c
F5	Pu = 2000	2500	2700	400	940	200	16 @ 100 c/c	16 @ 100 c/c
F6	Pu = 3220	3000	3500	600	1300	200	20 @ 100 c/c	20 @ 100 c/c
F7	Pu = 9860	5700	5700	1300	2600	200	25 @ 100 c/c	25 @ 100 c/c

Combined Footing

Footing No	Axial Load	Length	Breadth	d	D	L	Reinforcement	
CF1	Pu = 4000	3900	5200	1100	2200	-	16 @ 100 c/c	16 @ 100 c/c
CF2	Pu = 9620	5100	6300	1300	2600	-	25 @ 100 c/c	25 @ 100 c/c
CF3	Pu = 2400	2700	3000	400	870	-	16 @ 100 c/c	16 @ 100 c/c
CF4	Pu = 5400, 2100	7000	4500	-	700	-	12 @ 150 c/c	12 @ 150 c/c
CF5	Pu = 9600, 6400	7000	7000		1100	-	12 @ 150 c/c	12 @ 150 c/c
CF6	Pu = 6800, 10200	10500	9000		1000		12 @ 150 c/c	12 @ 150 c/c
CF7	Pu = 9860, 3000	8000	5700		900		12 @ 150 c/c	12 @ 150 c/c
CF8	Pu = 9600, 6400	7000	7000		1600		12 @ 150 c/c	12 @ 150 c/c
CF9	Pu = 9860	6500	7800		900		12 @ 150 c/c	12 @ 150 c/c
CF10	Pu = 9860	8000	6400		900		12 @ 150 c/c	12 @ 150 c/c

Columns

Column No	Load Case	Axial Force	Moment X	Moment Y	Size of column	Reinforcement	Stirrups
C1	1.5(DL+EQX)	374.77	223.8	140.15	230 x 230	6 of 20	8 @ 200 c/c
C1a	1.5(DL+EQX)	1307.33	340.03	769.88	400 x 800	12 of 25	8 @ 200 c/c
C2	1.5(DL+EQX)	925	334	228.15	350 x 800	12 of 25	8 @ 200 c/c
C3	1.2(DL+LL-EQX)	366.32	169.29	353.51	300 x 600	12 of 25	8 @ 200 c/c
C4	1.5(DL-EQX)	226.3	138.53	192.01	350 x 500	12 of 25	8 @ 200 c/c
C5	1.2(DL+LL-EQX)	830.8	59.63	224.86	350 x 600	10 of 25	8 @ 200 c/c
C6	1.5(DL-EQX)	560.34	563.33	210.34	400 x 900	16 of 25	8 @ 200 c/c
C7	1.5(DL-EQX)	4861.814	1817.69	0.23	800 x 800	30 of 32	8 @ 200 c/c
C8	1.5(DL-EQX)	895	324	240.15	300 x 800	12 of 25	8 @ 200 c/c
C9	1.5(DL+EQX)	717.13	112.72	60.37	300 x 400	8 of 25	8 @ 200 c/c
C3	1.2(DL+LL-EQX)	366.32	169.29	353.51	300 x 600	12 of 25	8 @ 200 c/c
C10	1.5(DL-EQX)	3015.88	918.44	2	550 x 850	22 of 25	8 @ 200 c/c

Plinth Beams

Beam No	Load Case	Moments		Shear		B	D	Ast Top	Ast Bottom	Shear Rft
		Support	Midspan	Support	Midspan					
PB1	1.5(DL-EQX)	380	127	152	151	250	600	6 of 25	4 of 25	8 @ 200

SLAB

	Thickness	Rft
GF,FF,SF,TF	120	12 @ 150 c/c

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

Beam No	Load Case	Moments			Shear		B	D	Ast Top	Ast Bottom	Shear Rft	Legs
		Support	Midspan	Torsion	Support	Midspan						
B-1	1.5(DL+EQX)	1053	331	150	233	218	400	900	12 of 25	5 of 25	12 @ 150	4
B-2	1.5(DL-EQX)	2065	855	125	908	884	800	900	18 of 25	8 of 25	12 @ 150	4
B-2A	1.5(DL-EQX)	1428	746	163	199	184	550	900	14 of 25	8 of 25	12 @ 150	2
B-3	1.5(DL-EQZ)	672	303	11	60	47	300	900	6 of 25	5 of 25	8 @ 200	4
B-4	1.5(DL-EQZ)	706	314	20	282	259	350	900	6 of 25	5 of 25	12 @ 150	2
B-5	1.5(DL-EQZ)						350	900	6 of 25	4 of 25	12 @ 150	2
B-6	1.5(DL+EQZ)	137	58	12	60	51	250	550	4 of 20	2of 20	8 @ 200	2
B-7	1.5(DL+EQX)	196	102	5	25	18	250	550	6 of 20	3 of 20	8 @ 200	2

SLAB Ramp

	Thickness	Rft
GF,FF,SF,TF	150	12 @ 150 c/c

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

Beam No	Load Case	Moments			Shear		B	D	Ast Top	Ast Bottom	Shear Rft	Legs
		Support	Midspan	Torsion	Support	Midspan						
RB1	1.5(DL+EQX)	235	150	0	125	120	300	500	4 of 25	2 of 25+1 of 20	12 @ 150	2
RB2	1.5(DL-EQX)	325	190	0	70	165	300	400	2 of 25 + 1 of 20	3 of 25+1 of 20	12 @ 150	2
RB3	1.5(DL-EQX)	803	360	0	377	270	300	850	8 of 25	3 of 25	12 @ 150	2
RB4	1.5(DL-EQZ)	283	120	0	90	139	300	600	4 of 25	3 of 25	8 @ 200	2

DESIGN OF FOOTING

DESIGN OF ISOLATED FOOTING F2

Design Parameters

Maximum factored axial load coming on footing =	**3200**	kN
Safe Bearing capacity of the soil =	**225**	kN/ m^2
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)	**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 =	**800**

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	m^2
	L	=	3.39	m
	B	=	2.89	m

Provide footing of size 3.4 m x 2.9 m

Projection beyond Column Faces	=	1.29	m
Net Upward Pressure on the foundation	=	306.812	kN/m^2
B.M @ Section XX = Mx	=	869.30	kNm
Factored Moment = Mux	=	1303.95	kNm
Equating Mu,lim to Mux = 0.138fckbd^2 = Mux			
$M_{u,lim}$	=	3312 d^2	
		627	mm
B.M @ Section YY = My	=	741	kNm
Factored Moment = Muy	=	1111	kNm
Equating Mu,lim to Muy = 0.138fckbd^2 = Muy			
$M_{u,lim}$	=	1242 d^2	
		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

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

Steel Reqd for Longer Direction

	Muy / bd^2	=	2.334	
	% 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 Reqd for Shorter Direction

	Mux / bd^2	=	0.995	
	% of steel	=	0.287 %	
Area of steel required		=	2940	mm^2
Reinforcment Reqd 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	kN
Factored Shear	Vu	=	21	kN
Overall depth of the critical section	D'	=	608	mm
Effective deph of the critical section	d'	=	548	mm
Breadth of the footing @ tp @this critical section	b'	=	3360	mm
Nominal shear stress	τ_{υ}	=	0.01	N/mm^2
Percentage of steel provided		=	0.15	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > 0.01 N/mm2

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/ m^2
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)	**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	m^2
	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/m^2
B.M @ Section XX = Mx	=	904.04	kNm
Factored Moment = Mux	=	1356.06	kNm
Equating Mu,lim to Mux = 0.138fckbd^2 = Mux			
$M_{u,lim}$	=	2484 d^2	
		739	mm
B.M @ Section YY = My	=	821	kNm
Factored Moment = Muy	=	1232	kNm
Equating Mu,lim to Muy = 0.138fckbd^2 = Muy			
$M_{u,lim}$	=	1242 d^2	
		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 Reqd for Longer Direction

Muy / bd^2	=	2.357	
% of steel	=	0.726 %	
Area of steel required	=	2875	mm^2

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

Steel Reqd for Shorter Direction

Mux / bd^2	=	1.259	
% of steel	=	0.367 %	
Area of steel required	=	2954	mm^2
Reinforcment Reqd for central band of 3.08 m	=	2238	mm^2

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	kN
Factored Shear	Vu	=	0	kN
Overall depth of the critical section	D'	=	700	mm
Effective deph of the critical section	d'	=	640	mm
Breadth of the footing @ tp @this critical section	b'	=	3280	mm
Nominal shear stress	τ_{υ}	=	0.00	N/mm^2
Percentage of steel provided		=	0.15	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > 0.00 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/ m^2
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)	**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	m^2
	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/m^2
B.M @ Section XX = Mx	=	190.29	kNm
Factored Moment = Mux	=	285.44	kNm
Equating Mu,lim to Mux = 0.138fckbd^2 = Mux			
$M_{u,lim}$	=	2070 d^2	
		371	mm
B.M @ Section YY = My	=	176	kNm
Factored Moment = Muy	=	265	kNm
Equating Mu,lim to Muy = 0.138fckbd^2 = Muy			
$M_{u,lim}$	=	1449 d^2	
		427	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 = 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 Reqd for Longer Direction

	Muy / bd^2	=	2.202	
	% 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 Reqd for Shorter Direction

	Mux / bd^2	=	1.575	
	% of steel	=	0.467 %	
Area of steel required		=	1405	mm^2
Reinforcment Reqd 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	kN
Factored Shear	Vu	=	166	kN
Overall depth of the critical section	D'	=	381	mm
Effective deph of the critical section	d'	=	323	mm
Breadth of the footing @ tp @this critical section	b'	=	1704	mm
Nominal shear stress	τ_υ	=	0.30	N/mm^2
Percentage of steel provided		=	0.33	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > 0.30 N/mm2

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/ m^2
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)	**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	m^2
	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/m^2
B.M @ Section XX = Mx	=	445.42	kNm
Factored Moment = Mux	=	668.13	kNm
Equating Mu,lim to Mux = 0.138fckbd^2 = Mux			
$M_{u,lim}$	=	2484 d^2	
		519	mm
B.M @ Section YY = My	=	404	kNm
Factored Moment = Muy	=	606	kNm
Equating Mu,lim to Muy = 0.138fckbd^2 = Muy			
$M_{u,lim}$	=	1449 d^2	
		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 Reqd for Longer Direction

	Muy / bd^2	=	2.308	
	% of steel	=	0.709 %	
Area of steel required		=	2149	mm^2

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

Steel Reqd for Shorter Direction

	Mux / bd^2	=	1.431	
	% of steel	=	0.421 %	
Area of steel required		=	2229	mm^2
Reinforcment Reqd for central band of 2.48 m		=	1905	mm^2

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	kN
Factored Shear	Vu	=	196	kN
Overall depth of the critical section	D'	=	482	mm
Effective deph of the critical section	d'	=	424	mm
Breadth of the footing @ tp @this critical section	b'	=	2364	mm
Nominal shear stress	τ_{υ}	=	0.20	N/mm^2
Percentage of steel provided		=	0.24	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > 0.20 N/mm2

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/ m^2
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)	**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 = **400**

Depth of the column (mm) D = **900**

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 =	2361.34	/	**225**	
		=	10.495	m^2
	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/m^2
B.M @ Section XX = Mx	=	906.66	kNm
Factored Moment = Mux	=	1359.99	kNm

Equating Mu,lim to Mux = 0.138fckbd^2 = Mux

$M_{u,lim}$ = 3726 d^2

604 mm

B.M @ Section YY = My	=	777	kNm
Factored Moment = Muy	=	1166	kNm

Equating Mu,lim to Muy = 0.138fckbd^2 = Muy

$M_{u,lim}$ = 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 Reqd for Longer Direction

Muy / bd^2	=	1.958	
% of steel	=	0.591 %	
Area of steel required	=	2883	mm^2

Provide 10 bars of 20 mm dia
Spacing of 20 mm dia bars 108 mm c/c

Steel Reqd for Shorter Direction

Mux / bd^2	=	0.983	
% of steel	=	0.283 %	
Area of steel required	=	3163	mm^2
Reinforcment Reqd 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	kN
Factored Shear	Vu	=	96	kN
Overall depth of the critical section	D'	=	632	mm
Effective deph of the critical section	d'	=	572	mm
Breadth of the footing @ tp @this critical section	b'	=	3380	mm
Nominal shear stress	τ_υ	=	0.05	N/mm^2
Percentage of steel provided		=	0.16	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > 0.05 N/mm2

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/ m^2
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)	**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 = 7230.67	/	**225**	
	=	32.137	m^2
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/m^2
B.M @ Section XX = Mx	=	5154.13	kNm
Factored Moment = Mux	=	7731.20	kNm

Equating Mu,lim to Mux = 0.138fckbd^2 = Mux

$M_{u,lim}$ = 3312 d^2

1528 mm

B.M @ Section YY = My	=	5154	kNm
Factored Moment = Muy	=	7731	kNm

Equating Mu,lim to Muy = 0.138fckbd^2 = Muy

$M_{u,lim}$ = 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

Muy / bd^2 = 1.531
% of steel = 0.453 %

Area of steel required = 9096 mm^2
Provide 19 bars of 25 mm dia
Spacing of 25 mm dia bars 100 mm c/c

Steel Reqd for Shorter Direction

Mux / bd^2 = 1.501
% of steel = 0.443 %

Area of steel required = 8994 mm^2

Reinforcment 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	kN
Factored Shear	Vu	=	-269	kN
Overall depth of the critical section	D'	=	1245	mm
Effective deph of the critical section	d'	=	1182	mm
Breadth of the footing @ tp @this critical section	b'	=	5875	mm
Nominal shear stress	τ_v	=	-0.04	N/mm^2
Percentage of steel provided		=	0.08	%
Permissible punching shear stress		=	0.25 x sqrt(fck)	

1.37 N/mm^2 > -0.04 N/mm2

Provided Section is adequate.

Combined Footing Analysis and Design		Issue:	Design	Page
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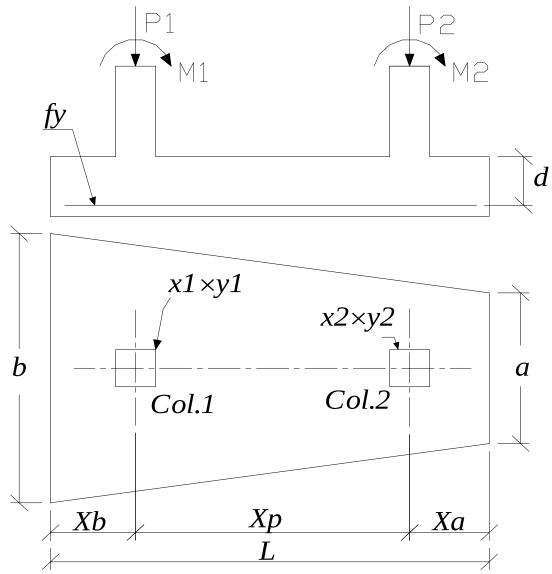
Project:	CF4	Revised by:		
		Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.8	0.35
Width, y (m)	0.8	0.8

Distance, Xp (m)	4.48	Left width, b (m)	4.5
Distance, Xb (m)	1.52	Right width, a (m)	3
Distance, Xa (m)	1	Length, L (m)	7
Eff. depth, d (m)	0.6	Area, (m2)	26.25



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	3857.14	0	0	0
P2 (kN)	2285.71	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

Checkings:

Allowable soil pressure, qa (kPa)	281.25	
Maximum soil pressure, qmax (kPa)	249.134	(qmax < qa) Ok
Minimum soil pressure, qmin (kPa)	162.55	(qmin > 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:

x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	4.5	20.241
0.7	1073.38	378.626	4.35	20.241
1.4	2096.81	1491.08	4.2	20.241
2.1	-2328.8	170.725	4.05	20.241
2.8	-1402.4	-1132.4	3.9	20.241
3.5	-523.14	-1803.6	3.75	22.1262
4.2	309.972	-1875.6	3.6	24.0316
4.9	1097.89	-1380.2	3.45	20.241
5.6	1841.57	-348.86	3.3	20.241
6.3	-658.05	227.874	3.15	20.241
7	0.00	0.00	3	20.241

Combined Footing Analysis and Design		Issue:	Design	Page
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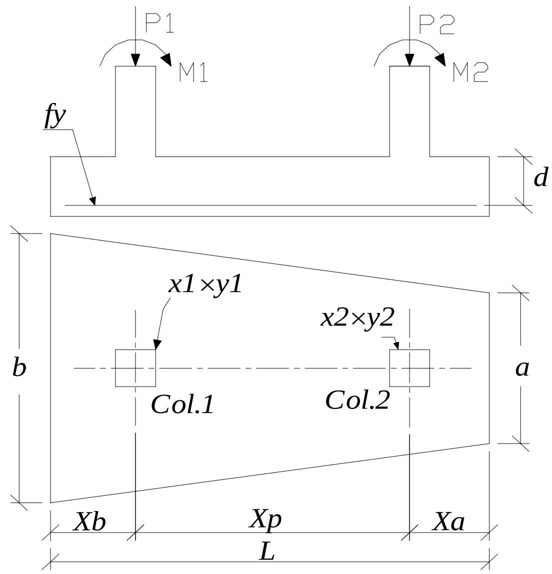
		Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.85	0.35
Width, y (m)	1.12	1.22

Distance, Xp (m)	4.48	Left width, b (m)	7
Distance, Xb (m)	1.525	Right width, a (m)	5
Distance, Xa (m)	0.995	Length, L (m)	7
Eff. depth, d (m)	1	Area, (m2)	42



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	6857.14	0	0	0
P2 (kN)	4571.43	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

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:

x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	7	33.7349
0.7	1824.61	641.485	6.8	33.7349
1.4	3599.86	2542.93	6.6	33.7349
2.1	-4274.4	149.732	6.4	33.7349
2.8	-2598.5	-2252.9	6.2	33.7349
3.5	-972.57	-3499.8	6	33.7349
4.2	603.176	-3626.2	5.8	33.7349
4.9	2128.52	-2667.2	5.6	33.7349
5.6	3603.24	-658.09	5.4	33.7349
6.3	-1372.9	477.518	5.2	33.7349
7	0.00	0.00	5	33.7349

Combined Footing Analysis and Design		Issue:	Design	Page
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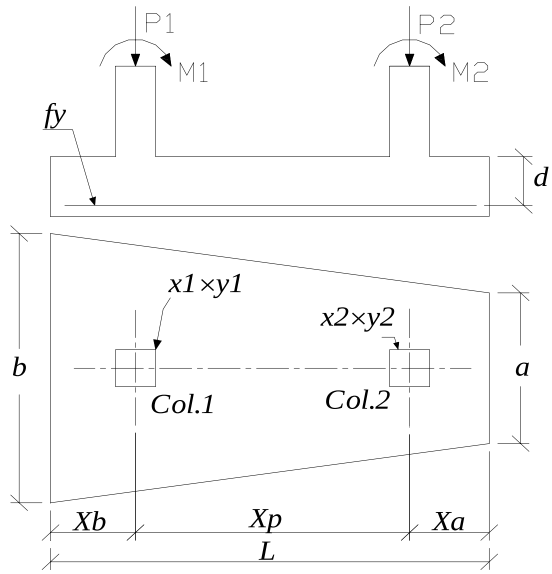
		Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.3	0.8
Width, y (m)	0.8	0.8

Distance, Xp (m)	4.555	Left width, b (m)	5
Distance, Xb (m)	3.585	Right width, a (m)	8
Distance, Xa (m)	2.36	Length, L (m)	10.5
Eff. depth, d (m)	1.2	Area, (m2)	68.25



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	4285.71	0	0	0
P2 (kN)	7714.29	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

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)	34	for bottom reinforcement
As (cm2)	40.4819	for top reinforcement

Details:

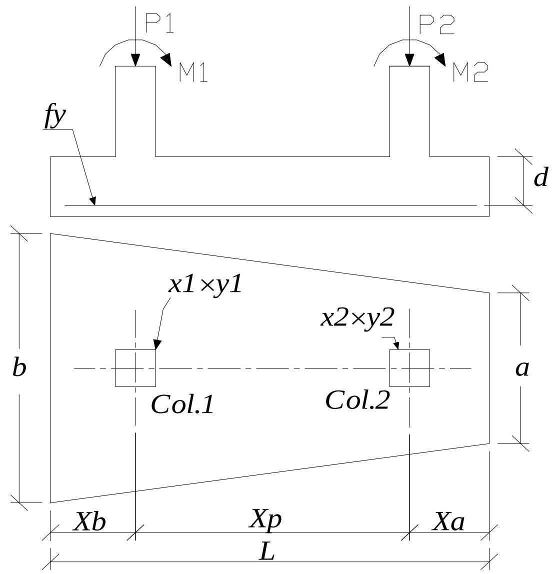
x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	5	40.4819
1.05	681.618	342.733	5.3	40.4819
2.1	1543.75	1494.58	5.6	40.4819
3.15	2601.9	3653.22	5.9	40.4819
4.2	-2128.4	3342.61	6.2	40.4819
5.25	-631.71	1872.99	6.5	40.4819
6.3	1107.53	2100.9	6.8	40.4819
7.35	3104.82	4289.12	7.1	40.4819
8.4	-5424.3	5908.75	7.4	40.4819
9.45	-2864.5	1531.17	7.7	40.4819
9.5	0.00	0.00	6.2	33.7349

Combined Footing Analysis and Design	Issue:	Design	Page
	Date:		1 of 1
Project: CF7	Revised by:		
	Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.3	0.8
Width, y (m)	0.8	0.8

Distance, Xp (m)	4.55	Left width, b (m)	4
Distance, Xb (m)	1.5	Right width, a (m)	6
Distance, Xa (m)	2.5	Length, L (m)	8.55
Eff. depth, d (m)	1.1	Area, (m2)	42.75



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	2142.86	0	0	0
P2 (kN)	7042.86	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

Checkings:

Allowable soil pressure, qa (kPa)	281.25	
Maximum soil pressure, qmax (kPa)	276	(qmax < qa) Ok
Minimum soil pressure, qmin (kPa)	108.756	(qmin > 0) Ok

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	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)	37.5	for bottom reinforcement
As (cm2)	37.1084	for top reinforcement

Details:

x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	4	37.1084
0.855	744.063	310.921	4.2	37.1084
1.71	-1408.2	671.903	4.4	37.1084
2.565	-450.42	-130.7	4.6	37.1084
3.42	623.572	-65.189	4.8	37.1084
4.275	1820.09	970.521	5	37.1084
5.13	3145.4	3083.87	5.2	37.1084
5.985	4605.77	6387.65	5.4	37.1084
6.84	-3652.5	3210.62	5.6	37.1084
7.695	-1903.2	824.806	5.8	37.1084
8.55	0.00	0.00	6	37.1084

Combined Footing Analysis and Design		Issue:	Design	Page
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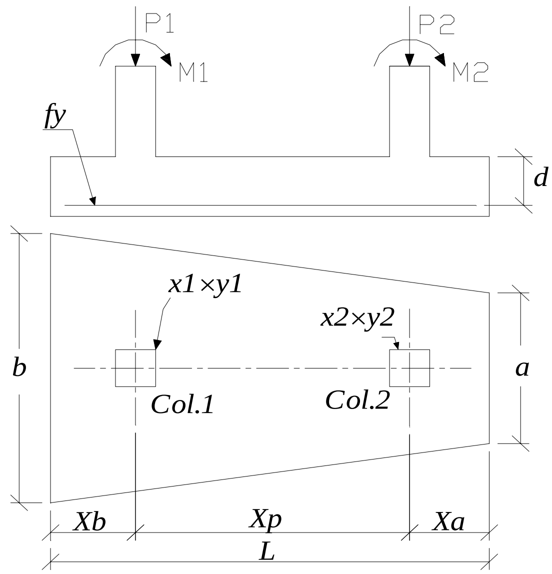
Project:	CF8	Revised by:		
		Checked by:		



Dimensions:

	Col.1	Col.2
Length, x (m)	0.35	0.85
Width, y (m)	1.22	1.12

Distance, Xp (m)	4.55	Left width, b (m)	7
Distance, Xb (m)	1.45	Right width, a (m)	5
Distance, Xa (m)	1	Length, L (m)	7
Eff. depth, d (m)	1.5	Area, (m2)	42

Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	6857.14	0	0	0
P2 (kN)	4571.43	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

Checkings:

Allowable soil pressure, qa (kPa)	281.25	
Maximum soil pressure, qmax (kPa)	280.014	(qmax < qa) Ok
Minimum soil pressure, qmin (kPa)	197.455	(qmin > 0) Ok

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	Vw < Vc1, OK
Punching shear strength, Vc2 (kN/m width)	3999.06	Vp < Vc2, OK

Area of Steel:

Use area of steel, As (cm2) 51 for bottom reinforcement

As (cm2) 50.6024 for top reinforcement

Details:

x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	7	50.6024
0.7	1887.82	664.59	6.8	50.6024
1.4	3709.94	2627.62	6.6	50.6024
2.1	-4133	-396.68	6.4	50.6024
2.8	-2440.3	-2693.6	6.2	50.6024
3.5	-811.43	-3828	6	50.6024
4.2	754.403	-3844.3	5.8	50.6024
4.9	2257.81	-2786.4	5.6	50.6024
5.6	3699.43	-697.82	5.4	50.6024
6.3	-1320.1	458.509	5.2	50.6024
7	0.00	0.00	5	50.6024

Combined Footing Analysis and Design		Issue:	Design	Page
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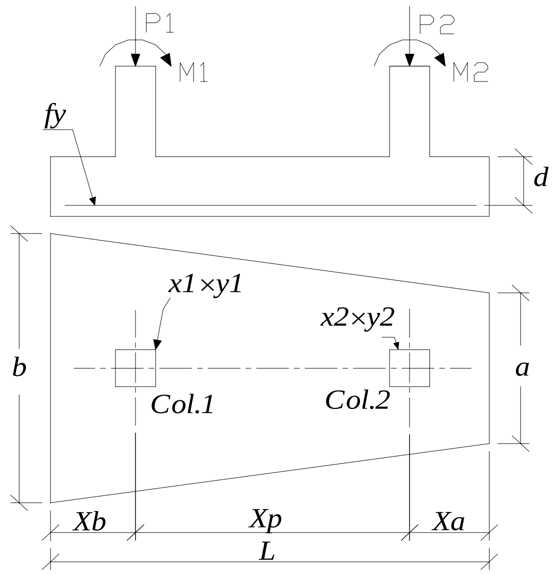
Project:	CF9	Revised by:		
		Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.8	0.8
Width, y (m)	0.8	0.8

Distance, Xp (m)	5.8	Left width, b (m)	6.5
Distance, Xb (m)	1	Right width, a (m)	6.5
Distance, Xa (m)	1	Length, L (m)	7.8
Eff. depth, d (m)	1.2	Area, (m2)	50.7



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	7042.86	0	0	0
P2 (kN)	7042.86	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

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:

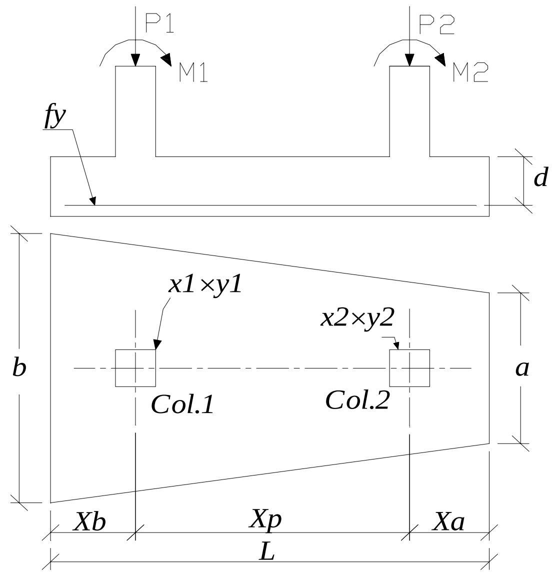
x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	6.5	40.4819
0.78	1972	769.08	6.5	40.4819
1.56	-5916	-2445.3	6.5	40.4819
2.34	-3944	-6290.7	6.5	40.4819
3.12	-1972	-8597.9	6.5	40.4819
3.9	0	-9367	6.5	40.4819
4.68	1972	-8597.9	6.5	40.4819
5.46	3944	-6290.7	6.5	40.4819
6.24	5916	-2445.3	6.5	40.4819
7.02	-1972	769.078	6.5	40.4819
9.8	0.00	0.00	6.6	26.988

Combined Footing Analysis and Design		Issue:	Design	Page
		Date:		1 of 1
Project:	CF10	Revised by:		
		Checked by:		

Dimensions:

	Col.1	Col.2
Length, x (m)	0.8	0.8
Width, y (m)	0.8	0.8

Distance, Xp (m)	4.555	Left width, b (m)	6.4
Distance, Xb (m)	1.722	Right width, a (m)	6.4
Distance, Xa (m)	1.722	Length, L (m)	8
Eff. depth, d (m)	1.15	Area, (m2)	51.2



Material Properties:

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

Loads:

	Working loads			
Col. Load	Dead	Live	Wind	Seismic
P1 (kN)	7042.86	0	0	0
P2 (kN)	7042.86	0	0	0
M1 (kN.m)	0	0	0	0
M2 (kN.m)	0	0	0	0

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:

x	V, kN	M, kN-m	b, m	As, cm2/m
0	0	0	6.4	38.7952
0.8	1972.67	789.076	6.4	38.7952
1.6	3945.18	3156.23	6.4	38.7952
2.4	-3942.4	416.25	6.4	38.7952
3.2	-1970.2	-1948.8	6.4	38.7952
4	1.84961	-2736.1	6.4	38.7952
4.8	1973.78	-1945.9	6.4	38.7952
5.6	3945.55	421.859	6.4	38.7952
6.4	-3942.8	3154.17	6.4	38.7952
7.2	-1971.3	788.528	6.4	38.7952
9.8	0.00	0.01	6.6	26.988

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**		

Dimesions of the Column

Unsupported length of column, L	=	3600	mm	
Least lateral dimension	=	230	mm	
Breadth of the column B (mm)	=	230		
Depth of the Column D (mm)	=	230		
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 , λ_{ex}	=	10.17	<12	column is Short
Slenderness ratio , λ_{ey}	=	10.17	<12	column is Short

Design Factors

Factored load, Pu	=	275	KN
Factored moment acting parallel to the larger dimension , M_{ux}	=	25	KN-m
Factored moment acting parallel to the shorter dimension, M_{uy}	=	3	KN-m

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

M_{ux}/p_u	=	90.91	mm
M_{uy}/p_u	=	327.27	mm

Both are more than 20mm minimum

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

$$d'/D = 0.2$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.07$$

$$\frac{P_u}{f_{ck} \times b \times D} = 0.17$$

From SP16 chart44

$\frac{P}{f_{ck}}$	=	**0.06**	From table
Assuming a higher value P/fck	=	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}

About X-axis

d'/D	=	0.22	
P/fck x b x D^2	=	0.17	
P/fck	=	0.09	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.13**	From table
M_{x1}	=	47.45	KN-m

About Y-axis

d'/D	=	0.22	
P/fck x b x D^2	=	0.17	
P/fck	=	0.09	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.13**	From table
M_{y1}	=	47.45	KN-m

4 Calculate α^n

$P_z = 0.45 f_{ck} A_c + 0.75 f_y A_s$

Pz	=	1301	KN
P/P_z	=	0.21	

By formula

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

α^n	=	1.02	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =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/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**		

Dimesions of the Column

Unsupported length of column, L	=	3600	mm	
Least lateral dimension	=	400	mm	
Breadth of the column B (mm)	=	400		
Depth of the Column D (mm)	=	700		
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 , λ_{ex}	=	5.85	<12	column is Short
Slenderness ratio , λ_{ey}	=	3.34	<12	column is Short

Design Factors

Factored load, Pu	=	1307.33	KN
Factored moment acting parallel to the larger dimension , M_{ux}	=	340.03	KN-m
Factored moment acting parallel to the shorter dimension, M_{uy}	=	769.88	KN-m

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

M_{ux}/p_u	=	260.10	mm
M_{uy}/p_u	=	68.84	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

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.06$$

$\frac{P_u}{f_{ck} \times b \times D}$	=	0.16	
From SP16 chart44			
$\frac{P}{f_{ck}}$	=	**0.05**	From table
Assuming a higher value P/fck	=	0.075	
Assumed , P	=	2.25	per cent
Area of steel, A_s	=	**6300.00**	mm^2
Use **16** no.s of **25** mm			
Area of steel provided	=	7850	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.07	
P/fck x b x D^2	=	0.16	
P/fck	=	0.075	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.12**	From table
M_{x1}	=	705.60	KN-m
About Y-axis			
d'/D	=	0.13	
P/fck x b x D^2	=	0.16	
P/fck	=	0.075	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.12**	From table
M_{y1}	=	403.20	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	6223	KN
P/P_z	=	0.21	

By formula

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

α^n	=	1.02	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.6903	< or = 1

Hence the column is safe

Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO C2

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**

Dimesions of the Column

Unsupported length of column, L	=	3750	mm
Least lateral dimension	=	350	mm
Breadth of the column B (mm)	=	350	
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

Slenderness ratio , λ_{ex}	=	7.80	<12	column is Short
Slenderness ratio , λ_{ey}	=	2.93	<12	column is Short

Design Factors

Factored load, Pu	=	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

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.06$$

$\dfrac{P_u}{f_{ck} \times b \times D}$	=	0.12

From SP16 chart44

$\dfrac{P}{f_{ck}}$	=	**0.05**	From table
Assuming a higher value P/fck	=	0.075	
Assumed , P	=	2.25	per cent
Area of steel, A_s	=	**5400.00**	mm^2

Use **12** no.s of **25** mm

Area of steel provided	=	5888	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis

d'/D	=	0.06	
P/fck x b x D^2	=	0.12	
P/fck	=	0.075	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.12**	From table
M_{x1}	=	691.20	KN-m

About Y-axis

d'/D	=	0.17	
P/fck x b x D^2	=	0.12	
P/fck	=	0.075	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.12**	From table
M_{y1}	=	269.20	KN-m

4 Calculate α^n

$P_z = 0.45 f_{ck} A_c + 0.75 f_y A_s$

Pz	=	5072	KN
P/P_z	=	0.18	

By formula

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

α^n	=	0.97

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.8613	< or = 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} (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)	**60**

Dimesions 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)	=	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 , λ_{ex}	=	7.80	<12	column is Short
Slenderness ratio , λ_{ey}	=	3.90	<12	column is Short

Design Factors

Factored load, Pu	=	366.32	KN
Factored moment acting parallel to the larger dimension , M_{ux}	=	162.29	KN-m
Factored moment acting parallel to the shorter dimension, M_{uy}	=	353.51	KN-m

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

M_{ux}/p_u	=	443.03	mm
M_{uy}/p_u	=	245.69	mm

Both are more than 20mm minimum

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

$$d'/D = 0.1$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.05$$

$\dfrac{P_u}{f_{ck} \times b \times D}$	=	0.07	
From SP16 chart44			
$\dfrac{P}{f_{ck}}$	=	**0.07**	From table
Assuming a higher value P/fck	=	0.105	
Assumed , P	=	3.15	per cent
Area of steel, A_s	=	**5670.00**	mm^2
Use **12** no.s of **25** mm			
Area of steel provided	=	5888	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.10	
P/fck x b x D^2	=	0.07	
P/fck	=	0.105	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.125**	From table
M_{x1}	=	405.00	KN-m
About Y-axis			
d'/D	=	0.20	
P/fck x b x D^2	=	0.07	
P/fck	=	0.105	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.125**	From table
M_{y1}	=	202.50	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	4262	KN
P/P_z	=	0.09	

By formula

α^n= 2/3[1+5/2 x P/Pz]

α^n	=	0.81	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.9919	< or = 1

Hence the column is safe

Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO	**C4**			
Load Case	1.2*(DL + LL - 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)		**60**		

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

Check for Slenderness ratio, L/D

Slenderness ratio , λ_{ex}	=	6.69	<12	column is Short
Slenderness ratio , λ_{ey}	=	4.68	<12	column is Short

Design Factors

Factored load, Pu	=	226.3	KN
Factored moment acting parallel to the larger dimension , M_{ux}	=	138.53	KN-m
Factored moment acting parallel to the shorter dimension, M_{uy}	=	192.01	KN-m

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

M_{ux}/p_u	=	612.15	mm
M_{uy}/p_u	=	397.70	mm

Both are more than 20mm minimum

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

$$d'/D = 0.1$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.05$$

$\dfrac{P_u}{f_{ck} \times b \times D}$	=	0.04	
From SP16 chart44			
$\dfrac{P}{f_{ck}}$	=	**0.07**	From table
Assuming a higher value P/fck	=	0.105	
Assumed , P	=	3.15	per cent
Area of steel, A_s	=	**5512.50**	mm^2
Use **12** no.s of **25** mm			
Area of steel provided	=	5888	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.12	
P/fck x b x D^2	=	0.04	
P/fck	=	0.06	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.13**	From table
M_{x1}	=	341.25	KN-m
About Y-axis			
d'/D	=	0.17	
P/fck x b x D^2	=	0.04	
P/fck	=	0.105	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.13**	From table
M_{y1}	=	238.88	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	4195	KN
P/P_z	=	0.05	
By formula			
α^n= 2/3[1+5/2 x P/Pz]			
α^n	=	0.76	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < 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 , 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**

Dimesions 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 , λ_{ex}	=	6.69	<12	column is Short
Slenderness ratio , λ_{ey}	=	3.90	<12	column is Short

Design Factors

Factored load, Pu	=	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_x)

$$d'/D = 0.1$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.02$$

$\frac{P_u}{f_{ck} \times b \times D}$	=	0.13	
From SP16 chart44			
$\frac{P}{f_{ck}}$	=	**0.05**	From table
Assuming a higher value P/fck	=	0.075	
Assumed , P	=	2.25	per cent
Area of steel, A_s	=	**4725.00**	mm^2
Use **10** no.s of **25** mm			
Area of steel provided	=	4906	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.08	
P/fck x b x D^2	=	0.13	
P/fck	=	0.075	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.12**	From table
M_{x1}	=	453.60	KN-m
About Y-axis			
d'/D	=	0.14	
P/fck x b x D^2	=	0.13	
P/fck	=	0.075	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.12**	From table
M_{y1}	=	264.60	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	4362	KN
P/P_z	=	0.19	

By formula

α^n= 2/3[1+5/2 x P/Pz]

α^n	=	0.99	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.4786	< or = 1

Hence the column is safe

Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO	**C6**			
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**		

Dimesions of the Column

Unsupported length of column, L	=	3600	mm
Least lateral dimension	=	400	mm
Breadth of the column B (mm)	=	400	
Depth of the Column D (mm)	=	900	
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 , λ_{ex}	=	5.85	<12	column is Short
Slenderness ratio , λ_{ey}	=	2.60	<12	column is Short

Design Factors

Factored load, Pu	=	560.34	KN
Factored moment acting parallel to the larger dimension , M_{ux}	=	563.33	KN-m
Factored moment acting parallel to the shorter dimension, M_{uy}	=	210.34	KN-m

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

M_{ux}/p_u	=	1005.34	mm
M_{uy}/p_u	=	160.62	mm

Both are more than 20mm minimum

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

$$\frac{d'}{D} = 0.1$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.06$$

$\frac{P_u}{f_{ck} \times b \times D}$	=	0.05	
From SP16 chart44			
$\frac{P}{f_{ck}}$	=	**0.045**	From table
Assuming a higher value P/fck	=	0.0675	
Assumed , P	=	2.03	per cent
Area of steel, A_s	=	**7290.00**	mm^2
Use **16** no.s of **25** mm			
Area of steel provided	=	7850	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.06	
P/fck x b x D^2	=	0.05	
P/fck	=	0.0675	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.12**	From table
M_{x1}	=	1166.40	KN-m
About Y-axis			
d'/D	=	0.13	
P/fck x b x D^2	=	0.05	
P/fck	=	0.0675	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.12**	From table
M_{y1}	=	518.40	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	7303	KN
P/P_z	=	0.08	

By formula

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

α^n	=	0.80	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < 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/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)		**60**

Dimesions 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

Slenderness ratio , λ_{ex}	=	2.93	<12	column is Short
Slenderness ratio , λ_{ey}	=	2.93	<12	column is Short

Design Factors

Factored load, Pu	=	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

M_{ux}/p_u	=	373.87	mm
M_{uy}/p_u	=	18.51	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

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.12$$

$\frac{P_u}{f_{ck} \times b \times D}$	=	0.25	
From SP16 chart44			
$\frac{P}{f_{ck}}$	=	**0.055**	From table
Assuming a higher value P/fck	=	0.0825	
Assumed , P	=	2.48	per cent
Area of steel, A_s	=	**15840.00**	mm^2
Use **20** no.s of **32** mm			
Area of steel provided	=	16077	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.08	
P/fck x b x D^2	=	0.25	
P/fck	=	0.0825	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.13**	From table
M_{x1}	=	1996.80	KN-m
About Y-axis			
d'/D	=	0.08	
P/fck x b x D^2	=	0.25	
P/fck	=	0.0825	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.13**	From table
M_{y1}	=	1996.80	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	13644	KN
P/P_z	=	0.36	
By formula			
α^n= 2/3[1+5/2 x P/Pz]			
α^n	=	1.27	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.9075	< or = 1

Hence the column is safe

Rectangular Short Column with Biaxial bending - Bresler method

COLUMN NO C8

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**		

Dimesions 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

Slenderness ratio , λ_{ex}	=	7.80	<12	column is Short
Slenderness ratio , λ_{ey}	=	2.93	<12	column is Short

Design Factors

Factored load, Pu	=	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

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 chart44			
$\frac{P}{f_{ck}}$	=	**0.05**	From table
Assuming a higher value P/fck	=	0.075	
Assumed , P	=	2.25	per cent
Area of steel, A_s	=	**5400.00**	mm^2
Use **12** no.s of **25** mm			
Area of steel provided	=	5888	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.06	
P/fck x b x D^2	=	0.12	
P/fck	=	0.075	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.12**	From table
M_{x1}	=	691.20	KN-m
About Y-axis			
d'/D	=	0.17	
P/fck x b x D^2	=	0.12	
P/fck	=	0.075	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.12**	From table
M_{y1}	=	259.20	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	5072	KN
P/P_z	=	0.18	

By formula

α^n= 2/3[1+5/2 x P/Pz]

α^n	=	0.97	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.8413	< 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 , 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**

Dimesions 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 , λ_{ex}	=	7.80	<12	column is Short
Slenderness ratio , λ_{ey}	=	5.85	<12	column is Short

Design Factors

Factored load, Pu	=	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)

$$d'/D = 0.1$$

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.08$$

$\dfrac{P_u}{f_{ck} \times b \times D}$	=	0.20	
From SP16 chart44			
$\dfrac{P}{f_{ck}}$	=	**0.07**	From table
Assuming a higher value P/fck	=	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/fck x b x D^2	=	0.20	
P/fck	=	0.105	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.07**	From table
M_{x1}	=	100.80	KN-m
About Y-axis			
d'/D	=	0.17	
P/fck x b x D^2	=	0.20	
P/fck	=	0.105	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.07**	From table
M_{y1}	=	75.60	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	2842	KN
P/P_z	=	0.25	

By formula

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

α^n	=	1.09	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.9000	< or = 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/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**

Dimesions 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 , λ_{ex}	=	4.25	<12	column is Short
Slenderness ratio , λ_{ey}	=	2.75	<12	column is Short

Design Factors

Factored load, Pu	=	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

M_{ux}/p_u	=	304.53	mm
M_{uy}/p_u	=	29.84	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

$$\frac{M_x}{f_{ck} \times b \times D^2} = 0.08$$

$\frac{P_u}{f_{ck} \times b \times D}$	=	0.22	
From SP16 chart44			
$\frac{P}{f_{ck}}$	=	**0.05**	From table
Assuming a higher value P/fck	=	0.075	
Assumed , P	=	2.25	per cent
Area of steel, A_s	=	**10518.75**	mm^2
Use **22** no.s of **25** mm			
Area of steel provided	=	10794	mm^2

3 Find the moment capacities M_{x1} and M_{y1}

About X-axis			
d'/D	=	0.06	
P/fck x b x D^2	=	0.22	
P/fck	=	0.075	
$M_{x1}/(f_{ck} \times b \times D^2)$	=	**0.125**	From table
M_{x1}	=	1490.16	KN-m
About Y-axis			
d'/D	=	0.09	
P/fck x b x D^2	=	0.22	
P/fck	=	0.075	
$M_{y1}/(f_{ck} \times D \times b^2)$	=	**0.125**	From table
M_{y1}	=	964.22	KN-m

4 Calculate α^n

$P_z=0.45f_{ck}A_c+0.75f_yA_s$

Pz	=	9671	KN
P/P_z	=	0.31	

By formula

α^n= 2/3[1+5/2 x P/Pz]

α^n	=	1.19	

5 Criteria for biaxial bending

$(M_x/M_{x1})^{\alpha n} + (M_y/M_{y1})^{\alpha n}$ < or =1.0	=	0.6207	< 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/mm^2)	**30**
Characteristic yield strength of steel , f_y (N/mm^2)	**415**
Unit weight of concrete , γ_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)	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 Mu(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

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\, b\, d^2$

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

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

$\Rightarrow$ $p_{t,lim} =$ (41.3 x 30 / 415 x 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	kNm
$M_{u,lim} =$	326.11	kNm

$\Rightarrow$ *ection cannot be designed as singly reinforced. Doubly Reinforced Section Neede*

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 x d)

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

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **2498** mm^2

∴ No of tension bars required (#)
(2498 / (Pi / 4 x 25^2) = 6.00

Actual percentage of steel , p_t (%)
(6 x Pi / 4 x 25 ^2 / 250 / 560 x 100) = 2.11

Actual area of steel , A_{st} (mm^2)
(6 x Pi / 4 x 25 ^2) = 2945

<u>Determining A_{sc}</u>

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 559.5) = 0.072
From the above table : by interpolation

<u>Design Check</u>

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 2.11

Actual p_c provided : p_c = 0.35

$\Rightarrow$ p_c^* = (0.87 x 415 x (2.106 - 1.433) / (354.73 - 0.447 x 30))

$\Rightarrow$ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 2292 / 2945) = 187.32 N/mm^2

F = 0.93

F_1 = 0.77

$F_2 =$ 0.90

$\therefore$ $(l / d)_{max} =$ (26 x 0.93 x 0.77 x 0.9) = 16.69

$(l / d)_{provided} =$ 19.30

$\Rightarrow$ *Not O.K*

Check for shear

Shear force at critical distance , V_{ud} (kN) 152

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 , τ_v

(152 x 1000 / (250 x 560)) = 1.09 N/mm^2

The maximum shear stress is given by : $Tc \max = 0.62 fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 2945 mm^2

Percentage of steel , p_t (%) 2.11

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.86 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.86 x 250 x 560 / 1000) = 120 kN

- *Design of " vertical " stirrups*

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

$\Rightarrow$ $V_{us} =$ (152 - 120) = 32 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

$\Rightarrow$ required spacing sv ≤ (0.87 x 415 x 101 x 560 / (32 x 1000))

$\Rightarrow$ Spacing , s_v = 635 mm

Check whether $\tau_v > 0.5\ \tau_c$

Nominal shear stress , τ_v (N/mm^2) 1.09
Design shear stress , τ_c (N/mm^2) 0.86

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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
300 mm

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 250) =	363	mm
ii)	≤	(0.75 x 559.5) =	420	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 560 / (32 x 1000)) =	635	mm

Beam PB1 Mid Span

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 , γ_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)	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

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)	289	
Equivalent Bending Moment , M_e (kNm)	289	
Shear force at critical distance , V_{ud} (kN)	179	
Equivalent Shear (kN)	179	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where $b =$ Breadth of the Section

$xu_{max} =$ Limiting depth of Neutral Axis

$d =$ Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\, fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 $f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 250 x 559.5^2 / 1000000) = 326.11 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 289.00 kNm
$M_{u,lim}$ = 326.11 kNm

⇒ *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 $A_{st,lim} = p_{t,lim}$ / 100 (b x d)

  ⇒ $A_{st,lim}$ (1.433 / 100 x 250 x 559.5) = 2004 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 1725 mm^2

∴ No of tension bars required (#)
(1725 / (Pi / 4 x 25^2) = 4.00

Actual percentage of steel , p_t (%)
(4 x Pi / 4 x 25 ^2 / 250 / 560 x 100) = 1.40

Actual area of steel , A_{st} (mm^2)
(4 x Pi / 4 x 25 ^2) = 1963

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

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 559.5) = 0.072
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \left(\frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \right) \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.40

Actual p_c provided : p_c = 0.35

⇒ p_c^* = (0.87 x 415 x (1.404 - 1.433) / (354.73 - 0.447 x 30))

⇒ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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.

$$f_s = 0.58\, f_y \, \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

⇒ f_{st} = (0.58 x 415 x 1806 / 1963) = 221.45 N/mm^2

F = 1.00

F_1 = 0.87

$F_2 =$ 0.90

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.87 x 0.9) = 20.28

$(l / d)_{provided} =$ 8.87

$\Rightarrow$ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (kN) 179

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 , τ_v

(179 x 1000 / (250 x 560)) = 1.28 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\,fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1963 mm^2

Percentage of steel , p_t (%) 1.40

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.74 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.74 x 250 x 560 / 1000) = 104 kN

- *Design of " vertical " stirrups*

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

$\Rightarrow$ $V_{us} =$ (179 - 104) = 75 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

⇒ required spacing sv ≤ (0.87 x 415 x 101 x 560 / (74.88 x 1000))

⇒ Spacing , s_v = 271 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.28

Design shear stress , τ_c (N/mm^2) 0.74

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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
300 mm

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 250) =	363	mm
ii)	≤	(0.75 x 559.5) =	420	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 560 / (74.88 x 1000)) =	271	mm

Beam B1 Support

Design Parameters

Load Case 13 [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)	**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)	4.96
Breadth of the beam , b (mm)	400
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	150	kN-m
Bending Moment Mu(kN-m)	1053	
Equivalent Bending Moment , M_e (kNm)	1340	
Shear force at critical distance , V_{ud} (kN)	233	
Equivalent Shear (kN)	833	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\mathrm{Mulim} = 0.362 fck * \frac{bxu_{max}}{d} * 0.416 xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} \, x \, \frac{Ast}{bd} \, x \, \frac{fck}{2\, fy} = 4.598 \, \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 $f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim} =$ (0.1389 x 30 x 400 x 859.5^2 / 1000000) = 1,231.33 kNm

⇒ $p_{t,lim} =$ (41.3 x 30 / 415 x 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

$M_u =$ 1,339.76 kNm

$M_{u,lim} =$ 1,231.33 kNm

⇒ *Section cannot be designed as singly reinforced. Doubly Reinforced Section Needed*

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 x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 400 x 859.5) = 4927 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Rightarrow A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\Rightarrow \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **5562** mm^2

∴ No of tension bars required (#)
(5562 / (Pi / 4 x 25^2) = 12.00

Actual percentage of steel , p_t (%)
(12 x Pi / 4 x 25 ^2 / 400 / 860 x 100) = 1.71

Actual area of steel , A_{st} (mm^2)
(12 x Pi / 4 x 25 ^2) = 5890

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{0.87\, f_y \,(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y \,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 859.5) = 0.047
From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \left(\frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}}\right)\left(p_t - p_{t,lim}\right)$$

Actual p_t provided : p_t = 1.71

Actual p_c provided : p_c = 0.14

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.713 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ p_c^* = 0.30

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

where
$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 5294 / 5890) = 216.31 N/mm^2

F = 1.00

F_1 = 0.79

$F_2 =$ 0.55

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.79 x 0.55) = 11.30

$(l / d)_{provided} =$ 5.77

$\Rightarrow$ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (kN) 833

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 , τ_v

(833 x 1000 / (400 x 860)) = 2.42 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 5890 mm^2

Percentage of steel , p_t (%) 1.71

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85 \sqrt{0.8 f_{ck}} \left(\sqrt{1 + 5\beta} - 1 \right)}{6 \beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.80 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.8 x 400 x 860 / 1000) = 275 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 kN

Using 12 mm bars and
No of legs 4

Area of stirrups , A_{sv} (mm^2) 452

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 452 x 860 / (558.2 x 1000))

$\Rightarrow$ Spacing , s_v = 251 mm

<u>Check whether τ_v > 0.5 τ_c</u>

Nominal shear stress , τ_v (N/mm^2)	2.42
Design shear stress , τ_c (N/mm^2)	0.80

$\tau_v > 0.5\ \tau_c$ *<u>Yes</u>*

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\left|\begin{array}{l} 0.75\ d \\ 300\ mm \end{array}\right.$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 452 / 400) =	1021	mm
ii)	$\leq$	(0.75 x 859.5) =	645	mm
iii)	$\leq$	300 mm	300	mm
iv)	$\leq$	(0.87 x 415 x 452 x 860 / (558.2 x 1000)) =	251	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} (N/mm^2)	**30**
Characteristic yield strength of steel , f_y (N/mm^2)	**415**
Unit weight of concrete , γ_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)	4.96
Breadth of the beam , b (mm)	400
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	156	kN-m
Bending Moment Mu(kN-m)	331	
Equivalent Bending Moment , M_e (kNm)	629	
Shear force at critical distance , V_{ud} (kN)	218	
Equivalent Shear (kN)	842	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2 fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\ b\ d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 400 x 859.5^2 / 1000000) = 1,231.33 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 629.24 kNm
$M_{u,lim}$ = 1,231.33 kNm

⇒ *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 $A_{st,lim} = p_{t,lim}$ / 100 (b x d)

  ⇒ $A_{st,lim}$ (1.433 / 100 x 400 x 859.5) = 4927 mm^2

- Assuming 25 mm bars for compression steel,

  d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Rightarrow A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\Rightarrow \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 2227 mm^2

$\therefore$ No of tension bars required (#)
(2227 / (Pi / 4 x 25^2) = 5.00

Actual percentage of steel , p_t (%)
(5 x Pi / 4 x 25 ^2 / 400 / 860 x 100) = 0.71

Actual area of steel , A_{st} (mm^2)
(5 x Pi / 4 x 25 ^2) = 2454

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		**d'/d**		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 859.5) = 0.047
From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.71

Actual p_c provided : p_c = 0.71

⇒ p_c^* = (0.87 x 415 x (0.714 - 1.433) / (354.98 - 0.447 x 30))

⇒ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

where

$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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.

$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

⇒ f_{st} = (0.58 x 415 x 2891 / 2454) = 283.49 N/mm^2

F = 1.00

F_1 = 1.08

$F_2 =$ 1.16

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 1.08 x 1.16) = 32.39

$(l / d)_{provided} =$ 5.77

$\Rightarrow$ *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 , τ_v

(842 x 1000 / (400 x 860)) = 2.45 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *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 , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89\, p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.57 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.57 x 400 x 860 / 1000) = 198 kN

- *Design of " vertical " stirrups*

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

$\Rightarrow$ $V_{us} =$ (842 - 198) = 644 kN

Using 12 mm bars and

No of legs 4

Area of stirrups , A_{sv} (mm^2) 452

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 452 x 860 / (644.46 x 1000))

$\Rightarrow$ Spacing , s_v = 218 mm

Check whether $\tau_v > 0.5\ \tau_c$

Nominal shear stress , τ_v (N/mm^2) 2.45
Design shear stress , τ_c (N/mm^2) 0.57

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \ \square \ \begin{array}{l} 0.75\ d \\ 300\ mm \end{array}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 452 / 400) =	1021	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 452 x 860 / (644.46 x 1000)) =	218	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/mm^2)	**30**
Characteristic yield strength of steel , f_y (N/mm^2)	**415**
Unit weight of concrete , γ_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)	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

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{\lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2 fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 800 x 859.5^2 / 1000000) = 2,462.66 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 kNm
$M_{u,lim}$ = 2,462.66 kNm

⇒ *Section can be designed as singly reinforced.*

Determining A_{st}

- Considering a ' balanced section ' (x_u = $x_{u,max}$)

  A_{st} = $A_{st,lim}$ + ΔA_{st}

  where $A_{st,lim}$ = $p_{t,lim}$ / 100 (b x d)

  ⇒ $A_{st,lim}$ (1.433 / 100 x 800 x 859.5) = 9854 mm^2

- Assuming 25 mm bars for compression steel,

  d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87 f_y (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87 f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 8670 mm^2

∴ No of tension bars required (#)
(8670 / (Pi / 4 x 25^2) = 18.00

Actual percentage of steel , p_t (%)
(18 x Pi / 4 x 25 ^2 / 800 / 860 x 100) = 1.29

Actual area of steel , A_{st} (mm^2)
(18 x Pi / 4 x 25 ^2) = 8836

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

$$p_c = \frac{0.87 f_y (p_t - p_{t,lim})}{f_{sc} - 0.447 f_{ck}}$$

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.

Grade of steel		**d'/d**		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, 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

$$p_c^* = \frac{0.87\ f_y}{f_{sc} - 0.447\ f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.29

Actual p_c provided : p_c = 0.14

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.285 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ p_c^* = -0.16

<u>*Section is not over reinforced*</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1 F_2$$

where
$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 9037 / 8836) = 246.19 N/mm^2

F = 0.93

F_1 = 0.85

F_2 = 0.55

∴ $(l / d)_{max}$ = (26 x 0.93 x 0.85 x 0.55) = 11.21

$(l / d)_{provided}$ = 12.57

⇒ _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 , τ_v

(1158 x 1000 / (800 x 860)) = 1.68 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ _Adopted section is adequate_

- _Design shear resistance at critical section_

At critical section , A_{st} is given by 8836 mm^2

Percentage of steel , p_t (%) 1.29

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

⇒ τ_c = 0.72 N/mm^2

⇒ V_{uc} = (0.72 x 800 x 860 / 1000) = 496 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} = (1158 - 496) = 662 kN

Using 12 mm bars and
No of legs 4

Area of stirrups , A_{sv} (mm^2) 452

⇒ required spacing sv ≤ (0.87 x 415 x 452 x 860 / (662.45 x 1000))

⇒ Spacing , s_v = 212 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.68
Design shear stress , τ_c (N/mm^2) 0.72

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \ \square \ \begin{cases} 0.75\ d \\ 300\ \text{mm} \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 452 / 800) =	510	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 452 x 860 / (662.45 x 1000)) =	212	mm

Beam B2 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 , γ_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)	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)	855	
Equivalent Bending Moment , M_e (kNm)	1011	
Shear force at critical distance , V_{ud} (kN)	884	
Equivalent Shear (kN)	1134	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389 \; f_{ck} \, b \, d^2$

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

⇒ $M_{u,lim} =$ (0.1389 x 30 x 800 x 859.5^2 / 1000000) = 2,462.66 kNm

⇒ $p_{t,lim} =$ (41.3 x 30 / 415 x 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.

<u>Check for the type of section</u>

$M_u =$ 1,011.25 kNm
$M_{u,lim} =$ 2,462.66 kNm

⇒ *Section can be designed as singly reinforced.*

<u>Determining A_{st}</u>

- 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 x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 800 x 859.5) = 9854 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 3506 mm^2

∴ No of tension bars required (#)
(3506 / (Pi / 4 x 25^2) = 8.00

Actual percentage of steel , p_t (%)
(8 x Pi / 4 x 25 ^2 / 800 / 860 x 100) = 0.57

Actual area of steel , A_{st} (mm^2)
(8 x Pi / 4 x 25 ^2) = 3927

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, 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

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.57

Actual p_c provided : p_c = 0.79

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.571 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

where
$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 4945 / 3927) = 303.12 N/mm^2

F = 0.93

F_1 = 1.18

F_2 = 1.19

∴ $(l / d)_{max}$ = (26 x 0.93 x 1.18 x 1.19) = 33.53

$(l / d)_{provided}$ = 12.57

⇒ *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 , τ_v

(1134 x 1000 / (800 x 860)) = 1.65 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 3927 mm^2

Percentage of steel , p_t (%) 0.57

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1 + 5\beta} - 1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89\, p_t}$ or 1 whichever is greater

For (M30 and Fe415)

⇒ τ_c = 0.52 N/mm^2

⇒ V_{uc} = (0.52 x 800 x 860 / 1000) = 361 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} = (1134 - 361) = 773 kN

Using 12 mm bars and

No of legs 4

Area of stirrups , A_{sv} (mm^2) 452

⇒ required spacing sv ≤ (0.87 x 415 x 452 x 860 / (773.14 x 1000))

⇒ Spacing , s_v = 182 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.65

Design shear stress , τ_c (N/mm^2) 0.52

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \le \begin{cases} 0.75\ d \\ 300\ mm \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 452 / 800) =	510	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 452 x 860 / (773.14 x 1000)) =	182	mm

Beam B2A 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 , γ_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)	4.96
Breadth of the beam , b (mm)	550
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	163	kN-m
Bending Moment Mu(kN-m)	1428	
Equivalent Bending Moment , M_e (kNm)	1681	
Shear force at critical distance , V_{ud} (kN)	199	
Equivalent Shear (kN)	673	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

$\Rightarrow$ $M_{u,lim}$ = (0.1389 x 30 x 550 x 859.5^2 / 1000000) = 1,693.08 kNm

$\Rightarrow$ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 1,680.78 kNm

$M_{u,lim}$ = 1,693.08 kNm

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

Determining A_{st}

- Considering a ' balanced section ' (x_u = $x_{u,max}$)

A_{st} = $A_{st,lim}$ + ΔA_{st}

where $A_{st,lim}$ = $p_{t,lim}$ / 100 (b x d)

$\Rightarrow$ $A_{st,lim}$ (1.433 / 100 x 550 x 859.5) = 6774 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **6749** mm^2

∴ No of tension bars required (#)

(6749 / (Pi / 4 x 25^2) = 14.00

Actual percentage of steel , p_t (%)

(14 x Pi / 4 x 25 ^2 / 550 / 860 x 100) = 1.45

Actual area of steel , A_{st} (mm^2)

(14 x Pi / 4 x 25 ^2) = 6872

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, 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

$$p_c^* = \left(\frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}}\right)\left(p_t - p_{t,lim}\right)$$

Actual p_t provided : p_t = 1.45

Actual p_c provided : p_c = 0.10

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.454 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ p_c^* = 0.02

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1 F_2$$

where

$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 6733 / 6872) = 235.82 N/mm^2

F = 1.00

F_1 = 0.82

$F_2 =$ 0.44

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.82 x 0.44) = 9.36

$(l / d)_{provided} =$ 5.77

$\Rightarrow$ *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 , τ_v

(673.181818181818 x 1000 / (550 x 860)) 1.42 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62 fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *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 , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.75 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.75 x 550 x 860 / 1000) = 356 kN

- *Design of " vertical " stirrups*

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

$\Rightarrow$ $V_{us} =$ (673 - 356) = 317 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 860 / (316.79 x 1000))

⇒ Spacing , s_v = 222 mm

<u>*Check whether τ_v > 0.5 τ_c*</u>

Nominal shear stress , τ_v (N/mm^2)	1.42
Design shear stress , τ_c (N/mm^2)	0.75

$\tau_v > 0.5\ \tau_c$ <u>*Yes*</u>

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{array}{l} 0.75\ d \\ 300\ mm \end{array}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 550) =	371	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 860 / (316.79 x 1000)) =	222	mm

Beam B2A Mid

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 , γ_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)	4.96
Breadth of the beam , b (mm)	550
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	163	kN-m
Bending Moment Mu(kN-m)	746	
Equivalent Bending Moment , M_e (kNm)	999	
Shear force at critical distance , V_{ud} (kN)	184	
Equivalent Shear (kN)	658	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\mathrm{Mulim} = 0.362\mathrm{fck} * \frac{\mathrm{bxu}_{\mathrm{max}}}{\mathrm{d}} * 0.416\mathrm{xu}_{\mathrm{max}}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$xu_{max} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2 fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389 \ f_{ck} b \ d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 550 x 859.5^2 / 1000000) = 1,693.08 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 998.78 kNm

$M_{u,lim}$ = 1,693.08 kNm

⇒ *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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 550 x 859.5) = 6774 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\ f_y\ (d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\ f_y\ \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 3597 mm^2

∴ No of tension bars required (#)
(3597 / (Pi / 4 x 25^2) = 8.00

Actual percentage of steel , p_t (%)
(8 x Pi / 4 x 25 ^2 / 550 / 860 x 100) = 0.83

Actual area of steel , A_{st} (mm^2)
(8 x Pi / 4 x 25 ^2) = 3927

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\ f_y)\ (\Delta A_{st})}{f_{sc} - 0.447\ f_{ck}}$$

or

$$p_c = \frac{0.87\ f_y\ (p_t - p_{t,lim})}{f_{sc} - 0.447\ f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, 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

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.83

Actual p_c provided : p_c = 0.62

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.831 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ p_c^* = -0.64

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 4426 / 3927) = 271.31 N/mm^2

F = 1.00

F_1 = 1.02

$F_2 =$ 1.11

$\therefore$ $(l/d)_{max}$ = (26 x 1 x 1.02 x 1.11) = 29.37

$(l/d)_{provided}$ = 5.77

$\Rightarrow$ *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 , τ_v

(658.181818181818 x 1000 / (550 x 860)) 1.39 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\,fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 3927 mm^2

Percentage of steel , p_t (%) 0.83

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89\, p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ τ_c = 0.61 N/mm^2

$\Rightarrow$ V_{uc} = (0.61 x 550 x 860 / 1000) = 288 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} = (658 - 288) = 370 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

$\Rightarrow$ required spacing sv ≤ (0.87 x 415 x 226 x 860 / (369.72 x 1000))

$\Rightarrow$ Spacing , s_v = 190 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.39

Design shear stress , τ_c (N/mm^2) 0.61

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \le \begin{cases} 0.75\ d \\ 300\ mm \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 550) =	371	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 860 / (369.72 x 1000)) =	190	mm

Beam B3 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 , γ_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

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$xu_{max} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^{2}}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 859.5^2 / 1000000) = 923.50 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 697.88 kNm
$M_{u,lim}$ = 923.50 kNm

⇒ *Section can be designed as singly reinforced.*

Determining A_{st}

- Considering a ' balanced section ' (x_u = $x_{u,max}$)

  A_{st} = $A_{st,lim}$ + ΔA_{st}

  where $A_{st,lim}$ = $p_{t,lim}$ / 100 (b x d)

  ⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 859.5) = 3695 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \left(d - d'\right)}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 2616 mm^2

∴ No of tension bars required (#)
(2616 / (Pi / 4 x 25^2) = 6.00

Actual percentage of steel , p_t (%)
(6 x Pi / 4 x 25 ^2 / 300 / 860 x 100) = 1.14

Actual area of steel , A_{st} (mm^2)
(6 x Pi / 4 x 25 ^2) = 2945

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{0.87\, f_y \left(\Delta A_{st}\right)}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y \left(p_t - p_{t,lim}\right)}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 859.5) = 0.047
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.14

Actual p_c provided : p_c = 0.38

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.142 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ p_c^* = -0.31

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

$$\text{where } \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 2932 / 2945) = 239.63 N/mm^2

F = 1.00

F_1 = 0.93

$F_2 =$ 0.93

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.93 x 0.93) = 22.37

$(l / d)_{provided} =$ 9.73

$\Rightarrow$ *Hence O.K.*

Check for shear

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

(118.666666666667 x 1000 / (300 x 860)) 0.46 N/mm^2

The maximum shear stress is given by : $$Tc\max = 0.62\, fck$$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 2945 mm^2

Percentage of steel , p_t (%) 1.14

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.69 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.69 x 300 x 860 / 1000) = 178 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 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

$\Rightarrow$ required spacing sv ≤ (0.87 x 415 x 101 x 860 / (-59.19 x 1000))

$\Rightarrow$ Spacing , s_v = -527 mm

<u>*Check whether τ_v > 0.5 τ_c*</u>

Nominal shear stress , τ_v (N/mm^2) 0.46
Design shear stress , τ_c (N/mm^2) 0.69

$\tau_v > 0.5\ \tau_c$ <u>*Yes*</u>

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \ \square \ \begin{cases} 0.75\ d \\ 300\ \text{mm} \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 300) =	302	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 860 / (-59.19 x 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 , γ_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

Effective depth , d (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.

Torsional Moment	11	kN-m
Bending Moment Mu(kN-m)	303	
Equivalent Bending Moment , M_e (kNm)	329	
Shear force at critical distance , V_{ud} (kN)	47	
Equivalent Shear (kN)	106	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\max}}{\text{d}} * 0.416\text{xu}_{\max}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$\text{xu}_{\max} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 862^2 / 1000000) = 928.88 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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.

<u>Check for the type of section</u>

M_u = 328.88 kNm

$M_{u,lim}$ = 928.88 kNm

⇒ *Section can be designed as singly reinforced.*

<u>Determining A_{st}</u>

- 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 x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 862) = 3706 mm^2

- Assuming 20 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 20 / 2) = 38 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\ f_y\ (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\ f_y\ \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **1124** mm^2

∴ No of tension bars required (#)
(1124 / (Pi / 4 x 20^2) = 4.00

Actual percentage of steel , p_t (%)
(4 x Pi / 4 x 20 ^2 / 300 / 862 x 100) = 0.49

Actual area of steel , A_{st} (mm^2)
(4 x Pi / 4 x 20 ^2) = 1257

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

$$p_c = \frac{0.87\ f_y\ (p_t - p_{t,lim})}{f_{sc} - 0.447\ f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (38 / 862) = 0.044
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

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.49

Actual p_c provided : p_c = 0.85

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.486 - 1.433) / (354.8 - 0.447 x 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1\, F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1689 / 1257) = 323.54 N/mm^2

F = 1.00

F_1 = 1.23

$F_2 =$ 1.21

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 1.23 x 1.21) = 38.53

$(l / d)_{provided} =$ 9.70

$\Rightarrow$ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (kN) 105.66667

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 , τ_v

(105.666666666667 x 1000 / (300 x 862)) 0.41 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1257 mm^2

Percentage of steel , p_t (%) 0.49

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.49 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.49 x 300 x 862 / 1000) = 127 kN

- *Design of " vertical " stirrups*

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

$\Rightarrow$ $V_{us} =$ (106 - 127) = -21 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 101 x 862 / (-21.27 x 1000))

$\Rightarrow$ Spacing , s_v = -1471 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 0.41
Design shear stress , τ_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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 300) =	302	mm
ii)	≤	(0.75 x 862) =	647	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 862 / (-21.27 x 1000)) =	-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 , γ_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

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 859.5^2 / 1000000) = 923.50 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 kNm

$M_{u,lim}$ = 923.50 kNm

⇒ *Section can be designed as singly reinforced.*

Determining A_{st}

- Considering a ' balanced section ' (x_u = $x_{u,max}$)

  A_{st} = $A_{st,lim}$ + ΔA_{st}

  where $A_{st,lim}$ = $p_{t,lim}$ / 100 (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 859.5) = 3695 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 2868 mm^2

∴ No of tension bars required (#)
(2868 / (Pi / 4 x 25^2) = 6.00

Actual percentage of steel , p_t (%)
(6 x Pi / 4 x 25 ^2 / 300 / 860 x 100) = 1.14

Actual area of steel , A_{st} (mm^2)
(6 x Pi / 4 x 25 ^2) = 2945

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 859.5) = 0.047
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.14

Actual p_c provided : p_c = 0.38

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.142 - 1.433) / (354.98 - 0.447 x 30))

$\Rightarrow$ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1\, F_2$$

where
$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 3119 / 2945) = 254.88 N/mm^2

F = 1.00

F_1 = 0.89

$F_2 =$ 0.93

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.89 x 0.93) = 21.40

$(l / d)_{provided} =$ 1.69

$\Rightarrow$ *__Hence O.K.__*

__Check for shear__

Shear force at critical distance , V_{ud} (kN) 388.666667

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 , τ_v

(388.666666666667 x 1000 / (300 x 860)) 1.51 N/mm^2

The maximum shear stress is given by : $$Tc\max = 0.62\, fck$$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *__Adopted section is adequate__*

- *__Design shear resistance at critical section__*

At critical section , A_{st} is given by 2945 mm^2

Percentage of steel , p_t (%) 1.14

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.69 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.69 x 300 x 860 / 1000) = 178 kN

- *__Design of " vertical " stirrups__*

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

$\Rightarrow$ $V_{us} =$ (389 - 178) = 211 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

⇒ required spacing sv ≤ (0.87 x 415 x 101 x 860 / (210.81 x 1000))

⇒ Spacing , s_v = 148 mm

Check whether $\tau_v > 0.5\ \tau_c$

Nominal shear stress , τ_v (N/mm^2)	1.51
Design shear stress , τ_c (N/mm^2)	0.69

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \ \square \ \begin{cases} 0.75\ d \\ 300\ mm \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 300) =	302	mm
ii)	≤	(0.75 x 859.5) =	645	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 860 / (210.81 x 1000)) =	148	mm

Beam B4 Mid

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 , γ_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)	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

Effective depth , d (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.

Torsional Moment	20	kN-m
Bending Moment Mu(kN-m)	314	
Equivalent Bending Moment , M_e (kNm)	361	
Shear force at critical distance , V_{ud} (kN)	259	
Equivalent Shear (kN)	366	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\mathrm{Mulim} = 0.362\mathrm{fck} * \frac{\mathrm{bxu}_{\max}}{\mathrm{d}} * 0.416\mathrm{xu}_{\max}$$

Where $\mathrm{b} = $ Breadth of the Section

$\mathrm{xu}_{\max} = $ Limiting depth of Neutral Axis

$\mathrm{d} = $ Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} \, x \, \frac{Ast}{bd} \, x \, \frac{fck}{2\, fy} \; = \; 4.598 \, \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 862^2 / 1000000) = 928.88 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 361.06 kNm
$M_{u,lim}$ = 928.88 kNm

⇒ *Section can be designed as singly reinforced.*

Determining A_{st}

- Considering a ' balanced section ' (x_u = $x_{u,max}$)

A_{st} = $A_{st,lim}$ + ΔA_{st}

where $A_{st,lim}$ = $p_{t,lim}$ / 100 (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 862) = 3706 mm^2

- Assuming 20 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 20 / 2) = 38 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **1243** mm^2

∴ No of tension bars required (#)
(1243 / (Pi / 4 x 20^2) = 5.00

Actual percentage of steel , p_t (%)
(4 x Pi / 4 x 20 ^2 / 300 / 862 x 100) = 0.49

Actual area of steel , A_{st} (mm^2)
(4 x Pi / 4 x 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

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		**d'/d**		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (38 / 862) = 0.044
  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

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.49
Actual p_c provided : p_c = 0.85

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.486 - 1.433) / (354.8 - 0.447 x 30))
$\Rightarrow$ p_c^* = -1.00

<u>*Section is not over reinforced*</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1 F_2$$

where
$$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1797 / 1257) = 344.26 N/mm^2

F = 1.00
F_1 = 1.13

$F_2 =$ 1.21

∴ $(l / d)_{max} =$ (26 x 1 x 1.13 x 1.21) = 35.62

$(l / d)_{provided} =$ 1.68

⇒ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (kN) 365.66667

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 , τ_v

(365.666666666667 x 1000 / (300 x 862)) 1.41 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62 fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1257 mm^2

Percentage of steel , p_t (%) 0.49

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

⇒ $\tau_c =$ 0.49 N/mm^2

⇒ $V_{uc} =$ (0.49 x 300 x 862 / 1000) = 127 kN

- *Design of " vertical " stirrups*

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

⇒ $V_{us} =$ (366 - 127) = 239 kN

Using 8 mm bars and

No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 101 x 862 / (238.73 x 1000))

$\Rightarrow$ Spacing , s_v = 131 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.41

Design shear stress , τ_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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V\ \square\ \begin{array}{|l} 0.75\ d \\ \hline 300\ mm \end{array}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 300) =	302	mm
ii)	≤	(0.75 x 862) =	647	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 862 / (238.73 x 1000)) =	131	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 , γ_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)	1.40
Breadth of the beam , b (mm)	250
Overall depth of the beam , D (mm)	550

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)	(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.

Torsional Moment	10	kN-m
Bending Moment Mu(kN-m)	245	
Equivalent Bending Moment , M_e (kNm)	264	
Shear force at critical distance , V_{ud} (kN)	55	
Equivalent Shear (kN)	119	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where $b =$ Breadth of the Section

$xu_{max} =$ Limiting depth of Neutral Axis

$d =$ Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 $f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 250 x 512^2 / 1000000) = 273.09 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 263.82 kNm
$M_{u,lim}$ = 273.09 kNm

⇒ *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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 250 x 512) = 1834 mm^2

- Assuming 20 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 20 / 2) = 38 mm

$$A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **1763** mm^2

∴ 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} (mm^2)
(6 x Pi / 4 x 20 ^2) = 1885

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y \,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, 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 $p_c \geq p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.47

Actual p_c provided : p_c = 0.25

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.473 - 1.433) / (354.61 - 0.447 x 30))

$\Rightarrow$ p_c^* = 0.04

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

where $\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$

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} = (0.58 x 415 x 1780 / 1885) = 227.32 N/mm^2

F = 1.00

F_1 = 0.83

F_2 = 0.75

$\therefore$ $(l / d)_{max}$ = (26 x 1 x 0.83 x 0.75) = 16.36

$(l / d)_{provided}$ = 2.73

$\Rightarrow$ *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 , τ_v

(119 x 1000 / (250 x 512)) = 0.93 N/mm^2

The maximum shear stress is given by : $$Tc\max = 0.62 fck$$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1885 mm^2

Percentage of steel , p_t (%) 1.47

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ τ_c = 0.76 N/mm^2

$\Rightarrow$ V_{uc} = (0.76 x 250 x 512 / 1000) = 97 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 kN

Using 8 mm bars and

No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

⇒ required spacing sv ≤ (0.87 x 415 x 101 x 512 / (22.05 x 1000))

⇒ Spacing , s_v = 843 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2)	0.93
Design shear stress , τ_c (N/mm^2)	0.76

$\tau_v > 0.5\, \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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
300 mm

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 250) =	363	mm
ii)	≤	(0.75 x 512) =	384	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 512 / (22.05 x 1000)) =	843	mm

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 , γ_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)	1.40
Breadth of the beam , b (mm)	250
Overall depth of the beam , D (mm)	550

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)	(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.

Torsional Moment	10	kN-m
Bending Moment Mu(kN-m)	118	
Equivalent Bending Moment , M_e (kNm)	137	
Shear force at critical distance , V_{ud} (kN)	55	
Equivalent Shear (kN)	119	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$\text{xu}_{\text{max}} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} \; x \; \frac{Ast}{bd} \; x \; \frac{fck}{2\,fy} \; = \; 4.598 \; \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389 \; f_{ck} \, b \, d^2$

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

⇒ $M_{u,lim} =$ (0.1389 x 30 x 250 x 512^2 / 1000000) = 273.09 kNm

⇒ $p_{t,lim} =$ (41.3 x 30 / 415 x 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 kNm

$M_{u,lim} =$ 273.09 kNm

⇒ *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 $A_{st,lim} = p_{t,lim} / 100 \, (b \times d)$

⇒ $A_{st,lim}$ (1.433 / 100 x 250 x 512) = 1834 mm^2

- Assuming 20 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 20 / 2) = 38 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 811 mm^2

∴ No of tension bars required (#)
(811 / (Pi / 4 x 20^2) = 3.00

Actual percentage of steel , p_t (%)
(3 x Pi / 4 x 20 ^2 / 250 / 512 x 100) = 0.74

Actual area of steel , A_{st} (mm^2)
(3 x Pi / 4 x 20 ^2) = 942

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

$$p_c = \frac{0.87\, f_y \,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (38 / 512) = 0.074
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\ f_y}{f_{sc} - 0.447\ f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.74

Actual p_c provided : p_c = 0.74

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.736 - 1.433) / (354.61 - 0.447 x 30))

$\Rightarrow$ p_c^* = -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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1\ F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1038 / 942) = 265.12 N/mm^2

F = 1.00

F_1 = 1.13

$F_2 =$ 1.16

$\therefore$ $(l/d)_{max} =$ (26 x 1 x 1.13 x 1.16) = 34.13

$(l/d)_{provided} =$ 2.73

$\Rightarrow$ *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 , τ_v

(119 x 1000 / (250 x 512)) = 0.93 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\sqrt{fck}$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 942 mm^2

Percentage of steel , p_t (%) 0.74

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.58 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.58 x 250 x 512 / 1000) = 74 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 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

⇒ required spacing sv ≤ (0.87 x 415 x 101 x 512 / (44.54 x 1000))

⇒ Spacing , s_v = 417 mm

<u>Check whether $\tau_v > 0.5\ \tau_c$</u>

Nominal shear stress , τ_v (N/mm^2) 0.93
Design shear stress , τ_c (N/mm^2) 0.58

$\tau_v > 0.5\ \tau_c$ *<u>Yes</u>*

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 250) =	363	mm
ii)	≤	(0.75 x 512) =	384	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 512 / (44.54 x 1000)) =	417	mm

Beam RB1 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 , γ_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)	235	
Equivalent Bending Moment , M_e (kNm)	235	
Shear force at critical distance , V_{ud} (kN)	125	
Equivalent Shear (kN)	125	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 $f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 459.5^2 / 1000000) = 263.95 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 235.00 kNm
$M_{u,lim}$ = 263.95 kNm

⇒ *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 $A_{st,lim} = p_{t,lim}$ / 100 (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 459.5) = 1975 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 1710 mm^2

$\therefore$ No of tension bars required (#)
(1710 / (Pi / 4 x 25^2) = 4.00

Actual percentage of steel , p_t (%)
(4 x Pi / 4 x 25 ^2 / 300 / 460 x 100) = 1.42

Actual area of steel , A_{st} (mm^2)
(4 x Pi / 4 x 25 ^2) = 1963

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{0.87\, f_y\, (A_{st} - A_{st,lim})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel	$\frac{d'}{d}$			
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 459.5) = 0.088
From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.42

Actual p_c provided : p_c = 0.36

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.424 - 1.433) / (353.38 - 0.447 x 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1784 / 1963) = 218.71 N/mm^2

F = 1.00

F_1 = 0.87

$F_2 =$ 0.90

$\therefore$ $(l / d)_{max} =$ (26 x 1 x 0.87 x 0.9) = 20.41

$(l / d)_{provided} =$ 11.63

$\Rightarrow$ <u>*Hence O.K.*</u>

<u>Check for shear</u>

Shear force at critical distance , V_{ud} (kN) 125

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

- <u>*Check for adequacy of section*</u>

Nominal shear stress , τ_v

(125 x 1000 / (300 x 460)) = 0.91 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\sqrt{fck}$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ <u>*Adopted section is adequate*</u>

- <u>*Design shear resistance at critical section*</u>

At critical section , A_{st} is given by 1963 mm^2

Percentage of steel , p_t (%) 1.42

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.75 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.75 x 300 x 460 / 1000) = 103 kN

- <u>*Design of " vertical " stirrups*</u>

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

$\Rightarrow$ $V_{us} =$ (125 - 103) = 22 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 460 / (21.84 x 1000))

⇒ Spacing , s_v = 1718 mm

<u>Check whether τ_v > 0.5 τ_c</u>

Nominal shear stress , τ_v (N/mm^2) 0.91
Design shear stress , τ_c (N/mm^2) 0.75

$\tau_v > 0.5\ \tau_c$ *<u>Yes</u>*

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V\ \square\ \begin{array}{|l|}0.75\ d\\ \hline 300\ mm\end{array}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 459.5) =	345	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 460 / (21.84 x 1000)) =	1718	mm

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 , γ_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* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\, fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\, b\, d^2$

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

$\Rightarrow$ $M_{u,lim} =$ (0.1389 x 30 x 300 x 459.5^2 / 1000000) = 263.95 kNm

$\Rightarrow$ $p_{t,lim} =$ (41.3 x 30 / 415 x 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

$M_u =$ 150.00 kNm

$M_{u,lim} =$ 263.95 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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

$\Rightarrow$ $A_{st,lim}$ (1.433 / 100 x 300 x 459.5) = 1975 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Rightarrow A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\Rightarrow \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 1006 mm^2

∴ No of tension bars required (#)
(1006 / (Pi / 4 x 25^2) = 3.00

Actual percentage of steel , p_t (%)
(3 x Pi / 4 x 25 ^2 / 300 / 460 x 100) = 1.07

Actual area of steel , A_{st} (mm^2)
(3 x Pi / 4 x 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

$$p_c = \frac{0.87\, f_y\,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 459.5) = 0.088
From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \left(\frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}}\right)\left(p_t - p_{t,lim}\right)$$

Actual p_t provided : p_t = 1.07

Actual p_c provided : p_c = 0.71

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.068 - 1.433) / (353.38 - 0.447 x 30))

$\Rightarrow$ p_c^* = -0.39

<u>Section is not over reinforced</u>

<u>Check for deflection control</u>

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1222 / 1473) = 199.78 N/mm^2

F = 1.00

F_1 = 1.10

$F_2 =$ 1.15

$\therefore$ $(l/d)_{max} = (26 \times 1 \times 1.1 \times 1.15) =$ 33.00

$(l/d)_{provided} =$ 11.63

$\Rightarrow$ *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 , τ_v

(120 x 1000 / (300 x 460)) = 0.87 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62 fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1473 mm^2

Percentage of steel , p_t (%) 1.07

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.67 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.67 x 300 x 460 / 1000) = 93 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 kN

Using 12 mm bars and
No of legs 4

Area of stirrups , A_{sv} (mm^2) 452

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 452 x 460 / (27.29 x 1000))

$\Rightarrow$ Spacing , s_v = 2750 mm

<u>Check whether τ_v > 0.5 τ_c</u>

Nominal shear stress , τ_v (N/mm^2) 0.87
Design shear stress , τ_c (N/mm^2) 0.67

$\tau_v > 0.5\ \tau_c$ *<u>Yes</u>*

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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$
$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 452 / 300) =	1361	mm
ii)	≤	(0.75 x 459.5) =	345	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 452 x 460 / (27.29 x 1000)) =	2750	mm

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} (N/mm^2)	**30**
Characteristic yield strength of steel , f_y (N/mm^2)	**415**
Unit weight of concrete , γ_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)	5.36
Breadth of the beam , b (mm)	300
Overall depth of the beam , D (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 (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 Mu(kN-m)	325	
Equivalent Bending Moment , M_e (kNm)	355	
Shear force at critical distance , V_{ud} (kN)	70	
Equivalent Shear (kN)	155	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\max}}{\text{d}} * 0.416\text{xu}_{\max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\ fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 f_{ck} b d^2

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 609.5^2 / 1000000) = 464.40 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 = 354.80 kNm
$M_{u,lim}$ = 464.40 kNm

⇒ *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 $A_{st,lim} = p_{t,lim}$ / 100 (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 609.5) = 2620 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 1880 mm^2

∴ No of tension bars required (#)
(1880 / (Pi / 4 x 25^2) = 4.00

Actual percentage of steel , p_t (%)
(4 x Pi / 4 x 25 ^2 / 300 / 610 x 100) = 1.07

Actual area of steel , A_{st} (mm^2)
(4 x Pi / 4 x 25 ^2) = 1963

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

$$p_c = \frac{0.87\, f_y\,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		**d'/d**		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 609.5) = 0.066
From the above table : by interpolation

Design Check

- To ensure $x_u \leq x_{u,max}$, it suffices to establish $p_c \geq p_c^*$

where p_c^* is given by

$$p_c^* = \left[\frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}}\right]\left[p_t - p_{t,lim}\right]$$

Actual p_t provided : p_t = 1.07

Actual p_c provided : p_c = 0.54

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.074 - 1.433) / (355.03 - 0.447 x 30))

$\Rightarrow$ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

where $\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$

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} = (0.58 x 415 x 2087 / 1963) = 255.82 N/mm^2

F = 1.00

F_1 = 0.91

F_2 = 1.06

$\therefore$ $(l/d)_{max}$ = (26 x 1 x 0.91 x 1.06) = 25.16

$(l/d)_{provided}$ = 13.72

$\Rightarrow$ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (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 , τ_v

(155.333333333333 x 1000 / (300 x 610)) 0.85 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\,fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1963 mm^2

Percentage of steel , p_t (%) 1.07

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89\,p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ τ_c = 0.67 N/mm^2

$\Rightarrow$ V_{uc} = (0.67 x 300 x 610 / 1000) = 123 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 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 610 / (32.12 x 1000))

⇒ Spacing , s_v = 1550 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 0.85
Design shear stress , τ_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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_v \leq \begin{cases} 0.75\ d \\ 300\ mm \end{cases}$$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 609.5) =	457	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 610 / (32.12 x 1000)) =	1550	mm

Beam RB2 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 , γ_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)	5.36
Breadth of the beam , b (mm)	300
Overall depth of the beam , D (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 (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 Mu(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

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2 fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389 \; f_{ck} b d^2$

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

$\Rightarrow$ $M_{u,lim} =$ (0.1389 x 30 x 300 x 609.5^2 / 1000000) = 464.40 kNm

$\Rightarrow$ $p_{t,lim} =$ (41.3 x 30 / 415 x 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

$M_u =$ 219.80 kNm

$M_{u,lim} =$ 464.40 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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

$\Rightarrow$ $A_{st,lim}$ (1.433 / 100 x 300 x 609.5) = 2620 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Rightarrow A_{st} = \frac{M_u - M_{u,lim}}{0.87\ f_y\ (d - d')}$$

$$\Rightarrow \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\ f_y\ \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 1088 mm^2

∴ No of tension bars required (#)
(1088 / (Pi / 4 x 25^2) = 3.00

Actual percentage of steel , p_t (%)
(3 x Pi / 4 x 25 ^2 / 300 / 610 x 100) = 0.81

Actual area of steel , A_{st} (mm^2)
(3 x Pi / 4 x 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

$$p_c = \frac{0.87\ f_y\ (p_t - p_{t,lim})}{f_{sc} - 0.447\ f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 609.5) = 0.066
  From the above table : by interpolation

Design Check

- To ensure $x_u \leq x_{u,max}$, it suffices to establish $p_c \geq p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.81

Actual p_c provided : p_c = 0.81

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.805 - 1.433) / (355.03 - 0.447 x 30))

$\Rightarrow$ p_c^* = -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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1 F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1430 / 1473) = 233.69 N/mm^2

F = 1.00

F_1 = 1.19

F_2 = 1.19

∴ $(l / d)_{max}$ = (26 x 1 x 1.19 x 1.19) = 36.82

$(l / d)_{provided}$ = 13.72

⇒ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (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 , τ_v

(250.333333333333 x 1000 / (300 x 610)) 1.37 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\, fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1473 mm^2

Percentage of steel , p_t (%) 0.81

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\ \ 0.8 f_{ck}\ \ 1\ \ 5\ \ 1}{6}$$

where $\frac{0.8 f_{ck}}{6.89 p_t}$ whichever is greater

1

For (M30 and Fe415)

⇒ τ_c = 0.60 N/mm^2

⇒ V_{uc} = (0.6 x 300 x 610 / 1000) = 110 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} = (250 - 110) = 140 kN

Using 12 mm bars and

No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 610 / (140.11 x 1000))

⇒ Spacing , s_v = 355 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.37

Design shear stress , τ_c (N/mm^2) 0.60

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 609.5) =	457	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 610 / (140.11 x 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 , γ_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

Effective depth , d (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 Mu(kN-m)	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* , $M_{u,lim}$ is given by

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$\text{xu}_{\text{max}} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

Where fck = Charcteristic Compressive strength of concrete

fy = Charcteristic 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 :

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim} =$ (0.1389 x 30 x 300 x 809.5^2 / 1000000) = 819.18 kNm

⇒ $p_{t,lim} =$ (41.3 x 30 / 415 x 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

$M_u =$ 816.53 kNm

$M_{u,lim} =$ 819.18 kNm

⇒ *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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 809.5) = 3480 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\ f_y\ (d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\ f_y\ \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 3486 mm^2

∴ No of tension bars required (#)
(3486 / (Pi / 4 x 25^2) = 8.00

Actual percentage of steel , p_t (%)
(8 x Pi / 4 x 25 ^2 / 300 / 810 x 100) = 1.62

Actual area of steel , A_{st} (mm^2)
(8 x Pi / 4 x 25 ^2) = 3927

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\ f_y)\ (\Delta A_{st})}{f_{sc} - 0.447\ f_{ck}}$$

or

$$p_c = \frac{0.87\ f_y\ (p_t - p_{t,lim})}{f_{sc} - 0.447\ f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 809.5) = 0.050
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\ f_y}{f_{sc} - 0.447\ f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.62

Actual p_c provided : p_c = 0.20

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.617 - 1.433) / (355.1 - 0.447 x 30))

$\Rightarrow$ p_c^* = 0.19

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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

where $$\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 3471 / 3927) = 212.73 N/mm^2

F = 0.93

F_1 = 0.83

$F_2 =$ 0.68

∴ $(l/d)_{max} =$ (26 x 0.93 x 0.83 x 0.68) = 13.48

$(l/d)_{provided} =$ 13.34

⇒ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (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 , τ_v

(409 x 1000 / (300 x 810)) = 1.68 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62 fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 3927 mm^2

Percentage of steel , p_t (%) 1.62

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

⇒ $\tau_c =$ 0.78 N/mm^2

⇒ $V_{uc} =$ (0.78 x 300 x 810 / 1000) = 190 kN

- *Design of " vertical " stirrups*

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

⇒ $V_{us} =$ (409 - 190) = 219 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

$\Rightarrow$ required spacing sv $\leq$ (0.87 x 415 x 226 x 810 / (218.81 x 1000))

$\Rightarrow$ Spacing , s_v = 302 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.68
Design shear stress , τ_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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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
300 mm

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 809.5) =	607	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 810 / (218.81 x 1000)) =	302	mm

Beam RB3 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 , γ_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

Effective depth , d (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 Mu(kN-m)	360	
Equivalent Bending Moment , M_e (kNm)	374	
Shear force at critical distance , V_{ud} (kN)	270	
Equivalent Shear (kN)	302	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

$$\text{Where } b = \text{Breadth of the Section}$$

$$xu_{max} = \text{Limiting depth of Neutral Axis}$$

$$d = \text{Effective depth of the Section}$$

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} \; x \; \frac{Ast}{bd} \; x \; \frac{fck}{2\,fy} \; = \; 4.598 \; \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ 0.1389 $f_{ck}\, b\, d^2$

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

⇒ $M_{u,lim}$ = (0.1389 x 30 x 300 x 809.5^2 / 1000000) = 819.18 kNm

⇒ $p_{t,lim}$ = (41.3 x 30 / 415 x 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 kNm

$M_{u,lim}$ = 819.18 kNm

⇒ *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 $A_{st,lim} = p_{t,lim}$ / 100 (b x d)

⇒ $A_{st,lim}$ (1.433 / 100 x 300 x 809.5) = 3480 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y\, (d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **1388** mm^2

∴ No of tension bars required (#)
(1388 / (Pi / 4 x 25^2) = 3.00

Actual percentage of steel , p_t (%)
(3 x Pi / 4 x 25 ^2 / 300 / 810 x 100) = 0.61

Actual area of steel , A_{st} (mm^2)
(3 x Pi / 4 x 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

$$p_c = \frac{0.87\, f_y\, (p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 809.5) = 0.050
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.61

Actual p_c provided : p_c = 0.81

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.606 - 1.433) / (355.1 - 0.447 x 30))

$\Rightarrow$ 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 F_2$$

where $\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$

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} = (0.58 x 415 x 1875 / 1473) = 306.48 N/mm^2

F = 0.93

F_1 = 1.11

F_2 = 1.19

$\therefore$ $(l / d)_{max}$ = (26 x 0.93 x 1.11 x 1.19) = 31.96

$(l / d)_{provided}$ = 13.34

$\Rightarrow$ *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 , τ_v

(302 x 1000 / (300 x 810)) = 1.24 N/mm^2

The maximum shear stress is given by : $Tc\,\max = 0.62\,fck$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1473 mm^2

Percentage of steel , p_t (%) 0.61

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ τ_c = 0.54 N/mm^2

$\Rightarrow$ V_{uc} = (0.54 x 300 x 810 / 1000) = 131 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 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 810 / (171.38 x 1000))

⇒ Spacing , s_v = 386 mm

Check whether τ_v > 0.5 τ_c

Nominal shear stress , τ_v (N/mm^2) 1.24
Design shear stress , τ_c (N/mm^2) 0.54

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 809.5) =	607	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 810 / (171.38 x 1000)) =	386	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_{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)	**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)	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	16	kN-m
Bending Moment Mu(kN-m)	283	
Equivalent Bending Moment , M_e (kNm)	311	
Shear force at critical distance , V_{ud} (kN)	90	
Equivalent Shear (kN)	175	

Singly reinforced or doubly reinforced section ?

The *limiting moment of resistance* , $M_{u,lim}$ is given by

$$\text{Mulim} = 0.362\text{fck} * \frac{\text{bxu}_{\text{max}}}{\text{d}} * 0.416\text{xu}_{\text{max}}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2\,fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\ b\ d^2$

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

$\Rightarrow$ $M_{u,lim} =$ (0.1389 x 30 x 300 x 559.5^2 / 1000000) = 391.33 kNm

$\Rightarrow$ $p_{t,lim} =$ (41.3 x 30 / 415 x 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

$M_u =$ 311.24 kNm
$M_{u,lim} =$ 391.33 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 $A_{st,lim} = p_{t,lim} / 100$ (b x d)

$\Rightarrow$ $A_{st,lim}$ (1.433 / 100 x 300 x 559.5) = 2405 mm^2

- Assuming 25 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 25 / 2) = 40.5 mm

$$\therefore A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\therefore \frac{p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd **=** **1811** mm^2

∴ No of tension bars required (#)

(1811 / (Pi / 4 x 25^2) = 4.00

Actual percentage of steel , p_t (%)

(4 x Pi / 4 x 25 ^2 / 300 / 560 x 100) = 1.17

Actual area of steel , A_{st} (mm^2)

(4 x Pi / 4 x 25 ^2) = 1963

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y\,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (40.5 / 559.5) = 0.072
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\, f_y}{f_{sc} - 0.447\, f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 1.17

Actual p_c provided : p_c = 0.29

$\Rightarrow$ p_c^* = (0.87 x 415 x (1.17 - 1.433) / (354.73 - 0.447 x 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} \times F_1 \times F_2$$

$$\text{where} \quad \left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$$

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} = (0.58 x 415 x 1978 / 1963) = 242.47 N/mm^2

F = 1.00

F_1 = 0.91

$F_2 =$ 0.82

$\therefore$ $(l/d)_{max} =$ (26 x 1 x 0.91 x 0.82) = 19.42

$(l/d)_{provided} =$ 9.83

$\Rightarrow$ *Hence O.K.*

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 , τ_v

(175.333333333333 x 1000 / (300 x 560)) 1.04 N/mm^2

The maximum shear stress is given by : $$Tc\max = 0.62 fck$$

$\Rightarrow$ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

$\Rightarrow$ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 1963 mm^2

Percentage of steel , p_t (%) 1.17

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$ or 1 whichever is greater

For (M30 and Fe415)

$\Rightarrow$ $\tau_c =$ 0.70 N/mm^2

$\Rightarrow$ $V_{uc} =$ (0.7 x 300 x 560 / 1000) = 117 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 kN

Using 12 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 226

⇒ required spacing sv ≤ (0.87 x 415 x 226 x 560 / (58.52 x 1000))

⇒ Spacing , s_v = 781 mm

Check whether $\tau_v > 0.5\ \tau_c$

Nominal shear stress , τ_v (N/mm^2)	1.04
Design shear stress , τ_c (N/mm^2)	0.70

$\tau_v > 0.5\ \tau_c$ *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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

s_V □ $\begin{cases} 0.75\ d \\ 300\ mm \end{cases}$

Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 226 / 300) =	681	mm
ii)	≤	(0.75 x 559.5) =	420	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 226 x 560 / (58.52 x 1000)) =	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 , γ_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 Mu(kN-m)	120	
Equivalent Bending Moment , M_e (kNm)	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* , $M_{u,lim}$ is given by

$$Mulim = 0.362fck * \frac{bxu_{max}}{d} * 0.416xu_{max}$$

Where b = Breadth of the Section

xu_{max} = Limiting depth of Neutral Axis

d = Effective depth of the Section

The limiting percentage of steel , $p_{t,lim}$ is given by

$$Pt_{lim} = 41.61 * \frac{fck}{fy} * \frac{xu_{max}}{d}$$

$$\text{Where } fck = \text{Charcteristic Compressive strength of concrete}$$

$$fy = \text{Charcteristic strength of steel}$$

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

$$\frac{Pt}{100} x \frac{Ast}{bd} x \frac{fck}{2 fy} = 4.598 \frac{R}{f_{ck}}$$

$$Where \quad R = \frac{Mu}{bd^2}$$

For (M30 and Fe415) $M_{u,lim}$ □ $0.1389\ f_{ck}\, b\, d^2$

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

$\Rightarrow$ $M_{u,lim} =$ (0.1389 x 30 x 300 x 562^2 / 1000000) = 394.84 kNm

$\Rightarrow$ $p_{t,lim} =$ (41.3 x 30 / 415 x 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 kNm

$M_{u,lim} =$ 394.84 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 $A_{st,lim} = p_{t,lim} / 100 (b \times d)$

$\Rightarrow$ $A_{st,lim}$ (1.433 / 100 x 300 x 562) = 2416 mm^2

- Assuming 20 mm bars for compression steel,

d' ≈ (20 mm clear cover + 8 mm stirrup + 20 / 2) = 38 mm

$$\Delta A_{st} = \frac{M_u - M_{u,lim}}{0.87\, f_y \,(d - d')}$$

$$\frac{\Delta p_t}{100} = \frac{R - R_{lim}}{0.87\, f_y \left(1 - \frac{d'}{d}\right)}$$

Mu = 0.87*fy**Ast*d(1-(Ast*fy)/b*d*fck)

Ast Reqd = 623 mm^2

∴ No of tension bars required (#)
(623 / (Pi / 4 x 20^2) = 2.00

Actual percentage of steel , p_t (%)
(2 x Pi / 4 x 20 ^2 / 300 / 562 x 100) = 0.37

Actual area of steel , A_{st} (mm^2)
(2 x Pi / 4 x 20 ^2) = 628

Determining A_{sc}

The compression steel , A_{sc} , is given by

$$A_{sc} = \frac{(0.87\, f_y)(\Delta A_{st})}{f_{sc} - 0.447\, f_{ck}}$$

or

$$p_c = \frac{0.87\, f_y \,(p_t - p_{t,lim})}{f_{sc} - 0.447\, f_{ck}}$$

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.

Grade of steel		$\frac{d'}{d}$		
	0.05	**0.10**	**0.15**	**0.20**
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

- Assuming $x_u = xu_{,max}$, for d' / d = (38 / 562) = 0.068
  From the above table : by interpolation

Design Check

- To ensure $x_u \le x_{u,max}$, it suffices to establish $p_c \ge p_c^*$

where p_c^* is given by

$$p_c^* = \frac{0.87\ f_y}{f_{sc} - 0.447\ f_{ck}} \left(p_t - p_{t,lim} \right)$$

Actual p_t provided : p_t = 0.37

Actual p_c provided : p_c = 0.93

$\Rightarrow$ p_c^* = (0.87 x 415 x (0.373 - 1.433) / (354.98 - 0.447 x 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 :

$$\left(\frac{l}{d}\right)_{max} = \left(\frac{l}{d}\right)_{basic} F_1 F_2$$

where $\left(\frac{l}{d}\right)_{basic} = \begin{cases} 7 \text{ for cantilever spans} \\ 20 \text{ for simply supported spans} \\ 26 \text{ for continuous spans} \end{cases}$

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} = (0.58 x 415 x 963 / 628) = 369.08 N/mm^2

F = 1.00

F_1 = 1.26

F_2 = 1.24

∴ $(l / d)_{max}$ = (26 x 1 x 1.26 x 1.24) = 40.32

$(l / d)_{provided}$ = 9.79

⇒ *Hence O.K.*

Check for shear

Shear force at critical distance , V_{ud} (kN) 139

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 , τ_v

(139 x 1000 / (300 x 562)) = 0.82 N/mm^2

The maximum shear stress is given by : $Tc\max = 0.62\,fck$

⇒ $\tau_{c,max}$ (0.62 x Sqrt (30)) = 3.40 N/mm^2

⇒ *Adopted section is adequate*

- *Design shear resistance at critical section*

At critical section , A_{st} is given by 628 mm^2

Percentage of steel , p_t (%) 0.37

The design shear strength of the concrete , τ_c , is given by :

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1+5\beta}-1\right)}{6\beta}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89\, p_t}$ or 1 whichever is greater

For (M30 and Fe415)

⇒ τ_c = 0.44 N/mm^2

⇒ V_{uc} = (0.44 x 300 x 562 / 1000) = 74 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} = (139 - 74) = 65 kN

Using 8 mm bars and
No of legs 2

Area of stirrups , A_{sv} (mm^2) 101

⇒ required spacing sv ≤ (0.87 x 415 x 101 x 562 / (65.03 x 1000))

⇒ Spacing , s_v = 314 mm

Check whether $\tau_v > 0.5\ \tau_c$

Nominal shear stress , τ_v (N/mm^2) 0.82
Design shear stress , τ_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 τ_v exceeds 0.5 τ_c. :

$$\frac{A_{sv}}{b_{sv}} = \frac{0.4}{0.87\ fy}\ When\ sv = 0.5tc$$

$$sv = \frac{2.175\ fyA_{sv}}{b}$$

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

$$s_V \le \begin{cases} 0.75\ d \\ 300\ mm \end{cases}$$

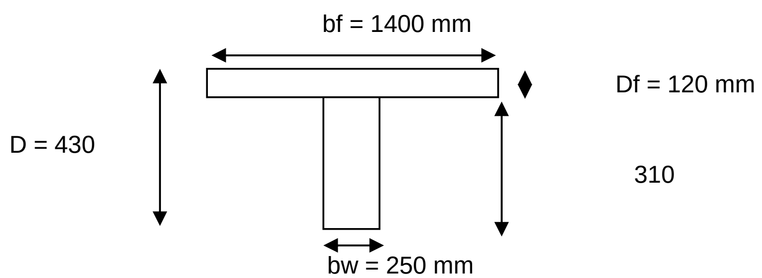
Code requirements for maximum spacing..

i)	<	(2.175 x 415 x 101 / 300) =	302	mm
ii)	≤	(0.75 x 562) =	422	mm
iii)	≤	300 mm	300	mm
iv)	≤	(0.87 x 415 x 101 x 562 / (65.03 x 1000)) =	314	mm

DESIGN OF SLAB

Design of Grid Slab

Grade of concrete		=	30	KN/m^2
Grade of steel		=	415	N/mm^2
Unit weight of concrete		=	24	kN/m^3
Live Load		=	3	kN/m^2
Cover		=	20	mm
Roof Finish Load		=	1	kN/m^2
L_y 8.36 m				
L_x 4.96 m				
Aspect Ratio	r	=	1.69	
		=	4	2
Grid Spacing		=		
	X - Dir	=	1.240	m
	Y - Dir	=	1.400	m
No of Beams in X - Direction		=	3	Nos
No of Beams in Y - Direction		=	5	Nos
Thickness of the Slab	Df	=	120	mm
Thickness of the Web	bw	=	250	mm
Depth of the Web	D	=	430	mm



Design of the Section :

Self weight of Slab		=	2.88	kN/m^2
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/m^2
Load per m^2	q	=	**8.9**	kN/m^2

Approximate Method (Moments)

If q1 & q2 are the moments shared in the x & y directions

$$q_1 = q\left(\frac{b_y{}^4}{a_x{}^4 + b_y{}^4}\right)$$

q1	=	**8.0**	kN/m^2

$$q_2 = q\left(\frac{a_x{}^4}{a_x{}^4 + b_y{}^4}\right)$$

q1	=	**1.0**

Moments in x & y directions at centre of grid for 2 m width is tained as :

$$Mx = \left(\frac{q_1 b_1 a^2}{8}\right)$$

Mx	=	**30.5**	kN.m

$$My = \left(\frac{q_2 a_1 b^2}{8}\right)$$

My	=	**12.1**	kN.m
Qx	=	**24.6**	kN
Qy	=	**5.8**	kN

Design of Reinforcement

Max working moment	Mw	=	**18.3**	kN.m/m
Moment resisted by central rib in x-direction over 1.24 m width		=	**22.7**	kN.m
Ultimate moment	Mu	=	**34.0**	kN.m

Moment capacity of flange section

$$M_{uf} = 0.36\, f_{ck}.b_f.D_f\,(d - 0.42 D_f)$$

Muf	=	**513.6**	kN.m

Mu<Muf 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**	mm^2
20	mm	=	1	Nos
	Ast Provided	=	**314**	mm^2
Max ultimate shear		=	**25**	kN
$\tau_v = \frac{V_u}{bd}$	τ_v	=	**0.3**	N/mm^2

Assuming 2 bars to be bent up near support

Ast at supports		=	**628**	mm^2
$\frac{100 A_{st}}{bd}$		=	**0.68**	
	τ_c	=	**0.56**	N/mm^2

Nominal Shear reinforcement is Required

8	mm	=	50	mm^2
No of stirrup legs	2	=	100.5	mm^2
$Sv = \frac{A_{sv} 0.87 f_y}{0.4 b}$	Sv	=	**363.0**	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/m^2
Grade of steel		=	415	N/mm^2
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

α_x+	0.069
α_y+	0.056
α_x-	
α_y-	

INTERPOLATION

1.2	1.16	1.3
0.072	**0.069**	0.079

M_x	=	1.55	KN-m				
M_y	=	1.26	KN-m				
Vux	=	6.14	KN				
Check for depth							
d	=	19.35	mm	d	=	110	mm
Total depth	=	39.35	mm				

Reinforcement (short and long span)

			Shorter span		Longer span	
	a	=	4.99		4.99	
	b	=	36105.00		36105.00	
	c	=	1549679.93		1258851.51	
	b^2-4ac	=	1272611364		1278421564	
	2a	=	9.99		9.99	
	SQ	=	35673.68		35755.02	
	Ast_1	=	43.18	mm^2	35.04	mm^2
	Ast_2	=	-7185.73627	mm^2	-7193.87945	

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

ζv = Vu / bd

= 0.0614 N/mm^2

Pt = 0.043

INTERPOLATION

0.5	0.04	0.75	Table19
0.48	**0.334**	0.56	

ζc = 0.334 N/mm^2

ζc > ζv 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/m^2
Grade of steel	=	415	N/mm^2
Live Load	=	4	kN/m2
Cover	=	20	mm
L_y 5.345 m			
L_x 3.6 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

α_x+ 0.092

α_y+ 0.056

α_x-

α_y-

INTERPOLATION

1.2	1.48	1.3
0.072	**0.092**	0.079

M_x	=	16.26	KN-m
M_y	=	9.90	KN-m
Vux	=	23.09	KN

Check for depth

d	=	62.67	mm	d	=	130 mm
Total depth	=	82.67	mm			

Reinforcement (short and long span)

		Shorter span		Longer span	
a	=	4.99		4.99	
b	=	46936.50		46936.50	
c	=	16258095.77		9903707.84	
b^2-4ac	=	1878229169		2005177767	
2a	=	9.99		9.99	
SQ	=	43338.54		44779.21	
Ast_1	=	360.19	mm^2	215.97	mm^2
Ast_2	=	-9037.40009	mm^2	-9181.625012	

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		150	mm c/c

Check for shear stress

Considering the short span & unit width of slab

ζv = Vu / bd

		=	0.1776	N/mm^2
	Pt	=	0.277	

INTERPOLATION

0.5	0.28	0.75	Table19
0.48	**0.409**	0.56	

ζc	=	0.409	N/mm^2

ζc > ζv Shear reinforcement is not reqd

Check for deflection Control

Modification Factor From IS 456

K = 1.2

BILL OF QUANTITIES

ARCHITECTURAL & STRUCTURAL WORKS						
Sl. No	**SOR Ref No**	**Description of Work**	**Unit**	**Quantity**	**Rate (Rs)**	**Amount (Rs)**
1.01	251a	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	cum	4,569.00	38.00	173,622.00
1.01a	254a	Extra for every additional 30m lead or part of 30m or for every additional 1.5m lift or part of 1.50m	cum	3,040.00	43.00	130,720.00
1.01b	254a	Extra for every additional 30m lead or part of 30m or for every additional 1.5m lift or part of 1.50m	cum	183.00	48.00	8,784.00
1.02	255a	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&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	cum	4,560.00	220.00	1,003,200.00
1.03	2.27 CPWD	Supplying and filling in plinth with Jamuna sand under floors including, watering, ramming consolidating and dressing complete.	cum	852.27	331.65	282,656.00
1.04	281	Cement concrete with 40mm gauge approved stone ballast, coarse sand& cement in the proportion of 8:4:1 including supply of all materials , labour, tools & plants etc. required for proper completion of the work.	cum	469.00	2,500.00	1,172,500.00
1.05	5.33 CPWD	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.	cum	21,148.00	4,147.40	87,709,215.00
1.05a	5.34.1 CPWD	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..**	cum	21,148.00	54.55	1,153,623.00
1.06	504	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.	MT	3,984.61	49,000.00	195,246,106.00
1.07		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.				
1.07a	303	In Foundation	cum	321.00	1,900.00	609,900.00
1.07b	310	Extra for Superstructure	cum	321.00	185.00	59,385.00
		Total Carried Forward				**287,549,711.00**

BILL NO.1 - ARCHITECTURAL & STRUCTURAL WORKS (Contd..)						
Sl. No	**SOR Ref No**	**Description of Work**	**Unit**	**Quantity**	**Rate (Rs)**	**Amount (Rs)**
		Total Brought Forward				**287,549,711.00**
1.08	13.7.2 CPWD	12 mm cement plaster finished with a floating coat of neat cement of mix :1:4 (1 cement: 4 fine sand)	sqm	1,604.00	97.90	157,032.00
1.09	13.8.2 CPWD	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)	sqm	1,604.00	110.70	177,563.00
1.10	13.16 CPWD	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	sqm	32,134.00	62.15	1,997,128.00
1.11	13.48.1 CPWD	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.	sqm	68,256.00	62.25	4,248,936.00
1.12	13.37.1 CPWD	White washing with whiting 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	sqm	32,134.00	6.75	216,905.00
1.13	11.9.5 CPWD	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.	sqm	8,566.00	313.35	2,684,156.00
1.14	16.64 CPWD	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.	sqm	8,208.00	63.15	518,335.00
1.15	12.15 CPWD	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	sqm	8,480.00	63.00	534,240.00
1.16	5.9.13 CPWD	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.	sqm	4,333.00	285.55	1,237,288.00
1.17	5.8 CPWD	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).	cum	432.00	3,928.75	1,697,220.00
1.18		Confirmatry Boreholes	Each	40.00	22,500.00	900,000.00
		Total Carried Forward				**301,918,514.00**

BILL NO.1 - ARCHITECTURAL & STRUCTURAL WORKS (Contd..)						
Sl. No	**SOR Ref No**	**Description of Work**	**Unit**	**Quantity**	**Rate (Rs)**	**Amount (Rs)**
		Total Brought Forward				**301,918,514.00**
1.19	5.9.1 CPWD	Centering and shuttering including strutting, propping etc. and removal of form for : Foundations, footings, bases of columns, etc. for mass concrete.	sqm	4,063.00	119.25	484,513.00
1.20	5.9.6 CPWD	Centering and shuttering including strutting, propping etc. And removal of form for : Columns, Pillars, Piers, Abutments, Posts and Struts.	sqm	5,714.00	238.40	1,362,218.00
1.21	5.9.3 CPWD	Centering and shuttering including strutting, propping etc. and removal of form for : Suspended floors, roofs, landings, balconies and access platform.	sqm	32,367.00	187.35	6,063,957.00
1.22	5.9.5 CPWD	Centering and shuttering including strutting, propping etc. and removal of form for : Lintels, beams, plinth beams, girders, bressumers and cantilevers.	sqm	52,948.00	162.65	8,611,992.00
1.23	5.9.7 CPWD	Centering and shuttering including strutting, propping etc. and removal of form for : Stairs, (excluding landings) except spiral-staircases.	sqm	1,394.00	204.00	284,376.00
1.24	5.27 CPWD	Providing and filling in position bitumen mix filler of Proportion 80 kg. of hot bitumen, 1 kg. of cement and 0.25 cubicmetre of coarse sand for expansion joints.	/cm depth / cm width / 100m	15.43	98.85	1,525.00
1.25	5.29.1.1 CPWD	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.	m	642.92	69.90	44,940.00
1.26	16.59.2 CPWD	Cautionary /warning sign boards of equilateral triangular shape having each side of 900mm with support length of 3650mm.	sqm	14.04	2,616.55	36,736.00
		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**

DETAILED QUANTITY ESTIMATE

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
1.01	251a	Excavation						cum	**4,569.00**		
		Column Raft									
			1	14.81	8.68	1.50			192.72	Raft	C1
			16	3.20	3.70	1.50			284.16	F2	C2
			5	3.30	3.60	1.50			89.10	F3	C3
			9	2.30	2.40	1.50			74.52	F4	C4
			1	2.80	3.00	1.50			12.60	F5	C5
			10	3.30	3.80	1.50			188.10	F6	C6
			38	6.00	6.00	1.50			2,052.00	F7	C7
			2	4.20	5.50	1.50			69.30	CF1	C9,C9
			10	5.40	6.60	1.50			534.60	CF2	C10,C10
			2	3.00	3.30	1.50			29.70	CF3	C4,C4
			8	3.95	7.20	0.80			182.02	CF4	C2,C7
			2	6.20	7.20	1.20			107.14	CF5	C10,C5
			1	8.20	10.70	1.10			96.51	CF6	C7,C8
			4	4.55	8.20	1.00			149.24	CF7	C7,C8
			2	6.20	7.20	1.70			151.78	CF8	C5,C10
			6	6.80	6.80	1.00			277.44	CF9	C7,C7
			2	6.20	6.20	1.00			76.88	CF10	C7,C7
		Stair case footing	1	1.50	0.90	0.75			1.01		
		Total							**4,568.81**		
1.01a	254a	Extra for every additional lift of 1.5m or part thereof in - all kinds of soil							**3,040.00**		
			1	14.81	8.68	1.25			160.60	Raft	C1
			16	3.20	3.70	1.25			236.80	F2	C2
			5	3.30	3.60	1.25			74.25	F3	C3
			9	2.30	2.40	1.25			62.10	F4	C4
			1	2.80	3.00	1.25			10.50	F5	C5
			10	3.30	3.80	1.25			156.75	F6	C6
			38	6.00	6.00	1.25			1,710.00	F7	C7
			2	4.20	5.50	1.50			69.30	CF1	C9,C9
			10	5.40	6.60	1.50			534.60	CF2	C10,C10
			2	3.00	3.30	1.25			24.75	CF3	C4,C4
		Total							**3,039.65**		
1.01b	254a	Extra for every additional lift of 1.5m or part thereof in - all kinds of soil							**183.00**		
			2	4.20	5.50	0.10			4.62		
			10	5.40	6.60	0.50			178.20		
		Total							**182.82**		
1.02	255a	Filling with good earth in plinth						cum	**4,560.00**		
			1	80.37	106.05	0.54			4,559.65		
		Total							**4,559.65**		
1.03	2.27 CPWD	River sand Filling							**852.27**		
			1	80.37	106.05	0.10			852.27		
		Total							**852.27**		
1.04	281	PCC 1:4:8						cum	**469.00**		
		Column Raft	1	14.81	8.68	0.10			12.85	Raft	C1
			16	3.20	3.70	0.15			28.42	F2	C2
			5	3.30	3.60	0.15			8.91	F3	C3
			9	2.30	2.40	0.15			7.45	F4	C4
			1	2.80	3.00	0.15			1.26	F5	C5
			10	3.30	3.80	0.15			18.81	F6	C6
			38	6.00	6.00	0.15			205.20	F7	C7
			2	4.20	5.50	0.15			6.93	CF1	C9,C9
			10	5.40	6.60	0.15			53.46	CF2	C10,C10
			2	3.00	3.30	0.15			2.97	CF3	C4,C4

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions Nos	Length	Width	Depth	Area	Unit	Total	Remarks	
			8	3.95	7.20	0.10			22.75	CF4	C2,C7
			2	6.20	7.20	0.10			8.93	CF5	C10,C5
			1	8.20	10.70	0.10			8.77	CF6	C7,C8
			4	4.55	8.20	0.10			14.92	CF7	C7,C8
			2	6.20	7.20	0.10			8.93	CF8	C5,C10
			6	6.80	6.80	0.10			27.74	CF9	C7,C7
			2	6.20	6.20	0.10			7.69	CF10	C7,C7
		Stair case footing	1	1.50	0.90	0.10			0.14		
		Below Pilnth beam	1	372.83	0.40	0.15			22.37		
		Total							**469.00**		
1.05	5.33 CPWD	M30 Concrete Building						cum	**21,148.00**		
		RCC M30 for all works upto Plinth level									
		RAFT	1	14.51	8.38	0.70			85.06	Raft	C1
			9	2.90	3.40	0.60			53.24	F2	C2
			9			0.74	5.07		33.77	F2	C2
			7	2.90	3.40	0.60			41.41	F2	C8
			7			0.74	5.26		27.25	F2	C8
			5	3.00	3.30	0.70			34.65	F3	C3
			5			0.70	5.22		18.27	F3	C3
			9	2.00	2.10	0.30			11.34	F4	C4
			9			0.36	2.36		7.65	F4	C4
			1	2.50	2.70	0.40			2.70	F5	C5
			1			0.54	3.67		1.98	F5	C5
			10	3.00	3.50	0.60			63.00	F6	C6
			10			0.70	5.67		39.69	F6	C6
			38	5.70	5.70	1.30			1,605.01	F7	C7
			38			1.30	16.85		832.39	F7	C7
			2	3.90	5.20	1.10			44.62	CF1	C5,C5
			2			1.10	10.63		23.39	CF1	C5,C5
			10	5.10	6.30	1.30			417.69	CF2	C10,C10
			10			1.30	16.88		219.46	CF2	C10,C10
			2	2.70	3.00	0.40			6.48	CF3	C4,C4
			2			0.47	4.46		4.19	CF3	C4,C4
			8	4.00	6.50	0.55			114.40	CF4	C2,C7
			2	5.25	6.50	0.55			37.54	CF5	C10(2),C5(2)
			1	8.00	10.50	1.00			84.00	CF6	C7(2),C8(2)
			4	4.00	10.50	1.00			168.00	CF7	C7,C8
			2	7.00	7.50	0.70			73.50	CF8	C5,C10
			6	6.60	6.60	0.90			235.22	CF9	C7,C7
			2	6.00	6.00	0.90			64.80	CF10	C7,C7
		Stair case footing	1	1.35	0.75	0.30			0.30		
		Pedestal									
			2	0.85	1.15	0.75			1.47	Raft	C10
			2	0.70	1.10	0.75			1.16	Raft	C1a
			6	0.53	0.53	0.75			1.26	Raft	C1
			9	0.65	1.10	0.75			4.83	F2	C2
			7	0.60	1.10	0.75			3.47	F2	C8
			5	0.60	0.90	0.75			2.03	F3	C3
			9	0.65	0.80	0.75			3.51	F4	C4
			1	0.65	0.90	0.75			0.44	F5	C5
			10	0.70	1.20	0.75			6.30	F6	C6
			38	1.10	1.10	0.75			34.49	F7	C7
			2	1.15	1.52	0.75			2.62	CF1	C5,C5
			10	1.15	1.42	0.75			12.25	CF2	C10,C10
			2	0.80	1.02	0.75			1.22	CF3	C4,C4
		Stair case pedestal	1	1.00	0.40	0.35			0.14		
		Columns upto Plinth Level	2	0.55	0.85	1.20			1.12	Raft	C10
			2	0.40	0.80	1.20			0.77	Raft	C1a
			6	0.23	0.23	1.20			0.38	Raft	C1
			9	0.35	0.80	0.66			1.66	F2	C2
			7	0.30	0.80	0.66			1.11	F2	C8

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Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
			5	0.30	0.60	0.60			0.54	F3	C3
			9	0.35	0.50	1.34			2.11	F4	C4
			1	0.35	0.60	1.06			0.22	F5	C5
			10	0.40	0.90	0.70			2.52	F6	C6
			38	0.80	0.80	-			-	F7	C7
			4	0.35	0.60	0.15			0.13	CF1	C5,C5
			20	0.55	0.85	0.55			5.14	CF2	C10,C10
			4	0.35	0.50	1.03			0.72	CF3	C4,C4
			8	0.35	0.80	0.15			0.34	CF4	C2
			8	0.80	0.80	0.15			0.77	CF4	C7
			4	0.35	0.60	0.15			0.13	CF5	C5(2)
			4	0.55	0.85	0.15			0.28	CF5	C10(2)
			2	0.80	0.80	0.15			0.19	CF6	C7(2)
			2	0.30	0.80	0.15			0.07	CF6	C8(2)
			4	0.80	0.80	0.15			0.38	CF7	C7
			4	0.30	0.80	0.15			0.14	CF7	C8
			4	0.35	0.60	0.15			0.13	CF8	C5
			4	0.55	0.85	0.15			0.28	CF8	C10
			12	0.80	0.80	0.15			1.15	CF9	C7,C7
			4	0.30	0.80	0.15			0.14	CF10	C7,C7
		Plinth Beam	2	80.60	0.25	0.60			24.18	PB1	
			12	80.60	0.25	0.60			145.07	PB1	
			12	106.05	0.25	0.60			190.89	PB1	
		Raft Beam	2	14.51	0.70	0.15			3.05	RFB1	
			2	8.38	0.40	0.15			1.01	RFB2	
			1	8.38	0.23	0.15			0.29	RFB3	
		Total							**4,812.00**		
		above basement upto 5th Floor hieght									
		For Walls, columns, pillars, posts and struts.									
		Column G.F	7	0.23	0.23	2.80			1.04		C1
			2	0.40	0.80	2.80			1.79		C1a
			17	0.35	0.80	2.80			13.33		C2
			5	0.30	0.60	2.80			2.52		C3
			13	0.35	0.50	2.80			6.37		C4
			13	0.35	0.60	2.80			7.64		C5
			10	0.40	0.90	2.80			10.08		C6
			68	0.80	0.80	2.80			121.86		C7
			13	0.30	0.80	2.80			8.74		C8
			32	0.55	0.85	2.80			41.89		C10
		First Floor	7	0.23	0.23	2.80			1.04		C1
			2	0.40	0.80	2.80			1.79		C1a
			17	0.35	0.80	2.80			13.33		C2
			5	0.30	0.60	2.80			2.52		C3
			13	0.35	0.50	2.80			6.37		C4
			13	0.35	0.60	2.80			7.64		C5
			10	0.40	0.90	2.80			10.08		C6
			68	0.80	0.80	2.80			121.86		C7
			13	0.30	0.80	2.80			8.74		C8
			32	0.55	0.85	2.80			41.89		C10
		Second Floor	7	0.23	0.23	2.80			1.04		C1
			2	0.40	0.80	2.80			1.79		C1a
			17	0.35	0.80	2.80			13.33		C2
			5	0.30	0.60	2.80			2.52		C3
			13	0.35	0.50	2.80			6.37		C4
			13	0.35	0.60	2.80			7.64		C5
			10	0.40	0.90	2.80			10.08		C6
			68	0.80	0.80	2.80			121.86		C7
			13	0.30	0.80	2.80			8.74		C8
			32	0.55	0.85	2.80			41.89		C10

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
		Third Floor	7	0.23	0.23	2.80			1.04		C1
			2	0.40	0.80	2.80			1.79		C1a
			17	0.35	0.80	2.80			13.33		C2
			5	0.30	0.60	2.80			2.52		C3
			13	0.35	0.50	2.80			6.37		C4
			13	0.35	0.60	2.80			7.64		C5
			10	0.40	0.90	2.80			10.08		C6
			68	0.80	0.80	2.80			121.86		C7
			13	0.30	0.80	2.80			8.74		C8
			32	0.55	0.85	2.80			41.89		C10
		Stair case columns									
			7	0.23	0.23	2.15			0.80		C1
			2	0.40	0.80	2.15			1.38		C1a
		Total							**864.00**		
		Roof Beam (Ground Floor, Ist, IInd IIIrd Floors)						Cum			
		Grid 2-11& L-J									
		B1	1	69.68	0.40	0.78			21.74		
		B2	2	69.68	0.80	0.78			86.95		
		B2	8	14.14	0.80	0.78			70.60		
		B3	1	14.14	0.30	0.78			3.31		
		B6	8	4.56	0.25	0.43			3.92		
		B4	1	14.14	0.35	0.78			3.86		
		B7	42	14.14	0.25	0.43			63.85		
			8	65.12	0.25	0.43			56.00		
		Grid 1-3 & G-F									
		B2	2	15.25	0.80	0.78			19.03		
			1	10.80	0.80	0.78			6.74		
		B3	2	10.80	0.30	0.78			5.05		
		B7	7	15.25	0.25	0.43			11.47		
		B7	10	10.80	0.25	0.43			11.61		
		Grid 2-11& A-B									
		B2	1	69.68	0.80	0.78			43.48		
		B2	8	5.47	0.80	0.78			27.30		
		B3	1	5.47	0.30	0.78			1.28		
		B4	1	5.47	0.35	0.78			1.49		
		B5	1	69.68	0.35	0.78			19.02		
		B6	3	4.56	0.25	0.43			1.47		
		B7	36	5.47	0.25	0.43			21.16		
			3	65.12	0.25	0.43			21.00		
		Grid 1-2& K-L									
		B1	1	10.69	0.40	0.78			3.34		
		B2	2	8.36	0.80	0.78			10.44		
			1	10.69	0.80	0.78			6.67		
		B6	5	10.69	0.25	0.43			5.75		
		B7	6	8.36	0.25	0.43			5.39		
		Grid 1-2& A-B									
		B2	1	10.69	0.80	0.78			6.67		
		B2	2	5.47	0.80	0.78			6.82		
		B6	3	10.69	0.25	0.43			3.45		
		B7	6	5.47	0.25	0.43			3.53		
		Grid 1'-2& K-J									
		B2	1	11.13	0.80	0.78			6.94		
		B6	3	5.35	0.25	0.43			1.72		
		B7	3	5.78	0.25	0.43			1.86		
		Grid 3-4& B-J									
		B2	5	10.22	0.80	0.78			31.89		
			7	10.80	0.80	0.78			47.17		
		B2A	4	10.22	0.55	0.78			17.54		
		B7	48	10.80	0.25	0.43			55.73		
			56	10.22	0.25	0.43			61.52		

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
		Grid 1'-3& G-J,B-F									
		B2	3	9.90	0.80	0.78			18.53		
			7	10.80	0.80	0.78			47.17		
		B2A	4	9.90	0.55	0.78			16.99		
		B7	49	9.90	0.25	0.43			52.15		
			49	10.80	0.25	0.43			56.89		
		B3	7	10.80	0.30	0.78			17.69		
		Grid 10-11& B-J									
		B2	8	10.80	0.80	0.78			53.91		
			5	4.48	0.80	0.78			13.98		
		B2A	4	4.48	0.55	0.78			7.69		
		B7	16	10.80	0.25	0.43			18.58		
			56	4.48	0.25	0.43			26.97		
		B4	8	10.80	0.35	0.78			23.59		
		For Span 10.8x8.4									
		B2	24	8.40	0.80	0.78			125.80		
		B2	48	10.80	0.80	0.78			323.48		
			24	8.40	0.80	0.78			125.80		
		B7	168	8.40	0.25	0.43			151.70		
			120	10.80	0.25	0.43			139.32		
		For Span 10.8x8.4									
		B2	24	8.40	0.80	0.78			125.80		
		B2	48	10.80	0.80	0.78			323.48		
		B2A	24	8.40	0.55	0.78			86.49		
		B7	168	8.40	0.25	0.43			151.70		
			120	10.80	0.25	0.43			139.32		
		Lift Room Beam									
		MRB1	4	1.77	0.23	0.30			0.49		
		MRB2	1	8.41	0.23	0.45			0.87		
		MRB3	1	3.83	0.23	0.40			0.35		
		MRB4	1	8.41	0.30	0.80			2.02		
		Total							**2,827.52**		
		For lift room upper roof beam (2.5m above IIIrd floor)									
		LRB1	1	43.21	0.23	0.23			2.24		
		For ramp portion									
		Entry ramp (To I- Floor ,									
		RB1	4	5.35	0.30	0.35			2.24		
		RB2	6	5.35	0.30	0.25			2.41		
		RB4	1	32.83	0.30	0.45			4.43		
		RB3	1	32.83	0.30	0.70			6.89		
		Exit ramp (from I- Floor ,									
		RB1	7	5.35	0.30	0.35			3.93		
		RB2	5	5.35	0.30	0.25			2.00		
		RB3	1	43.15	0.30	0.70			9.06		
		RB4	1	43.15	0.30	0.45			5.83		
		Entry ramp (To II- Floor ,									
		RB1	5	5.35	0.30	0.35			2.81		
		RB2	4	5.35	0.30	0.25			1.60		
		RB3	1	32.42	0.30	0.70			6.81		
		RB4	1	32.42	0.30	0.45			4.38		
		Exit ramp (from II- Floor ,									
		RB1	6	5.35	0.30	0.35			3.37		
		RB2	4	5.35	0.30	0.25			1.60		
		RB3	1	37.34	0.30	0.70			7.84		
		RB4	1	37.34	0.30	0.45			5.04		
		Entry ramp (To III- Floor ,									
		RB1	5	5.35	0.30	0.35			2.81		
		RB2	4	5.35	0.30	0.25			1.60		
		RB3	1	32.42	0.30	0.70			6.81		
		RB4	1	32.42	0.30	0.45			4.38		
		Exit ramp (from III- Floor ,									

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
		RB1	6	5.35	0.30	0.35			3.37	
		RB2	4	5.35	0.30	0.25			1.60	
		RB3	1	37.34	0.30	0.70			7.84	
		RB4	1	37.34	0.30	0.45			5.04	
		Stair case beam								
		SB1	9	1.70	0.25	0.15			0.57	
			5	1.00	0.25	0.15			0.19	
		Steps	54	1.70			0.02		1.72	
			18	1.00			0.02		0.34	
		Total For Three Floors							**11,419.00**	
		Roof Slab (Ground Floor, Ist, IInd IIIrd Floors)								
			1	75.02	106.05	0.12			954.70	
			1	5.35	8.36	0.12			5.36	
			1	5.35	10.81	0.12			6.93	
			1	5.35	11.33	0.12			7.27	
		deduction for lift room	1	8.41	3.60	0.12			(3.63)	
		Total							**970.64**	
		deduction for ground slab								
			1	5.35	41.32	0.12			(26.50)	
		for lift room III rd floor	1	3.70	1.77	0.12			0.79	
		For lift room upper roof	1	8.41	3.60	0.12			3.63	
		For ramp Slab								
		Entry ramp (To I- Floor ,								
		Grid 1'-2& G'-K	1	38.44	5.35	0.15			30.82	
		Exit ramp (from I- Floor ,								
		Grid 1-1'& B'-G	1	43.53	5.35	0.15			34.90	
		Entry ramp (To II- Floor , Grid 1-2 & G-L)								
		Grid 1-1' & K-G'	1	38.35	5.35	0.15			30.75	
		Exit ramp (from II- Floor , Grid 1-1'& A-G)								
		Grid 1-1'& B'-E"	1	32.49	5.35	0.15			26.05	
		Entry ramp (To III- Floor , Grid 1-2 & G-L)								
		Grid 1-1' & K-G'	1	38.35	5.35	0.15			30.75	
		Exit ramp (from III- Floor , Grid 1-1'& A-G)								
		Grid 1-1'& B'-E"	1	38.35	5.35	0.15			30.75	
		Stair case Slab	1	26.50	1.50	0.15			5.96	
		Landing Slab	9	1.70	1.00	0.15			2.30	
		Total For Three Floors							**4,053.00**	
1.05a	5.34.1 CPWD	Add or deduct for providing richer or leaner mixes							**21,148.00**	
1.06	504	**HYSD Steel**						MT	**3,984.61**	
	Footing			50.00	kg/cum				142.89	
	Combined footing			100.00	kg/cum				149.33	
	Raft Footing			200.00	kg/cum				17.01	
	Column	upto 1st Floor								
	Stair pedestal			165.00	kg/cum				0.02	
	C1			315.00	kg/cum				0.84	
	C2			188.00	kg/cum				3.79	
	C1a			165.00	kg/cum				0.61	
	C3			284.00	kg/cum				1.44	
	C4			291.00	kg/cum				3.70	
	C5			208.00	kg/cum				2.35	

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
	C6			140.00	kg/cum				2.65		
	C7			342.00	kg/cum				54.37		
	C8			218.00	kg/cum				2.95		
	C10			220.00	kg/cum				13.73		
	1st floor & IInd floor										
	C1			315.00	kg/cum				0.65		
	C2			188.00	kg/cum				5.01		
	C1a			165.00	kg/cum				0.59		
	C3			284.00	kg/cum				1.43		
	C4			291.00	kg/cum				3.71		
	C5			208.00	kg/cum				3.18		
	C6			140.00	kg/cum				2.82		
	C7			342.00	kg/cum				83.35		
	C8			218.00	kg/cum				3.81		
	C10			220.00	kg/cum				18.43		
	IIIrd Floor										
	C1			315.00	kg/cum				0.58		
	C2			188.00	kg/cum				2.51		
	C1a			165.00	kg/cum				0.52		
	C3			284.00	kg/cum				0.72		
	C4			291.00	kg/cum				1.85		
	C5			208.00	kg/cum				1.59		
	C6			140.00	kg/cum				1.41		
	C7			342.00	kg/cum				41.67		
	C8			218.00	kg/cum				1.90		
	C10			220.00	kg/cum				9.22		
	Plinth Beam PB1			210.00	kg/cum				75.63		
	Roof Beam										
	B1			245.00	kg/cum				24.57		
	B2			200.00	kg/cum				1,217.53		
	B2A			235.00	kg/cum				120.98		
	B3			240.00	kg/cum				26.24		
	B4			215.00	kg/cum				24.89		
	B5			215.00	kg/cum				16.36		
	B6			186.00	kg/cum				12.13		
	B7			235.00	kg/cum				986.78		
	Ramp Beam										
	RB1			205.00	kg/cum				3.80		
	RB2			245.00	kg/cum				2.65		
	RB3			185.00	kg/cum				8.37		
	RB4			185.00	kg/cum				5.38		
	Raft Beam			1,500.00	kg/cum				6.51		
	Lift roof beams										
	MRB1			215.00	kg/cum				2.34		
	MRB2			185.00	kg/cum				0.64		
	MRB3			180.00	kg/cum				0.25		
	MRB4			180.00	kg/cum				1.45		
	stair case beams			245.00	kg/cum				0.69		
	Slab			200.00	kg/cum				810.60		
	RCC Parapet wall			130.00	kg/cum				56.16		
	Total								**3,984.61**		
1.07		Brick work									

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
1.07a	303	**Ground Floor**						cum	**321.00**		
			2	80.37	0.20	2.80			90.01		
			2	106.05	0.20	2.80			118.78		
		Parapet wall on terrace	2	80.37	0.20	1.50			48.22		
			2	106.05	0.20	1.50			63.63		
		Total							**321.00**		
1.07b	310	Extra for brick work above plinth upto floor five level							**321.00**		
		Qty as the BW of all works upto plinth level							321.00		
		Total							**321.00**		
1.08	13.7.2 CPWD	Plastering with CM 1:4 mix (one cement and 4 sand) 12mm thick For outside walls						Sqm	**1,604.00**		
		Ground Floor	2	80.37		2.80			450.04		
			2	106.05		2.80			593.88		
		Parapet wall on terrace	2	80.37		1.50			241.10		
			2	106.05		1.50			318.15		
		Total							**1,604.00**		
1.09	13.8.2	Plastering with CM 1:4 mix						Sqm	**1,604.00**		
		Ground Floor									
			2	80.37		2.80			450.04		
			2	106.05		2.80			593.88		
		Parapet wall on terrace	2	80.37		1.50			241.10		
			2	106.05		1.50			318.15		
		Total							**1,604.00**		

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions: Nos	Length	Width	Depth	Area	Unit	Total	Remarks
1.10	13.16 CPWD	1:3 Plastering for Ceiling						Sqm	**32,134.00**	
		Roof Slab (Ground Floor, Ist, IInd IIIrd Floors)								
			1	75.02	106.05				7,955.87	
			1	5.35	8.36				44.69	
			1	5.35	10.81				57.78	
			1	5.35	11.33				60.55	
		deduction for lift room	1	8.41	3.60				(30.23)	
		Total							**8,088.66**	
		deduction for ground slab on grid 1'-2 & G'-L	1	5.35	41.32				(220.87)	
		for three floors							**32,134.00**	
1.11	13.48.1 CPWD	Two coats of exterior type emulsion paint							**68,256.00**	
		Ground Floor								
			2	80.37		3.60			578.63	
			2	106.05		3.60			763.56	
		First Floor	2	80.37		1.00			160.73	
			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							**3,020.00**	
		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	
			4	32.68		1.00			130.72	
			4	32.24		1.00			128.96	
		Third Floor								
			2	80.37		1.00			160.73	
			2	106.05		1.00			212.10	
			2	32.68		1.00			65.36	
			2	32.24		1.00			64.48	
			4	80.37		1.50			482.19	
			4	106.05		1.50			636.30	
		Parapet wall on terrace	2	80.37		1.50			241.10	
			2	106.05		1.50			318.15	
		For beams & 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	

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		First Floor	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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Second Floor	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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Third Floor	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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Stair case columns	7	0.92		2.15			13.85	
			2	2.40		2.15			10.32	
		Beams								
		B1	4	80.37		1.56			501.48	
		B2	4	2,248.16		1.56			14,028.52	
		B2A	4	300.00		1.56			1,872.00	
		B3	4	116.81		1.56			728.89	
		B4	4	106.01		1.56			661.50	
		B5	4	69.68		1.56			434.77	
		B6	4	151.66		0.86			521.71	
		B7	4	9,765.27		0.86			33,592.52	
		Ramp Beams								
		RB1	1	176.39		0.70			123.47	
		RB2	1	144.32		0.50			72.16	
		RB3	1	215.50		1.40			301.70	
		RB4	1	215.50		0.90			193.95	
		Stair case beam								
		SB1	1	20.30		0.30			6.09	
		Steps	1	109.80		0.40			43.92	
		Lift Room Beam								
		MRB1	4	7.08		0.60			16.99	
		MRB2	4	8.41		0.90			30.28	
		MRB3	4	3.83		0.80			12.26	
		MRB4	4	8.41		1.60			53.82	
		For lift room upper roof beam (2.5m above IIIrd floor)								
		LRB1	1	43.21		0.45			19.44	
		Ramp Slab								
			1	38.44	5.35				205.46	
			1	43.53	5.35				232.69	
			1	38.35	5.35				204.98	
			1	32.49	5.35				173.66	
			1	38.35	5.35				204.98	

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
			1	38.35	5.35				204.98	
		Stair case Slab	1	26.50	1.50				39.75	
		Landing Slab	18	1.70	1.00				30.60	
		Total							**65,236.00**	
1.12	13.37.1 CPWD	Two coats of cement paint(white) for ceiling						Sqm	**32,134.00**	
									32,134.00	
		Total							**32,134.00**	
1.13	11.9.5 CPWD	40mm thick marble chip flooring							**8,566.00**	
			1	80.60	106.28				8,565.64	
		PCC 1:2:4 Flooring						Cum	**821.00**	
			1	80.60	106.28	0.10			856.56	
		Deduction for plinth Beams								
			2	80.60	0.25	0.10			(4.03)	
			12	106.05	0.25	0.10			(31.82)	
		Total							**821.00**	
		Chequers of approved pattern on floors						Sqm	**8,207.00**	
			1	80.60	106.28				8,565.64	
			2	80.60	0.25				(40.30)	
			12	106.05	0.25				(318.15)	
		Total							**8,207.00**	
1.14	16.64 CPWD	40 mm brick Aggregates below 1:2:4 pcc Flooring						Sqm	**8,208.00**	
			1	80.60	106.28				8,565.64	
		Deduction for plinth Beams							-	
			2	80.60	0.25				(40.30)	
			12	106.05	0.25				(318.15)	
		Total							**8,208.00**	
1.15	12.15 CPWD	Painting with bitumen on terrace						Sqm	**8,480.00**	
			1	80.14	105.82				8,479.89	
		Total							**8,480.00**	
1.16	5.9.13 CPWD	Shuttering of parapet wall							**4,333.00**	
		Entry ramp parapet	4	32.83		1.00			131.30	
		Exit ramp parapet	4	42.63		1.00			170.50	
		First Floor	4	80.37		1.00			321.46	
			4	106.05		1.00			424.20	
			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							-	
			4	80.37		1.00			321.46	
			4	106.05		1.00			424.20	
			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	
		Third Floor							-	
			4	80.37		1.00			321.46	
			4	106.05		1.00			424.20	
			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	
			4	0.20		1.00			0.80	

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			2	0.20		1.00			0.40	
			2	0.20		1.00			0.40	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			2	0.20		1.00			0.40	
			2	0.20		1.00			0.40	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
			2	0.20		1.00			0.40	
			2	0.20		1.00			0.40	
			4	0.20		1.00			0.80	
			4	0.20		1.00			0.80	
		Total							**4,333.00**	
1.17	5.8 CPWD	Parapet Wall RCC M30							**432.00**	
		Entry ramp parapet	2	32.83	0.20	1.00			13.13	
		Exit ramp parapet	2	42.63	0.20	1.00			17.05	
		First Floor							-	
			2	80.37	0.20	1.00			32.15	
			2	106.05	0.20	1.00			42.42	
			1	72.99	0.20	1.00			14.60	
			1	93.79	0.20	1.00			18.76	
		Entry ramp parapet	2	32.68	0.20	1.00			13.07	
		Exit ramp parapet	2	32.24	0.20	1.00			12.90	
		Second Floor							-	
			2	80.37	0.20	1.00			32.15	
			2	106.05	0.20	1.00			42.42	
			1	72.99	0.20	1.00			14.60	
			1	93.79	0.20	1.00			18.76	
		Entry ramp parapet	2	32.68	0.20	1.00			13.07	
		Exit ramp parapet	2	32.24	0.20	1.00			12.90	
		Third Floor								
			2	80.37	0.20	1.00			32.15	
			2	106.05	0.20	1.00			42.42	
			1	72.99	0.20	1.00			14.60	
			1	93.79	0.20	1.00			18.76	
		Entry ramp parapet	2	32.68	0.20	1.00			13.07	
		Exit ramp parapet	2	32.24	0.20	1.00			12.90	
									432.00	

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks	
			Nos	Length	Width	Depth	Area				
		Shuttering Works									
1.19	5.9.1 CPWD	For Foundation							**4,063.00**		
		RAFT FOUNDATION									
			1	45.77		0.70			32.04		
			9	12.60		1.34			151.96		
			7	12.60		1.34			118.19		
			5	12.60		1.40			88.20		
			9	8.20		0.66			48.71		
			1	10.40		0.94			9.78		
			10	13.00		1.30			169.00		
			38	22.80		2.60			2,252.64		
			2	18.20		2.20			80.08		
			10	22.80		2.60			592.80		
			2	11.40		0.87			19.84		
			8	21.00		0.55			92.40		
			2	23.50		0.55			25.85		
			1	37.00		1.00			37.00		
			4	29.00		1.00			116.00		
			2	29.00		0.70			40.60		
			6	26.40		0.90			142.56		
			2	24.00		0.90			43.20		
			1	4.20		0.30			1.26		
		Total							**4,063.00**		
1.20	5.9.6 CPWD	For Columns & struts							**5,714.00**		
			2	4.00		0.75			6.00		
			2	3.60		0.75			5.40		
			6	2.12		0.75			9.54		
			9	3.50		0.75			23.63		
			7	3.40		0.75			17.85		
			5	3.00		0.75			11.25		
			9	2.90		0.75			19.58		
			1	3.10		0.75			2.33		
			10	3.80		0.75			28.50		
			38	4.40		0.75			125.40		
			2	5.34		0.75			8.01		
			10	5.14		0.75			38.55		
			2	3.64		0.75			5.46		
			1	2.80		0.35			0.98		
			2	2.80		1.20			6.72		
			2	2.40		1.20			5.76		
			9	2.30		0.66			13.66		
			7	2.20		0.66			10.16		
			5	1.80		0.60			5.40		
			9	1.70		1.34			20.50		
			1	1.90		1.06			2.01		
			10	2.60		0.70			18.20		
			4	1.90		0.15			1.14		
			20	2.80		0.55			30.80		
			4	1.70		1.03			7.00		
			8	2.30		0.15			2.76		
			8	3.20		0.15			3.84		
			4	1.90		0.15			1.14		
			4	2.80		0.15			1.68		
			2	3.20		0.15			0.96		
			2	2.20		0.15			0.66		
			4	3.20		0.15			1.92		
			4	2.20		0.15			1.32		
			4	1.90		0.15			1.14		
			4	2.80		0.15			1.68		
			12	3.20		0.15			5.76		
			4	2.20		0.15			1.32		

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
		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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		First Floor		-		-			-	
			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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Second Floor	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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Third Floor								
			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	
			10	2.60		2.80			72.80	
			68	3.20		2.80			609.28	
			13	2.20		2.80			80.08	
			32	2.80		2.80			250.88	
		Stair case columns				-				
						-				
			7	0.92		2.15			13.85	
			2	2.40		2.15			10.32	
		Total							**5,714.00**	
1.21	5.9.3 CPWD	For Slabs							**32,367.00**	
			1	75.02	106.05				7,955.87	
			1	5.35	8.36				44.69	
			1	5.35	10.81				57.78	
			1	5.35	11.33				60.55	
			1	362.14	0.12				43.46	
		deduction for lift room	1	8.41	3.60				(30.23)	
		Total							**8,132.12**	
		deduction for ground slab on grid 1'-2 & G'-L	1	5.35	41.32				(220.87)	
			1	3.70	1.77				6.55	
			1	8.41	3.60				30.23	
		Landing Slab	9	1.70	1.00				15.30	
			9	5.40	0.15				7.29	
									32,367.00	
1.22	5.9.5 CPWD	For Beams							**52,948.00**	
		B1	1	80.37		1.96			157.52	
		B2	1	2,248.16		2.36			5,305.66	
		B3	1	116.81		1.86			217.27	

ARCHITECTURAL & STRUCTURAL WORKS

Item No	Code No.	Description	Dimensions					Unit	Total	Remarks
			Nos	Length	Width	Depth	Area			
		B4	1	106.01		1.91			202.48	
		B5	1	69.68		1.91			133.08	
		B6	1	151.66		1.11			168.34	
		B7	1	9,765.27		1.11			10,839.45	
		MRB1	1	7.08		0.83			5.88	
		MRB2	1	8.41		1.13			9.50	
		MRB3	1	3.83		1.03			3.94	
		MRB4	1	8.41		1.90			15.98	
		Total							**17,059.09**	
		LRB1	1	43.21		0.68			29.38	
		Plinth Beam	1	1,433.79		1.20			1,720.55	
			28	0.25		0.60			4.20	
		Raft beam	2	14.51		0.30			8.71	
			2	8.38		0.30			5.03	
			1	8.38		0.30			2.51	
		Total for three floors							**52,948.00**	
1.23	5.9.7 CPWD	For ramp Slab							**1,394.00**	
			1	38.44	5.35				205.46	
			1	43.53	5.35				232.69	
			1	38.35	5.35				204.98	
			1	32.49	5.35				173.66	
			1	38.35	5.35				204.98	
			1	38.35	5.35				204.98	
			2	229.51	0.20				91.80	
		Stair case								
			1	26.50	1.50				39.75	
			1	53.00	0.15				7.95	
		SB1	1	20.30		0.55			11.17	
		Steps	72	1.50		0.15			16.20	
		Total							**1,394.00**	
1.24	5.27 CPWD	Expansion Joint						per cm depth	**15.43**	
		total length for three floors	2	80.37				per cm	642.92	
		Total						width	**15.43**	
								per 100m		
1.25	5.29.1.1 CPWD	Sheet covering							**642.92**	
			2	80.37					642.92	
1.26	16.59.2 CPWD	Sign boards						Sqm	**14.04**	
			10				0.35		14.04	

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