Certification

Palestine Polytechnic University

PPU Campus

This Project Written By

Ayman Ersheid
Yousif AL-Ja'bary
Mansor AL-Sa'di
Zaher Mahmoud

Under the direction of the supervisors and approved by all examining committee members has been presented for and accepted by the chairman of civil engineering dept.
In partial fulfillment of the requirements for the degree of Bachelor of engineering

Project Supervisor

Dr.Arch.Ghassn AL-Dweik
Dr.Eng.Maher Amro

Department Chairman

Dr.Eng.Majed Abu-Sharkh

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List of Abbreviations

- **Ac** = area of concrete section resisting shear transfer.
- **As** = area of nonprestressed tension reinforcement.
- Ag = gross area of section.
- Av = area of shear reinforcement within a distance (S).
- At = area of one leg of a closed stirrup resisting torsion within a (S)
- b = width of compression face of member.
- bw = web width, or diameter of circular section.
- DL = Dead loads.
- d = distance from extreme compression fiber to Centroid of tension
- Ec = modulus of elasticity of concrete.
- Fy = specified yield strength of non prestressed reinforcement.
- h = overall thickness of member.
- I = moment of inertia of section resisting externally applied factored loads
- Ln = length of clear span in long direction of two- way construction, measured face-to-face of supports in slabs without beams and face to face of beam or other supports in other cases.
- LL = live loads, or related internal moments and forces.
- Ld = development length.
- M = bending moment .
- Mu = Ultimate moment .
- Mn = Nominal moment.
- Pn = nominal axial load
- S = Spacing of shear or in direction parallel to longitudinal reinforcement.
- T c = nominal tensional concrete moment strength provided by concrete.
- Tn = nominal torsional moment strength.
- T s = nominal torsional moment strength provided by torsion reinforcement
- Vc = nominal shear strength provided by concrete.
- Vn = nominal shear stress.

- **Vs** = nominal shear strength provided by shear reinforcement.
- **V** $_u$ – factored shear force at section.
- **Wc** = weight of concrete, kg/m3.
- **W** = wind load, or related internal moments.
- **W** = Width of beam or rib.
- **Wu** = factored load per unit area.
- **X** = shorter overall dimension of rectangular part of cross section.
- **Y** = longer overall dimension of rectangular part of cross section.
- **Φ** = strength reduction factor.

Abstract

The main goal of this project is to design the library building after determining its location on campus of PPU in two sides architectural and structural side to execute that in the stage one of construction of the campus.

In this project, there are seven chapters to satisfy all the aims of the project Chapter one is an introduction that gives us guidelines for the project while chapter two and three are for campus design in analytical and architectural studies .

The other chapters are divided into two main sides ; architectural and structural for the library design building.

In the architectural design, the site of the library was determined according to the site of campus. The architectural drawings were prepared after studying all of the architectural requirements for all parts of library areas.

In the structural design, a structural study was made to determine what members to design and all structural requirements that must be considerd in this project.

The structural analyses were calculated at the end according to the ACI code for all of the structural members from roof to footings. Then structural drawings were prepared to illustrate the distrubution of steel in structural members

Chapter 1

INTRODUCTION

CHAPTER 1

INTRODUCTION

The purpose of this chapter is to provide guidelines to the readers for all contents of this project in both sides, the architectural and the structural design. This chapter includes information of the reasons for choosing such a project and its objectives, and the methodology to satisfy its required aims .

1.1 General

This project is divided into two branches:

First is a primary study of the PPU campus and its architectural design including site planning.

Second is the architectural and structural design for the suggested library building of a university campus with its selected site.

This research is a primary study of the different aspects of the PPU campus design. The current needed area amounts to 46400m² including the administration, classrooms, students' restaurant, and gymnasium and activities hall. However, the required area for the coming 10 years is estimated to be 64400m².

The objective of this research is to embody the feasibility of providing the necessary buildings to absorb the university facilities, such as laboratories and workshops at one site. The reason is to avoid the recurring transportation of the students and professors among the different 5 sites dispersing in Hebron. The project is to provide proper buildings that meet the standards and specifications of educational buildings, and consequently fitting a comfortable studying atmosphere for both students and professors. The second reason is to avoid the current and future leases of those 5 buildings so that leasing fees can be a budget for developing all facilities and requirements of the campus.

Most of the PPU buildings are leased and located are not united buildings. Not to mention that those buildings are designed as domiciles, which results in a loss of 25% of the total area. Also this causes a financial burden in addition to the lease and transportation fees. The meetings with both professors and students showed that the principal obstacle in developing the PPU is the unavailability of proper educational buildings.

On the other hand, the design of the university PPU library, which is selected from a campus, depends on several factors that affect the design, for example, the architectural study of the required area, the possibilities of expansion, and the shape of library must be similar to real campus buildings.

Other essential factor in this project is, a structural study of library building that contains the design of all structural members according to the ACI code and the required structural drawings.

Finally, this project is an engineering procedure to solve all problems in both side; campus design and library design, in architectural & structural design criteria.

1.2 Problems

There are some reasons for choosing this project.

The main two reasons are there is no one comprehensive campus for the PPU, and the other reason is to design a specific building (library) in two aspects, architectural and structural design.

1.2.1 Reasons for choosing this Project for campus

1. 6o% of the current PPU buildings area is leased from private sector against 8% of the PPU budget.
2. There are no accommodated buildings; the current total area of the PPU buildings is 11000m² approximately while the actual needed area is 46400m² in order to absorb the present facilities and equipment as per the architectural standards. Add to that the improperness of such buildings especially the lecture-rooms for academic and educational functions.
3. The PPU campus is divided into diverse buildings, which causes a difficulty in transportation; this project is to save time, exertion and money for both students and professors.
4. The restriction in the students' number due to the limited area; the area must be increased in order to be able to absorb the students applying to PPU for its distinguished curricula.

1.2.2 Reasons for choosing this Project for library

1. The Library is a central building in a university campus.
2. A new kinds of architectural design with different functional requirements.
3. To design that architectural and structural
4. To solve the problem for students for needed more areas to reading and continuing their studies.

1.3 Objectives

The objectives of this project are divided into two types ,main and minor objectives. The main objectives are to design a university campus for PPU in Halhol city, and to design a library building of the university from two sides, architectural and structural sides.

There are minor objectives that can be summarized as follows:

1.3.1 The site planning of the campus of the PPU to meet all the requirements.

1.3.2 Architectural and structural drawings.

1.4 Scope

In this project, there are specific procedures to satisfy all the previous objectives, The following are the steps of this project:

1.4.1 Making analytical study for the project's site aspects to get the best results for the site planning together with a comprehensive architectural and geological study for the site.

1.4.2 Concerning the architectural planning, the researchers will make a design for the faculty and the designs for site planning, leveling, views, facades and sectioning after choosing the faculty place on the site plan.

1.4.3 About the constructional planning, the researchers will analyze all requirements of the building and some of the fundamental constructional elements.

1.4.4 Determining the architectural and constructional requirements for planning the Main Library.

1.4.5 Designing all architectural drawing of the library building including plans, elevations, sections, site plan and other important architectural details

1.4.6 Structural analysis are making to start all structural calculations and design of all structural drawing that illustrate the design of structural members.

This project is reported in seven chapters including:

Chapter one: discusses a general information of the project, problems, objectives and scope.

Chapter two: is an analytical study of a master plan campus, including determine students number, kinds of colleges and building and areas required.

Chapter three: introduces to architectural study of a master plan, this chapter contains some important factors that take considerably before designing campus such as functional requirements and other factors.

Chapter four will discuss the architectural study of the library building design, where the design criteria and areas required will be shown, and all architectural drawing are design in this chapter.

Chapter five will discuss the structural requirements, load cases stability of building, And the selection design member.

Chapter six will show all structural calculation of all structural members, with tables and structural key drawings.

Chapter seven is a chapter contains all conclusions and design recommendations of this project.

Ch

2

ANALYTICAL STUDY

2.1 Analyzing And Determining The Students' Number

2.2 Analyzing And Determining The Professors' Number

2.3 Adopted mathematical way in calculating the required space

2.4Construction stages for campus project

CHAPTER 2

ANALYTICAL STUDY

It is essential that all detailed analysis of existing buildings in a campus be performed according to the same basic set of rules. In order for the university to receive complete and consistent collections of information, that assist in designing of all parts of campus facilities, the following "analytical studies" will be used.

2.1 *Analyzing And Determining The Students' Number*

As the PPU does not have such statistics, it was necessary to find a way to estimate the numbers of students and professors and needed areas in order to consider them in the final design.

A questionnaire was given to each dean of the 4 faculties. The questionnaire is requesting details about the number of departments in each faculty, number of specializations, expected new specializations during the Project period, and number of professors and students in each specialization. The questionnaire also included details about each specialization such as the number of credited hours, and the percentage of both theoretical and practical hours. The results clarify the curricula classified as Lab hours, technical drawings, workshops and field training for each specialization.

Consequently, the number of actual students at each department was calculated then summed up to general and preparatory engineering.

2.1.1 Faculty of Engineering and Technology

1. After calculating the number of current and expected specializations for the 5-year-PhaseI of the Project, the mean of students' number in one branch (MS/B) was timed by 4 (4 educational levels for each specialization). The number of 1st year students in 5-year-educational levels was summed up with the Applied Science Faculty. (As it will be mentioned later).

2. The number of students in one department was calculated then the number of students in Engineering and technology Faculty could be calculated.

2.1.2 Faculties of Applied Professions and Administrative science:

The number of current and expected specializations was calculated, timed by the MS/B then timed by the number of educational levels.

2.1.3 Faculty of Applied Science:

The same way in calculating the number of students in Applied Professions was used here then added to the number of preparatory engineering students as follows:

1. Total of engineering faculty students' number divided by 4, so the outcome is the number of General engineering students plus 25 (reserve).
2. The number of general engineering students during 5 years is 350 students; 300 = admission students and 50 = preparatory students taking into consideration that the 350 students will be reduced to

280-300 because some of the students shift there specialization, others quit studying, and other does not finish all prerequisites of general engineering.

3. The number of general engineering and preparatory was added to the number of Applied Science Faculty students. The preparatory students are estimated to be 2 branches during Phase I and 3 branches during Phase II rating 25 students/branch and 40-credited hours/ 1 year (2 semesters). As a conclusion, the number of students is estimated as 75 students yearly.

4. Whereas the total number of students in Engineering Faculty is 1500 students, the number of students in general engineering can be concluded by dividing 1500 by 4 (number of specialization educational level) including preparatory students.

2.2 Analyzing And Determining The Professors' Number:

The researchers adopted the deans' way in calculating the appropriate number of professors that is:

1. The rate of credited hours for each student/ semester * the rate of one teaching hour, lab, and technical drawing.

   For example, the rate of credited hours for each student/ semester for Science students equal to 18 ,with factor of safety = 1.25 , then the credited hour for student is (18)(1.25) = 22.5 → a.

2. the rate of credited hours (a) multiply by total of educational levels equals the total number of students(b) for faculty

Note : All these faculties are 4 educational levels: Engineering and Technology, Administrative Science, while Applied science has only 2 educational levels: Applied professions.

3. To determine Number of professors(c) in each specialization , it can be divided the total number of students(b) for faculty to rate of teaching hours load per one professor .Assume that the rate of teaching hours load per one professor = 12 hours then, b/12 → c
4. Total number of professors of specializations(d) in each department is a number of professors(c) in each specialization multiply by number of specializations.
5. Finally, sum up the number of professors in 1 department to estimate the appropriate ideal number of professors

2.3 Adopted mathematical way in calculating the required space

In order to calculate the required spaces and areas (classrooms, technical drawings, Labs, etc) for each specialization, the following way was adopted:

As an example , the numbers and areas required for classrooms by using the credited hours of each curriculum are calculated as following steps:

1. Considering that the number of semesters per year is 2,So the number of credited hours(a) for each curriculum is divided by 2
2. Considering that the number of attendance (b) days per week is 5 days then by dividing number of credited hours(a) by five days.
3. Considering that there is 12 studying hours/ day - 8:00-20:00- after deducting 1 hour, then b divided by 11 hours → c
4. Considering all curricula in one department, round up the sum of a + b + c.
5. In order to calculate the space required for technical drawings and labs, the previous arithmetical way is adopted except for step 3 in which every 3-credited-hours are considered as one credited hours.

6. Considering that the lectures taken at labs cannot be repeated more than once a week, the hours and days of the week are arranged in way to allow all departments to get benefit of the lab.

7. As for engineering workshops, they are not calculated but decided by the related departments because they are limited due to the equipment and costs.

8. Other supplies and facilities for each faculty are calculated on basis of each department needs.

All analytical tables for numbers of students, numbers of professors ,and areas required for PPU campus are founded in appendix (A).

2.4 Construction stages for campus project :

It is suggested to execute this project in two main stages as following:

1. Stage one

This stage is for designing and construction of the current fourth colleges including engineering and technology college, applied professions, administrative science college, applied science and management building of the university. In addition to the other important facilities that service the university campus, such as student's restaurant, central library and gymnasium building .

The expected number of students in the university in this stage is 4655 students male and female with 60 program that colleges offered. and the areas required are 46400 m^2

This stage with the mentioned current collages is constructed through the first five years of the time of project.

2. Stage Two

Stage two is a completing for the previous stage .So, it is the final image for the campus PPU project construction. Such that this stage contains eight colleges ,the first four collage are mentioned in the above stage with adding new programs, and the other forth collages are a new suggested collages with new programs to cover all of society needs.

Through this stage, the expected number of students in university is around 10000 students, 6400 in the first forth collages and 3600 in the new suggested collages.

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Ch

3

ARCHITECTURAL STUDY OF CAMPUS

CHAPTER 3

CAMPUS ARCHITECTURAL STUDY

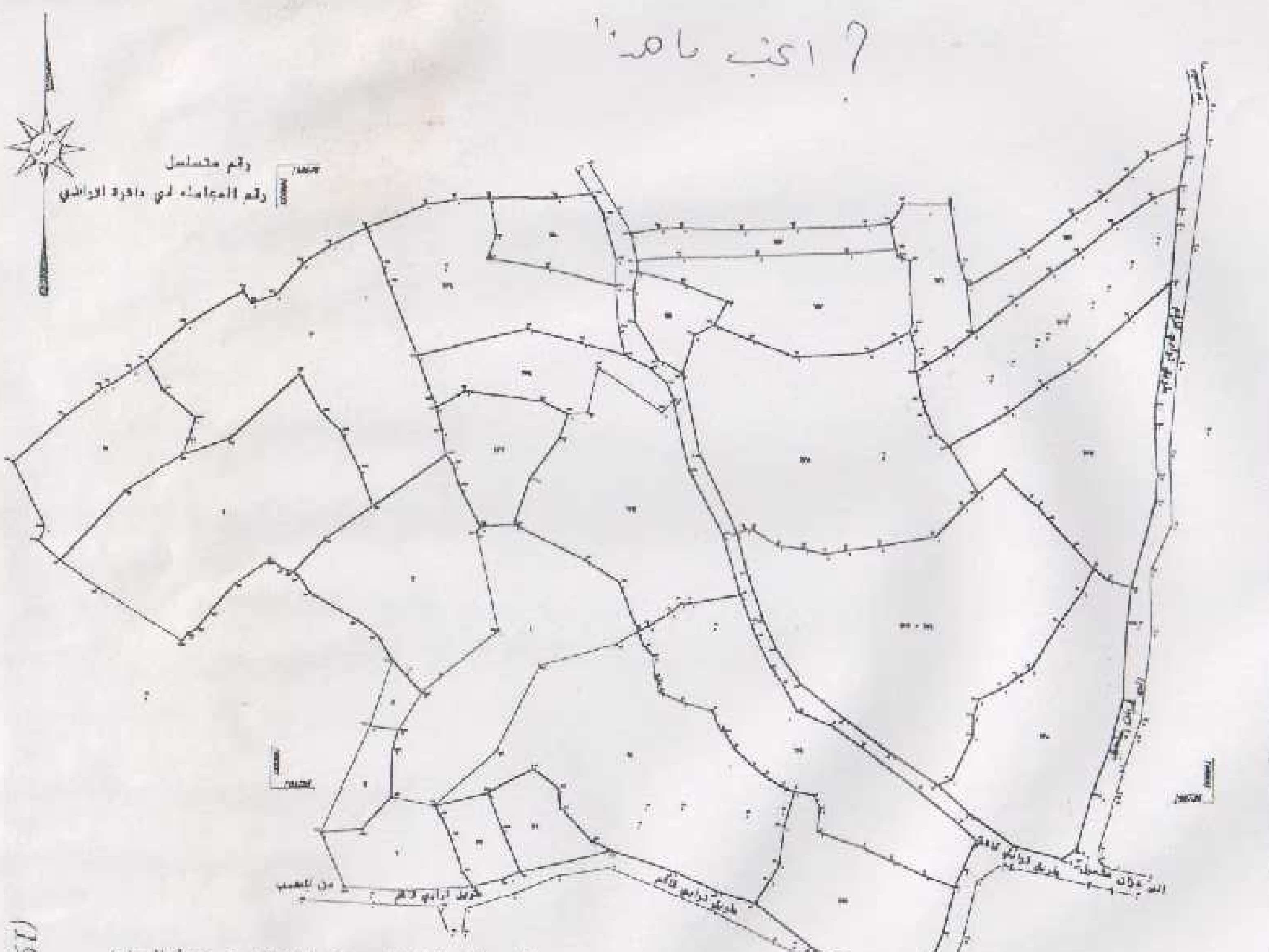
Ith this chapter will discuss all the factors that affect the architectural design of PPU campus. This chapter is divided into two main parts, one is to discuss campus site in Tel Babas in Halhol City, and the other discusses the functional requirements of a campus including general requirements concerning educational and administrative buildings and different facilities as decided by The Planning And Development Department Of PPU, studying and allocating buildings on site planning. Such as site topography study and environmental study.

3.1 *Campus Location*

Tel Babas site in Halhul is authorized by the PPU as a recommended site for the PPU campus. As the PPU is a subsidiary of UGU, that formed a committee to evaluate a number of Hebron lands from technical, financial, engineering and other points of view. The table found in appendix taken from Planing and Developing Department shows that Tel Babas of Halhul got the highest votes:

The total area of the recommended land is 92 000 m^2. It is extending on a hill that overlooks the street next to the new water tank hill that is 500m-distance of the highway and around 1.5km-distance of east of Camp quarter. The site is provided by electric power and water supply but lacks drainage system as the whole city of Halhol. Below are the site photos that show the boundaries and location in Halhol and west - bank maps.

Figure 3.1 Halhoul City in West Bank in Palestine



General Notes

البلد : الخليل

القضاء : الخليل

المدينة : حلحول

الموقع : واد ماضي، ظهر أبو دعيل ، واد قرين

تقسيم بطلب السيد

السلطة الوطنية الفلسطينية

جامعة بوليتكنك فلسطين

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Site Location

Supervisor Dr. Ghassan Al-dwiek

Dr. Maher Amro

Desined by :

Ayman Erahoid

Mansor AL- Sa'di

Zahir Mahmoud

Yousif AL-Jabary

SCALE	no.of DRAW	DATE
ToFit	Figure 3.2	7/2002

أشهد أن هذه نسخة صحيحة وطبق الأصل من خارطة المساحة التي نقلتها في هذا اليوم الظاهر من شهر أيار من العام ٢٠٠١ وأنها تمثل تماما حدود الأملاك العقارية الغير منقولة في التاريخ المذكور أعلاه حسبما بينت واضعي لدى مسح الأرض .

3.2 Functional Requirements of the Campus:

3.2.1 General Requirements: *Concerning Educational And Administrative Buildings As Decided By The Planning And Development Department Of PPU:*

1. General Administration Building
2. Central Library Building
3. Faculty of Engineering and Technology Building
4. Faculty of Applied Science Building
5. Faculty of Applied Professions Building
6. Faculty of Administrative Science Building

3.2.2 General Requirements: *Concerning Different Facilities As Decided By The Planning And Development Department Of PPU:*

1. Multipurpose rooms
2. Close Gymnasium
3. Playgrounds (Tennis, Basket Ball, Soccer).
4. Clinics
5. Main restaurant Building
6. Workshops Building

7. Parking for cars and taxis

8. Horizontal expanding areas

9. Tiled internal squares

10. Green and open areas

3.2.3 Studying And Allocating Buildings On Site Planning:

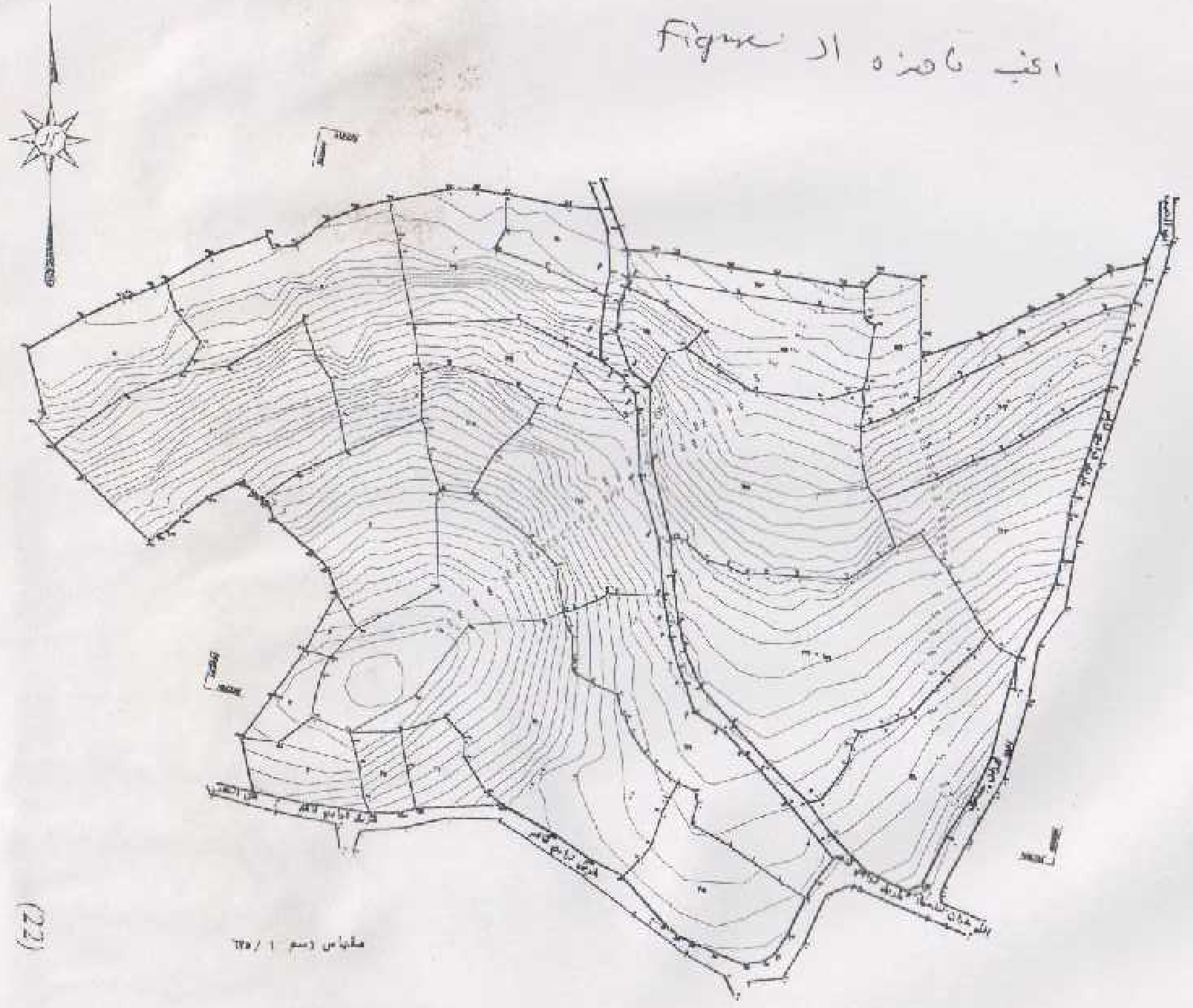
3.2.3.1: Building Study Factors

A - Site Topographical Study:

The topography is an important factor in determining the site elevations, and consequently it affects the work and site and how to organize it.

The researchers made a survey at site using a theodolite and a destomate then draft it using the Soft Disk Software. The Figure 3.3 and Figure 3.4 show the boundaries and the contour's line in one meter interval.

It is noticed that the contours varies between 945m ~ 994m from sea level, so that the difference between the highest and lowest point is 49m. It is also noticed that the contour lines diverge at the high area, converge at the center then re-diverge at the bottom; this indicates different inclinations and slopes. Accordingly, the lowest inclination percentage of the site as per the land sections is 3.9% while the highest percentage is 100%.



General Notes

اللواء : الخليل

القضاء : الخليل

السلطة الوطنية الفلسطينية

جامعة بوليتكنك فلسطين

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Contours lines 1 m period

Supervisor Dr. Ghassan Al-dwiek

Dr. Maher Amro

Dedicated by :

Ayman Nrshats

Mansur AL- Sa'idi

Zaher Mahmod

Yousif AL-Jabary

SCALE	no.of DRAW	DATE
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مقياس رسم ١ / ٣٥

General Notes

اللواء : الخليل

القضاء : الخليل

البلدية : حلحول

الموقع : راس حاضر ، شعب أبو رميل ، راس قوين

تقدمت بطلب التسوية

السلطة الوطنية الفلسطينية

جامعة بوليتكنك فلسطين

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Contours lines 1 m period

Supervisor Dr. Ghassan Al-dwiek
Dr. Maher Amro

Desined by :

Ayman Nrsheid

Mansur AL- Sa'dH

Zaher Mahmod

Yousif AL-Jubary

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The map of contour lines of Figure (3.5) shows the inclinations classified in 4 directions; area A goes to east direction, B goes to north direction, C goes to north east while D goes to north west .

There is a different section in a site map to show the topography of the site.The following figures (3.6) to (3.8) are to illustrate both contours' line direction and some sections in the site

B – Environmental Study:

It is always difficult to make those conditions naturally available without referring to other methods. However, the natural resources should be highly exploited in order to improve the internal climate conditions because this will reduce power consumption and consequently reduce the cost and of the project.

This study aims at studying the climate of Halhul and Hebron cities and especially the Project site because the climate conditions are very important in determining the building plan from the following points of view:

- General planning and choosing the activities locations.
- Buildings converging to create an appropriate internal climate
- Buildings' designing that affects the climatic function

This study includes the climate influence on man's comfort and activities in addition to the relationship between climate and domicile and architecture, and the importance of climate conditions for an architect. It also includes an analytical study

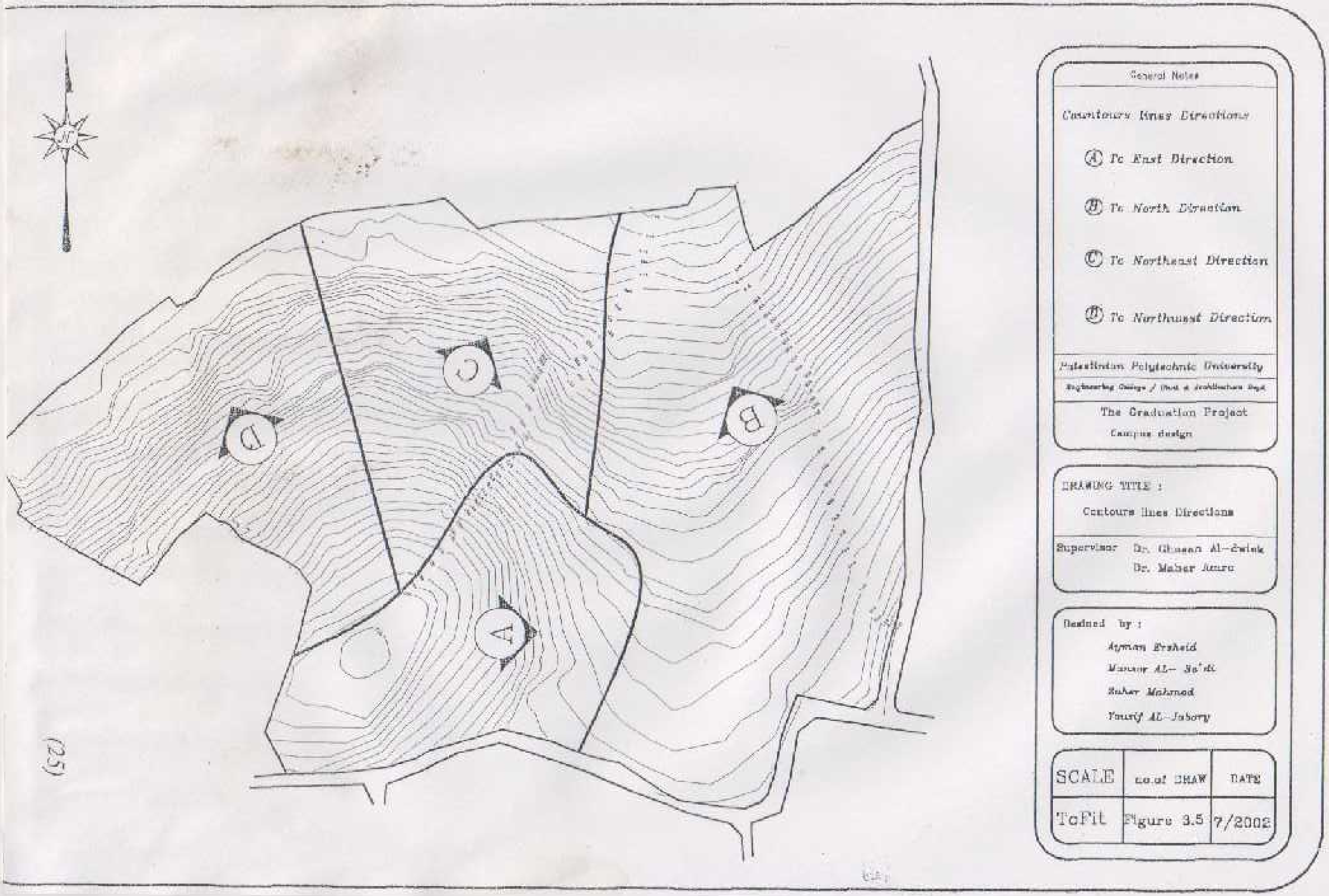
A
B
C
D
General Notes
Countours lines Directions
(A) To East Direction
(B) To North Direction
(C) To Northeast Direction
(D) To Northwest Direction
Palestinian Polytechnic University
Engineering College / Civil & Architecture Dept.
The Graduation Project
Campus design
DRAWING TITLE :
Contours lines Directions
Supervisor Dr. Ghasan Al-Jwiek
Dr. Maher Amro
Desined by :
Ayman Ersheid
Mansor AL- Sa'di
Suher Mahmod
Yousif AL-Jabory
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Figure 3.5
7/2002

General Notes

Section A-A

Section B-B

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Sections on Site

Supervisor Dr. Ghassan Al-dwiek

Dr. Maher Amro

Desined by :

Ayman Ershaid

Mansor AL- Sa'di

Zaher Mahmoud

Yousif AL-Jabary

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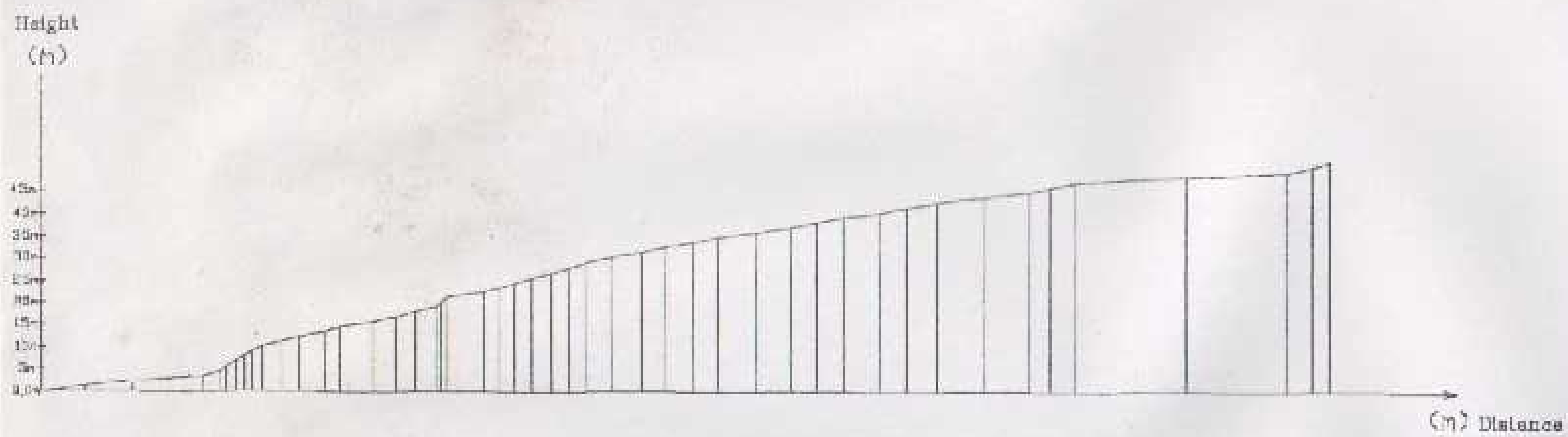
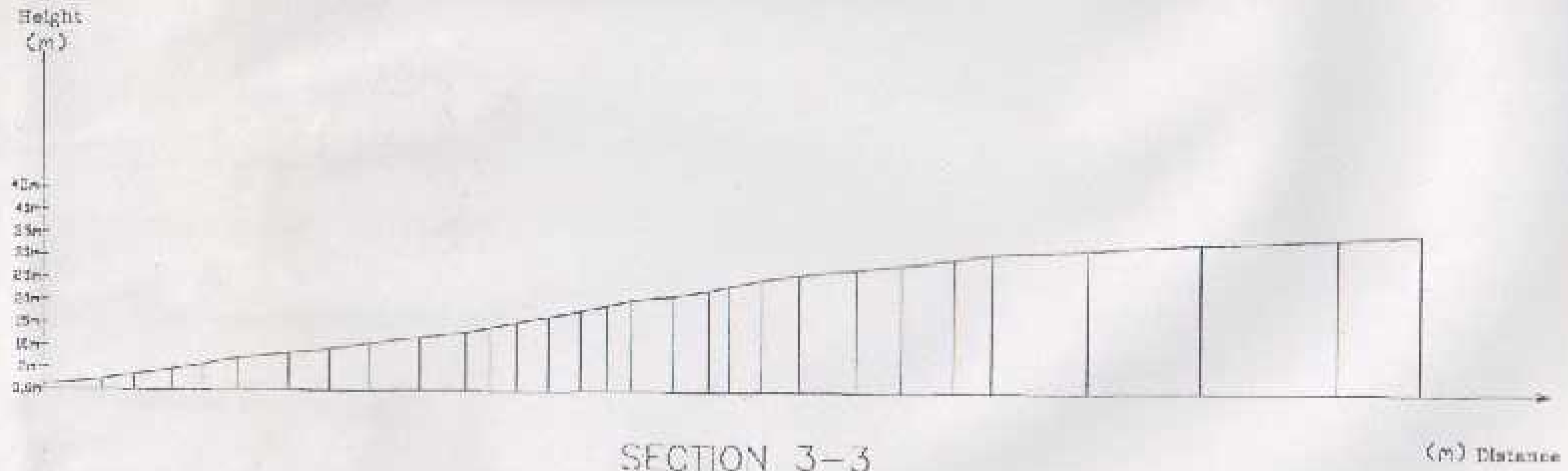


SECTION 2-2



SECTION 3-3

General Notes

Section 2-2

Section 3-3

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Sections on Site

Supervisor Dr. Ghasan Al dwiek

Dr. Maher Amro

Desined by :

Ayman Erzheid

Mansor AL- Sa'di

Zaher Mahmod

Yousif AL-Jabary

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C– Determine the Entrances and Exits

The entrances and exits of the site land are determined as showed in figure 3.13

D– Study of Streets And Roads Going To Site:

It is noticed that the recommended site is located in between of many roads connecting the cities, districts and city quarters. Also one can reach to site from the western side passing by Shaheera Hospital and Halhol Police Station. There are also two entrances at the eastern and south-east ward coming from the street around Hebron and Halhol. Another entrance is located at the northern side coming from Al-Nabbi Younes Quarter in Halhol.

Jerusalem-Hebron Main Street is one of the vital streets in this region especially and Palestine generally because it connects Jerusalem with the southern cities; that's why the main gateway is to be in the western side and the other secondary gateways are to be in the other entrances sides.

Please refer to figure 3.14 that shows the external roads map.

E– Study of Internal Routes And Walkways

The internal routes and transportation depend on the topography of the site.

Halhol is located to the east of Hebron-Jerusalem Highway and in-between of latitudes n° 109370 to 109700 and longitudes n° 160100 to 160600 of the Palestine.

Actually, the scientific location of Halhol is on the intersection of northward latitude n° 31.35 and eastward longitude n° 35.6.

The climate of Halhol is affected by the Mediterranean; it is hot and dry in summer, cold and raining in winter. Rainfalls decrease gradually towards the east but increase opposite to the west on top of mountains where temperature becomes lower than 0°c in winter.

This research stresses climate conditions and factors influencing on them in Halhul:

B-1 Influencing Factors On Halhul's Climate:

Many interfering factors influence on climate so that one cannot distinguish which factor causes what. One of the most important factors is the geographical location. So we find the temperate climate affected by the Mediterranean because it is located on the northern latitude n° 31.6.

The topography is another important factor. So we find some of the mountain chains in Halhul extending from the northeastward to the southwestward exceeding 1000m above sea level, which affects the summer climate and makes it more temperate, and at the same time it makes the winter climate more raining. Not to mention that the western wind helps moderating the temperature in summer and lowering it in winter.

B-2 Climate Elements:

In order to the study the climate of Halhul, one should analyze the climate data to determine the main points that might be changed due to special circumstances in the region.

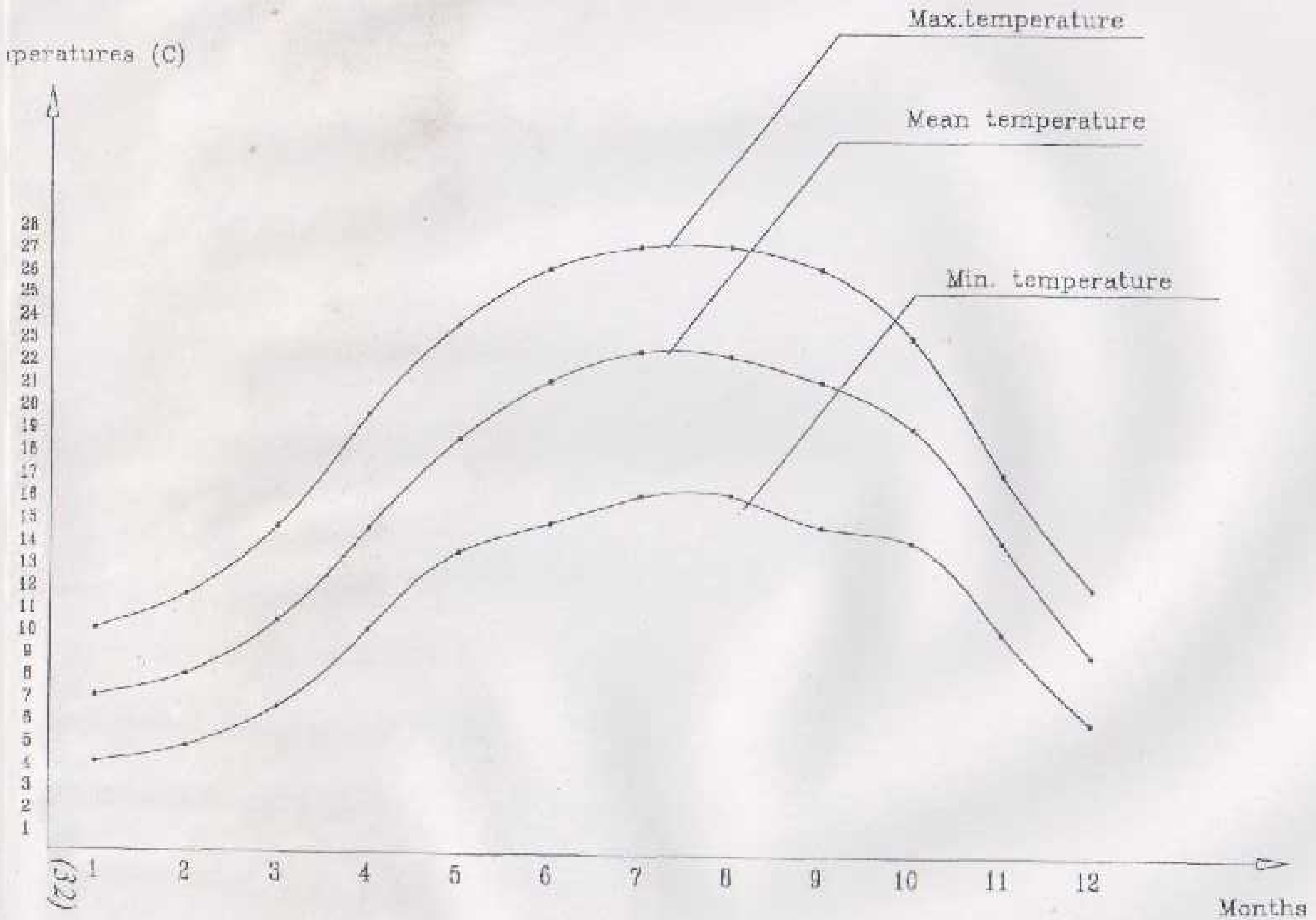
B-3 The analytical study includes the following elements:

1. *Temperature*
2. *Relative humidity*
3. *Sunrays projection angles*
4. *Wind direction*
5. *Rain quantity*

1. Temperature

Usually, the temperature affects the climate conditions and the design of any building. In Halhul, the green covered mountains of Halhul are considered a summer resort in the south of Palestine and this is due to their effect of moderating the temperature.

The following table 3.1 shows the mean, maximum and minimum temperatures since1975 till 1997:



General Notes

Maximim and minimum

temperatures since

1975 to 1997

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Temperature /month diagram

Supervisor Dr, Ghassan Al-dwiek

Dr. Maher Amro

Designed by :

Ayman Ersheid

Mansor AL- Sa'di

Zaher Mahmod

Yousif AL-Jubory

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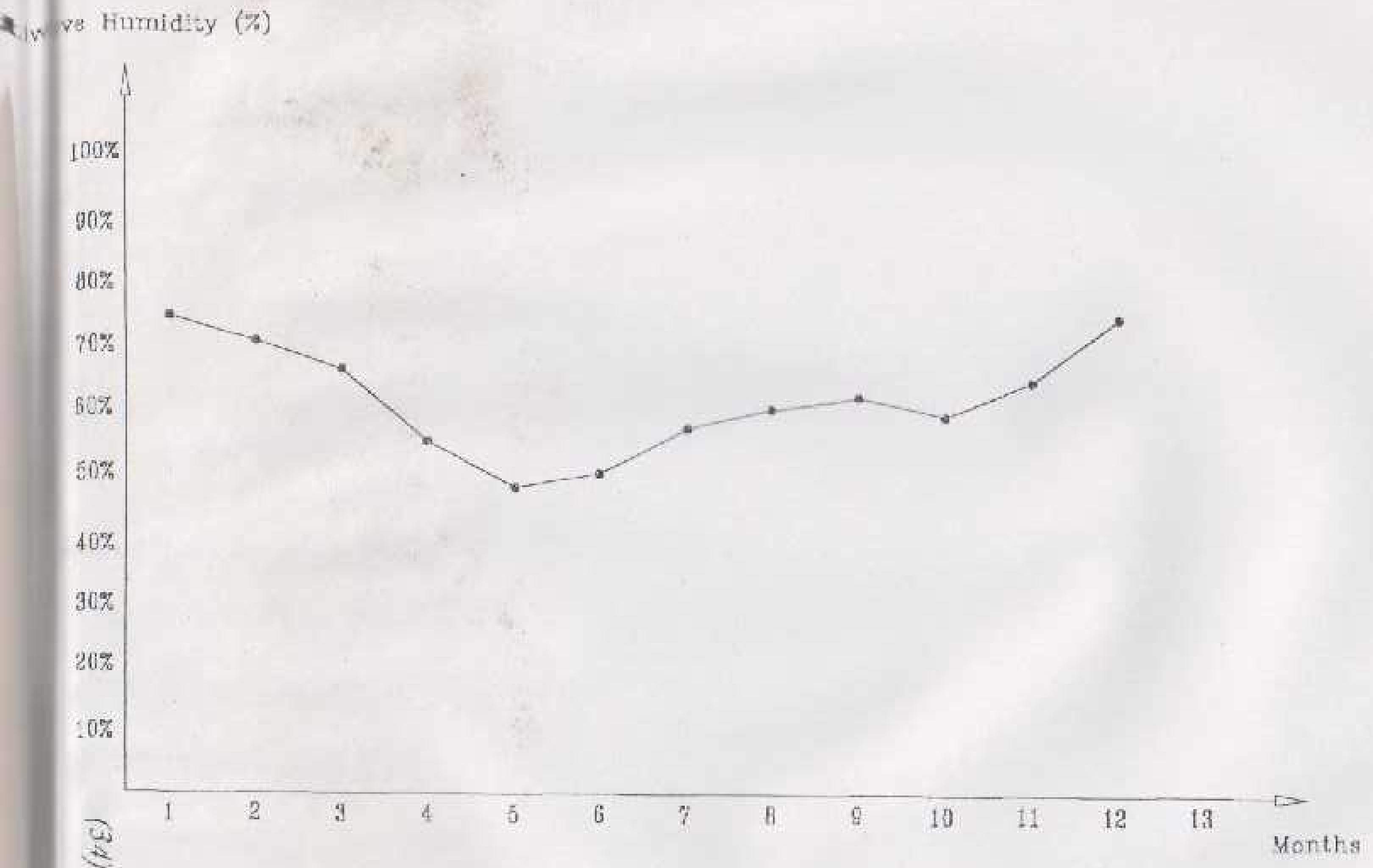


General Notes

Mean Relative Humidity Since 1975 to 1997

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Relative Humidity /month diagram

Supervisor Dr. Ghasan Al-dwiek

Dr. Maher Amro

Desined by :

Ayman Arshaid

Mansor AL- Sa'di

Zaher Mahmud

Yousif AL-Jabary

SCALE	no.of DRAW	DATE
ToFit	Figure 3.10	7/2002

2. *Relative Humidity*

The following table 3.2 shows the mean relative humidity percentage since1975 till 1997:

Table 3.2 *Mean relative humidity*

Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean RH Percent.	74%	72%	66%	55%	48%	51%	57%	60%	62%	59%	64%	73%

the following graphic figure 3.10 showing the relationship between the different relative humidity and months:

Conclusions of the previous graphic analysis:

the general mean RH since 1975 still 1997 between (48-73) percentage

RH rate increases in winter varying from 66% to 74% especially in December as it reaches 74% and in winter in February as it reaches 72% .the reason of this increase is the decrease of temperature at this period.

It is noticed that the RH decreases in summer to 51% in June and 48% in May due to the hot and dry weather

The max mean RH is in December as it reaches74%

The min mean RH is in May as it reaches48%

3. Sunrays Projection Angels:

The sunrays and daylight are the most important climatic elements as they affect the humanity and environmental aspects because they are related to the temperature, humidity, rains, wind and evaporation.

Although the interaction between those elements give us a clear idea about the prevailing climate of a region, the effectiveness of one element more than the others urge us to find the reasons for the difference of climates from one region to the other.

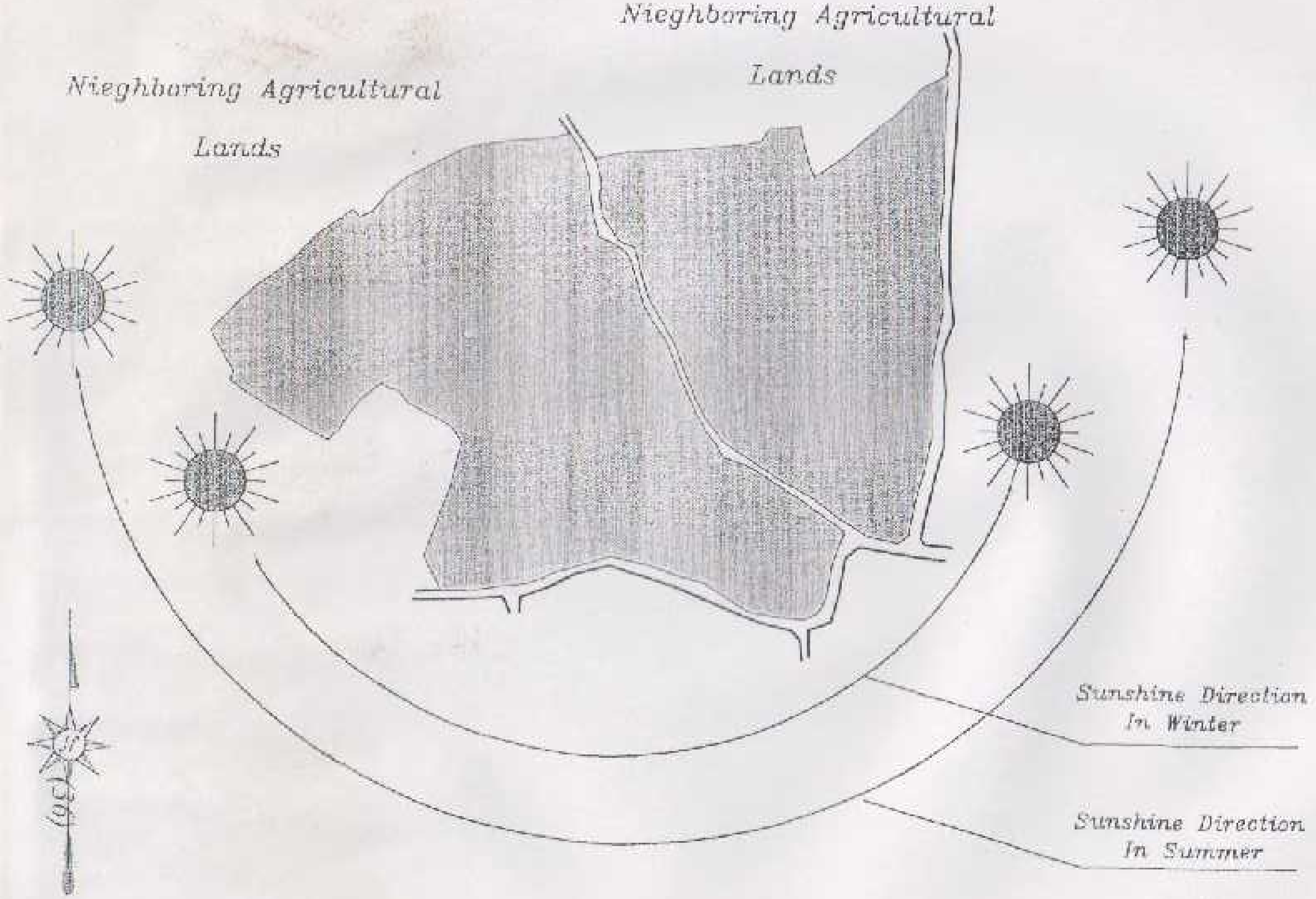
One of the most important reasons for the difference is the sun and its rays projection angles.

The following table 3.3 shows the daylight period in Hebron and Halhul throughout one year:

Table 3.3 ***Sunshine Period IN Hebron City***

Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
H/Day	4.7	4.8	6.4	8.1	9.0	8.3	9.6	10.9	10.3	9.8	7.0	4.7

The following Figure 3.11 shows the sunshine direction in summer and winter and the sun radiation of the university site Halhul throughout one year.



General Notes

Sun Radiation

And Sunshine Direction

In summer And Winter

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Sun Radiation

Supervisor Dr. Ghasan Al-dwiek

Dr. Maher Amro

Desined by :

Ayman Ershaid

Mansar AL- Sa'di

Zaher Mahmod

Yousif AL-Jabary

SCALE	no.of DRAW	DATE
ToFit	Figure 3.11	7/2002

C– Determine the Entrances and Exits

The entrances and exits of the site land are determined as showed in figure 3.13

D– Study of Streets And Roads Going To Site:

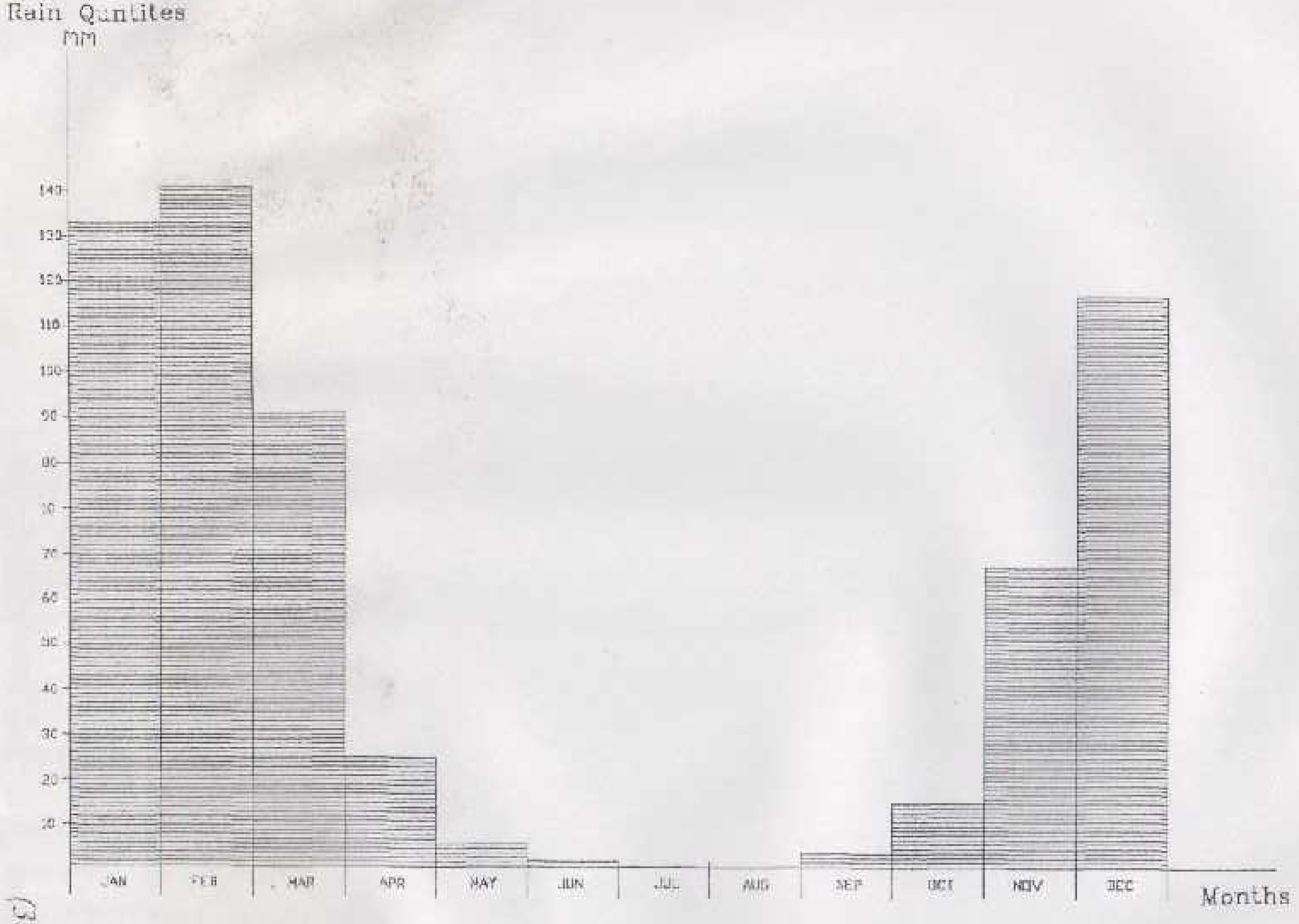
It is noticed that the recommended site is located in between of many roads connecting the cities, districts and city quarters. Also one can reach to site from the western side passing by Shaheera Hospital and Halhol Police Station. There are also two entrances at the eastern and south-east ward coming from the street around Hebron and Halhol. Another entrance is located at the northern side coming from Al-Nabbi Younes Quarter in Halhol.

Jerusalem-Hebron Main Street is one of the vital streets in this region especially and Palestine generally because it connects Jerusalem with the southern cities, that's why the main gateway is to be in the western side and the other secondary gateways are to be in the other entrances sides.

Please refer to figure 3.14 that shows the external roads map.

E– Study of Internal Routes And Walkways

The internal routes and transportation depend on the topography of the site.



General Notes

Rain Quntities

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Rain Quntities

Supervisor Dr. Ghassan Al-dweik

Dr. Maher Amro

Desined by :

Ayman

Mansur AL- Sa'di

Zuhur Mahmud

Yousef AL-Jabary

SCALE	no.of DRAW	DATE
ToFit	Figure 3.12	7/2002

Ch 4

Architectural study of the library

4.1 Introduction

4.2 Libraries

4.3 Functional Requirements

4.4 Functions

4.5 Architectural drawings

CHAPTER 4

Architectural study of the library

Until now, much of our study of project has involved analytical studies and campus architectural study. In this chapter we will discuss one of the main part of this project "architectural study of the library building".

4.1 Introduction

An architect is unlikely to design a satisfactory library building without first understanding clearly its function and the proposed methods of carrying out that function. The information he receives from the librarian will guide him in detail, in providing the information, the librarian will assume that the architect is very familiar with the type of library he is to design.

Different types of libraries have different spatial and environmental phases and it is only too easy for an architect to assume that his experience in designing one type of library can be used every time, with only small adjustment, in designing another.

The following paragraphs are meant for architects: they give a very general outline of the purposes of the main categories of library, but it must be emphasized that there can be enormous variations within each category. "Fundamentally a library is not a building but a service organization".

4.2 Libraries

4.2.1 Types of Libraries

Libraries in general are of three basic types:

- Lending libraries with minimal or no reader
- Reference libraries with large reader areas and few or no Lending facilities.
- Libraries with reference/study areas plus lending facilities

Most libraries have separate sections of both main types, but there are several kinds of libraries as following:

1. University and college libraries
2. School Libraries
3. Public Libraries
4. Hospital and welfare Libraries
5. Prison Libraries
6. Special Libraries
7. National types of Libraries

In this project, the selected type of library, that will be designed, is a university or college library which will introduce a specific information that needed to design architectural requirements to satisfy all of needed functions and requirements.

4.2.2 University Libraries

A new university, given space in which to build, usually takes the form of a campus, with the library occupying a key site, easily accessible from the main street and with all-weather approaches.

The program of transforming colleges into university can mean that a library has to be designed to serve those working in growing and often isolated buildings within a campus, which is at the planning stage.

The main function of the university library is to store bibliographical and audio-visual materials and to make them available swiftly to students, faculty and researchers. A few years ago most universities would have assessed their main role as a service to research, but today it is generally accepted that the library is an active participant in teaching and learning program at all levels.

All university libraries have the duty to provide bibliographical tools necessary not only for subject interests but also for education and development of the whole university body as human beings. So, the library will serve as a tool to assist learning, teaching and research, and for student's interests.

4.3 Functional Requirements

In any project the first time should be spent to discussing the requirements and assessing if they can be realized within the project.

4.3.1 Site or Location of Library

Libraries which is a form part of larger organization in a building complex will seldom have much say in the choice of site .In a university, the master plan or campus will probably have fixed the library site.

On the other hand, it must to take the question of site selection very seriously because so much of its success in attracting readers will depend upon its location.

Even when the question is only where to place the library within a building, conflicts of interest will a rise.

1. Site Selection Conditions

The library will need to be central to:

- Easily accessible for all major activities of the university.
- Close to a main entrance of university
- Close to a main entrance of university parking
- A library should be placed in a silent area in a campus for readers.

These will also be the requirements of most other special interests within the organization.

So, some interesting studies (called site selection study) have been made of criteria needed when choosing sites for any types of libraries.

The conditions here, are ideal for a clear study of principles because the areas are unencumbered by existing buildings.The studies have been particularly directed to the importance of the sitting relationships between the libraries and other surrounded buildings or colleges in a university campus.

It is obviously of the greatest importance that, if choice of site is possible, the architectural designer should be closely involved in decision.

2. Direction Of Building

According to the library site and university master plan, it can see that the direction of library building is suitable .because this direction is a result taking some considerable points and requirments which affect the architectural design of the library.

The reasons for a suitable direction for library are the following:

- ***Wind Direction:***

The effect of wind diection is not a big problems for building .because,the Hebron City is far serious variaty in climate and weather as result as it is far about the sea and desearl .and the hieght of Hebron City about the sea leavel diven that the ideal critira.

- ***Sunlight Direction:***

The sunlight is very stronge in summer which rise in the bulding tempreture ,So this problem is solved in architectural design by using a windows that prevent the heat transfear between building and surrounded areas.but in winter the direction of sun rays are perpendicular to building from the east.

3. Relationships with other factors

☙ Relation with campus buildings

The site of library is central in master plan in order to use library with all of students in different colleges, and is near the main parking of university for the outside visitors.

☙ Relation with Hebron City

The relation between university and city is a major factor in designing to make a contact between a sosity needs and the university,the library is similar condition,so it must take care in designing the factor of the image of building and its critira which must be similar and suitable with the organization of architectural critrias of hebron city.

ℭ Relation with Visitors

This library is not only for universitie's students but its for any visitors and resarches and some intresting people, so it must take this point in mind when designing library and where its place.

ℭ Relation with other libraries

The relation between library and local and international libraries are important to improve all services offered by library.when starting a design this point are solved by selection of the site which is close to main roads and university entrance and by designing a place which is for international with other library and socites .

4.3.2 Design Checklist

The following checklist covers the kinds of information that may be needed to draw up a design brief. It is relevant mainly (but not only) to public libraries.

There are several types of data or information that needed to begin the design:

1. **Users services**
2. **Staff services to users**
3. **Technical services**

The tables (4.1) showing all needed information for all types of the pervious data :

Table [illegible] Design check list data

Users services					Staff services to users	Technical services	
Opening hours	No. of readers	Associated activities	Reader's facilities	Refreshment areas			
Peak-use times	Bibliographical	Meeting rooms	Catalogue reference	Storage of readers' belongings	Number of staff	types of staff	Areas for Accessioning
Hours	General reference	Lecture rooms	Document copying	Coats	Security points	Offices	Cataloguing
Days of the week	Lending system	Typing room	CD-ROM viewing/listening	Bags	Book issue and return	Administrative	Processing
Times of the year for educational libraries)	Periodicals, newspapers	Exhibition area	Accessing remote databases	Lavatories	Reader enquiries	Executive	Receipt and despatch
	Special reference		Microform viewing	Telephones		Network manager	Post and packing
			Video cassette viewing/listening Audio reproduction	Bookshop		Printing	Photography
			Poster display			CD-ROM making	Binding

4.3.3 Area Required

Space requirements vary considerably, and to determine the area needed for library for each part, it must be first know and study all types of spaces or parts that Library contain it to analysis and specified total area of library and all parts of its. So the following points are the types of area required:

1) Control and counter areas
2) Reading areas
3) Staff areas
4) Services areas
5) Storing areas
6) Other areas and facilities

4.3.3.1 Control areas

In multi-story or sectionalized libraries such a center or counter area may be required for each major division. The centers will have three main functions:

Control of the issue and return of books

Bibliographical assistance to readers;

Supervision of user activities and security control.

In larger libraries the borrowing and return counter will provide some security control and will be quite separate from the readers' adviser's desk. In very large libraries the counter may be placed in an entrance lobby so that the inevitable noise of reader traffic will be isolated from quieter reading areas.

The detailed design of the counter is of great importance to the library because of the critical operations, which take place there. When drawings are prepared, the designer must study them with great care because errors in, planning counters and deciding the areas of counter, can affect the efficiency of control system.

1.Counters shape

☙ In the smallest library a single control, Figure 4.2, covers both issue and return. A desk may be used, but a counter has the advantage that it is at standing height and can have cupboards fitted. This layout has the disadvantage that the inevitable crossing of traffic routes can be troublesome at peak times.

☙ The layout in Figure 4.3 offers more security control and fewer traffic problems, but two members of staff are necessary at all times to operate it.

☙ This shape, Figure 4.4, with slight variation, is the commonest of all. It shields the staff and allows one person to serve at quiet periods. In larger libraries, a variation can be this shape doubled so that a reserved book and enquiry area can be manned at quiet times with minimal staffing, (figure 4.5)

☙ In the largest libraries, particularly in universities, the counter acts as a barrier between readers and staff working areas. Here the shape can fit in with that of the building, but it is usually straight, (Figure 4.6). In some university libraries such a counter can be more than 20 m long.

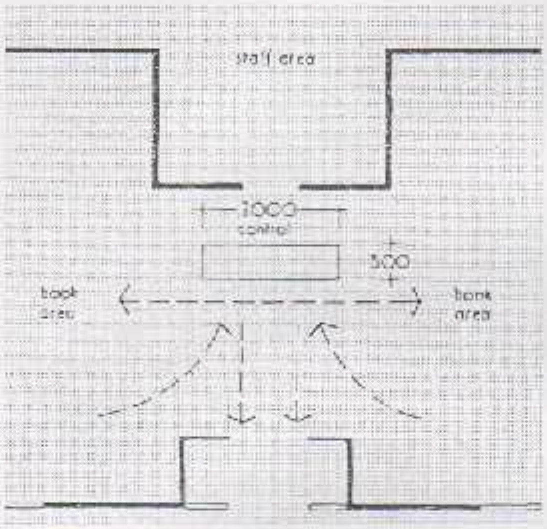


Figure 4.2 *Control counter for small library at busy times*

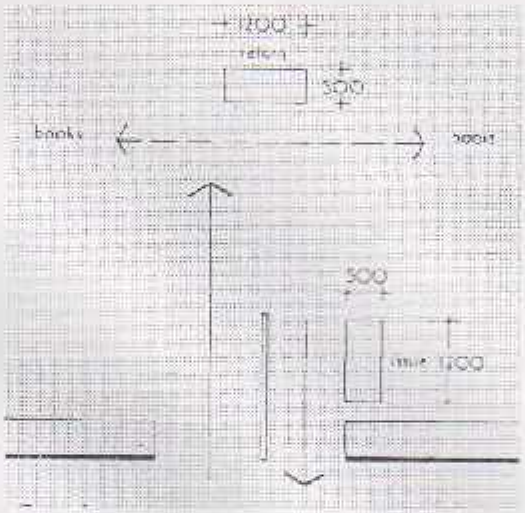


Figure 4.3 *Control arrangements for slightly larger library*

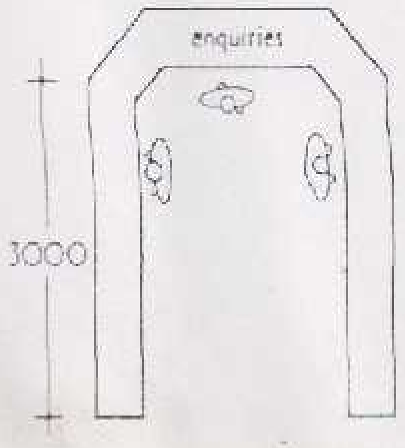


Figure 4.4 *Most common type of Control counter*

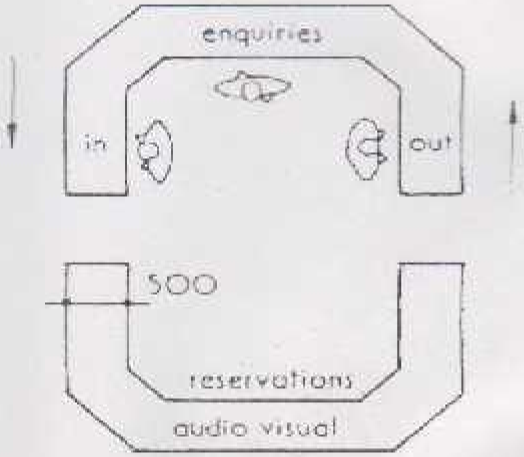


Figure 4.5 Variation of figure 4.4 for large library

Figure 4.6 Type of control counter
preferred in very large libraries

2. Counters Width

Counters are usually 500 mm wide and sometimes
Also have a 150mm bag rail on the outside of the in counter.

4.3.3.2 Readings areas

Although reading is a basic activity in libraries of all kinds, there is a fundamental difference between serious study and casual browsing: entirely different space. Allowances have to be made for the two activities, the proportions depending on the aims of the library. Academic libraries will concentrate on serious study but they will not neglect to use the appeal of books in an informal setting to catch the student's interest and widen their horizons. In addition, spaces taken by library readers are given in the following table:

Table 4.2 *reader space requirements*

User	Floor Area m^2
Student Or General Reader	*2.3*
Research Worker	*3.25*
Carrel User	*3.70*
Actual Floor area occupied by reader at table	*0.93 to 1.20*

The total Reading areas are calculated according to determine the area of the following parts:

1. Structural grid
2. Layout
3. Bookshelf capacity
4. Shelf depth and spaces
5. Closed –access system
6. Open-access system
7. Shelf heights
8. Tables
9. Carrels
10. Card catalog capacity

1.Structural grid

Book stacks and shelving systems are rigidly standardized on a distance of 900 mm. Spacing of book stacks, and therefore the capacity of the library, will be radically affected by the chosen structural grid, and also by the dimensions of the vertical structural elements and other facilities such as service ducts. In this building type, if in no other, careful integration of all structural and service elements is essential.

2.layout

Layout for stacks within a 6900mm structural grid is given in Figure 4.7 and Table 4.3 lists the book capacity of structural grids ranging from 5.6 to 8.4 m.

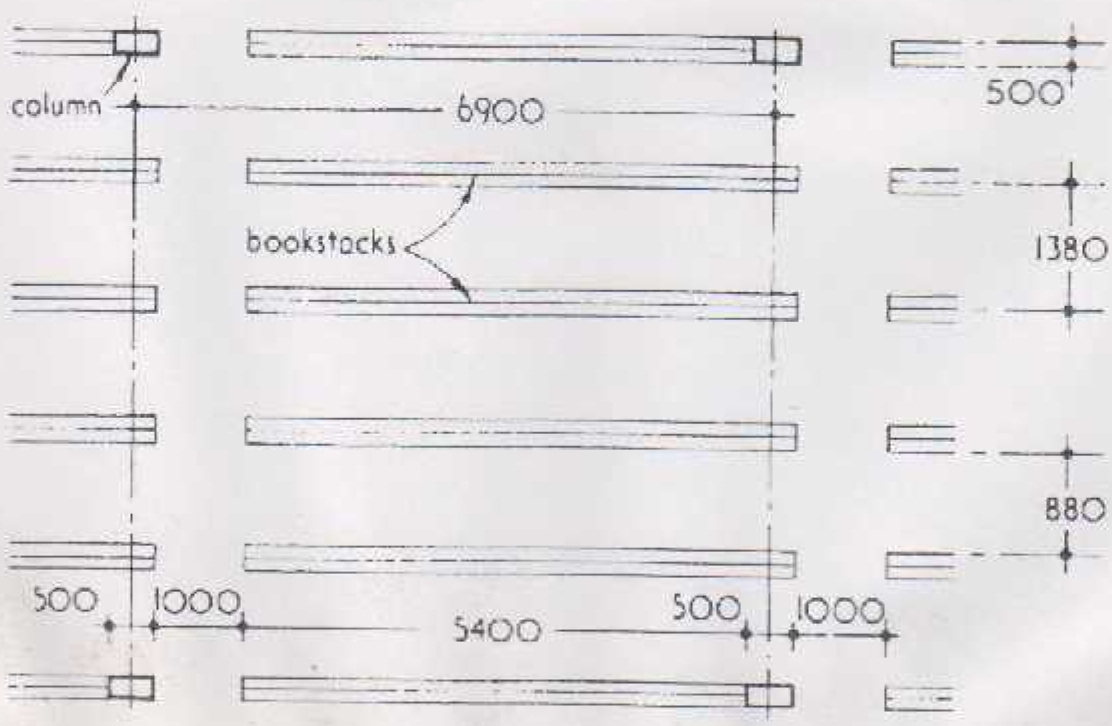


Figure 4.7 *Capacities of book stacks within a 6900 mm structural*

Table 4.3 Book per 300 mm run of shelf

Type	Number	Recommended Shelf depth
Children's books	*10-12*	*200-300*
Loan and fiction stock in public libraries	*8*	*200*
Literature and history, politic and economics	*7*	*200*
Scientific and technical	*6*	*250*
Medical	*5*	*250*
Law	*4*	*200*

3. Bookshelf capacity

The capacity of standard 900mm bookshelves to hold books periodicals and reports is indicated in ***Figure 4.8***. These shelves are assumed to be only three-quarters full to allow for expansion and book movement. The average space requirements of each type of book are given in Table 4.4.

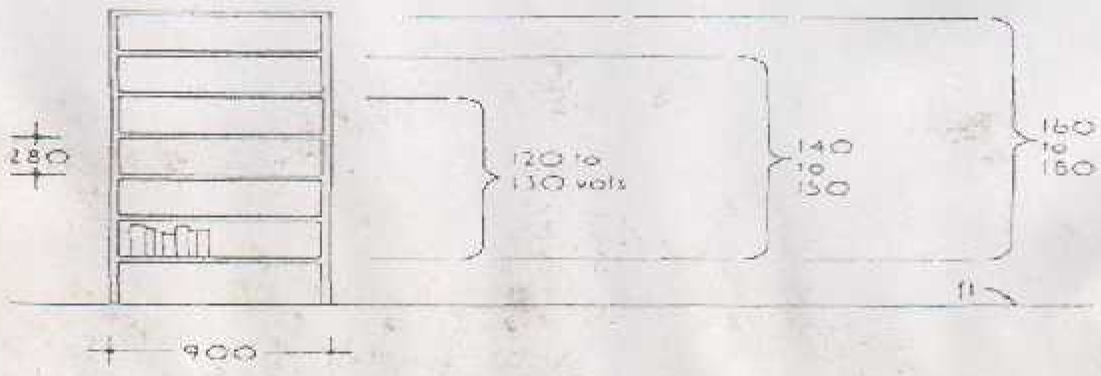


Figure 4.8 *Capacities of shelves to hold books*

Table 4.4 *Book capacity at various sizes of structural grid*

Grid Size m	Spacing Of stacks	No. Of double-sided Stacks	Books per Structural bay
5.6	1.4	3	5012
6	1.5	3	5460
6	1.2	4	6860
6.5	1.1	4	6160
7	1.35	4	8310
7.25	1.45	4	8610
7.2	1.2	5	10276
7.5	1.25	5	10780
7.7	1.1	6	12992
7.8	1.56	4	9380
7.8	1.3	5	11290
8.4	1.68	4	10220
8.4	1.2	6	14364
8.4	1.4	5	12292

4 . Shelf depth and spaces

Table 4.5 gives the recommended shelf depth and the average spacing along the shelf for the main types of book.

Table 4.5 *Shelf depth and spacing*

Type of book	% Of Total	Spacing mm	Depth Mm
Popular (light novels)	50	225	230
General	97	280	230
Bound periodicals	-	300	230
Over size books	3	500	300-400

5.Closed –access system

Figure 4.9 show the limitations of various narrow aisle widths. Anything less than 610 mm makes it difficult to bend to reach the Lower shelves. 810mm is the minimum if a trolley is to be used.

As space is often at a premium, sliding stacks may be used. These provide for only one such 900 mm aisle for, say, ten stacks, depending on the frequency of use and the numbers of staff. Sliding stacks are generally confined to the lowest level of the building due to the heavy structural load involved. If used on suspended floors there are also onerous limitations of deflection that could cause jamming.

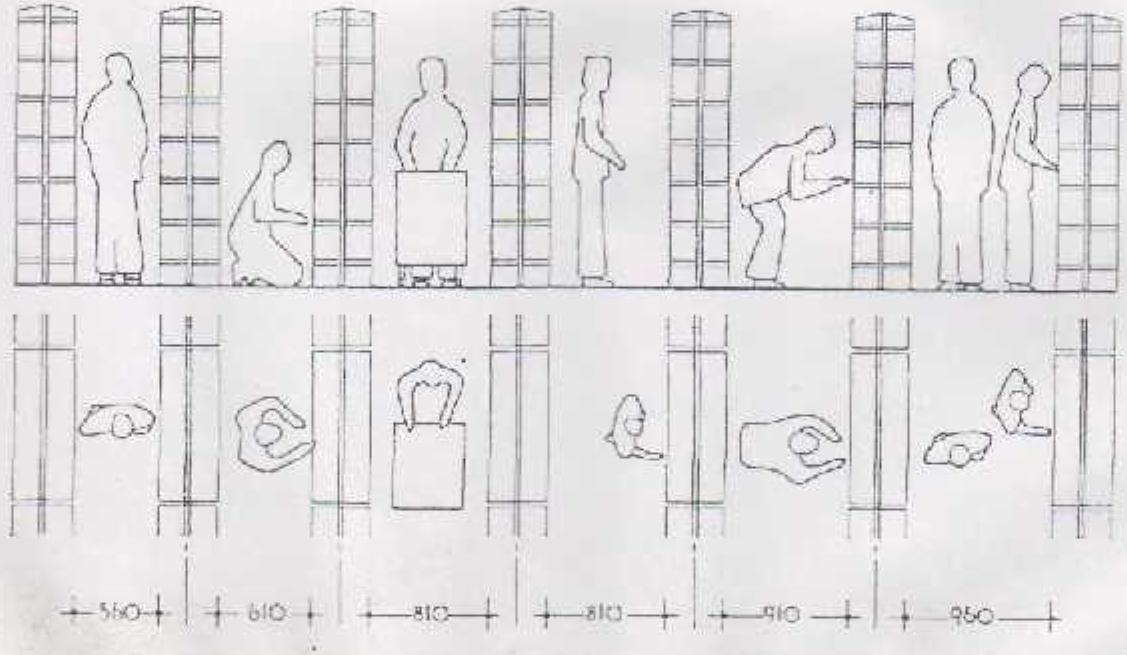


Figure 4.9 *Minimum clearances in shelving areas for various attitudes: narrow aisles*

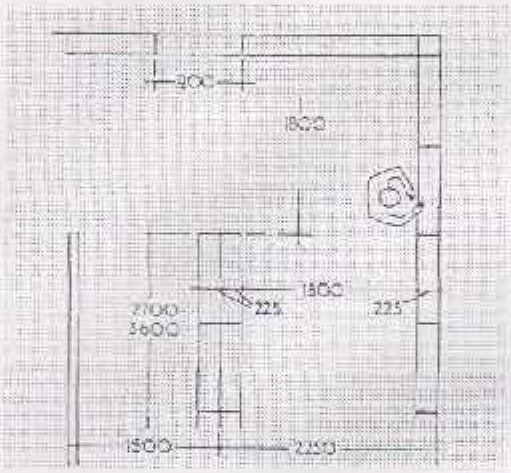


Figure 4.11 *Recommended minima in open -access bookshelf areas*

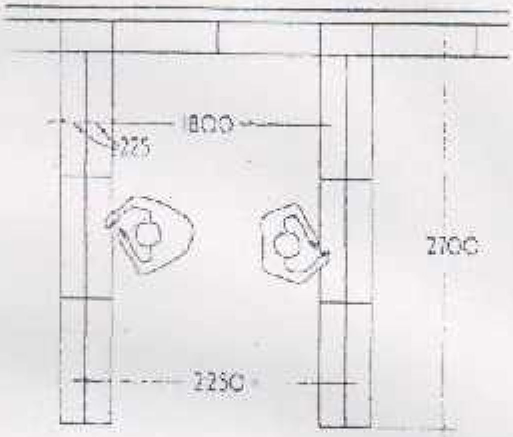


Figure 4.12 *Recommended minima in open -access arranged as alcoves*

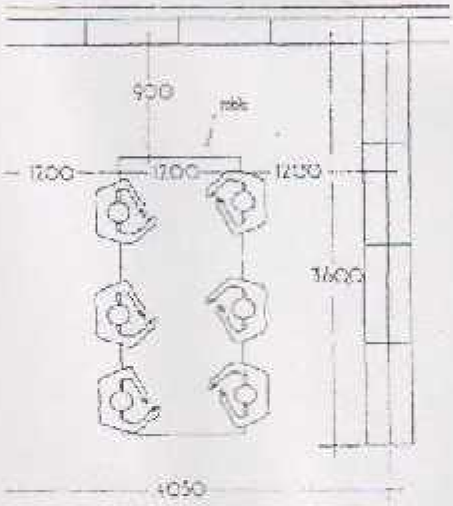


Figure 4.13 *Recommended minima in open -access arranged as alcoves containing reading tables*

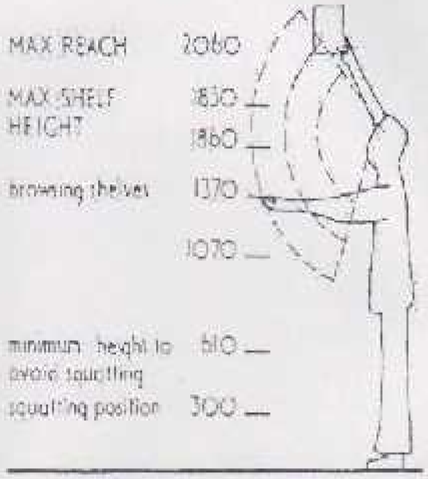
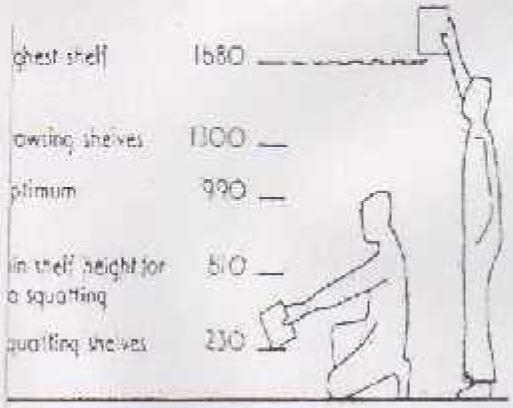


Figure 4.14 *Optimum shelf heights for adults*

ıelf Heights

The easiest shelves to access are those at the user's eye height. The previous e 4.14 gives height criteria for adults. For libraries, such as in schools, used y by teenagers Figure 4.15 should be followed, and Figure 4.16 for small ren. People in wheelchairs will need assistance from staff in normal libraries to the higher and possibly the lower shelves. In libraries designed to cater partic- y for wheelchair users, only shelves between 400 and 1200mm above the floor ld be used.



ıre 4.15 *Optimum shelf heights for agers*

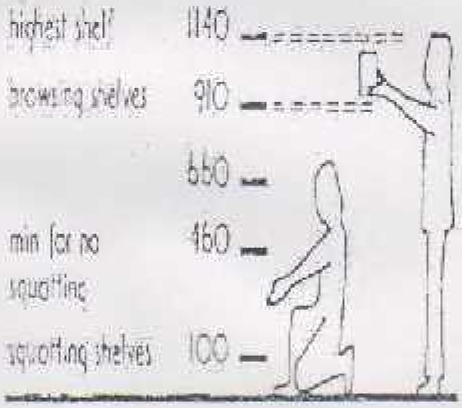


Figure 4.16*Optimum shelf heights for children*

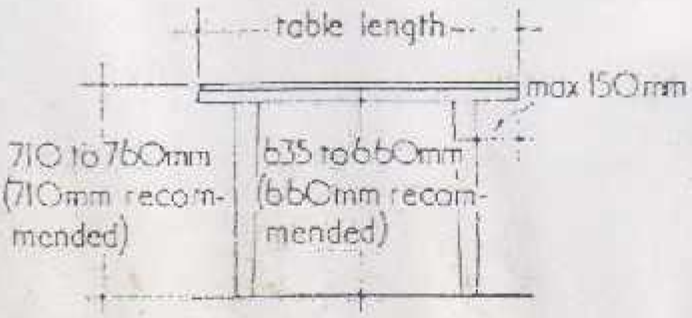


Figure 4.17 *reading table height for adults*

8. Tables

Most readers are expected to use communal tables, Figure 4.17. The design parameters for these are given in Figure 4.18.

In reference libraries tables are commonly arranged in rows in areas separate from the bulk of the books. Figure 4.19 shows reading tables for up to eight people at a table. Double-sided tables are not popular unless there is a low screen down the middle to ensure a measure of privacy. The size of table must reflect the material likely to be consulted — obviously maps and newspapers require more space.

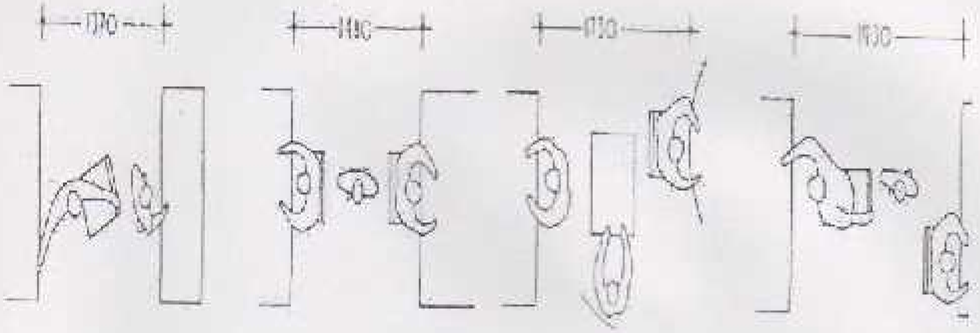


Figure 4.18 *Minimum clearance in reading areas*

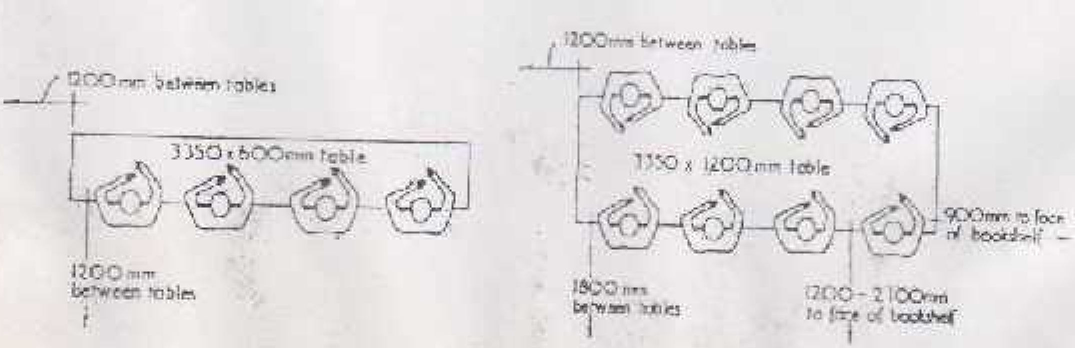


Figure 4.19 *Minimum clearance of tables in reading areas*

9. Carrels

Where users will require more privacy they can be accommodated in:

- Individual tables, Figure 4.20
- Dual-reading tables with screens, Figure4.21
- Open carrels, these can be placed within book stack areas, Figure 4.22, or
- Closed carrels, Figure 4.23

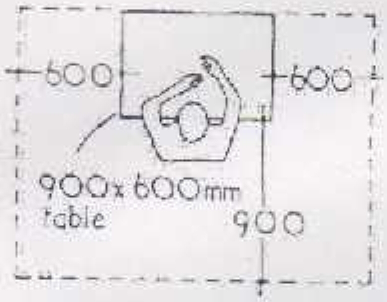


Figure 4.20 *Recommended minima for one –person reading tables*

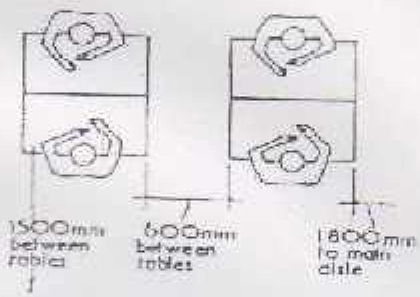


Figure 4.21 *Minima for dual-reading tables*

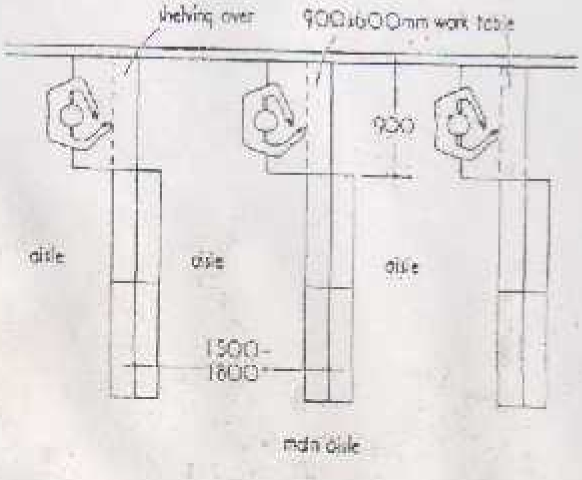


Figure 4.22 *Arrangement for open carrels in bookshelf areas*

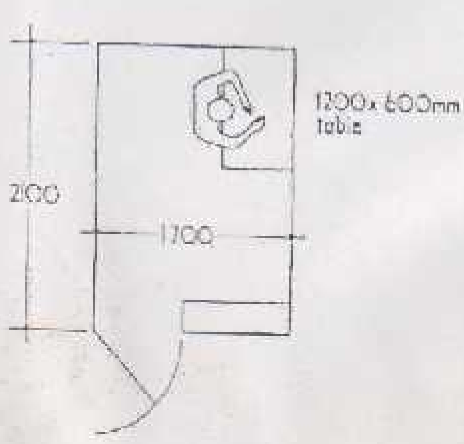


Figure 4.23 *Recommended single-person enclosed carrel*

10. Card catalog capacity

An essential part of any library is the catalogue. In the past this was kept almost invariably in the form of index cards, usually 6 in X 4 in kept captive in drawers in cabinets. Some libraries used paper slips in binders as an alternative method. The design criteria for such facilities shown in previous editions have been omitted, as they are now completely obsolete.

Nowadays, catalogues are kept on computer and this can be combined with monitoring the lending and acquisition functions. Monitors and keyboards will be required for all staff positions, and additional ones for the use of the public. These can be dumb terminals for a central computer, or more commonly standard workstations on a network. Such a network will require a network manager and storage for back-up material which may be tapes or
CD-ROMS.

4.3.3.3 Staff areas

This area is essential for staff library to service all kind of library users

1) *Lending and return book areas*
2) *Central information area*
3) *Manager of library (manager room, secretary*
4) *Meeting room*
5) *Arrival of material room*
6) *Processing room*
7) *Cataloging and classifying rooms*
8) *Maintainance room*
9) *International communication areas*
10) *Staff entrance*

1. Lending and return book areas

The area of lending and return book is determined according the number of students or readers that use library, and it may be combined with control and counters area.

2. Central information area

This areas is used to get some information about the materials of library and all essential information that important to readers and readerships

3. Manager of library (manger room, secretary,)

Administrative offices should include a combination library's office-trustee room; spaces for the assistant librarian and a secretary-receptionist; business office; and ether related office. Workroom areas should be provided for technical processing; reference, circulation, extension, end other departments; subject specialists; end supply storage.

4. Meeting rooms

With the exception of the very smallest libraries, much public library should provide some group meeting space, at least one multipurpose meeting room. At the other extreme, a smell auditorium end a series of conference rooms may be required. The service proposed by the library together with community needs for facilities of this type will be the final determinants.

5. *Arrival of material room*

Van access should preferably be under cover with a ramp to facilitate loading and unloading without lifting heavy packages.

If a ramp is not possible, a pulley should be fitted. Tables for unpacking will have to be provided and, particularly where satellite libraries have to be served and transfers of book stock organized, areas of immediate shelving (industrial shelving will be adequate). Space for storing packing cases, which may have to be

Reused, for disposal of rubbish, and access, direct or by hoist, to the accessioning area will be important.

6. *Processing room*

All library materials need some form of treatment before they are ready for use by readers. At the very least books have to be labeled and lettered, and although in very small libraries this can be done in the general workroom, in most cases there will need to be a special place in the flow line for this work. Books may have to be jacketed, laminated or strengthened, and this may call fairly heavy (and therefore static) machinery. In the largest libraries there will be a bindery; this will require very heavy equipment and will be laid out on the lines of a small factory.

7. *Cataloging and classifying rooms*

The layout of these sections will depend on whether the two processes are to be carried out together or separately.

The architect will know about the recommended allocation of space per office worker but it must be emphasized that cataloguers need space not only to work on the books in hand but also for the large number of bibliographical tools they use in their craft: 14 m^2 per cataloguer is a reasonable overall figure.

All the present tendencies are towards a reduction of the attention paid in any single library to the tasks of cataloguing and classification because centralized information can now be supplied through computers in tape form. For this reason alone flexibility in planning becomes more important, as does the sitting of accessions, cataloguing and classification in a single large area where space saved on one process can be used for another.

8. *Maintance room*

In large establishments there will be a workshop for those who have to maintain the elaborate service machinery, but in a small library if there is a caretaker-handyman who has to "make-and mend" then he should be provided with the space and the equipment to do his work properly. Benches, tool housing and storage for materials will be needed, as well as space for the inevitable chairs awaiting repair. In many libraries broken chairs are piled in odd corners where they are a hindrance and may be a fire hazard.

Cleaners will need space for storing materials as well as for vacuum cleaners and brushes (which can be tall). There should be a sink where buckets can be filled and dirty water poured away:

Washbasins should never be used for either purpose. These provisions should be made in each major area; cleaners too often have to haul buckets of water and cleaning materials for long distances because these operations had not been thought out at the planning stage.

9. *International communication areas*

A requirement often met with is a committee or boardroom for occasional use by the authority. Although this will increase the overall allocation it can hardly be grudged by the authority and moreover, if the room contains shelves it can also become a "special collections" room or be used for seminars or as an occasional office. It is very useful to have at least one room, which has no specific function at the outset; a need will certainly arise, even before the opening takes place.

10. *Staff entrance*

This is always necessary. It should be approachable by a well-paved path; there should be facilities for staff car, motorcycle and bicycle parking, preferably separate from the public provision, and under cover.

4.3.3.4 Services areas

These areas are divided into four main areas:

1) *Viewing areas*
2) *Networking areas*
3) *Photographic and printing areas*
4) *Lecture rooms*

1. Viewing areas:

Although the technology moves on all the time, it leaves its detritus in its wake. As many libraries are archive depositories, they will need to be able to access many obsolete forms of record. In particular, microform readers and printers of different types will be needed. So it must saved areas to use this facilities.

2. Networking areas

Much information is readily and often cheaply available through the Internet and other network facilities. Again, the same

Workstations may be used for this purpose, although they will need to be fitted with an appropriate monitoring and metering device to ensure that any fees incurred can be correctly attributed.

3. Photographic and printing areas

The range of work in this field can vary from the dry-copier, which requires little except the space on which it stands (plus an electric power point), to a full photographic laboratory complex employing many staff. Any library holding large and/or fragile material, which may have to be photocopied, will find that no single machine will economically do the whole range of work. Camera machines need professional photographers and if one of these is to be employed then his work must be scheduled so that his time is used most economically. Unless he is given non-professional assistance he will necessarily spend valuable time on tasks below his capability (and salary level). A large library therefore may need a full photographic department, the tasks to be carried out have to be scheduled, and operation movements visualized (preferably with the assistance of a professional photographer). This information will be handed to the architect who will plan the appropriate area.

This is the theory; in practice I believe that there are three areas for which the architect is not the best person to produce a layout— photographic department, the library bindery and printing department. In each case it is better to have the layout designed by the professional concerned, preferably the one who is to run the department.

4. Lecture rooms

Many libraries of all kinds need rooms for meetings, lectures, play-readings and similar community activities, often in the late evenings when the main library services have ceased for the day. The rooms may need separate approaches from outside the building and special arrangement must be made for security and public safety. Public lavatories may have to be associated with these rooms and in some cases cloakrooms, manned or unmanned. In the latter case the possibility of key-operated lockers should be explored. The County Libraries Section of the Library Association says, "Lecture rooms [in public libraries] as such, are not recommended for communities of under 25000 population" and goes on to recommend flexibility in the fittings of main rooms of small libraries so that these can be adapted for lectures after library hours. Many librarians will disagree strongly with this idea

4.3.3.5 Storing areas

There is no doubt that in most libraries there will be a continuing need for accommodation of conventional books for some time to come. So it must be to design area for storage books, bookshelf, tables and any another materials.

4.3.3.6 Other areas

These other areas are for both staff and readers to use that; the following points are some of main important areas:

1) *Toilets areas*
2) *Stairs*
3) *Elevators*

4.3.3 Car parking

A clear parking policy for each area is an essential when starting to any design or project.

There are no statutory requirements and few guidelines for the scale of parking provision. In appendix (B) Table (4.6) gives recommendations, but each specific case should be examined to determine expected requirements.

Once the scale of provision has been decided, the form will depend on the size and the shape of available area, and also on the type of vehicles expected.

So, we must take in our mind, car parking spaces and number of car required for university library are determined and collected in appendixes () which show the number of car needed for every building in a campus including library building.

4.4 Functions

There are some functions that must be considered in design. The most important one is slope of the rain on the roof and inside the floors, and the other is the fire protection that it must to save emergency exit and the last function is lighting in library.

The following points are the parts of this section:

4.4.1 Water Drainage

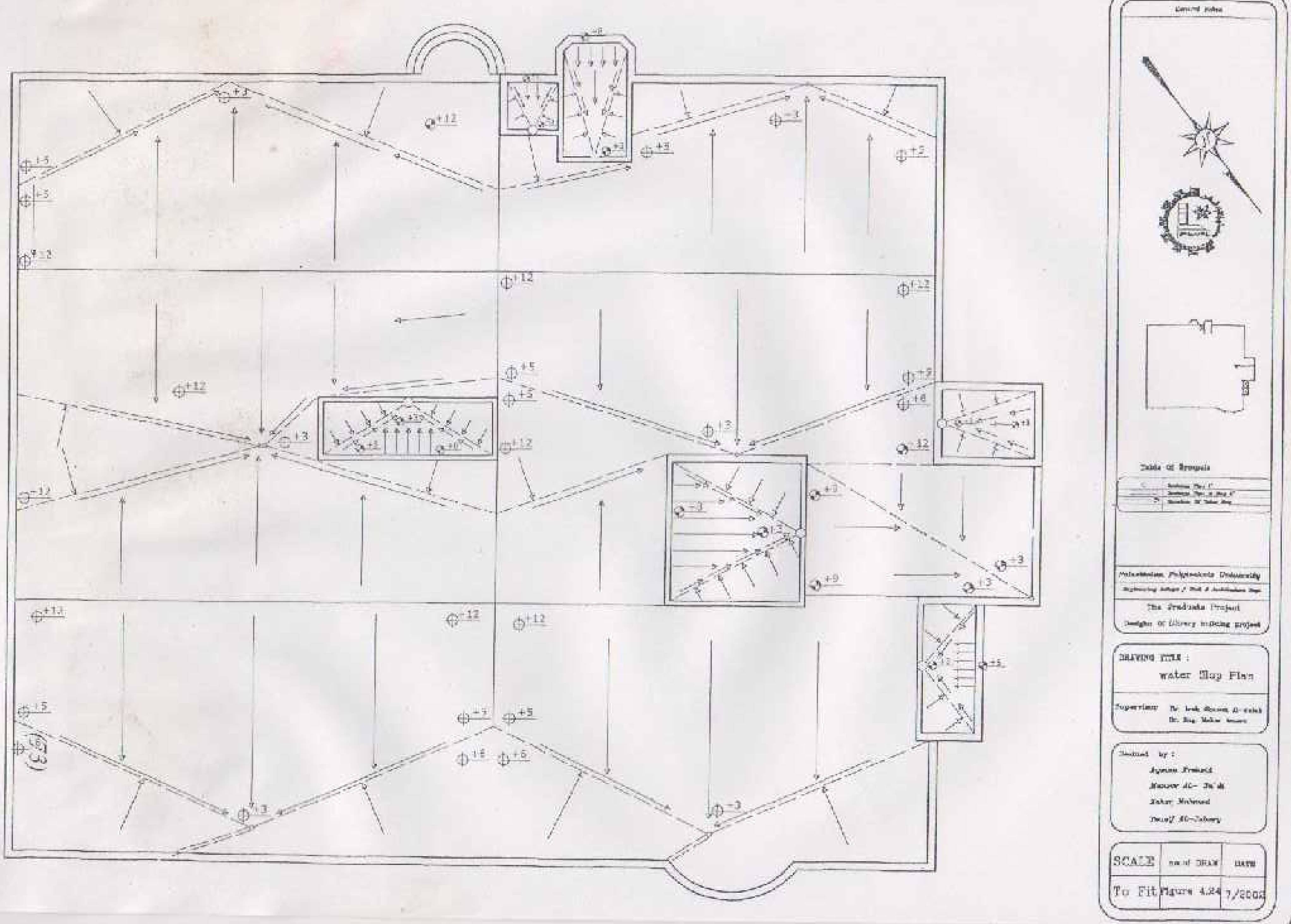
4.4.2 Fire Protection

4.4.3 Lighting

4.4.1 Water Drainage

We considered some solutions for this problems for library, where we divided the slab library roof to seven parts, so some ranges were made with slopes which lead the rain to gutters which installed in various positions as shown in figure (4.24). This way is the most effective one to deal with the rain, and it is cheep in comparison with other ways.

For Stairs and elevators roofs, the drainage system expends on drainage the water to the main roof of library and to pipes. So a sloping area designed to cover this distance and the slope lodes the water to a gutter next the passage to convey the water outside along the pips which come from the roof.



The Graduate Project
DRAWING TITLE :
water Slop Plan
SCALE
DATE
To Fit
Figure 4.24
7/2002

4.4.2 Fire Protection

When designing a building with fire safety in mind, particularly ease of escape, it is important not to forget the needs also for usability and security. If the fire measures are seen to be too onerous. The following subsections are very important points to design a building resists the fire hazard effects.

4.4.2.1 Principles of Fire Protection:

specific recommendations relating to periods of fire resistance for specific uses, structural elements etc are given in various codes only some of which are referred to here:

- Unrestricted growth and spread of a fire within a building that will cause extensive damage and may result in its collapse.
- Rapid spread of fire across surfaces within the building, ignition of adjacent fuels, means of escape prejudiced,
- Spread of fire, smoke and hot gases in a building through ducts, voids and shafts affecting the means of escape, access for firefighters and causing extensive damage to decorations and property,
- Spread of fire to adjacent buildings affecting life safety and property,
- Loss of contents, disruption of work, loss of trade/production.

4.4.2.2 Influencing Factors

The aim of fire precautions within a building is to inhibit the growth and to restrict the spread of any fire. The influencing factors are:

- The size of building — area, height, volume
- the layout and configuration within the building
- The uses accommodated, and the requirements of the occupants
- The construction materials, linings and cladding
- The type of construction
- The services installed
- The furniture.

4.4.2.3 the precautions are:

- Protection of load bearing structure to prevent untimely collapse, limitation of combustibility of key structural elements
- Adequate and appropriate provisions for means of escape. Access for fire-fighters up to and through the building to reach the seat of the fire and promptly extinguish it.
- Compart mentation and separation to restrict spread of fire, maintenance of these by protection of openings, fire stopping and cavity barriers within concealed spaces
- Safe installation and maintenance of services, heat-producing equipment and user equipment.
- Separation of different uses to protect, for example, a risk to sleepers from commercial uses.
- Enclosure of high risks with fire-resisting construction to protect adjacent areas.
- Active fire-extinguishing installations to detect and/or contain fire in its early stages and restrict its spread and growth.
- Limitation of flame spread by selective use of materials.

ↂ Fire-resisting external walls and/or space separation to prevent spread of fire to adjacent properties, protection of openings in external walls, limited flame spread across external walls and roofs, use of insulation with limited combustibility to restrict ignition and spread.

ↂ The provision of natural or mechanical ventilation, smoke extraction and/or smoke control measures to facilitate means of escape and firefighting.

ↂ Management training and procedures for evacuation, maintenance of fire precautions, risk analysis, management policy.

4.4.2.4 Fire Protection Methods:

The requirements for fire safety or methods for fir protection are:

1. *Means of escape:*

The building shall be designed and constructed so that there are means of escape in case of fire from the building to a place of safety outside the building capable of being safely and effectively used at all material times.

2. *Internal fire spread (linings)*

a. To inhibit the spread of fire within the building the internal linings shall

ↂ resist the spread of flame over their surfaces; and

ↂ have, if ignited, a rate of heat release which is reasonable in the circumstances.

b. In this paragraph 'internal linings' means the materials lining any partition, wall, ceiling or other internal structure.

3. *Internal fire spread (structure)*

a- The building shall be designed and constructed so that, in the event of fire, its stability will be maintained for a reasonable period.

b- A wall common to two or more buildings shall be designed and constructed so that it resists the spread of fire between those buildings. For the purposes of this subparagraph a house in a terrace and a semi-detached house are each to be treated as a separate building.

c- To inhibit the spread of fire within the building, it shall be subdivided with fire-resisting construction to an extent appropriate to the size and intended use of the building.

d- The building shall be designed and constructed so that the unseen spread of fire and smoke within concealed spaces in the structure and fabric is inhibited.

4. *External fire spread.*

a -The external walls of the building shall resist the spread of fire over the walls and from one building to another, having regard to the height, use and position of the building.

b- The roof of the building shall resist the spread of fire over the roof and from one building to another, having regard to the use and position of the building.

5. *Access and facilities for the fire service*

a- The building shall be designed and constructed so as provide facilities to assist firefighter in the protection of life.

b- Provision shall be made within the site of the building enable fire appliances to gain access to the building.

4.4.3 lighting

Choosing the best lighting for a library is a particularly complex problem because the lighting has to do several entirely different things:

1. to allow reading to take place in comfort
2. to contribute to the internal appearance of the buildings
3. to a lesser extent
4. to the external impact upon the passer-by.

For each of these there will be available artificial light, which is entirely controllable, and natural light which is very much less controllable. Because the human response to light is largely subjective, or at any rate conditioned, there are no absolute standards by which success can be judged and the librarian's experience can be as sound a guide as the architect's. Even a specialist lighting consultant will be expert on the best methods rather than the most acceptable results. -
to sole the lighting problem in the library : there are two aspect we can control that : natural lighting and artificial lighting

4.4.3.1 Natural Lighting

Any plan for natural lighting depends upon the architect's ideas on fenestration. Natural light is free, but it has three great disadvantages:

1. Whether through wall or roof, it imposes severe restrictions upon the flexible and economic use of floor and wall space.

2. Protection has to be provided against the concomitant heat, cold and glare: this can be extremely expensive. Window glass transmits not only light but heat. Solar radiant heat can be partially deflected and its absorbance and

transmittance reduced by the installation of special glass: many alternatives with varying properties are on the market but inevitably they will add to the cost.

3. There are enormous variations in intensity: a clear summer day can be twenty-five times brighter than a cloudy winter day, and as the human eye is very sensitive to change, variation of a tenth of this amount is unacceptable in continuous reading conditions. Also the continuous change in the angle of the light, although predictable, is often disturbing to the serious reader.

In this project ,to solve this problems and to use a large amount of natural lighting ,the aluminum glass windows are used in two elevations of library ,front and side elevations, for reading areas especially .

4.4.3.2 Artificial Lighting :

At present the most usual form of general lighting for reading areas consists of fluorescent lights recessed into a false ceiling and covered by diffusers. The effect is bright, efficient, cold and rather soulless. Among the advantages are flexibility, comparative cleanliness (and therefore economy of maintenance) absence of shadows and low consumption of current.

The architect will certainly wish to vary the lighting conditions in different parts of the building in order to indicate changes of environment and to add sparkle and interest. Combinations of lighting will be an important part of his aesthetic concept and he will break up regular and rather flat functional lighting by spots and chandeliers. In doing so he must consider carefully the effect which these lights may have upon serious readers and must design and position them to avoid glare. Lighting fittings exist to provide adequate visibility and to add to the attraction of the building, but only in exceptional cases should they draw attention to themselves.

4.5 architectural drawings :

This section is in appendix (C) which is an attachment with this project

Ch

5

STRUCTURAL STUDY

5.1 Requirments

5.2 *ACI Code for Design*

5.3 Reinforcement Requirements

5.4 Development Length

5.5 *Retaining Wall*

5.6 Loads

CHAPTER 5

STRUCTURAL STUDY

The structural design of library will be discussed in this chapter to find solutions for all members, all elements of a structure are reinforced concrete .

According to A.C.I code, the structure must be designed to resist and meet the required strength and serviceability.

5-1 Requirements

There are few requirements that must be taken as a case of a library design.

5-1-1 Cracking

Concrete, or rigid, structure are made from a mix of portland cement water and aggregate. Concrete structure have a long life and require little maintenance. Concrete is generally laid as a single thick layer directly over a base course. Concrete is usually laid in long sections or slabs of varying length. Metal bars or dowels inserted into the edges of the slabs to connect the joints where one slab ends and another begins.

Concrete is a strong material and can withstand compression, but it has a poor tensile strength (resistance to being pulled). When the ground underneath expands and contracts from seasonal or weather changes, the concrete becomes prone to cracking. Cracks can occur at or near the joints where concrete slabs meet or on the slabs themselves. Deep cracks can allow the broken concrete slabs to move upward or downward, creating an uneven slab surface. Metal bars or dowels inserted between the slabs help hold the slabs together. Reinforced concrete contains steel bars or mesh imbedded within the concrete layer. The steel helps hold concrete together over time, even if cracks occur. Unreinforced concrete may be used when cost is a factor, or where weather conditions are more mild. Unreinforced slabs have several shallow grooves cut into them, allowing the concrete to crack at defined points. The cracked slabs are kept in place by pressure and by the grainy texture of the concrete itself.

In ACI code, shrinkage and temperature reinforcement is required at right angle to the principle reinforcement to minimize cracking and to tie the structure together to assure its acting as assumed in the design.(1)

5-1-2 Fire-Resistance Requirements

The provision of structural fire-resistance requirements in building design and building codes is intended primarily to ensure building integrity for a certain period of time under fire conditions and to permit evacuation of occupants and access for firefighters.

To meet fire-resistance requirements specified in building codes, the fire resistance of individual building elements has to be determined. Often, subjecting specimens to costly and time-consuming fire tests does this. Recent developments, however, have made it possible in many cases to calculate fire resistance using mathematical models. Developed to simulate the behavior of building elements exposed to fire, such as beams, columns, floors and walls, these models make it possible to predict the fire resistance of most building elements for a wide range of practical conditions.(5)

The fire-resistance class for concrete structures can be achieved by choosing the value of concrete cover for constructive measure. Isolating heat and preventing it from reaching iron is achieved by using the concrete cover. Both iron and the concrete cover have the same thermal expansion coefficient (αt). The strength of the iron is reduced and as a result members suffer from weakness. The building has been designed in such away that the member would not be exposed to heat for a period of 90 minutes.

5-2 ACI Code for Design :

The strength of a structure depends on the strength of the materials from which it is made. For this purpose, minimum material strengths are specified in standardized ways.

Actual material strengths cannot be known precisely. Structural strength depends, on the care with which a structure is built, which in turn reflects the quality of supervision and inspection. Members sizes may differ from specified dimensions, reinforcement may be out of position, poorly placed concrete may show voids, etc, this can reduce the strength of the structure.

The design strength provided by a member, its connections to other members, and its cross-sections in terms of flexure, and load, shear, and torsion is taken as the nominal strength calculated in accordance with the requirements and assumptions of ACI-code.

5-2-1 Strength Design Method:

In the strength design method, the service load are increased sufficiently by factors to obtain the load at which failure is considered to be (imminent).

This load is called factored load or factored service load .The structure or structural element is then proportioned such that the strength is reached when the factored load is acting . The computation of this strength takes into account the non-liner stress-strain behavior of concrete.

The strength design method may be expressed by the following :

Provide Strength ≥ Required Strength to carry factored loads

$\Phi M_n \geq M_u$ Bending Moment

$\Phi P_n \geq P_u$ Axial Force

$\Phi V_n \geq V_u$ Shear Force

Where the provided strength is computed in accordance with rules and assumption of behavior prescribed by a building code and the required strength is that obtained by performing a structural analysis using factored load.

The provided strength has been commonly referred to by practitioners as (ultimate strength) , However it is a code defined value for strength , and is not necessarily (ultimate) in the sense of being a value above which it is impossible to reach .

The ACI code uses a conservative definition of strength thus the modifier (ultimate) is not appropriate when the strength design method is used , the comparison of provided strength with required strength (that is, axial force , shear , or bending moment caused by factored loads) does not imply that any material (yields) or (fails) under service load conditions .

In fact, at service loads the behavior of the structure is essentially elastic. The use of the term (imminent) under factored loads is only a device for establishing adequate safety parameters.

5-2-2 Safety Factors :

Structures and structural members must be designed to carry some reserve load above what is expected under normal use . Such reserve capacity is provided to account for a variety of factors that may be grouped in two general categories :

1 - Factors relating to over load.

2 - Factors relating to under strength .

The purpose of a safety provision is to limit the probability of failure and yet permit economical structure , obviously , if cost is no object, it is easy to design a structure whose probability of failure is nil .

Table 5.1 *Over Load Factor*

Load	Factor
Dead load	1.4
Live load	1.7

Table 5.2 *Under Strength Factors :*

Capacity	Φ
Flexure	0.90
Shear	0.85
Compression , spirally	0.75
other	0.70
Plain Concrete	0.65

5-2-3 Design of Selected Member according to ACI Code :

Design of members for flexure and axial loads is based on satisfying the application of equilibrium and compatibility of strains

5-2-3-1 *One-Way Ribbed Slab :*

Any structural system consists of various elements such as slabs , beams , columns and footings .These elements are combined in various ways to create structural systems. The types of flooring systems are the one way and two way systems, in this project the type of flooring system is chosen to be one-way ribbed slab only.

In the selection of flooring system, the designer should take into account various circumstances such as :

1. The type of building, and the use of floor.
2. The dimension of existing panels.
3. Economical considerations.
4. Availability of workmanship.
5. Serviceability requirements.

The slab system is designed as ribbed one-way slab.

Dimensions of the rib

To select the dimensions of one-way ribbed slab, slab thickness should be taken into account, where slab thickness must be sufficient to control deflection in addition to strength requirements.

This table 5.2 gives minimum values of slab thickness required to control deflection according to ACI code if there are no non-structural members that may be damaged by excessive deflections.

Table 5.3 Minimum values of slab thickness

Member	Thickness			
	Simply supported	One end continuous	Both end continuous	Cantilever
Solid one way slab	L/20	L/24	L/28	L/10
Beams or ribbed slab	L/16	L/18.5	L/21	L/8

However, when the depth of slab cannot control deflections, dropped beams are used , since these beams increase the rigidity of the slab.

For one way ribbed slab:

H min = L/18.5 (one end continuous)

H min = 5.3/18.5

H min = 28.64 cm

Take H min = 30 cm

Width of rib (b_w) is grater than 10 cm.

Take = 12cm

5-2-3-2 Beam

Beams are structural elements carrying transverse loads (that cause bending moments and shear forces along their length) from slabs and transfer them to the girders or directly to the columns.

There are two main kinds of beams are flanged or rectangular according to shape, sections having reinforcement only in the tension side are called singly reinforced section but sections having reinforcement in both tension and compression side are called doubly reinforced section.

Usually in ribbed slabs, the beams have the same height of the slab are named hidden beams, they are designed as rectangular beam for both negative and positive moments.

Sometimes, the depth required to control deflection is greater than the depth of slab in which case dropped beams can be used. In this project where the depth of slab control the deflection, hidden beams are used in the design.

Design for flexural reinforcement

In flexural design, the preliminary dimensions are assumed to be the actual dimensions since the calculations approximately yields the same dimensions.

All sections for positive and negative moments are rectangular sections.

The following steps are used in the design process: -

1 - The negative and positive moments are calculated.
2 - The nominal flexural strength Mn is determined.
3 - Strength coefficient of resistance Rn is calculated.

4 - Ratio of reinforcement area to gross concrete area is determined and compared with maximum ratio of reinforcement from table (5.4) to know if section needs doubly or singly reinforcement.

Table 5.4 *Maximum reinforcement ratio*

f_y	$f'_c = 20(MPa)$ $\beta_1 = 0.85$	$f'_c = 25(MPa)$ $\beta_1 = 0.85$	$f'_c = 30(MPa)$ $\beta_1 = 0.85$	$f'_c = 35(MPa)$ $\beta_1 = 0.81$	$f'_c = 40(MPa)$ $\beta_1 = 0.77$
300 MPa	0.0241	0.0301	0.0361	0.0402	0.0436
350 MPa	0.0196	0.0244	0.0293	0.0326	0.0354
400 MPa	0.0163	0.0203	0.0244	0.0271	0.0295
f_y	$f'_c = 200$ (Kg/cm^2) $\beta_1 = 0.85$	$f'_c = 240$ (Kg/cm^2) $\beta_1 = 0.85$	$f'_c = 280$ (Kg/cm^2) $\beta_1 = 0.85$	$f'_c = 320$ (Kg/cm^2) $\beta_1 = 0.82$	$f'_c = 360$ (Kg/cm^2) $\beta_1 = 0.79$
2800 Kgf/cm^2	0.0266	0.0319	0.0372	0.0410	0.0444
3500 Kgf/cm^2	0.0197	0.0236	0.0276	0.0304	0.0330
4200 Kgf/cm^2	0.0153	0.0184	0.0214	0.0236	0.0256

Beams design Data :

$f'_c = 30(MPa)$

$f_y = 420(MPa)$

$\rho_{min} = 0.0033$

$\rho_{max} = 0.023$

5-2-3-3 Columns

Columns are compression elements, failure collapses of the adjoining floors and the ultimate total collapse of the entire structure. Thus, extreme care needs to Columns are vertical compression members that transmit loads from the upper floors to the lower levels and then to soil trough the foundations. Since be in column design, with higher reserve strength than the case of beams and other horizontal structural elements, particularly since compression failure provides little visual warning.

A column has been defined as a member used primarily to support axial compressive *load with a ratio of height to least lateral dimension greater than 3* .

all columns are subjected to some bending moment, which may result from: -

1. Unbalanced floor loads on both exterior and interior columns.
2. Eccentric loads such as crane loads in industrial buildings.
3. Lateral loading such as from wind or earthquake.

In our project we consider the column is unloaded by bending moment force

Reinforced concrete columns are principally of two types classified according to the manner in which the longitudinal reinforcing bars are laterally supported: -

1 - Tied columns : usually square, rectangular and circular in shape.

2 - spirally reinforced column, usually of square or circular shapes one in which the longitudinal reinforcing bars are arranged in a circle and wrapped.

The ACI code classifies columns according to their behavior in two main types:

1. *short column*: - Is the column that reaches its ultimate compressive strength before buckling takes place.
2. *Long column*:- is the column whose strength is controlled by buckling ;

(ex : buckling takes place before the column reaches its ultimate compressive strength).

For design the column (axially loaded)

$$P_n = \frac{P_u}{\Phi} \qquad \text{where : } \Phi = 0.56$$

$$0.01 \le \rho \le 0.08$$

$$\Phi P_{n(max)} = 0.8\Phi\left(0.85 f_c'\left(A_g - A_{st}\right)\right)$$

$A_g = a \times b$ gross area

$A_{st} = \rho \times a \times b$ area of steel

$a \,\&\, b$ dimensions of column

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

48 d_{bt}

16 d_b

Least dimension of column

5-2-3-4 *Foundation:*

Building loads are transmitted by columns or bearing walls (Shear walls) to foundation, A foundation is the lower part of a structure which transmits load to the under laying soil without causing a shear failure of soil or excessive settlement.

The foundation is classified as shallow and deep foundation according to the depth of construction. The stability of foundation depends upon the safety of a soil against :

(a) Shear failure.

(b) Excessive settlement.

The main purpose of foundations is: -

1) To distributed weight of super structure to the deep soil layer.

2) To prevent deferential settlement due to uneven pressure distribution.

3) To increase stability of super structure by preventing overturning failure.

Site exploration must be made to investigate the type of soil or rock in the building site, the main objectives of site exploration: -

1) To determine safe bearing capacity of soil.

2) To determine the foundation depth.

3) To select the most appropriate and economical type of foundation.

The allowable bearing capacity of soil is assuming (σ = 3.5 Kg/cm2) .

Type of foundation :

Most building footing may be classified as one of the following types: -

1 - Isolated spread footings under individual columns, these may be square, rectangular or circular in plane .

2 - Wall footing , rather flats or stripped, which support bearing walls.

3 - Combined footing support two or more column loads .

4 - Mat foundation which is one large continuous footing supporting all the columns of the structure, this is used when soil conditions are poor

5 - Pile caps, structural elements that tie a group of piles together.

Foundation Design :

1 – Footing Area :

assume depth of the footing (H) and compute

- Service Load
- Footing Weight
- Earth Pressure
- Total Weight

Area (A) = Total Weight / Soil Pressure

2 – Shear Strength :

- Check this depth for one way action :

$$P_{net} = \frac{P_u}{Area}$$

$$V_u = P_{net} \times d \times W$$

When No shear reinforcement is used :

$$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d$$

$$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d$$

$$\Phi V_c > V_u \qquad \text{OK.}$$

- Check this depth for two way action *(punching)* :

$$V_u = P_{net} \times \left((W) \times (L) - (a+d)(b+d) \right)$$

When No shear reinforcement is used :

$$V_c = \frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d$$

$$V_c = \frac{1}{12}\left(\frac{\alpha_s}{b_o / d} + 2\right)\sqrt{f_c'}\, b_o d$$

$$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \qquad \text{The minimum is control}$$

$$\Phi V_c > V_u$$

3 – Bending Moment :

$$\text{Mu} = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$$\text{Mn} = \frac{Mu}{\Phi}$$

$$\text{Rn} = \frac{Mn}{bd^2}$$

Req. A_s

4 - Development Length (L_d) :

$$L_d = \frac{f_y}{2\sqrt{f_c'}} \times d_b$$

5 - Dwell Req.

Minimum Dwells Req. = 0.005 (a)(b)

5-3 Reinforcement Requirement:

5-3-1 Balanced Conditions :

A balanced strain condition exists at cross section when the maximum strain at the extreme compression fiber just reaches (0.003) simultaneously with the first yield strain (f_y / E_s) in the tension reinforcement . The reinforcement ratio (ρ bd) which produces balanced conditions under flexure depends on the shape of the cross section and the location of the reinforcement .(3)

Maximum Reinforcement Ratio (ρ max) for beams :

The maximum amount of tension reinforcement in flexural members is limited to insure a level of ductile behavior .

$\rho_{max} = 0.75\rho_b$ ACI 10.3.3

Minimum Reinforcement Ratio (ρ min) :

The provision for a minimum amount of reinforcement applies to beams for architectural or other reasons are much larger in cross section than required by strength considerations with a very small amount of tensile reinforcement . The computed moment strength a reinforcement concrete section becomes less than that of the corresponding plain concrete section computed from modulus of rupture . Failure in such a case can be quite sudden .

The minimum reinforcement ratio is :

$$\rho_{min} = \frac{1.4}{f_y} \qquad \text{but not less than}$$

$$\rho_{min} = \frac{\sqrt{f'_c}}{4 f_y}$$

5-3-2 Design Assumptions :

The strength of a member computed by the strength design method of the code requires that two basic condition be satisfied :

1. Static Equilibrium
2. Equilibrium between compressive and tensile force acting on the cross section at nominal strength must be satisfied.

Design For Bending Moment :

The dimensions are limitted by the architectural drawing . The procedure to be used in the strength the design of rectangular sections with tension reinforcement only involves the following steps :

1- Compute $m = \dfrac{f_y}{0.85 f'_c}$

2- Compute $R_n = \dfrac{M_u}{bd^2}$

3- Compute $\rho = \dfrac{1}{m}\left(1 - \sqrt{1 - \dfrac{2mR_n}{f_y}}\right)$

4- Compute $A_s = \rho bd$

5- Select reinforcement and check the static equilibrium and the equilibrium between compressive and tensile force acting on the cross section at nominal strength.

C = T

$$0.85 f'_c ab = A_s f_y$$

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad \text{then}$$

Check the steel is yield or not ?

$$X = \frac{a}{\beta_1} \qquad \text{where}$$

$$\beta_1 = 0.85 \qquad \text{for} \quad f'_c \le 30MPa$$

$$M_{n(provided)} = (Cc) or (T)\left(d - \frac{a}{2}\right)$$

Check :

$$M_{n(provided)} \ge M_{n(required)}$$

5-3-3 Shear Strength :

High shear strength on a beam results in the formation of inclined cracks. In order to prevent the formation of inclined cracks, transverse reinforcement (known as shear reinforcement) .

In the form of closed or U shaped stirrups is used. Normally in the vertical direction to enclose the main longitudinal reinforcement along the face of the beam.

Shear Reinforcement :

The shear reinforcement in abeam can be of three different kinds :

1. *vertical stirrups*
2. *bent-up bars*
3. *combination of both (1) & (2)*

Spacing For Shear Stirrups :

The actual spacing of the stirrups will be decided by the designer from a shear force diagram with respect (ACI 11.5.4)

The shear strength is based on average shear stress on the full effective cross section (bw*d) . In a member with our shear reinforcement shear is assumed to be carried by the concrete web . In a member with shear reinforcement, a portion of the shear is assumed to be provided by the concrete and the remainder by the shear reinforcement .

The shear strength provided by concrete (Vc) is assumed to be the same for beams with and without shear reinforcement and is taken as the shear causing significant inclined cracking.

Design of shear reinforcement:

1 – Minimum shear reinforcement :

$$V_{s(min)} = \frac{1}{3}(MPa)b_w d$$

2 - Maximum shear reinforcement :

$$V_{s(max)} = \frac{2}{3}\sqrt{f'_c}\,b_w d$$

$$\Phi V_{s(Required)} = V_u - \Phi V_c$$

where :

$\Phi = 0.85$

V_u : ultimate shear force at critical section (a way (d/2) from the face of support)

$V_c = \frac{\sqrt{f'_c}}{6} b_w d$ (concrete force which resist the shear force)

$V_c = \frac{\sqrt{f'_c}}{6} b_w d$ (concrete force which resist the shear force)

5-4 Development Length

A basic requirement in reinforced concrete construction is that three is adequate means for transfer of the force in the reinforcement to the surrounding concrete. This transfer may arise from the adhesion at the surface area of the reinforcing bar as well as from bearing of the raised ribs, or "lugs", of the deformed bars against the concrete. The interacting force preventing slip of the longitudinal bars relative to the surrounding concrete has traditionally been referred to as bond

The more common failure mode is "splitting" of the surrounding concrete , resulting from excessive bearing of the reinforcing bar deformations against that concrete In this case, the development length is roughly proportional to the bar area, Since splitting depends on the ability of the concrete to resist tension, favorable condition are increased concrete cover and larger spacing between bars, as the confinement capabilities of any ties and stirrups.

The moment strength (capacity) of a beam is, therefore, a three dimensional relationship involving not only the cross-sectional properties at a location along the span, but also the embedment lengths of the steel bars in both directions there from

Development length for bottom bars :

$$L_d = \frac{f_y}{2\sqrt{f'_c}} \alpha\beta\lambda(d_b) \quad \textbf{for} \quad \Phi \le 20mm \quad \textbf{bars}$$

$$L_d = \frac{5f_y}{8\sqrt{f'_c}} \alpha\beta\lambda(d_b) \quad \textbf{for} \quad \Phi \ge 22mm \quad \textbf{bars}$$

Development length for top bars

$L_d = 0.3 \times L_n$

where : L_n : Clear spacing between supports

5-5 Retaining Wall

Earth retaining walls are required for a variety of uses in many civil engineering schemes, to retain the lower portion of embankments and cuttings, around the perimeter of leveled sites, wing walls to bridges, basement walls to buildings, etc..

In principle, a reinforced concrete retaining wall consists of a vertical cantilever with a base to provide stability against overturning. The base should also incorporate some means of preventing the whole wall from sliding forward under load.

5-6 Loads :

The design method divides the load idea in two categories, the service dead and live loads (or dead load and live load effects), and in the design a constant load is taken into consideration , Loads that act on structures can be divided into three broad categories: dead loads, live loads, and environmental loads .

The design loading for a structure is generally specified in codes. In general, the structural engineer works with two types of codes: general building codes and design codes. *General building codes* specify the requirements governmental bodies for minimum design loads on structures and minimum standards for construction .

In these codes reference is often made to *design codes,* where more detailed technical standards are established for the actual structural design that codes provide only a general guide for design . The ultimate responsibility for an appropriate judgment lies with the structural engineer.

A structure is generally subjected to two types of loads; dead loads and live loads. A brief discussion of these loadings will now be presented to illustrate how one must consider their effects in practice.

5-6-1 Dead loads :

Dead loads consist of the weights of the various structural members and the weights of any objects permanently attached to the structure.

Hence, for a building, the dead loads include the weights of the columns, beams and girders, the floor slab, roofing, walls, windows, plumbing, electrical fixtures, and other miscellaneous attachments.

The actual design of a structure begins with those elements first subjected to the primary loads that the structure is intended to carry and proceeds in sequence to the various supporting members until one reaches the foundation.

Thus, a building floor slab would be designed first, followed by the supporting beams, columns, and last, the foundation footings. Because of this sequence, the actual weight of the slab will generally not be that important in the design of the slab, however, an accurate estimate of this loading becomes important when designing the supporting beams. Likewise, the beam weights are generally neglected in their design; however, several beams supported by a single column may require an accurate estimate of their weights for the design of the column.

5-6-2 Live loads:

Live loads can vary both in their magnitude and location. They may be caused by the weights of objects temporarily placed on a structure, moving vehicles, or natural forces.

The minimum live loads specified in codes are determined from studying the history of their effects on existing structures. Usually, these loads include an allowance for some protection against excessive deflection or sudden overload.

Various types of live loads will now be considered :

5-6-2-1 Building loads:

The floors of buildings are assumed to be subjected to uniform live loads, which depend on the purpose for which the building is designed. These loadings are generally tabulated in local, state, or national codes. A representative example of such loadings, taken from Table (5-5) in appendix.

5-6-2-2 Wind loads:

In the case of library building ,by this we mean, load which may result from the wind force. The process of loading the wind — load kill is shown when we design the shear walls. Shear walls are members used to give the structure the required stability against horizontal loads and consequently the building wont move . Stairs have shear walls, and these shear walls stabilize the building and resist the horizontal force , these stairs will be reinforced according to the design of shear walls.

5-6-2-3 Snow loads :

In some parts of the country roof loading due to snow can be quite severe and therefore protection against possible failure is of primary concern . Design loadings typically depend on the building's general shape and roof geometry, wind exposure, and location. Like wind, snow loads are generally determined from a zone map reporting 50-year recurrence intervals of an extreme snow depth.

For example, in some western states and in the northeast (45 Ib/ft2) is commonly used for design . No single code can cover all the implications of this type of loading. Instead, the engineer must use judgment regarding the possibility of additional effects caused by rain, snow drifting or movement, and whether the building to be designed is to be heated.

5-6-2-4 Earthquake loads:

The effect of an earthquake on a structure depends on the amount and type of ground accelerations and the mass and stiffness of the structure. Some local codes require that specific attention be given to earthquake design, especially in areas of the country where strong earthquakes predominate .

During an earthquake the ground vibrates both horizontally and vertically. The vertical motion is slight, and is usually neglected in design. As a consequence of the horizontal accelerations, shear forces in a building supporting columns try to put the building in sequential motion with the ground. Once the earthquake response spectrum is specified, these forces can be determined using the theory of structural dynamics. The analysis is often quite elaborate and requires the use of a computer. Although this is the case, such an analysis becomes mandatory if the structure is large. For small structures, however, many codes allow a more simplistic approach to earthquake analysis, which is based on determining the column shear force V using an equivalent static load .

5-6-2-5 Hydrostatic and Soil Pressure :

When structures are used to retain water, soil or granular materials, the pressure developed by those loadings becomes an important criterion for their design. Examples of such types of structures include tanks, dams, ships, bulkheads, and retaining walls. Here the laws of hydrostatics and soil mechanics are applied to define the intensity of the loadings on the structure.

5-6-2-6 Other National loads :

Several other types of live loads may also have to be considered in the design of a structure, depending on its location or use .These include the effect of blast, temperature changes, and differential settlement of the foundation . Like the other effects mentioned above, design codes usually specify the limitations of these loadings and which combinations of them should be applied simultaneously so as to cause the maximum, yet realistic live-load effects on the structure.

Ch

6

DESIGN STRUCTURAL ANALYSES

CHAPTER 6

DESIGN STRUCTURAL ANALYSES

Chap 6

In this ~~project, all the~~ design calculations for all structural members were made upon structural system which chosen in the previous chapter. All slabs are ribbed slab system and were analyzed by using finite element method, with the aid of a computer program called (mb software) to find internal forces and moment, and then manual calculations were made to find the steel required for all members.

Also, design of columns, beams, foundation, and stairs were made in this chapter, and can be shown in the following sections in details.

The design process started from the top to the bottom of structure, so numbers were given to each member for simplifying and classifying these members. The key plans which show the keys for each member can be shown in the following figures, then the calculation started step by step from the roof to the foundation.

6.1Structural Key Plans

These structural key plans are attachement in appendix (E) .

6.2 Ribs Design

6-2-1 Loads Calculation

The main loads acting on the structure are dead & live loads , Dead Load is calculated based on the density for each material used in the slab:

Tile	(0.03)(0.62)(20.0) = 0.372 KN/m
Coarse Sand Fill	(0.10)(0.62)(20.0) = 1.240 KN/m
Concrete Cover	(0.10)(0.62)(24.0) = 1.488 KN/m
Concrete Rib	(0.12)(0.20)(24.0) = 0.576 KN/m
Block	(0.50)(0.20)(9.00) = 0.900 KN/m
Plaster	(0.03)(0.62)(20.0) = 0.372 KN/m
Partitions	(1.25)(0.62) = 0.775 KN/m

Nominal Total Dead Load = 5.73 KN/m

Factored Total Dead Load = 1.4(5.73) = 8.022 KN/m

= 8.022/0.62 = 12.94 KN/m ^{2}

$\cong$ 13 KN/m ^{2}

Live load for top roof = (1 KN/m^2)(0.62 m) = 0.62 KN/m

Factored live load for top roof = 1.7 (0.62 KN/m) = 1.06 KN/m

$\cong 1.78$ KN/m^2

Live load for other roof = (7.2 KN/m^2)(0.62 m) = 4.46 KN/m

Factored live load for other roof = 1.7 (0.62 KN/m) = 7.59 KN/m

$\cong 12.3$ KN/m^2

6-2-2 Slab (Cover) Design :

1 - For Top Roof :

Live load = 1.78 KN/m^2 = 0.178 Ton/m^2

Dead load = 1.3 – 1.4(0.0576/0.62) – 1.17 Ton/ m^2

$W_u = 1.17 + 1.7(0.178) = 1.47$ Ton/ m^2

Assume slab is fixed at support point (ribs)

$$M_u = \left(\frac{1.47 \times 0.5^2}{12}\right) = 0.0306 \text{ Ton.m}$$, for 1 m wide strip

According to ACI (9.5.2.3)

$$f_r = 0.7\sqrt{f'_c}(MPa) = 0.7\sqrt{30} = 3.83(MPa) = 38.3(Kg/cm^2)$$

$$Mn = (f_r)(s)$$

$$s = \frac{bh^2}{6} = \frac{100 \times 10^2}{6} = 1666.67 cm^3$$ ……. for a rectangular X-section

ΦMn = 0.65 (38.3)(1666.67) = 41491.75 Kg.cm , Φ = 0.65 for plain concrete

= 0.415 Ton.m

ΦMn = 0.415 Ton.m > Mu = 0.0306 Ton.m

∴Provide Shrinkage & Temperature Reinforcement :

$A_s = 0.0018(100)(10) = 1.8\ cm^2/1m$

Use Φ 8 @ 25 cm on center both side

Provided $A_s = 2\ cm^2/1m$

2 - For Other Roof :

Live load = 12.3 KN/m^2 = 1.23 Ton/m^2

Dead load = 1.3 − 1.4(0.0576/0.62) =1.17 Ton/ m^2

$W_u = 1.17 + 1.7(1.23) = 3.62$ Ton/ m^2

Assume slab is fixed at support point (ribs)

$M_u = \left(\frac{3.62 \times 0.5^2}{12}\right) = 0.0754$ Ton.m , for 1 m wide strip

According to ACI (9.5.2.3)

$$f_r = 0.7\sqrt{f'_c}(MPa) = 0.7\sqrt{30} = 3.83(MPa) = 38.3(Kg/cm^2)$$
$$Mn = (f_r)(s)$$

$s = \frac{bh^2}{6} = \frac{100 \times 10^2}{6} = 1666.67 cm^3$ .... for a rectangular X-section

ΦMn = 0.65 (38.3)(1666.67) = 41491.75 Kg.cm , Φ = 0.65 for plain concrete

= 0.415 Ton.m

ΦMn = 0.415 Ton.m > Mu = 0.0754 Ton.m

∴Provide Shrinkage & Temperature Reinforcement :

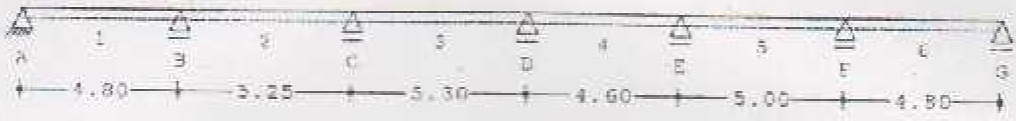
$A_s = 0.0018(100)(10) = 1.8\ cm^2/1m$

Use Φ 8 @ 25 cm on center both side

Provided $A_s = 2\ cm^2/1m$

6-2-3 Ribs Design

RIB (1)



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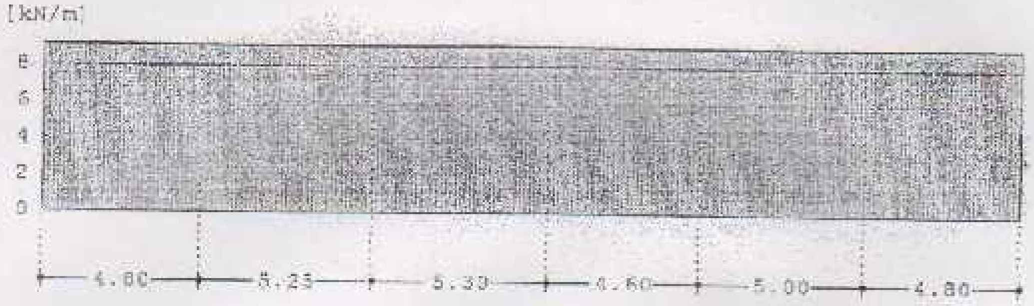
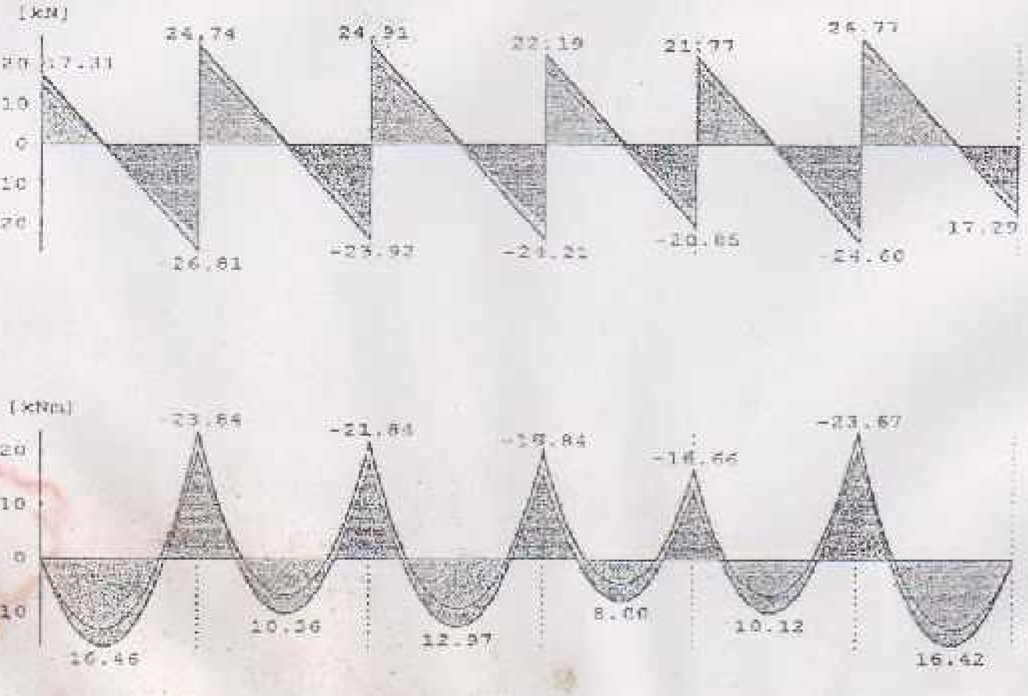


Loads



Shear & Moment Envelope Diagram
Fig. (6-1)

1 - Design For Positive Moment :

Effective Flange width (b_E) according to ACI code 8.10.2 :

b_E of T- section is the smallest of the following :

b_E = L / 4 = 530 / 4 = 132.5 cm

$b_E = b_w + 16\,t = 12 + 16\,(10) = 172$ cm

b_E = C/C = 62 .... control

Mu = 16.46 KN.m As shown in Fig. (6-1)

Mn = 16.46 / 0.9 = 18.29 KN.m = 1.83 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

$C = 0.85\ fc\,t\,b_E = 0.85\,(0.3)\,(10)\,(62) = 158.1$ Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required = 1.83 Ton.m

Design as a rectangular with b_E = 62 cm

As max.= ρ bd

As max. = 0.023 (62) (26) = 37.08 cm^2

$$As\ \min = \frac{\sqrt{30}}{4(420)}(12)(26) \geq \frac{1.4}{420}(12)(26)$$

As min = 1.02 ≥ 1.04

As min = 1.04 cm^2

$$m = \frac{fy}{0.85\,fc'} = \frac{420}{0.85(30)} = 16.47$$

$$Rn = \frac{Mn}{bd^2} = \frac{1.829\ (10)^5}{(62)(26)^2} = 4.36\ Kg\ /\ cm^2$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mRn}{fy}}\right) = 0.00105$$

As = 0.00105 (62) (26) = 1.69 cm^2 > As min ∴ OK

Use 2 Φ 12 mm As = 2.26 cm^2

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C = 0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) = 9.49 Ton

Actual a = 0.6 cm

Actual x = a / β = 0.6 / 0.85 = 0.71 cm

ε_s = 0.106 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mn = 2.44 Ton.m > 1.83 Ton.m

2 - *Design For Negative Moment :*

Mu = 23.84 KN.m As shown in Fig. (6-1)

Mn = 23.84 / 0.9 = 26.48 KN.m = 2.65 Ton.m

m = 16.47

Rn = 6.32 Kg / cm^2

ρ = 0.00152

As = 0.00152 (62) (26) = 2.46 cm^2

Use 1 Φ 12 mm & 1 Φ 14 mm As = 2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) = 11.21 Ton

Actual a = 0.71 cm

Actual x = 0.83 cm

$\varepsilon_s = 0.09098 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 3.64 Ton.m > 2.65 Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 2.7$ Ton

V_u @ critical point = 2.4 Ton

$$\Phi V_c = 0.85\left(\frac{\sqrt{fc'}}{6}\right)bd = 0.85\left(\frac{\sqrt{30}}{6}\right)(62)(26)\left(\frac{10}{1000}\right) = 12.51 \text{ Ton}$$

$0.5\ \Phi V_c = 6.25 \quad > \quad V_u = 2.4$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

use Φ6 for 30 cm

4 - Development length (l_d) :

L_d for bottom bars (Φ 12) :

$$l_d = \frac{f_y}{2\sqrt{f_c}}\alpha\beta\gamma\ d_b = \frac{420}{2\sqrt{30}}(1)(1)(1)\ (1.2) = 46 \text{ cm}$$

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ………………. control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

106.1+ 26 ≥ 46 cm

132.1 cm ≥ 46 cm ∴ available development > Req.

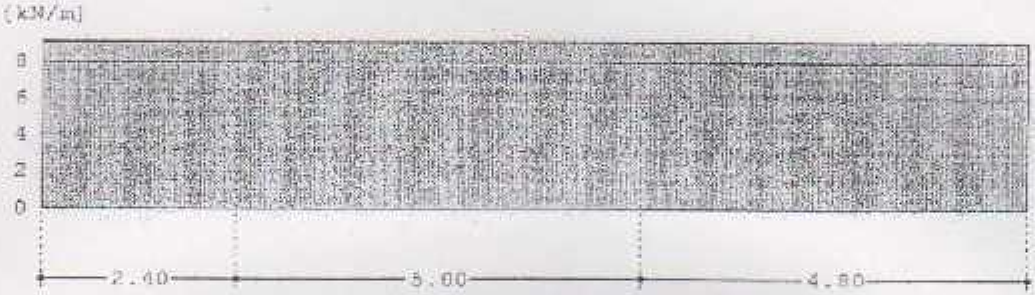
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 − 2(0.15)) = 1.5 m

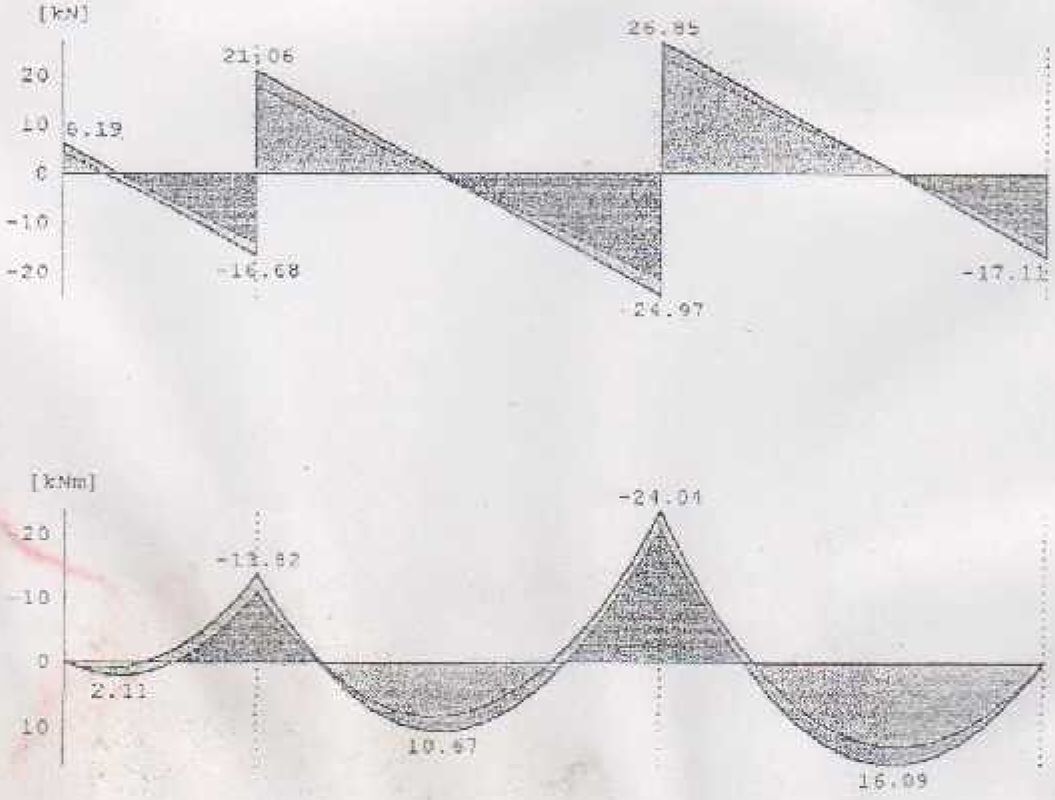
The total length of the top bars = 2 (1.5) + 0.8 = 3.8 m Use L = 3.8 m

RIB (2) :

System



Loads



Shear & Moment Envelope Diagram

Fig (6.7)

1 - Design For Positive Moment :

Mu =16.1 KN.m As shown in Fig. (6-2)

Mn =16.1 / 0.9 =17.9 KN.m =1.79 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 f_c t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =1.79 Ton.m
Design as a rectangular with b_E = 62 cm

A.s max. = 37.08 cm^2

A.s min = 1.04 cm^2

m = 16.47 Rn =4.27 Kg/cm^2 ρ =0.001025

A.s =0.001025 (62) (26) =1.65 cm^2 > A.s min ∴OK

Use 2Φ 12 mm A.s =2.26 cm^2

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) =9.5 Ton

Actual a =0.6 cm

Actual x = a / β =0.6 / 0.85 =0.71 cm

ε_s =0.1068 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.44 Ton.m >1.79 Ton.m ∴ OK.

2 - Design For Negative Moment :

Mu =24.6 KN.m As shown in Fig. (6-2)

Mn =24.6 / 0.9 =26.8 KN.m =2.68 Ton.m

$m = 16.47$ Rn =6.39 Kg/cm^2 $\rho = 0.00154$

As =0.00154 (62) (26) =2.48 cm^2

Use 1 Φ 12 mm & 1 Φ 14 mm As =2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) =11.21 Ton

Actual a =0.71 cm

Actual x =0.83 cm

$\varepsilon_s = 0.0914 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.88 Ton.m >2.68 Ton.m ∴ OK.

3 - Design For Shear :

V_u =2.7 Ton

V_u @ critical point =2.5 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =2.5 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (l_d) :

L_d for bottom bars (Φ 12) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ……………… control

$\frac{Mn}{V_u} + L_a \geq L_d$

123.6 cm ≥ 46 cm ∴ available development > Req.

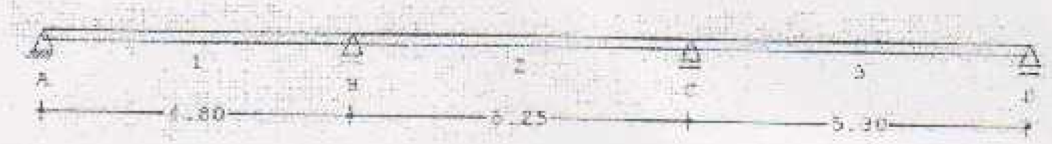
L_d for Top bars:

0.3 Ln from face of support = 0.3(5 - 2(0.15)) = 1.41 m

The total length of the top bars = 2 (1.41) + 0.8 = 3.62 m

Use L = 3.7 m

RIB (3)



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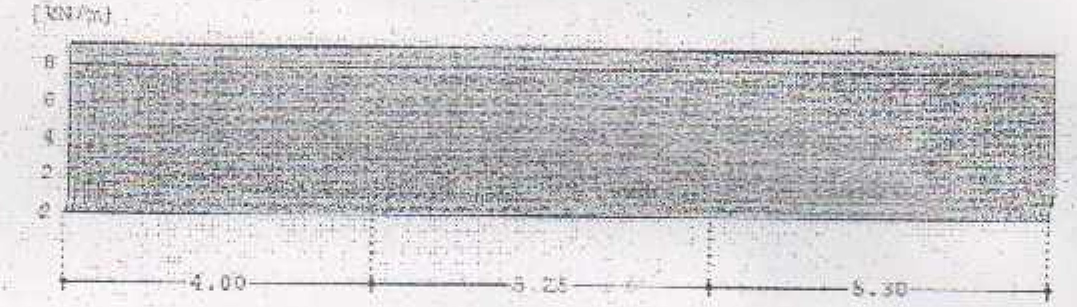
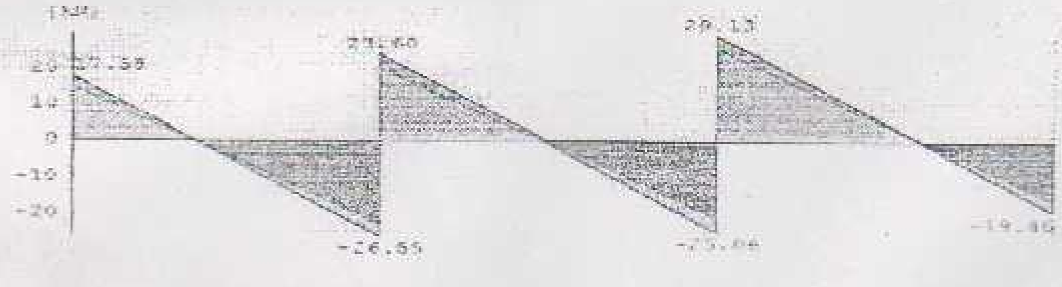
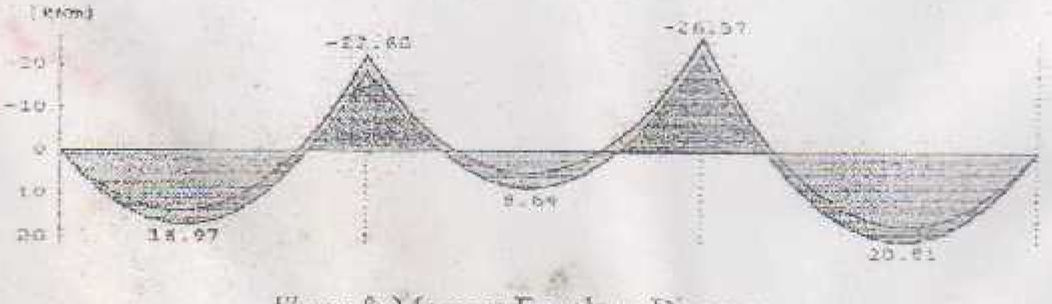


Loads





Shear & Moment Envelope Diagram
Fig. (6-3)

1 - Design For Positive Moment :

Mu =20.81 KN.m As shown in Fig. (6-3)

Mn =20.81 / 0.9 =23.12 KN.m =2.31 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 fc t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m >Mn required =2.31 Ton.m

Design as a rectangular with b_E = 62 cm

As max.= 37.08 cm^2

As min = 1.04 cm^2

m = 16.47 Rn =5.52 Kg / cm^2 ρ =0.001328

As =0.001328 (62) (26) =2.14 cm^2 > As min ∴ OK

Use 2Φ 12 mm As =2.26 cm^2

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) =9.49 Ton

Actual a =0.6 cm

Actual x = a / β =0.6 / 0.85 =0.71 cm

ε_s=0.106 > ε_y= 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.44 Ton.m > 2.31 Ton m

2 - Design For Negative Moment :

Mu =26.57 KN.m As shown in Fig. (6-3)

Mn =26.57 / 0.9 =29.52 KN.m =2.95 Ton.m

$m = 16.47$ Rn =7.03 Kg / cm^2 ρ =0.00169

As =0.00169 (62) (26) =2.736 cm^2

Use 3 Φ 12 mm As =3.4 cm^2

2Φ 12 from positive reinforcement & 1Φ 12 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (3.4)(4.2) =14.3 Ton

Actual a =0.9 cm

Actual x =1.1 cm

$\varepsilon_s = 0.068 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK

Actual Mn =3.64 Ton.m > 2.95 Ton.m ∴ OK

3 - Design For Shear :

V_u =2.9 Ton

V_u @ critical point =2.7 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =2.7 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

L_d = 46 cm

L_a = 12 d_b =12(1.2) =14.4 cm

= d = 26 cm ………………… control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

90.37 cm ≥ 46 cm ∴ available development > Req.

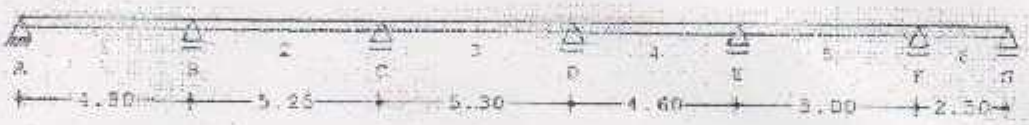
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

Use L =3.8 m

RIB (4)



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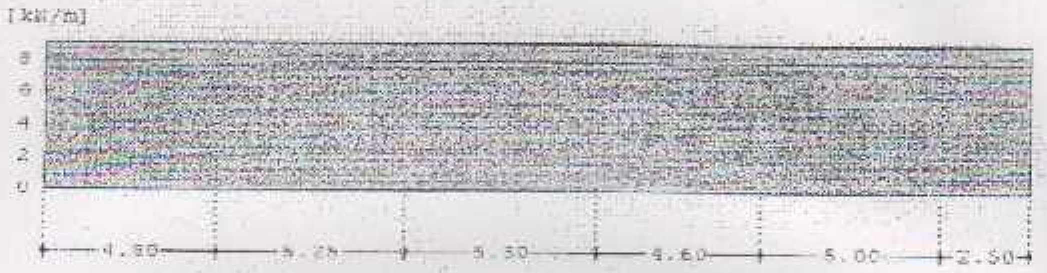

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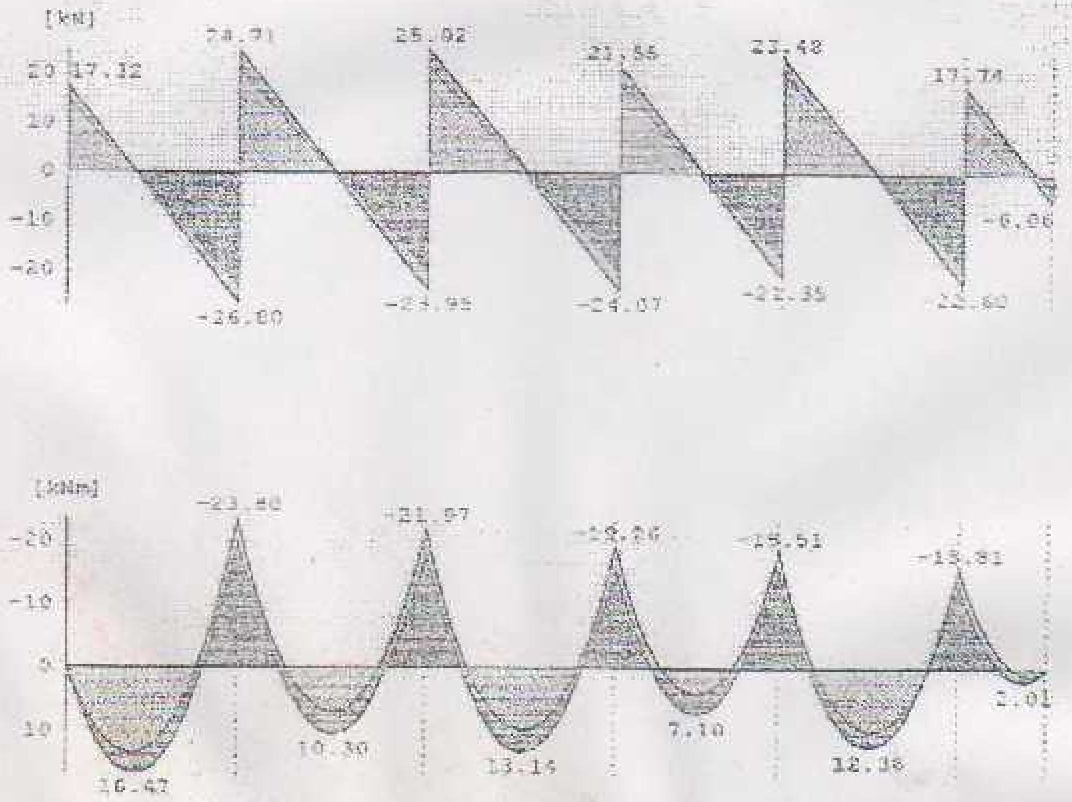


Shear & Moment Envelope Diagram
Fig. (6-4)

1 - Design For Positive Moment :

Mu =16.47 KN.m As shown in Fig. (6-4)

Mn =16.47 / 0.9 =18.3 KN.m =1.83 Ton.m

Determine whether the rib will act as rectangular or T – section :

For $a = t = 10$ cm

$C = 0.85\ f_c\ t\ b_E = 0.85\ (0.3)\ (10)\ (62) = 158.1$ Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required = 1.83 Ton.m

Design as a rectangular with $b_E = 62$ cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

$m = 16.47$ Rn =4.36 Kg/cm^2 $\rho = 0.001047$

$As = 0.001047\ (62)\ (26) = 1.68\ cm^2 > As\ min$ ∴ OK

Use 2Φ 12 mm $As = 2.26\ cm^2$

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) =9.49 Ton

Actual a =0.6 cm

Actual $x = a / \beta = 0.6 / 0.85 = 0.71$ cm

$\varepsilon_s = 0.106 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.44 Ton.m >1.79 Ton.m

2 - Design For *Negative* Moment :

Mu =23.8 KN.m   As shown in Fig. (6-4)

Mn =23.8 / 0.9 =26.44 KN.m   =2.64 Ton.m

$m = 16.47$ Rn =6.31 Kg/cm^2 ρ=0.00152

As =0.00152 (62) (26) =2.45 cm^2

Use 1 Φ 12 mm &   1 Φ 14 mm   As =2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) =11.2 Ton

Actual a =0.71 cm

Actual x =0.83 cm

ε_s =0.00914 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.87 Ton.m >2.64 Ton.m   ∴ OK.

3 - Design For Shear :

V_u =2.5 Ton

V_u @ critical point =2.3 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25   > V_u = 2.3 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ………………. control

$\frac{Mn}{V_u} + L_a \geq L_d$

132.1cm ≥ 46 cm ∴ available development > Req.

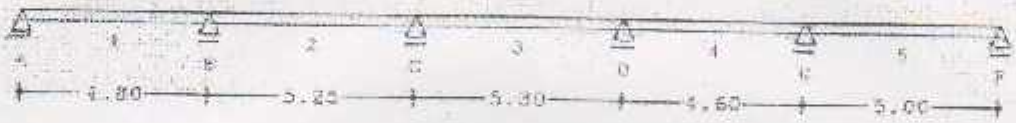
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

Use L =3.8 m

RIB (5)



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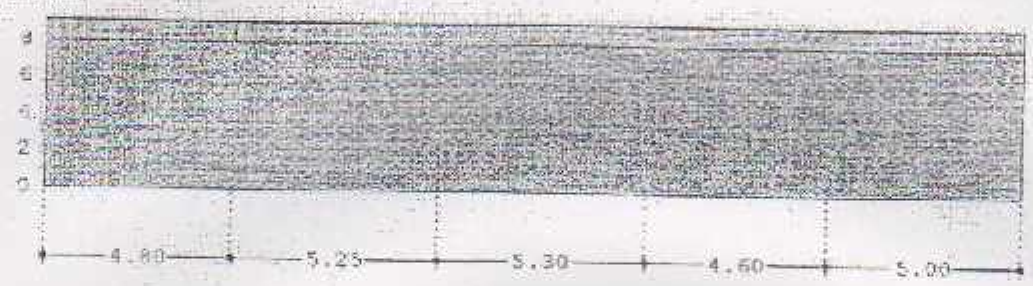

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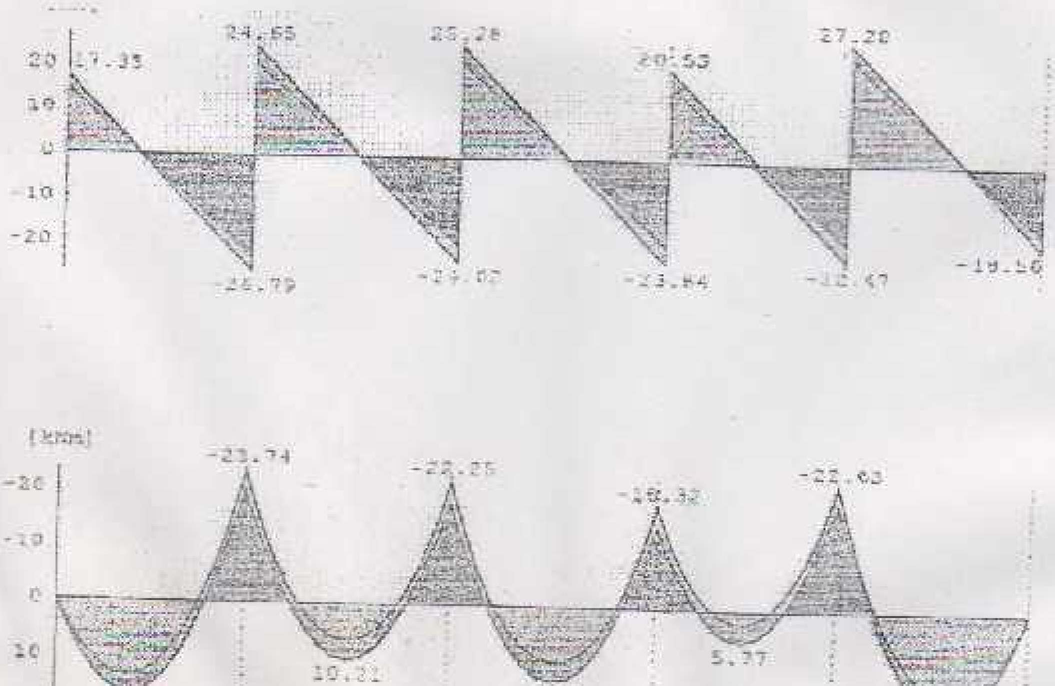


Shear & Moment Envelope Diagram

Fig. (6-5)

1 - Design For Positive Moment :

Mu =18.92 KN.m As shown in Fig. (6-5)

Mn =18.92 / 0.9 =21.02 KN.m = 2.1 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 $fc\,t\,b_E$ = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =1.83 Ton.m

Design as a rectangular with b_E = 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16 .47   Rn = 5.01 Kg/cm^2 ρ =0.0012

As =0.0012 (62) (26) =1.94 cm^2 > As min   ∴ OK

Use 2Φ 12 mm As =2.26 cm^2

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) =9.49 Ton

Actual a =0.6 cm

Actual x = a / β =0.6 / 0.85 =0.71 cm

ε_s =0.106 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.44 Ton.m > 2.1 Ton.m   ∴ OK

2 - Design For **Negative** Moment :

Mu =23.74 KN.m As shown in Fig. (6-5)

Mn =23.74 / 0.9 =26.37 KN.m =2.64 Ton.m

$m = 16.47$ Rn =6.3 Kg/cm^2 ρ =0.001518

As =0.001518 (62) (26) =2.45 cm^2

Use 1 Φ 12 mm & 1 Φ 14 mm As =2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) =11.2 Ton

Actual a =0.71 cm

Actual x =0.83 cm

ε_s =0.00914 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.87 Ton.m >2.64 Ton.m ∴ OK.

3 - Design For Shear :

V_u =2.7 Ton

V_u @ critical point =2.5 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =2.5 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ……………… control

$$\frac{Mn}{V_u} + L_a > L_d$$

116.37 cm $\geq$ 46 cm $\therefore$ available development > Req.

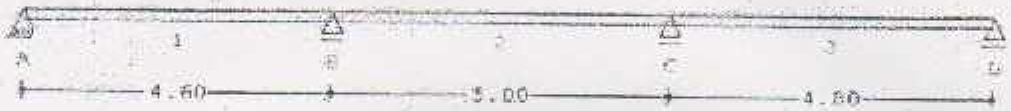
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

Use L =3.8 m

RIB (6)



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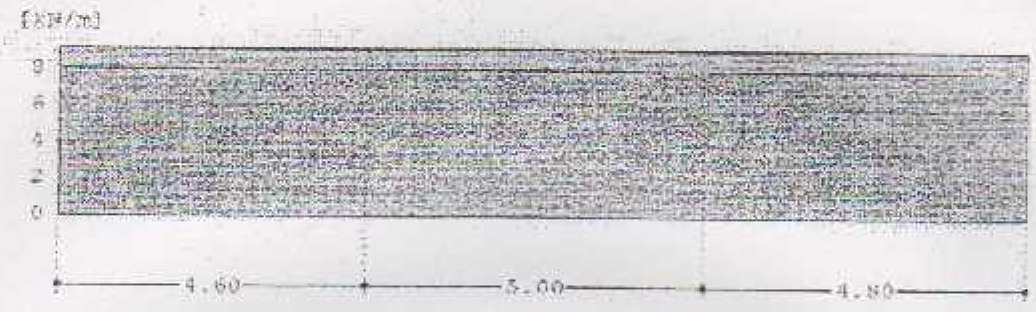

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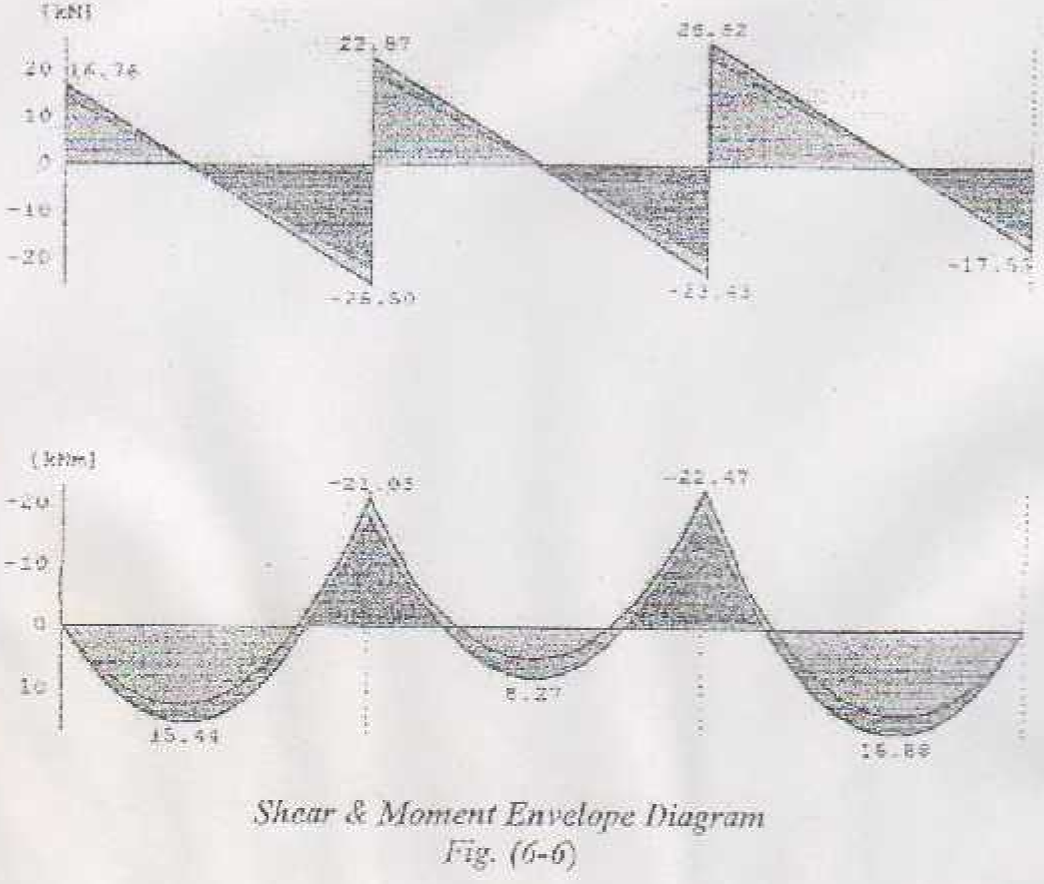


Shear & Moment Envelope Diagram
Fig. (6-6)

1 - Design For **Positive** Moment :

Mu =16.88 KN.m As shown in Fig. (6-6)

Mn =16.88 / 0.9 =18.76 KN.m =1.87 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

$C = 0.85\ fc\ t\ b_e = 0.85\ (0.3)\ (10)\ (62) = 158.1$ Ton

$Mn = T$ or $C\ (d - 0.5\ a) = 158.1\ (26 - 0.5\ (10)) / 100 = 33.2$ Ton.m

Mn available = 33.2 Ton.m > Mn required =1.87 Ton.m
Design as a rectangular with $b_e = 62$ cm

$As\ max. = 37.08\ cm^2$

$As\ min = 1.04\ cm^2$

$m = 16.47$ $Rn = 4.48\ Kg/cm^2$ $\rho = 0.00108$

$As = 0.00108\ (62)\ (26) = 1.73\ cm^2 > As\ min$ $\therefore$ OK

Use 2Φ12 mm $As = 2.26\ cm^2$

1 Φ 12 straight & 1Φ12 bent

Check : tension steel is yield ?

$C = 0.85(0.3)(62)a = 15.81\ a$

Actual $T = (2.26)(4.2) = 9.49$ Ton

Actual a = 0.6 cm

Actual $x = a / \beta = 0.6 / 0.85 = 0.71$ cm

$\varepsilon_s = 0.10686 > \varepsilon_y = 0.0021$

$\therefore$ Tension steel is yielding $\therefore$ OK.

Actual Mn = 2.43 Ton.m > 1.87 Ton.m

2 - Design For **Negative** Moment :

Mu =22.47 KN.m As shown in Fig. (6-6)

Mn =22.47 / 0.9 =24.97 KN.m =2.5 Ton.m

$m = 16.47$ Rn =5.96 Kg/cm^2 ρ =0.00144

As =0.00144 (62) (26) =2.31 cm^2

Use 1 Φ 12 mm & 1 Φ 14 mm As =2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) =11.21 Ton

Actual a =0.71 cm

Actual x =0.83 cm

ε_s =0.09098 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.87 Ton.m > 2.5 Ton.m ∴ OK.

3 - Design For **Shear** :

V_u =2.65 Ton

V_u @ critical point =2.27 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =2.27 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm .... control

$\frac{Mn}{V_u} + L_a \geq L_d$

117.63 cm $\geq$ 46 cm $\therefore$ available development > Req.

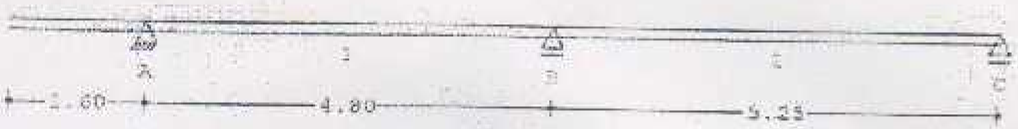
L_d for Top bars:

0.3 Ln from face of support = 0.3(5 - 2(0.15)) =1.41 m

The total length of the top bars = 2 (1.41)+ 0.8 =3.62 m

Use L =3.7 m

RIB (7)



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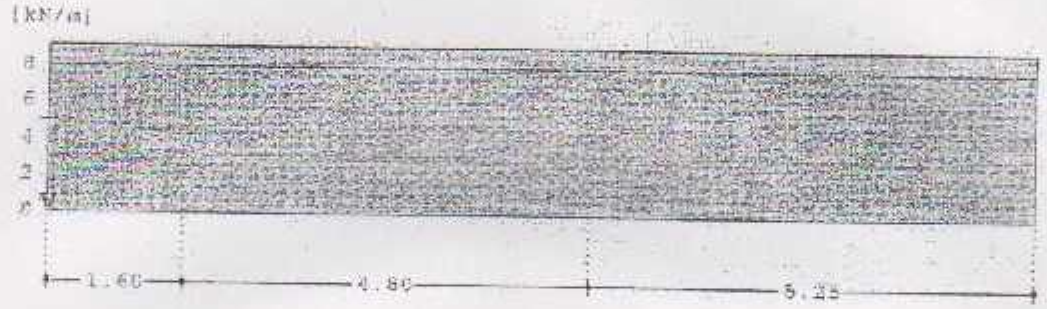

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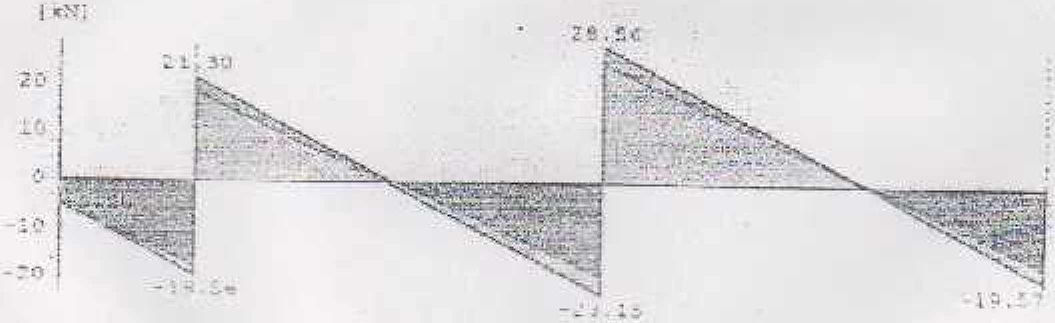
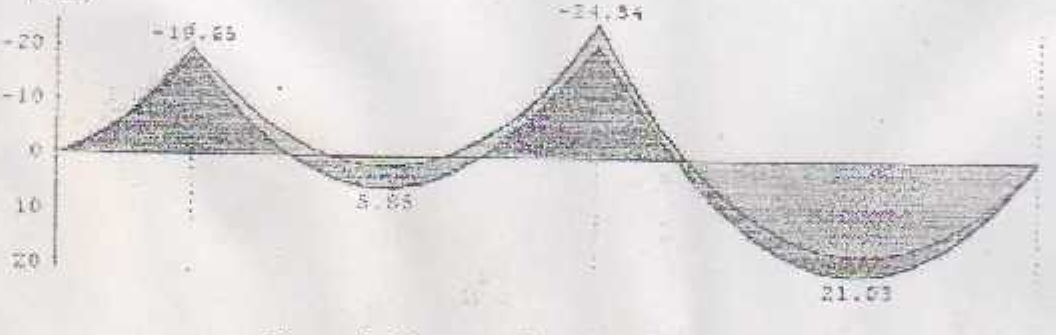




Shear & Moment Envelope Diagram
Fig. (6-7)

1 - Design For **Positive** Moment :

Mu =21.03 KN.m As shown in Fig. (6-7)

Mn =21.03 / 0.9 =23.37 KN.m =2.34 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

$C = 0.85\ fc\ t\ b_E = 0.85\ (0.3)\ (10)\ (62) = 158.1$ Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =2.34 Ton.m

Design as a rectangular with $b_E = 62$ cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

$m = 16.47$ Rn =5.78 Kg / cm^2 $\rho = 0.00139$

$As = 0.00139\ (62)\ (26) = 2.24\ cm^2 > As$ min ∴ OK

Use 2Φ 12 mm $As = 2.26\ cm^2$

1 Φ 12 straight & 1 Φ 12 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (2.26)(4.2) =9.49 Ton

Actual a =0.6 cm

Actual x = a / β =0.6 / 0.85 =0.71 cm

$\varepsilon_s = 0.107 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK

Actual Mn =2.43 Ton.m >2.34 Ton.m

2 - Design For **Negative** Moment :

Mu =24.54 KN.m As shown in Fig. (6-7)

Mn =24.54 / 0.9 =27.27 KN.m =2.73 Ton.m

$m = 16.47$ Rn =6.51 Kg/cm^2 $\rho = 0.00157$

As =0.00157 (62) (26) =2.53 cm^2

Use 1 Φ 12 mm & 1 Φ 14 mm As =2.67 cm^2

1 Φ 12 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (2.67)(4.2) =11.21 Ton

Actual a =0.71 cm

Actual x =0.84 cm

$\varepsilon_s = 0.0898 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =2.875 Ton.m >2.73 Ton.m ∴ OK.

3 - Design For **Shear** :

V_u =2.3 Ton

V_u @ critical point =2.1 Ton

ΦV_c = 12.51 Ton

$0.5\ \Phi V_c = 6.25 > V_u = 2.1$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

131.6cm ≥ 46 cm ∴ available development > Req.

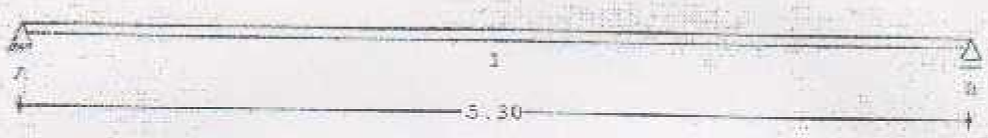
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.25 - 2(0.15)) =1.48 m

The total length of the top bars = 2 (1.48)+ 0.8 =3.77 m

Use L =3.8 m

RIB (8)



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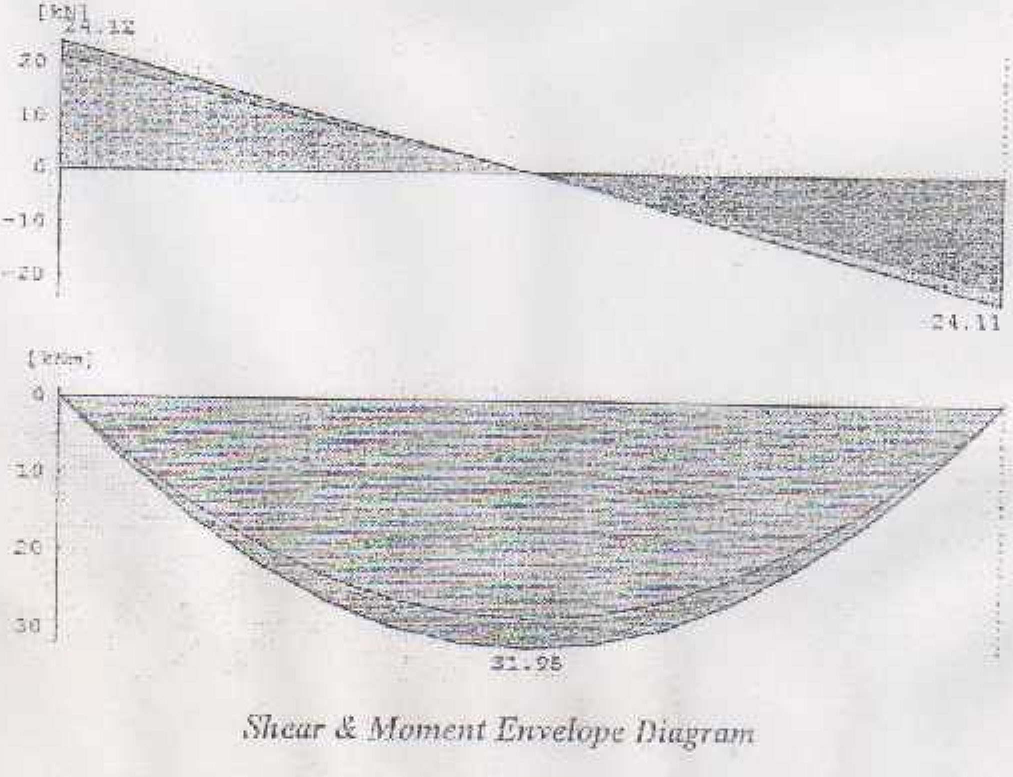
Loads



Shear & Moment Envelope Diagram

Fig. (6-8)

1 - Design For Positive Moment :

Mu =31.95 KN.m As shown in Fig. (6-8)

Mn =31.95 / 0.9 =35.5 KN.m =3.55 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.55 Ton.m

Design as a rectangular with b_E = 62 cm

$m = 16.47$ Rn =8.47 Kg/cm^2 ρ =0.00205

As =0.00205 (62) (26) =3.31 cm^2 > As min ∴ OK

Use 2Φ 16 mm As =4.02 cm^2

1 Φ 16 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (4.02)(4.2) =16.88 Ton

Actual a =1.07 cm

Actual x = a / β =1.07 / 0.85 =1.26 cm

ε_s =0.0589 > ε_y = 0.0021

Actual Mn =4.3 Ton.m > 3.55 Ton.m

2 - Design For Shear :

V_u =2.4 Ton

V_u @ critical point =2.2 Ton

0.5 ΦV_c = 6.25 > V_u =2.2 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required Use Φ6 for 30 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 16) :

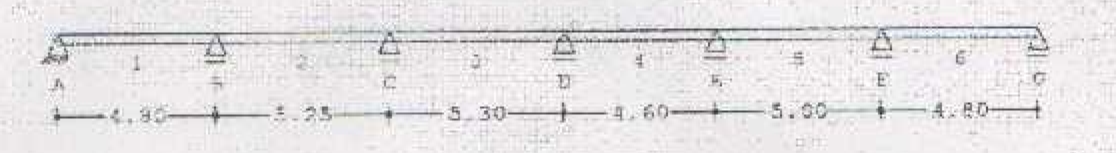
$L_d = 61.34$ cm

L_a = d = 26 cm

$$\frac{Mn}{V_u} + L_a \geq L_d$$

205.2cm ≥ 61.34 cm ∴ available development > Req.

RIB (9)



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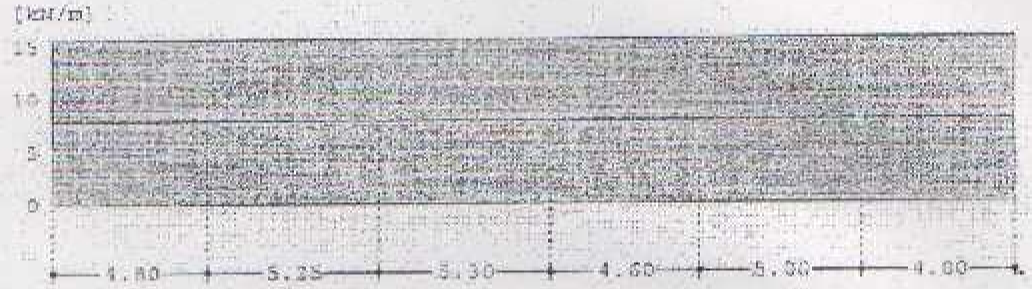

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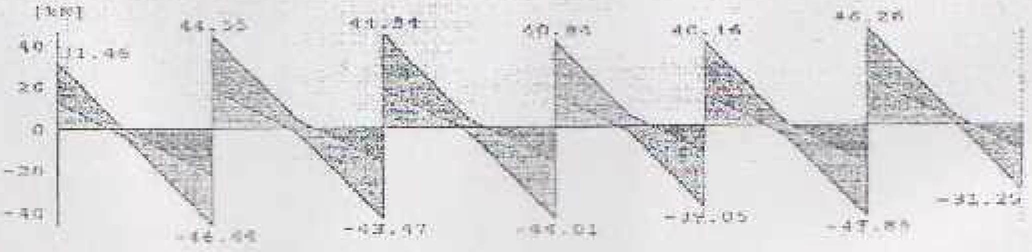
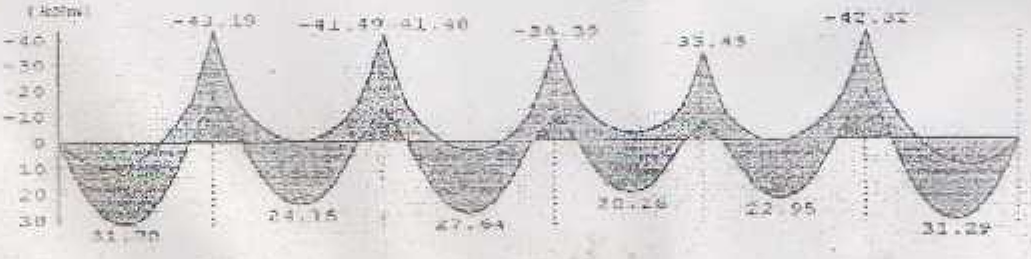




Shear & Moment Envelope Diagram
Fig. (6-9)

1 - Design For Positive Moment :

Mu =31.72 KN.m   As shown in Fig. (6-9)

Mn =31.72 / 0.9 =35.24 KN.m   =3.53 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 f_c t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.53 Ton.m

Design as a rectangular with b_E = 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16.47   Rn =8.41 Kg / cm^2 ρ =0.00204

As =0.00204 (62) (26) =3.28 cm^2 > As min   ∴ OK

Use 2Φ16 mm As =4.02 cm^2

1 Φ 16 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (4.02)(4.2) =16.88 Ton

Actual a =1.07 cm

Actual x = a / β =1.07 / 0.85 =1.26 cm

ε_s =0.0589 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mn =4.3 Ton.m > 3.53 Ton.m

2 - Design For **Negative** Moment :

Mu =43.19 KN.m As shown in Fig. (6-9)

Mn =43.19 / 0.9 =48 KN.m =4.8 Ton.m

$m = 16.47$ Rn =11.45 Kg/cm^2 $\rho = 0.00279$

$As = 0.00279\,(62)\,(26) = 4.5\ cm^2$

Use 1 Φ 16 mm & 1 Φ 18mm $As = 4.55\ cm^2$

1 Φ 16 from positive reinforcement & 1 Φ 18 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.55)(4.2) =19.13 Ton

Actual a =1.21 cm

Actual x =1.42cm

$\varepsilon_s = 0.05006 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.86 Ton.m > 4.8 Ton.m ∴ OK.

3 - Design For **Shear** :

$V_u = 4.64$ Ton

V_u @ critical point =4.24 Ton

$\Phi V_c = 12.51$ Ton

$0.5\ \Phi V_c = 6.25 \quad > \quad V_u = 4.24$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 16) :

$L_d = 46$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ………………. control

$\frac{Mn}{V_u} + L_a \geq L_d$

118.67 cm $\geq$ 46 cm $\therefore$ available development > Req.

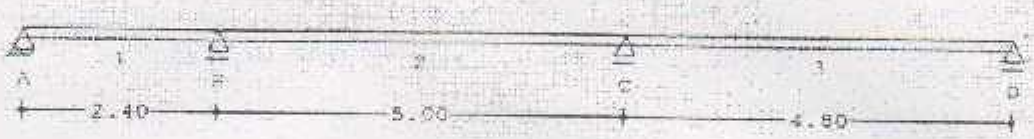
L_d for Top bars:

0.3 Ln from face of support $= 0.3(5.3 - 2(0.15)) = 1.5$ m

The total length of the top bars $= 2\ (1.58) + 0.8 = 3.8$ m

Use L = 3.8 m

RIB (10)



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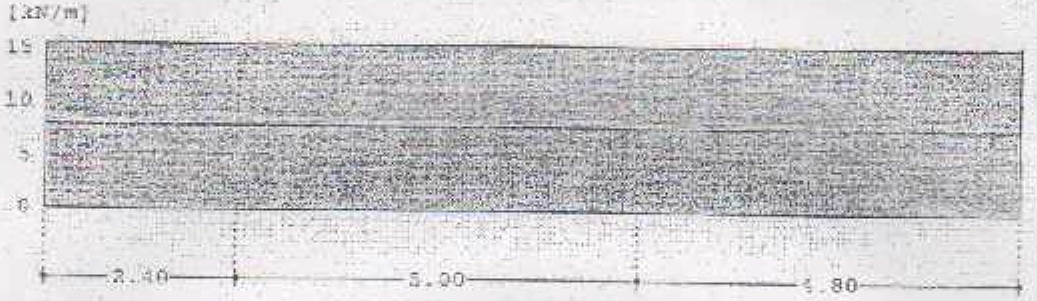
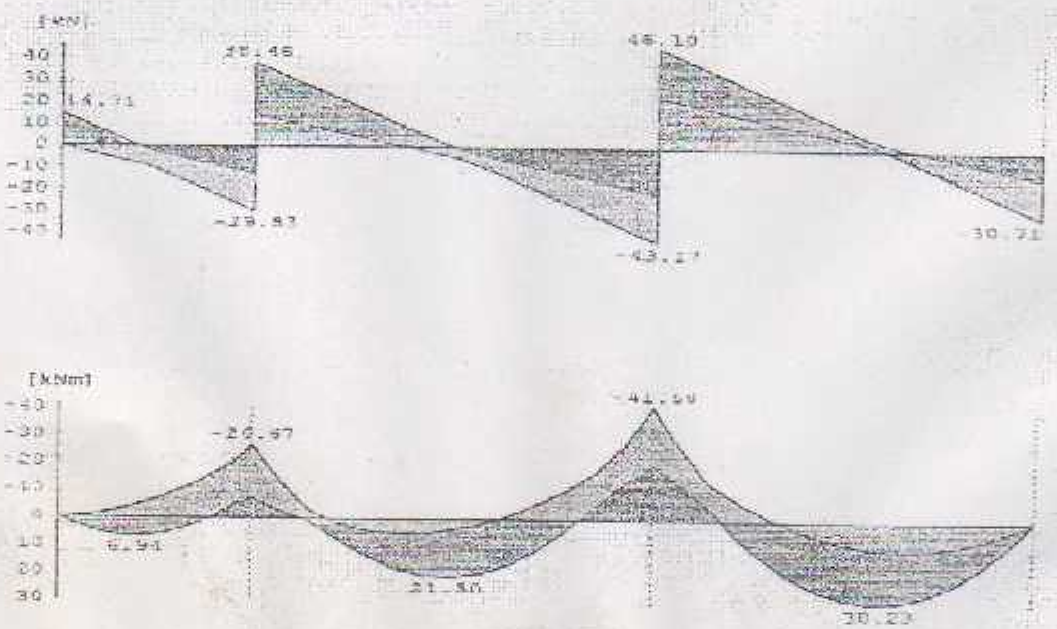


Loads



Shear & Moment Envelope Diagram
Fig. (6-10)

1 - Design For **Positive** Moment :

Mu =30.23 KN.m As shown in Fig. (6-10)

Mn =30.23 / 0.9 =33.59 KN.m =3.4 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

$C = 0.85\ f_c t\ b_E =$ 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.4 Ton.m

Design as a rectangular with $b_E =$ 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16.47 Rn =8.1 Kg/cm^2 ρ =0.00194

As =0.00194 (62) (26) =3.12 cm^2 > As min ∴ OK

Use 1Φ 14 mm & 1 Φ 16 mm As =3.55 cm^2

1 Φ 14 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (3.55)(4.2) =14.91 Ton

Actual a =0.94 cm

Actual x = a / β =0.94 / 0.85 =1.11 cm

ε_s =0.06727 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =3.81 Ton.m >3.4 Ton.m

2 - Design For Negative Moment :

Mu =41.58 KN.m As shown in Fig. (6-10)

Mn =41.58 / 0.9 =46.2 KN.m =4.62 Ton.m

$m = 16.47$ Rn = 11.02 Kg / cm^2 $\rho = 0.00268$

As =0.00268 (62) (26) =4.33 cm^2

Use 1 Φ 18 mm & 1 Φ 16 mm As =4.55 cm^2

1 Φ 16 from positive reinforcement & 1 Φ 18 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.55)(4.2) =19.11 Ton

Actual a =1.21 cm

Actual x =1.42 cm

$\varepsilon_s = 0.05193 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.85 Ton.m > 4.62 Ton.m ∴ OK.

3 - Design For Shear :

V_u =4.61 Ton

V_u @ critical point =4.22 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =4.22 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 16) :

$L_d = 61.3$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

= d = 26 cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

108.8 cm ≥ 46 cm ∴ available development > Req.

L_d for Top bars:

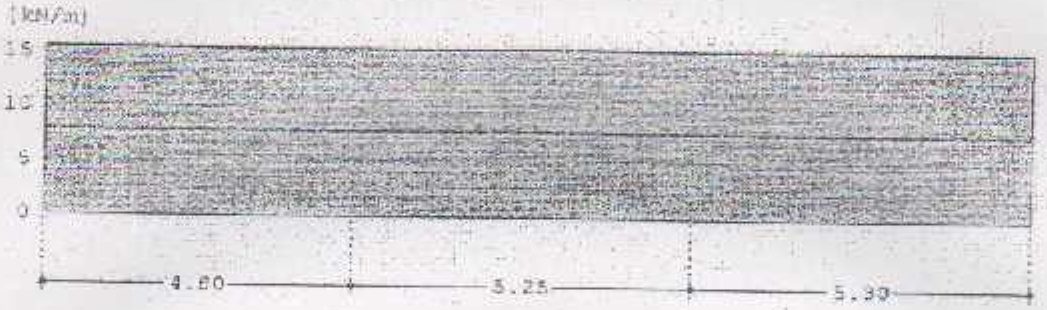
0.3 Ln from face of support = 0.3(5 - 2(0.15)) =1.41 m

The total length of the top bars = 2 ()+ 0.8 =3.62 m

Use L =3.7 m

RIB (11)

System



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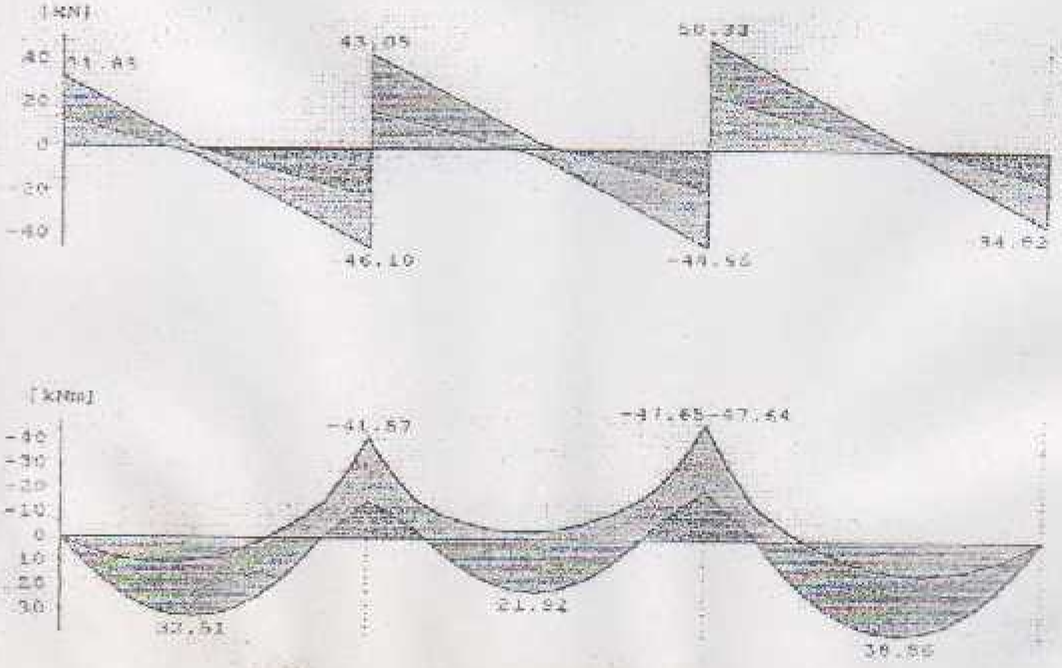


Shear & Moment Envelope Diagram

Fig. (6-11)

1 - Design For Positive Moment :

Mu =38.86 KN.m   As shown in Fig. (6-11)

Mn =38.86 / 0.9 =43.18 KN.m =4.32 Ton.m

Determine whether the rib will act as rectangular or T – section :

For $a = t = 10$ cm

$C = 0.85\ fc\ t\ b_E = 0.85\ (0.3)\ (10)\ (62) = 158.1$ Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =4.32 Ton.m

Design as a rectangular with $b_E = 62$ cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

$m = 16.47$ Rn =10.63 Kg / cm^2 $\rho = 0.00250$

$As = 0.0025\ (62)\ (26) = 4.04\ cm^2 > As\ min$ ∴ OK

Use 2Φ 16 mm $As = 4.02\ cm^2$

1 Φ 16 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (4.02)(4.2) =16.88 Ton

Actual a =1.07 cm

Actual $x = a / \beta = 1.07 / 0.85 = 1.26$ cm

$\varepsilon_s = 0.0859 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.3 Ton.m >= 4.32 Ton.m

2 - Design For Negative Moment :

Mu =47.65 KN.m   As shown in Fig (6-11)

Mn =47.65 / 0.9 =52.9 KN.m =5.3 Ton.m

$m = 16.47$ Rn =12.62 Kg/cm^2 $\rho = 0.00308$

As =0.00308 (62) (26) =4.97 cm^2

Use 3Φ 16 As =6.03 cm^2

2 Φ 16 from positive reinforcement & 1 Φ 16 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (6.03)(4.2) =25.33 Ton

Actual a =1.6 cm

Actual x =1.9 cm

$\varepsilon_s = 0.038 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =6.3 Ton.m > 5.3 Ton.m   ∴ OK.

3 - Design For Shear :

V_u =5.1 Ton

V_u @ critical point =4.71 Ton

ΦV_c = 12.51 Ton

$0.5\ \Phi V_c = 6.25 \quad > \quad V_u = 4.71$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 16) :

L_d = 61.3 cm

L_a = 12 d_b =12(1.2)=14.4 cm

= d = 26 cm .... control

$\frac{Mn}{V_u} + L_a \geq L_d$

110.3 cm $\geq$ 46 cm $\therefore$ available development > Req.

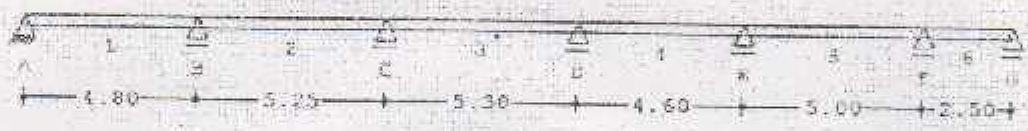
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

Use L =3.8 m

RIB (12)



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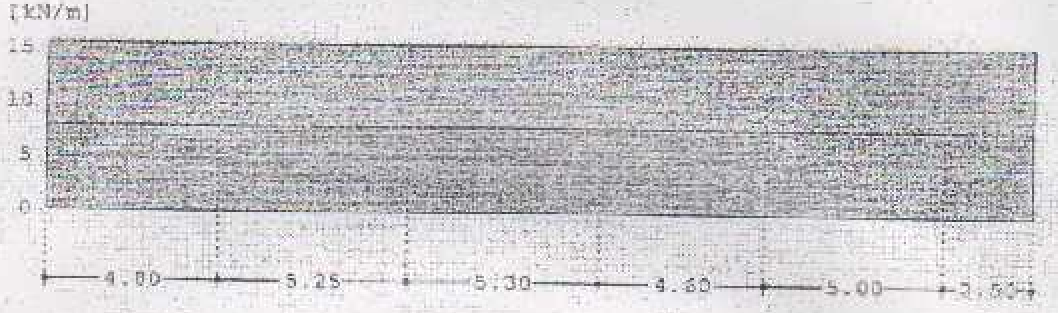

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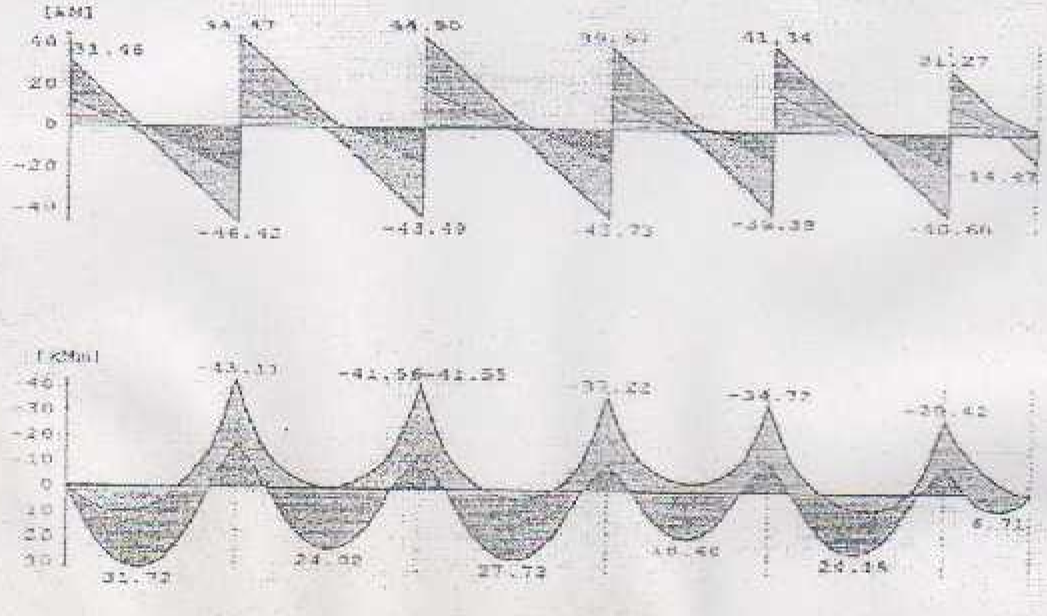


Shear & Moment Envelope Diagram
Fig. (6-12)

I - Design For Positive Moment :

Mu =31.72 KN.m   As shown in Fig. (6-12)

Mn =31.72 / 0.9 =35.24 KN.m =3.52 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

$C = 0.85\ f'c\ t\ b_E = 0.85\,(0.3)\,(10)\,(62) = 158.1$ Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.52 Ton.m

Design as a rectangular with $b_E = 62$ cm

$As\ max. = 37.08\ cm^2$

$As\ min = 1.04\ cm^2$

$m = 16.47$ $Rn = 8.41\ Kg/cm^2$ $\rho = 0.00204$

$As = 0.00204\,(62)\,(26) = 3.28\ cm^2 > As\ min$ ∴ OK

Use 1 Φ 14 mm & 1 Φ 16 mm $As = 3.55\ cm^2$

1 Φ 14 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (3.55)(4.2) =14.91 Ton

Actual a =0.94 cm

Actual x = a / β =0.94 / 0.85 =1.11 cm

$\varepsilon_s = 0.06727 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =3.81 Ton.m >3.52 Ton.m

2 - Design For **Negative** Moment :

Mu =43.11 KN.m As shown in Fig. (6-12)

Mn =43.11 / 0.9 =47.9 KN.m =4.8 Ton.m

$m = 16.47$ Rn=11.43 Kg/cm^2 ρ=0.00279

As =0.00279 (62) (26) =4.49 cm^2

Use 1 Φ 16 mm & 1 Φ 18 mm As =4.55 cm^2

1 Φ 16 from positive reinforcement & 1 Φ 18 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.55)(4.2) =19.11 Ton

Actual a =1.21 cm

Actual x =1.42 cm

ε_s =0.05193 > ε_y =0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.85 Ton.m >4.8 Ton.m ∴ OK.

3 - Design For **Shear** :

V_u =4.64 Ton

V_u @ critical point =3.98 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =3.98 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 14) :

L_d = 53.68 cm

$L_a = 12\ d_b$ =12(1.2) =14.4 cm

= d = 26 cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

108.08 cm ≥ 53.68 cm ∴ available development > Req.

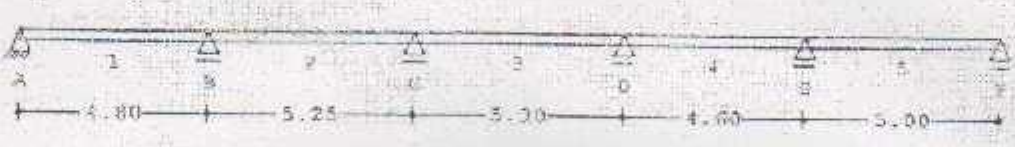
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

Use L =3.8 m

RIB.(13)



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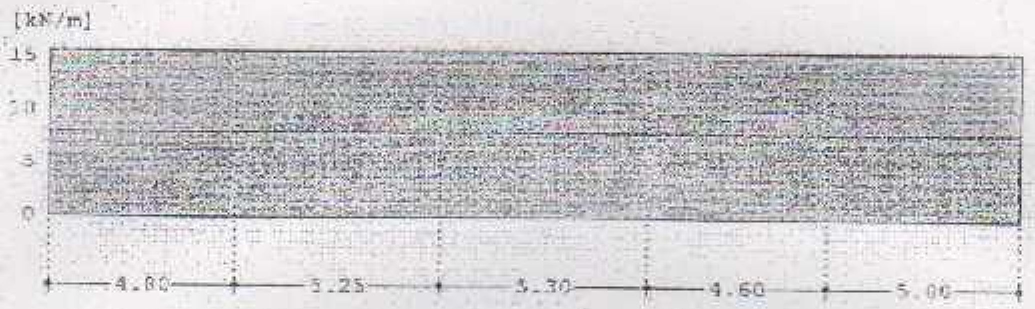
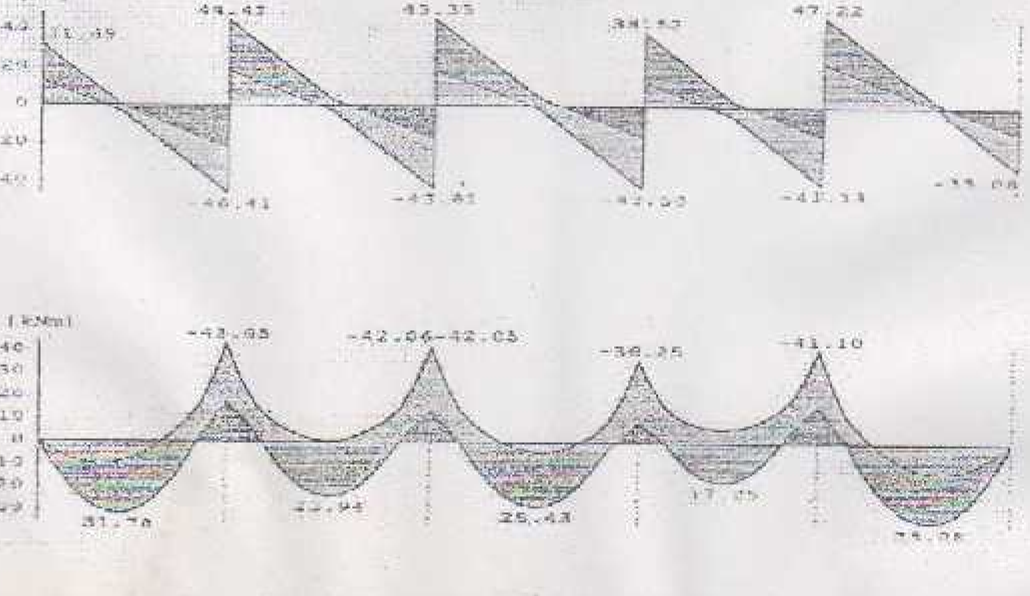


Loads



Shear & Moment Envelope Diagram

Fig. (6-13)

I - Design For Positive Moment :

Mu =35.08 KN.m As shown in Fig. (6-13)

Mn =35.08 / 0.9 =38.98 KN.m =3.9 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 f_c t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.9 Ton.m
Design as a rectangular with b_E = 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16 .47 , Rn =9.3 Kg/cm^2 ρ =0.00226

As =0.00226 (62) (26) =3.64 cm^2 > As min ∴ OK

Use 2Φ16 mm As =4.02 cm^2

1 Φ 16 straight & 1 Φ 16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (4.02)(4.2) =16.88 Ton

Actual a =1.07 cm

Actual x = a / β =1.07 / 0.85 =1.26 cm

ε_s =0.0589 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.3 Ton.m >3.9 Ton.m

2 - Design For Negative Moment :

Mu =43.05 KN.m As shown in Fig. (6-13)

Mn =43.05 / 0.9 =47.83 KN.m =4.8 Ton.m

$m = 16.47$ Rn =11.41 Kg/cm^2 $\rho = 0.00278$

As =0.00278 (62) (26) =4.48 cm^2

Use 1 Φ 16 mm & 1 Φ 18 mm As =4.55 cm^2

1 Φ 16 from positive reinforcement & 1 Φ 18 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.55)(4.2) =19.11 Ton

Actual a =1.21 cm

Actual x =1.42 cm

$\varepsilon_s = 0.05193 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.85 Ton.m > 4.8 Ton.m ∴ OK.

3 - Design For Shear :

V_u =4.7 Ton

V_u @ critical point =4.1 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25 > V_u =4.1 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (l_d) :

L_d for bottom bars (Φ 16) :

L_d = 61.34 cm

L_a = 12 d_b =12(1.2) =14.4 cm

= d = 26 cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

117.06cm > 61.34 cm ∴ available development > Req.

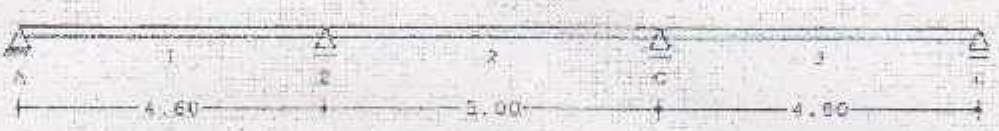
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) =1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 =3.8 m

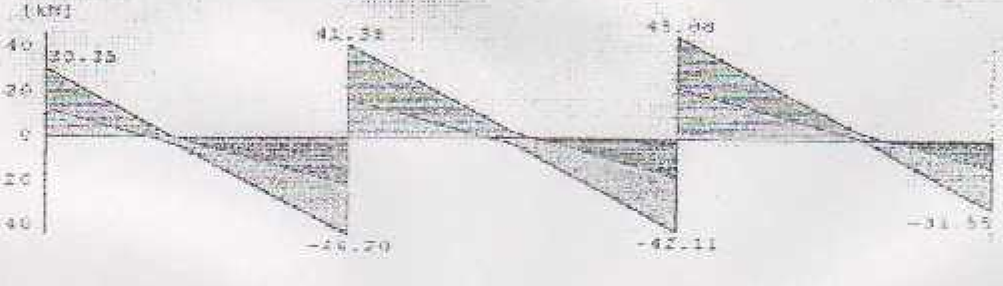
Use L =3.8 m

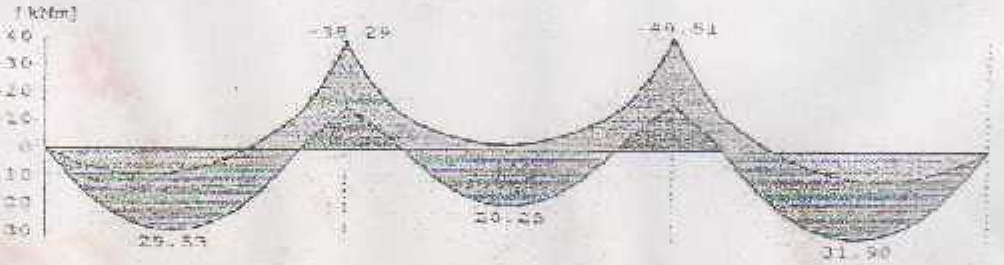
RIB (14)



System

Loads





Shear & Moment Envelope Diagram
Fig. (6-14)

1 - Design For Positive Moment :

Mu =31.9 KN.m As shown in Fig. (6-14)

Mn =31.9 / 0.9 =35.44 KN.m =3.54 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 fc t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =3.54 Ton.m
Design as a rectangular with b_E = 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16 .47 Rn =8.46 Kg/cm^2 ρ =0.00205

As =0.00205 (62) (26) =3.3 cm^2 > As min ∴ OK

Use 1Φ14 mm & 1 Φ16 mm As =3.55 cm^2

1 Φ14 straight & 1 Φ16 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (3.55)(4.2) =14.91 Ton

Actual a =0.94 cm

Actual x = a / β =0.94 / 0.85 =1.11 cm

ε_s =0.06727 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =3.81 Ton.m > 3.54 Ton.m

2 - Design For **Negative** Moment :

Mu =40.51 KN.m   As shown in Fig. (6-14)

Mn =40.51 / 0.9 =45.01 KN.m   =4.5 Ton.m

$m = 16.47$ Rn =10.74 Kg / cm^2 ρ=0.00261

As =0.00261 (62) (26) =4.21 cm^2

Use 1 Φ 16 mm &   1 Φ 18 mm   As =4.55 cm^2

1 Φ 16 from positive reinforcement & 1 Φ18 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.55)(4.2) =19.11 Ton

Actual a =1.21 cm

Actual   x =1.42 cm

ε_s =0.05193 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =4.85 Ton.m >4.5 Ton.m   ∴ OK

3 - Design For **Shear** :

V_u =4.6 Ton

V_u @ critical point =3.9 Ton

ΦV_c = 12.51 Ton

0.5 ΦV_c = 6.25   > V_u =3.9 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 14) :

L_d = 53.68 cm

L_a = 12 d_b =12(1.2) =14.4 cm

= d = 26 cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

109.04cm ≥ 53.68 cm ∴ available development > Req.

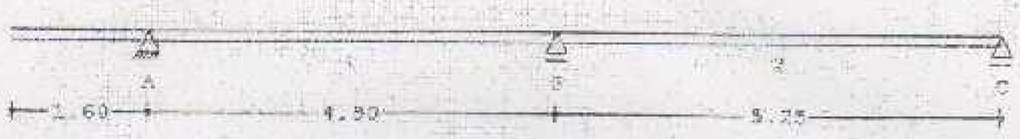
L_d for Top bars:

0.3 Ln from face of support = 0.3(5 - 2(0.15)) =1.41 m

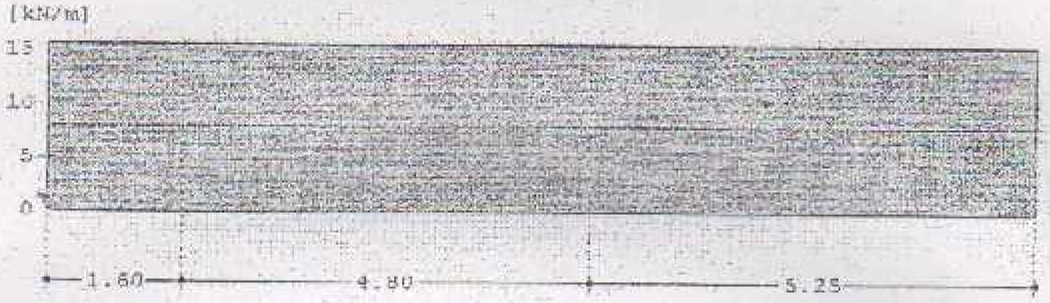
The total length of the top bars = 2 (1.41)+ 0.8 =3.62 m

Use L =3.7 m

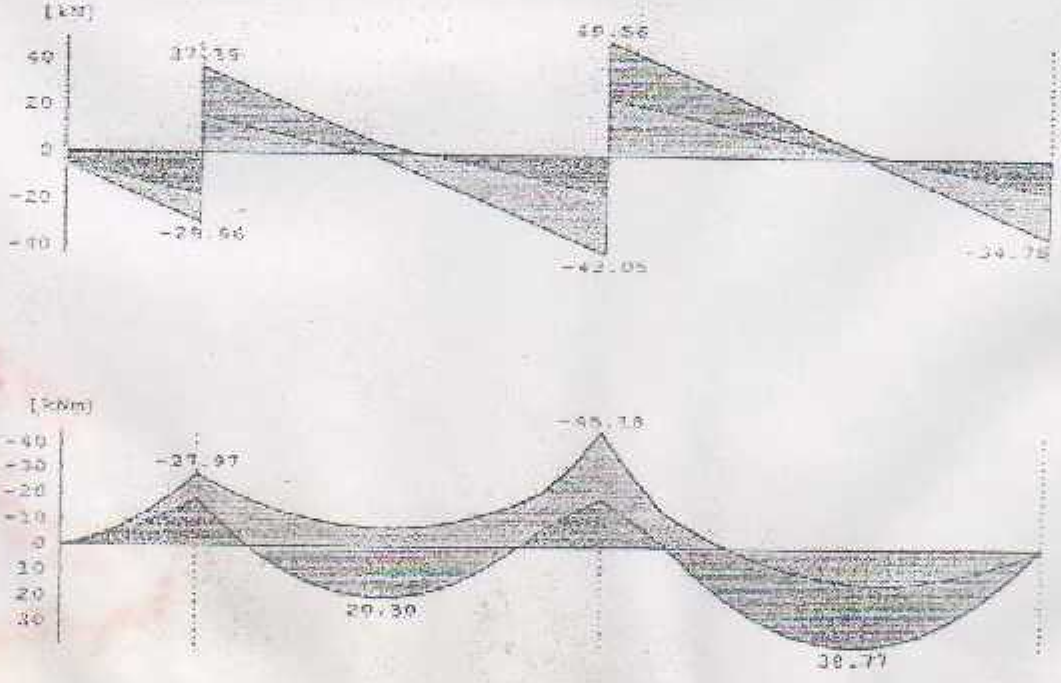
RIB (15)



System



Loads



Shear & Moment Envelope Diagram
Fig. (6-15)

1 - Design For **Positive** Moment :

Mu =38.77 KN.m   As shown in Fig. (6-15)

Mn =38.77 / 0.9 =43 KN.m =4.3 Ton.m

Determine whether the rib will act as rectangular or T - section :

For a = t = 10 cm

C = 0.85 fc t b_E = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d - 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required = 4.3 Ton.m

Design as a rectangular with b_E = 62 cm

A_s max. = 37.08 cm^2

A_s min = 1.04 cm^2

m = 16.47   Rn =10.25 Kg/cm^2 ρ =0.00149

A_s =0.00149 (62) (26) = cm^2 > A_s min   ∴ OK

Use 2Φ 18 mm A_s =5.09 cm^2

1 Φ 18 straight & 1 Φ 18 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (5.09)(4.2) =21.4 Ton

Actual a =1.35 cm

Actual x = a / β =1.35 / 0.85 =1.6 cm

ε_s=0.046 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mn =5.42Ton.m >4.3 Ton.m

2 - Design For **Negative** Moment :

Mu =45.18 KN.m As shown in Fig. (6-15)

Mn =45.18 / 0.9 =50.2 KN.m =5.02 Ton.m

$m = 16.47$ Rn =11.97 Kg/cm^2 ρ =0.00292

$As = 0.00292(62)(26) = 4.7\ cm^2$

Use 2Φ 18 mm $As = 5.09\ cm^2$

2 Φ 18 from positive reinforcement

Check : tension steel is yield ?

C = 15.81 a

Actual T = (5.09)(4.2) =21.9 Ton

Actual a =1.35 cm

Actual x =1.6 cm

$\varepsilon_s = 0.046 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =5.42 Ton.m >5.02 Ton.m ∴ OK

3 - Design For **Shear** :

V_u =4.95 Ton

V_u @ critical point =4.5 Ton

ΦV_c = 12.51 Ton

$0.5\ \Phi V_c = 6.25 > V_u = 4.5$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 18) :

$L_d = 69$ cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

112.9 cm $\geq$ 46 cm $\therefore$ available development > Req.

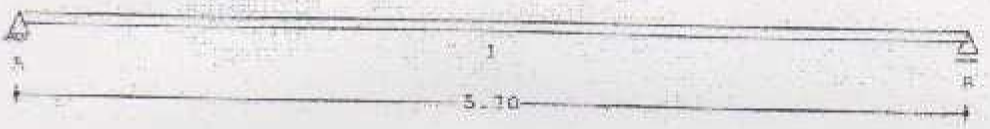
L_d for Top bars:

0.3 Ln from face of support = 0.3(5.25 - 2(0.15)) =1.48 m

The total length of the top bars = 2 (1.48)+ 0.8 =3.77 m

Use L =3.8 m

RIB (16)



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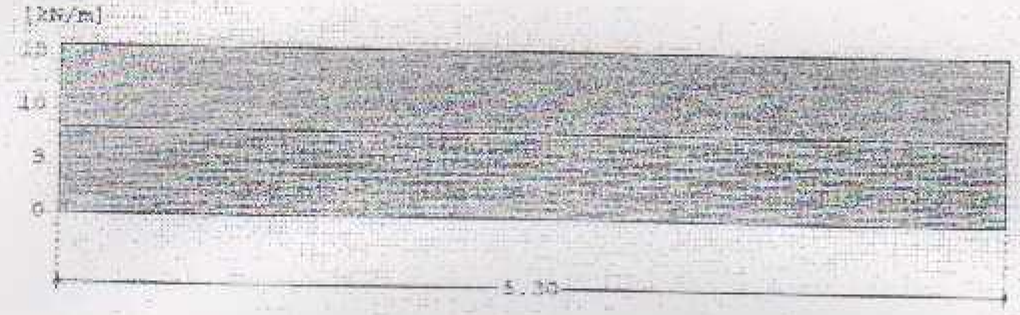

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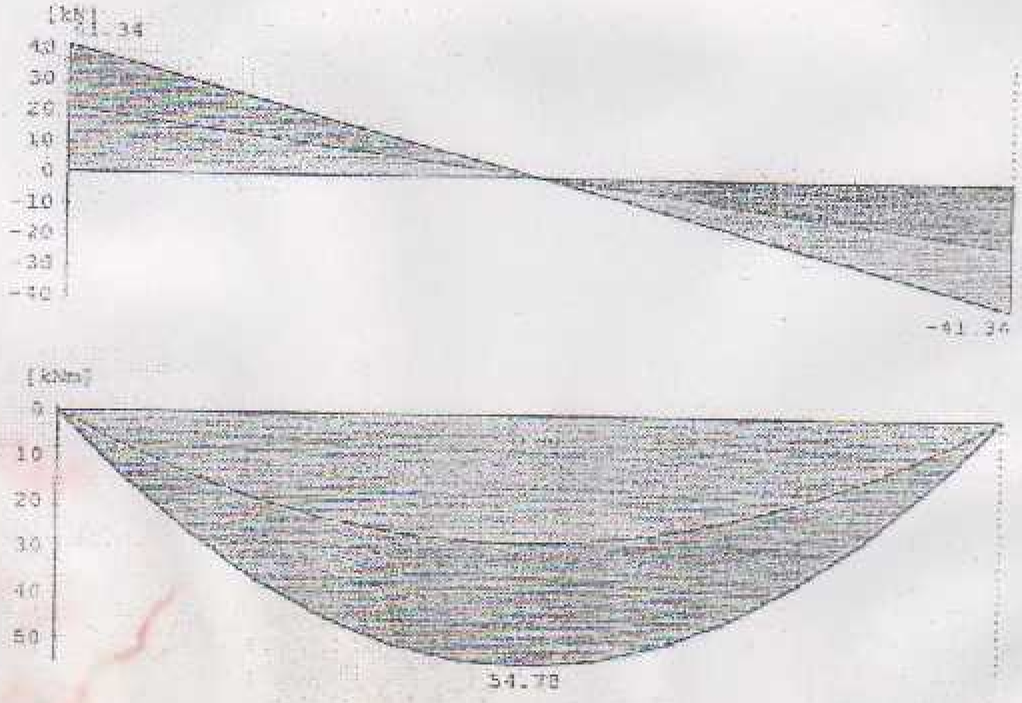


Shear & Moment Envelope Diagram

Fig. (6-16)

1 - Design For **Positive** Moment :

Mu =54.78 KN.m As shown in Fig (6-16)

Mn =57.78 / 0.9 =60.9 KN.m =6.1 Ton.m

Mn available = 33.2 Ton.m > Mn required =6.1 Ton.m

Design as a rectangular with b_E = 62 cm

m = 16.47 Rn =14.66 Kg/cm^2 ρ =0.00356

As =0.00356 (62) (26) =5.75 cm^2 > As min ∴ OK

Use 3Φ 18 mm As =7.65 cm^2

1 Φ 18 straight & 2 Φ18 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (7.65)(4.2) =32.13 Ton

Actual a =2.03 cm

Actual x = a / β =2.03 / 0.85 =2.4 cm

ε_s =0.0295 > ε_y = 0.0021

Actual Mn =8 Ton.m > 6.1 Ton.m

2 - Design For **Shear** :

V_u =4.2 Ton

V_u @ critical point =3.87 Ton

0.5 ΦV_c = 6.25 > V_u =3.75 Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required . Use Φ6 for 30 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 18) :

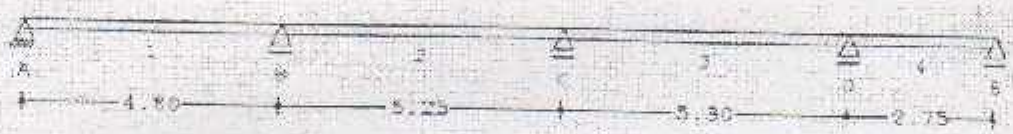
$L_d = 69$ cm

$L_a = d = 26$ cm

$$\frac{Mn}{V_u} + L_a \geq L_d$$

155 cm $\geq$ 69 cm $\therefore$ available development > Req.

RIB (17) :



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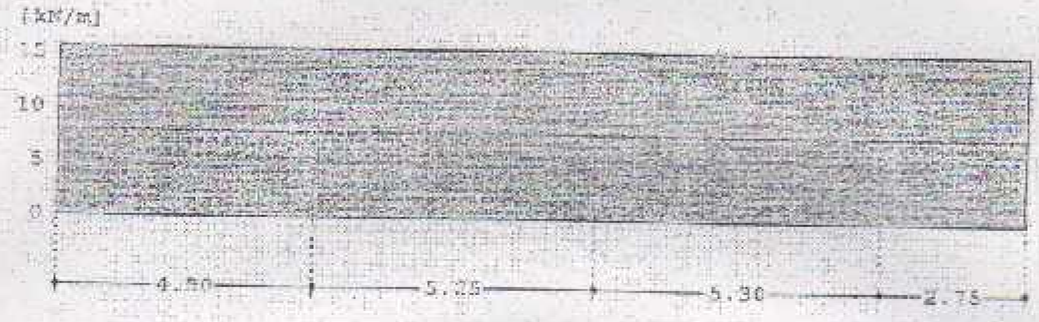

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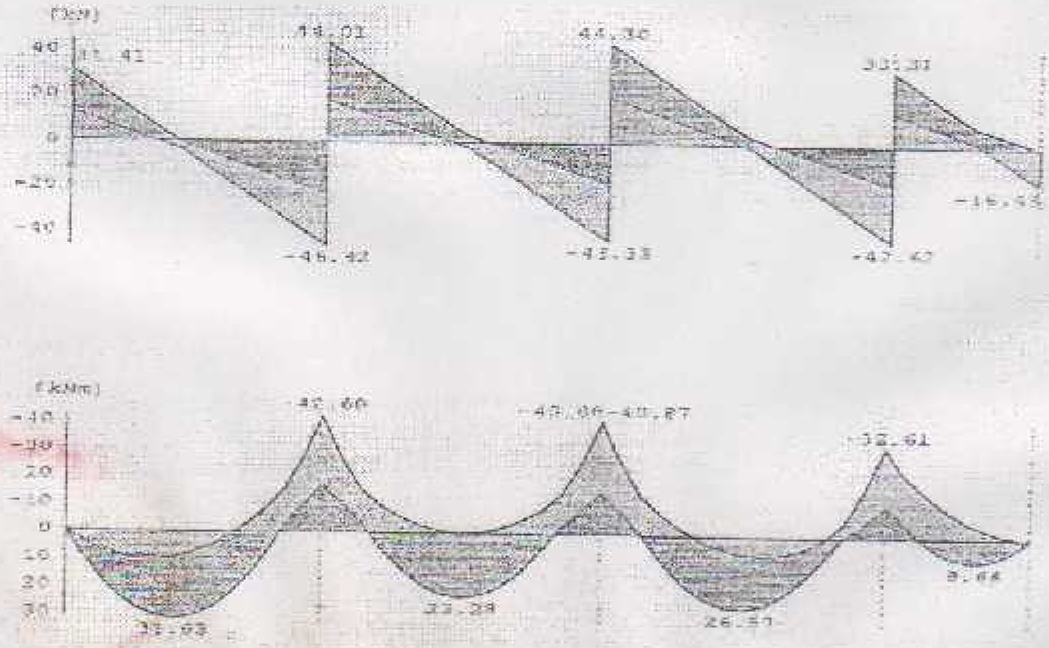


Shear & Moment Envelope Diagram

Fig. (6-17)

1 - Design For **Positive** Moment :

Mu =31.63 KN.m   As shown in Fig. (6-17)

Mn =31.63 / 0.9 =35.14 KN.m =3.51 Ton.m

Determine whether the rib will act as rectangular or T – section :

For a = t = 10 cm

C = 0.85 $fc t\ b_E$ = 0.85 (0.3) (10) (62) = 158.1 Ton

Mn = T or C (d – 0.5 a) = 158.1 (26 – 0.5 (10)) / 100 = 33.2 Ton.m

Mn available = 33.2 Ton.m > Mn required =2.9 Ton.m

Design as a rectangular with b_E = 62 cm

As max. = 37.08 cm^2

As min = 1.04 cm^2

m = 16 .47   Rn =8.37 Kg/cm^2 ρ =0.00203

As =0.00203 (62) (26) = 3.27 cm^2 > As min   ∴ OK

Use 1Φ 16 mm & 1 Φ 14 mm As = 3.55 cm^2

1 Φ 16 straight & 1 Φ 14 bent

Check : tension steel is yield ?

C =0.85(0.3)(62)a = 15.81 a

Actual T = (3.55)(4.2) =14.91 Ton

Actual a = 0.94 cm

Actual x = a / β = .94 / 0.85 =1.11 cm

ε_s =0.06331 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =3.81 Ton.m >3.51 Ton.m

2 - Design For **Negative** Moment :

Mu =42.60 KN.m   As shown in Fig. (6-17)

Mn =42.60 / 0.9 =47.33 KN.m   =4.73 Ton.m

$m = 16.47$ Rn =11.29 Kg/cm^2 $\rho = 0.00275$

$A_s = 0.00275\,(62)\,(26) = 4.43\ cm^2$

Use 3 Φ 14 mm $A_s = 4.62\ cm^2$

2 Φ 14 from positive reinforcement & 1 Φ 14 straight

Check : tension steel is yield ?

C = 15.81 a

Actual T = (4.62)(4.2) =19.40 Ton

Actual a =1.23 cm

Actual x =1.44 cm

$\varepsilon_s = 0.0516 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 4.92 Ton.m > 4.73 Ton.m   ∴ OK.

3 - Design For **Shear** :

$V_u = 4.62$ Ton

V_u @ critical point = 4.13 Ton

$\Phi V_c = 12.51$ Ton

$0.5\ \Phi V_c = 6.25 > V_u = 4.2$ Ton

according to category (2) , (No shear reinforcement is required) , but the design to provide minimum shear even if not required .

Use Φ6 for 30 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 12) :

$L_d = 61.3$cm

$L_a = 12\ d_b = 12(1.2) = 14.4$ cm

$= d = 26$ cm ……………… control

$\frac{Mn}{V_u} + L_a \geq L_d$

121.3 cm $\geq$ 61.3 cm ∴ available development > Req.

L_d for Top bars:

0.3 Ln from face of support = 0.3(5.3 - 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5)+ 0.8 = 3.8 m

Use L = 3.8 m

6-3 Beams Design :

6-3-1 Loads Calculation

How the loads transported to the beams ?

Assume that :

- the beam is a middle beam
- L_1 is the rib length from the one side
- L_2 is the rib length from the other side

Factored Total Dead Load $= \left(\frac{L_1 + L_2}{2}\right) \times 13$

Factored live load for top roof $= \left(\frac{L_1 + L_2}{2}\right) \times 1.78$

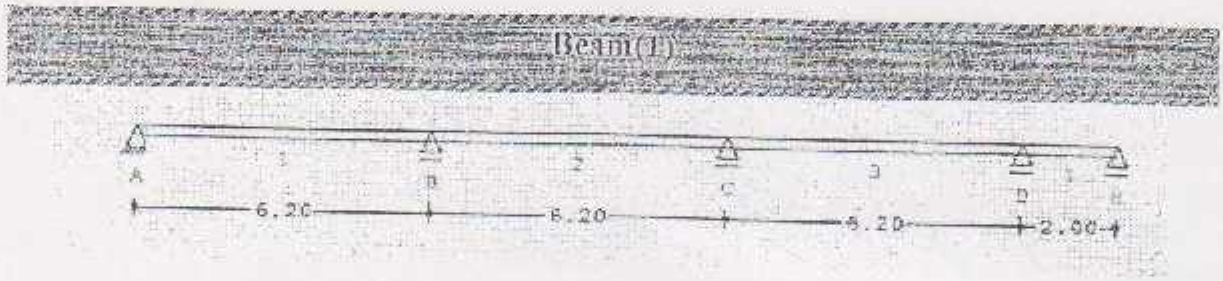
Factored live load for other roof $= \left(\frac{L_1 + L_2}{2}\right) \times 12.3$

For Beam (3) For Example :

Dead Load $= \left(\frac{4.8 + 5}{2}(m)\right) \times 13\ KN/m^2 = 63.7\ KN/m$

Live Load $= \left(\frac{4.8 + 5}{2}(m)\right) \times 1.78\ KN/m^2 = 8.72\ KN/m$

6-3-2 Design for the beams



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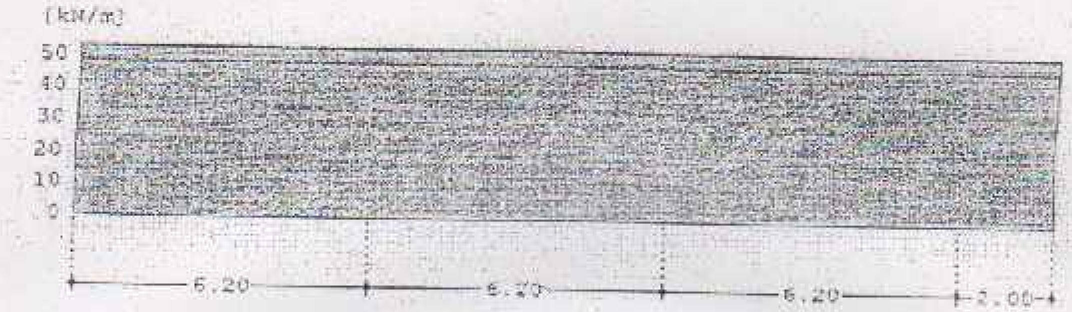

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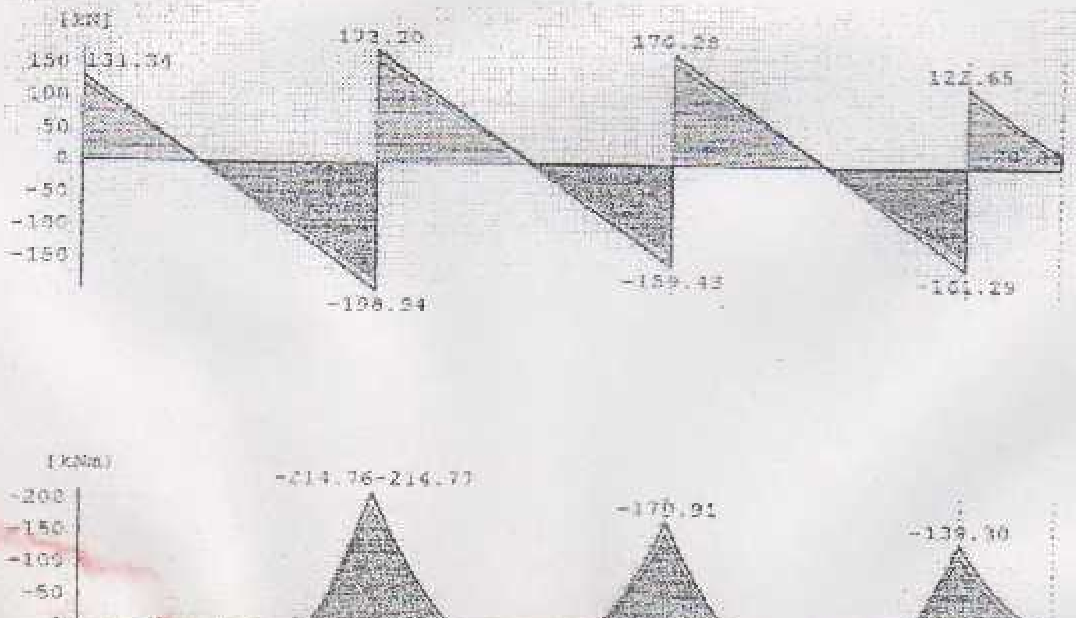
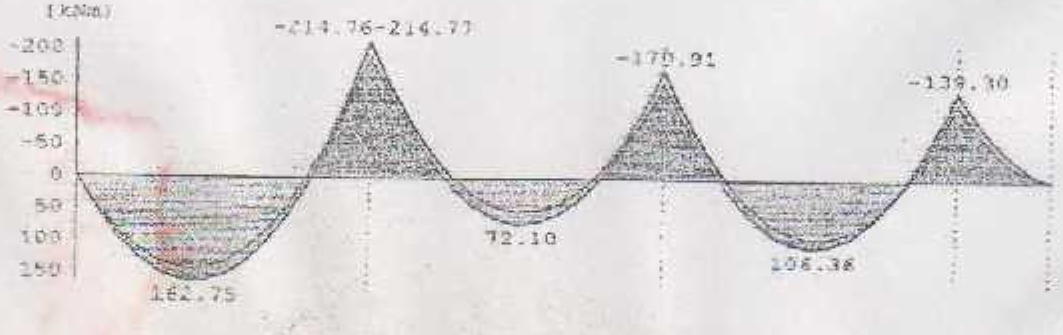




Shear & Moment Envelope Diagram

Fig. (6-18)

B = 50 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu = 162.75 KN.m As shown in Fig. (6-18)

Mn = 162.75 / 0.9 = 180.83 KN.m = 18.1 Ton.m

$As\ max. = \rho\ bd$

$As\ max. = 0.023\ (50)\ (26) = 29.9\ cm^2$

Determine X_b:

$$\frac{0.003}{X_b} = \frac{0.0021}{26 - X_b}$$

$X_b = 15.29$ cm $X max. = 0.75\ X_b = 11.47$ cm

$a\ max. = \beta X_b = 0.85\ (11.47) = 9.75$ cm

$T max. = As\ fy = (29.9)(4.2) = 125.58$ Ton

Mn = T or C (d – 0.5 a)

Mn max. = 125.58(26 – 0.5(9.75)) /100 = 26.53 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$$m = \frac{f_y}{0.85 f'_c} = \frac{420}{2(30)} = 16.47$$

$$R_n = \frac{M_n}{bd^2} = \frac{18.1 \times 10^5}{50 \times 26^2} = 53.55 (Kg / cm^2)$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right)$$

$$= \frac{1}{16.47}\left(1 - \sqrt{1 - \frac{2 \times 16.47 \times 53.55}{4200}}\right)$$

$\rho = 0.01448$ $\rho_{min.} < \rho < \rho_{max.}$

$As = .01448(50)\ (26) = 18.82\ cm^2$

Use 6 Φ 20 mm $As = 18.85\ cm^2$

Check : tension steel is yield ?

$C = 0.85 f'_c ab$

$C = 0.85(0.3)(50)a = 12.75$ a

Actual $T = A_s f_y$

Actual $T = (18.85)(4.2) = 79.17$ Ton

Actual $a = 79.17 / 12.75 = 6.21$ cm

Actual $x = a / \beta = 6.21 / 0.85 = 7.3$ cm

$\varepsilon_s = 0.00779 > \varepsilon_y = 0.0021$

$\therefore$ Tension steel is yielding $\therefore$ OK.

Actual Mn = 18.13 Ton.m

2-Design For Negative Moment :

Mu = 214.76 KN.m As shown in Fig. (6-18)

Mn = 214.76 / 0.9 = 238.62 KN.m = 23.86 Ton.m

$As\ max. = \rho\ bd$

$As\ max. = 0.023\ (50)\ (26) = 29.9\ cm^2$

Determine X_b:

$$\frac{0.003}{X_b} = \frac{0.0021}{26 - X_b}$$

$X_b = 15.29$ cm $\qquad$ $X max. = 0.75\ X_b = 11.47$ cm

$a max = \beta X_b = 0.85\ (11.47) = 9.75$ cm

$T max. = As\ fy = (29.9)(4.2) = 125.58$ Ton

Mn = T or C (d − 0.5 a)

Mn max. = 125.58(26 − 0.5(9.75)) /100 = 26.53 Ton.m

Mn max. > Req. Mn $\qquad$ $\therefore$ singly reinforcement

$$m = \frac{f_y}{0.85 f'_c} = \frac{420}{2(30)} = 16.47$$

$$R_n = \frac{M_n}{bd^2} = \frac{23.86 \times 10^5}{50 \times 26^2} = 70.41 (Kg / cm^2)$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right)$$

$$= \frac{1}{16.47}\left(1 - \sqrt{1 - \frac{2 \times 16.47 \times 70.41}{4200}}\right)$$

$\rho = 0.02009$ $\rho_{min.} < \rho < \rho_{max.}$

$As = .02009(50)(26) = 26.11 \text{ cm}^2$

Use 8 Φ 20 mm & 2 Φ 14 $As = 28.27 \text{ cm}^2$

Check : tension steel is yield ?

$C = 0.85\ fc'ba$

$C = 0.85(0.3)(50)a = 12.75\ a$

Actual $T = A_s f_y$

Actual T = (28.27)(4.2) = 119.36 Ton

Actual a = 119.36 / 12.75 = 9.3 cm

Actual x = a / β = 9.3 / 0.85 = 10.9 cm

$\varepsilon_s = 0.00415 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 25.48 Ton.m > 23.86 Ton.m ∴ OK.

3-Design For Shear :

$V_u = 19.84$ Ton

V_u @ critical point = 18.57 Ton

$$\Phi V_c = 0.85\left(\frac{\sqrt{fc'}}{6}\right)bd \ =0.85\left(\frac{\sqrt{30}}{6}\right)(50)(26)\left(\frac{10}{1000}\right) = 10.1 \text{ Ton}$$

$$\min\ \Phi V_s = 0.85\frac{1}{3}MPa(bd) = 0.85\frac{1}{3}(50)(26)\left(\frac{10}{1000}\right) = 3.68 \text{ Ton}$$

$$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd \ =0.85\left(\frac{\sqrt{30}}{3}\right)(50)(26)\left(\frac{10}{1000}\right) = 20.17 \text{ Ton}$$

complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 18.57 - 10.1 = 8.47$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = \frac{0.85(1.57)(4.2)(26)}{8.47} = 17.21 \text{ cm}$$

Use Φ 10 (2 leg) , S = 17 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

$$L_d = \frac{f_y}{2\sqrt{f_c}}\alpha\beta\gamma\ d_b = \frac{420}{2\sqrt{30}}(1)(1)(1)(2) = 76.68 \text{ cm}$$

$L_a = 12\ d_b = 12(2) = 24$ cm

$= d = 26$ cm .... control

$$\frac{Mn}{V_u} + L_a \geq L_d$$

$$\frac{18.1(100)}{19.89} + 26 \geq 76.68$$

117.15 cm $\geq$ 76.68 cm $\therefore$ available development > Req.

L_d for Top bars (Φ 20) :

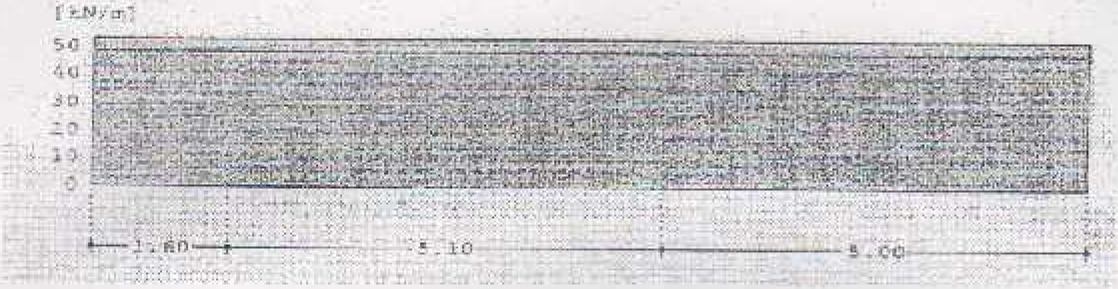
0.3 Ln from face of support $= 0.3(6.2 - 2(0.15)) = 1.77$ m

The total length of the top bars $= 2\ (1.77) + 0.6 = 4.14$ m

Use $L = 4.2$ m

Beam(2)

system



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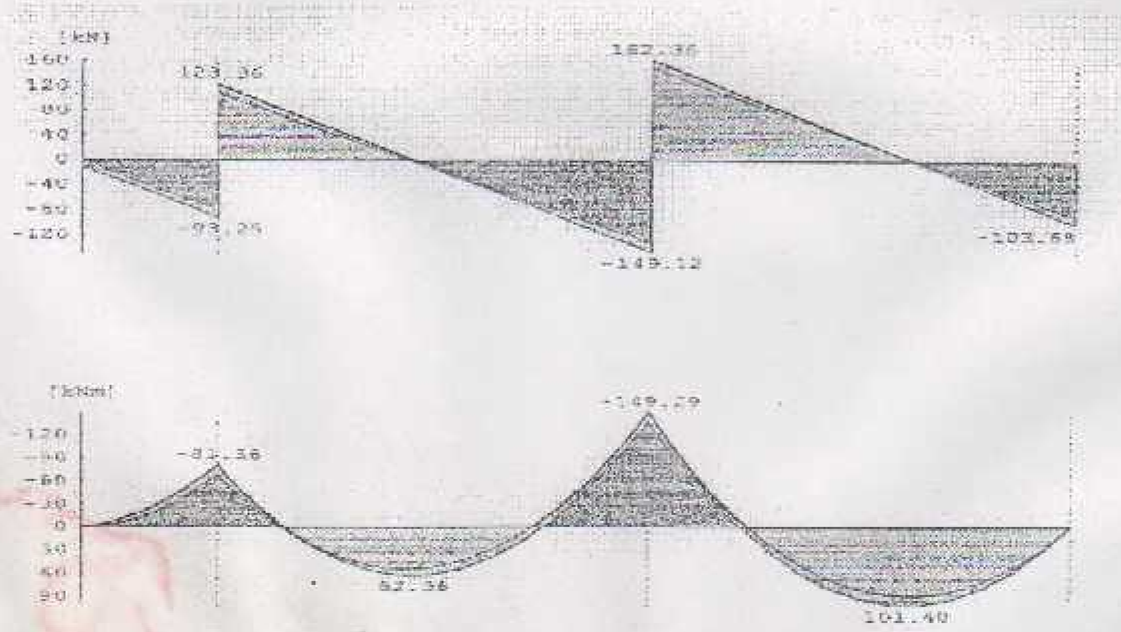


Shear & Moment Envelope Diagram

Fig. (6-19)

B =50 cm H = 30 cm d = 26 cm

1-*Design For Positive Moment :*

Mu =101.40 KN.m As shown in Fig. (6-19)

Mn =101.40 / 0.9 =112.67 KN.m =11.27 Ton.m

As max. =0.023 (50) (26) =29.9 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

Tmax. = As fy = (29.9)(4.2) = 125.58 Ton

Mn max. = 125.58(26 – 0.5(9.75)) /100 = 26.53 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 33.43 Kg/cm^2 ρ = 0.00854 $\rho_{min.} < \rho < \rho_{max.}$

As = 0.00854(50) (26) = 11.13 cm^2

Use 4 Φ 20 mm As = 12.72 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50)a =12.7 a

Actual T = (12.72)(4.2) = Ton

Actual a = 4.2 cm

Actual x = a / β = 4.2/ 0.85 = 4.95cm

ε_s = 0..0127 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =12.77 Ton.m >11.27 Ton.m OK

2-*Design For Negative Moment :*

Mu =149.29 KN.m As shown in Fig. (6-19)

Mn =149.29 / 0.9 =156.88 KN.m =15.67 Ton.m

$m = 16.47$ Rn = 46.36 Kg/cm^2 $\rho = 0.0123$ $\rho_{min} < \rho < \rho_{max}$

As =0.0123 (50) (26) =16 cm^2

Use 5 Φ 20 mm & 2Φ 14 As =18.85 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50)a =12.75 a

Actual T = (18.85)(4.2) =79.17 Ton

Actual a = 79.17 / 12.75 = 6.2cm

Actual x = a / β = 6.2/ 0.85 = 7.3cm

$\varepsilon_s = 0.00768 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =18.12 Ton.m >15.67 Ton.m ∴ OK.

3-Design For Shear :

V_u = 14.91Ton

V_u @ critical point = 13.71Ton

ΦV_c = 10.1 Ton

min ΦV_s =3.68 Ton

complies with category (3)

Req. $\Phi V_s = V_u - \Phi V_c$ =13.71-10.1=3.61 Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 25.9 \text{ cm}$$

Use Φ 10 (2 leg) , S =25 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_e = 12 d_b =12(2) =24 cm

= d = 26 cm ……………… control

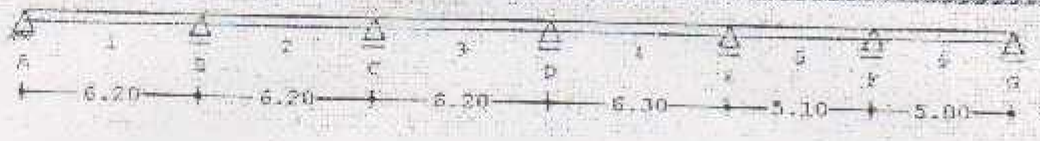
111.7 cm > 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(5.1 − 2(0.15)) = 1.44m

The total length of the top bars = 2 (1.44) + 0.6 =3.48m Use L = 3.5m

Beam (8)



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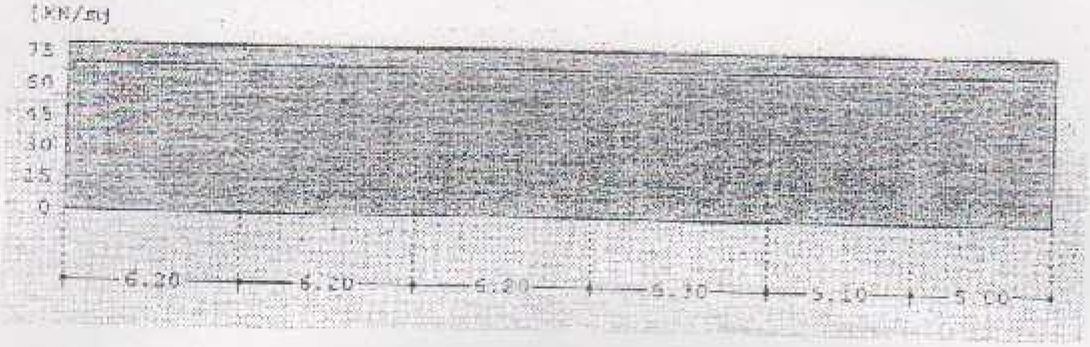

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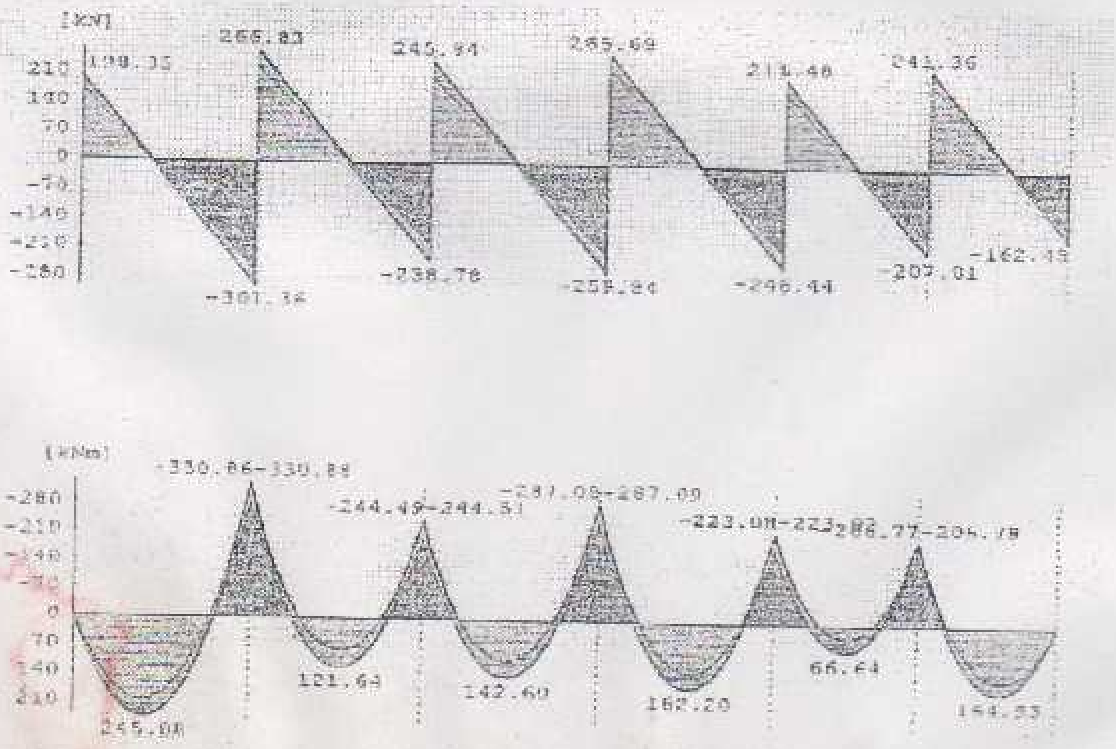


Shear & Moment Envelope Diagram

Fig. (6-20)

B =70 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =245.88 KN.m As shown in Fig. (6-20)

Mn =245.88 / 0.9 =273.2 KN.m =27.32 Ton.m

As max. =0.023 (70) (26) =41.86 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

C=.85(0.3)(9.75)(70) =174.03Ton

Mn max. =(174.03)(26-.5(9.75))/100 =36.76 Ton

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =67.35 Kg/cm^2 $\rho = 0.019$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.019 (70) (26) =34.5 cm^2

Use 12 Φ 20 mm As =37.68 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(70)a =17.85 a

Actual T = (37.68)(4.2) = 158.26Ton

Actual a = 8.87 cm

Actual x = a / ß = 8.87/ 0.85 = 10.44 cm

ε_s =0.00447 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =34.13 Ton.m > 27.32 Ton.m

2-Design For Negative Moment :

Mu =330.86 KN.m As shown in Fig. (6-20)

Mn =330.86 / 0.9 =367.6 KN.m =36.76 Ton.m

$m = 16.47$ Rn = 77.68 Kg/cm^2 $\rho = .0227$ $\rho_{min.} < \rho < \rho_{max.}$

$As = .0227\,(70)\,(26) = 41.58\ cm^2$

Use 12Φ 20 mm & 4Φ 14 mm $As = 43.98\ cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(70)a =17.85 a

Actual T = (43.98)(4.2) =184.716 Ton

Actual a = 184.716 / 17.85= 10.3 cm

Actual x = a / β = 10.3 / 0.85 = 11.5 cm

$\varepsilon_s = .00378 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =38.51 Ton.m >36.76 Ton.m ∴ OK.

3-Design For Shear :

$V_u = 30.14$ Ton

V_u @ critical point = 26.83 Ton

$\Phi V_c = 14.12$ Ton

min $\Phi V_s = 5.2$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 28.24$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 26.83 - 14.12 = 12.71$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 22.93 \text{ cm}$$

Use Φ 10 (4 leg) , S = 20 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_b =12(2)=24 cm

= d = 26 cm .... control

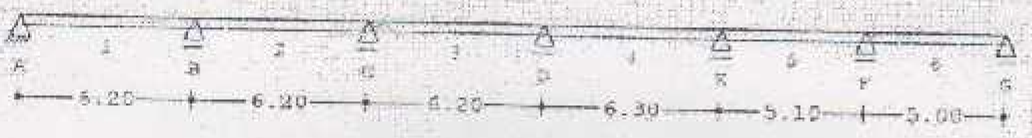
131.61 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3– 2(0.15)) = 1.8m

The total length of the top bars = 2 (1.8) + 0.6 =4.2 m Use L =4.2 m

Beam (4)



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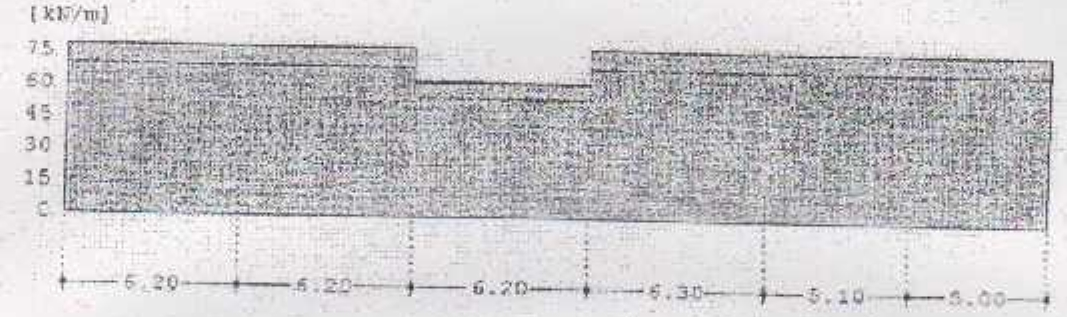

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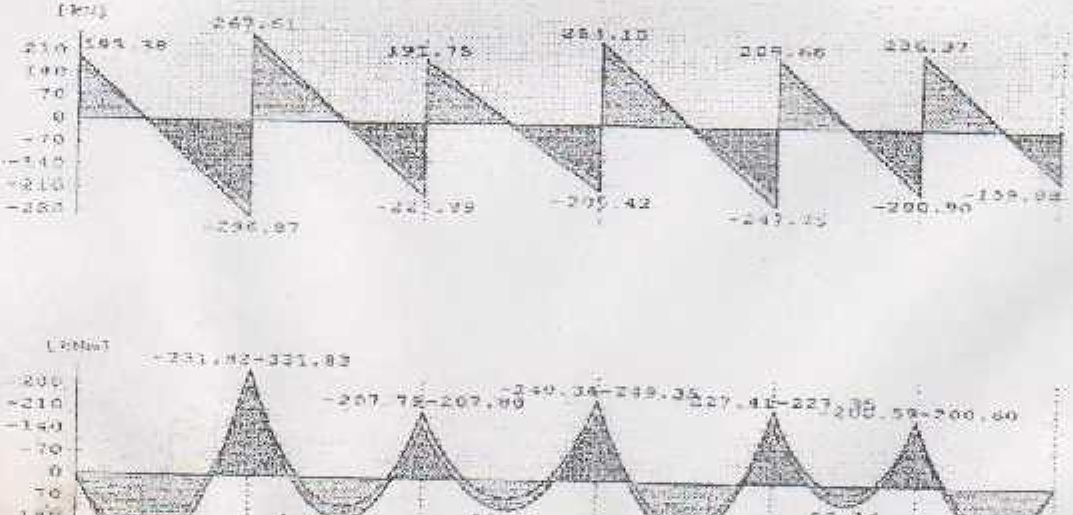


Shear & Moment Envelope Diagram

Fig. (6-21)

B =70 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =238.18 KN.m As shown in Fig. (6-21)

Mn =238.18 / 0.9 =264.64 KN.m = 26.5 Ton.m

As max. =0.023 (70) (26) =41.86 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C=.85(0.3)(9.75)(70) =174.03 Ton

Mn max. =(174.03)(26-.5(9.75))/100 =36.76 Ton

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 65.33 Kg/cm^2 ρ =0.0183 $\rho_{min} < \rho < \rho_{max.}$

As =0.0183 (70) (26) = 33.31 cm^2

Use 11Φ 20 mm As =33.79 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(70)a =17.85 a

Actual T = (33.79)(4.2) =141.9 Ton

Actual a = 7.95 cm

Actual x = a / β = 7.95 / 0.85 = 9.35 cm

ε_s = .00534 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn 31.26 = Ton.m > 26.5 Ton.m

2-Design For Negative Moment :

Mu =331.83 KN.m   As shown in Fig. (6-21)

Mn =331.83 / 0.9 =368.7 KN.m =36.87 Ton.m

$m = 16.47$ Rn = 77.9 Kg/cm^2 ρ =.0228 $\rho_{min.} < \rho < \rho_{max.}$

A_s =0.0228 (70) (26) =41.49 cm^2

Use 12 Φ 20 mm & 4 Φ 14 mm A_s =43.98 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(70)a = 17.85a

Actual T = (43.98)(4.2) =184.716 Ton

Actual a = 184.716 / 17.85 = 10.3 cm

Actual x = a / β = 10.3/ 0.85 = 12.1cm

ε_s =0.00344 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =38.5Ton.m > 36.87 Ton.m   ∴ OK.

3-Design For Shear :

V_u = 29.7 Ton

V_u @ critical point = 27.73 Ton

ΦV_c = 14.12 Ton

min ΦV_s =5.2 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =28.24 Ton   complies with category (4)

Req $\Phi V_s = V_u - \Phi V_c$ =27.73-14.12 =13.61 Ton

$S = \frac{0.85 A_v f_y d}{\Phi V_s}$ = 10.7 cm

Use Φ 10 (2 leg) , S =10 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_b =12(2)=24 cm

= d = 26 cm .... control

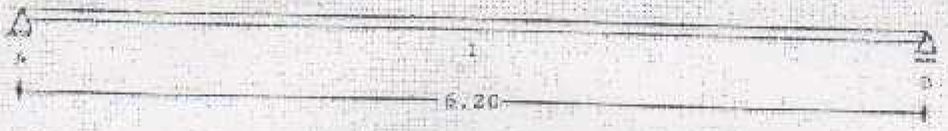
131.25cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3−2(0.15)) =1.8 m

The total length of the top bars = 2 (1.8) + 0.6 =4.2 m <u>Use L =4.2 m</u>

Beam (5)



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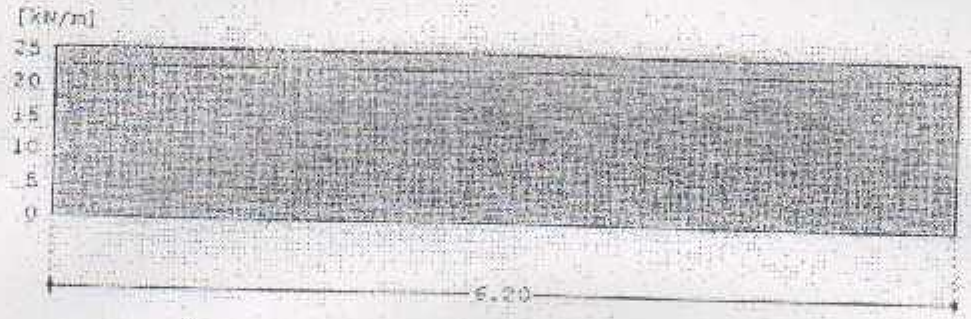

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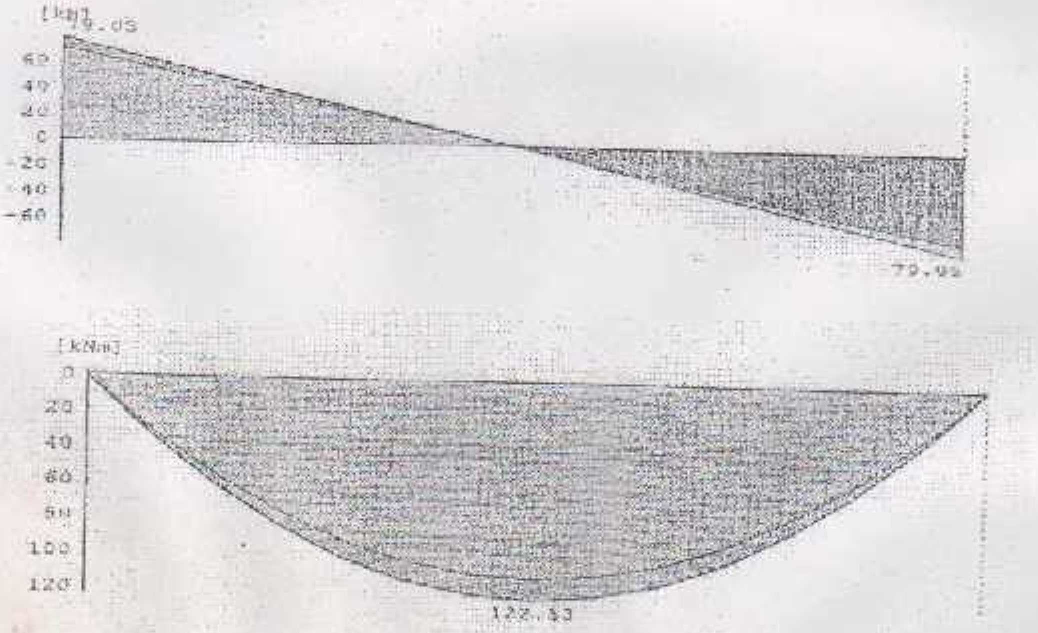


Shear & Moment Envelope Diagram

Fig. (6-22)

B =50 cm H = 30 cm d = 26 cm

1-Design For **Positive** Moment :

Mu =122.57 KN.m As shown in Fig. (6-22)

Mn = 122.57 / 0.9 = 136.19 KN.m = 13.62 Ton.m

As max. = 0.023 (50) (26) = 29.9 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

amax. = βX_b = 0.85 (11.47) = 9.75 cm

C=.85(0.3)(50)(9.75) =124.3

Mn max. = 124.3(26 – 0.5(9.75)) /100 = 26.26 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn =40.29 Kg/cm^2 ρ =0.0105 $\rho_{min.} < \rho < \rho_{max.}$

As =0.0105(50) (26) =13.65 cm^2

Use 5Φ 20 mm As =15.71 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50)a =12.75 a

Actual T = (15.71)(4.2) =65.98 Ton

Actual a = 5.17 cm

Actual x = a / β = 5.17 / 0.85 = 6.1 cm

ε_s = 0.00978 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =15.44 Ton.m > 13.62 Ton.m

2-Design For **Shear** :

V_u = 7.91 Ton

V_u @ critical point = 7.4 Ton

ΦV_c =10.1 Ton

complies with category (2) {minimum shear}

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 30 \text{ cm}$$

Use Φ 10 (2 leg) , S =30 cm

3-Development length (L_d) :

L_d for bottom bars (Φ 20) :

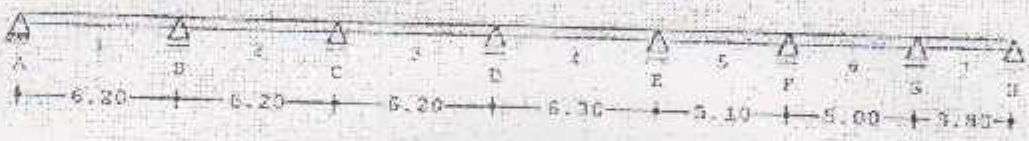
L_d=76.68 cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

221.4cm ≥ 76.68 cm ∴ available development > Req.

Beam (6):



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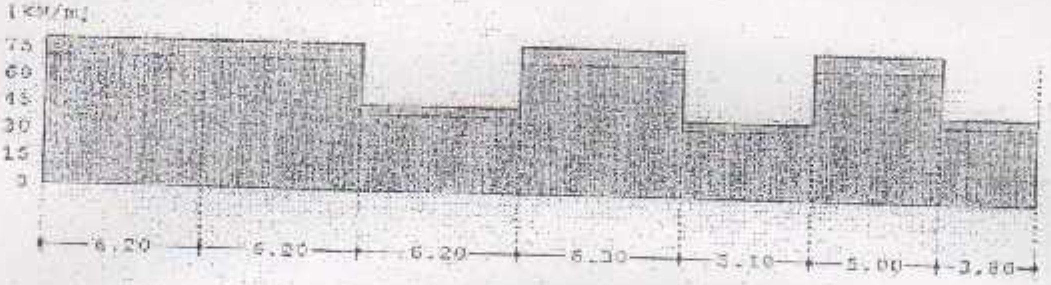

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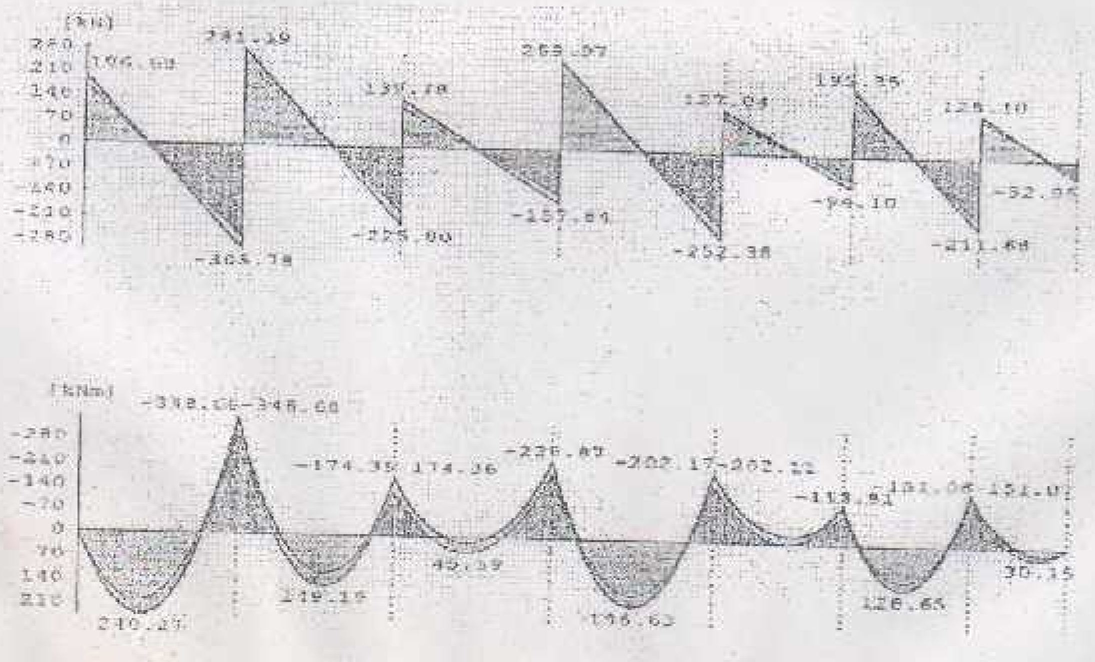


Shear & Moment Envelope Diagram

Fig. (6-23)

B =80 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =240.27 KN.m As shown in Fig. (6-23)

Mn =240.27 / 0.9 =266.9 KN.m =26.7 Ton.m

As max. =0.023 (80) (26) =47.84 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(80) =198.9 Ton

Mn max. =198.9 (26 – 0.5(9.75)) /100 = 42.02 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =49.37 Kg/cm^2 ρ =0.0132 $\rho_{min.} < \rho < \rho_{max.}$

As =0.0132(80) (26) =27.456 cm^2

Use 9Φ 20 mm As =28.27 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (28.27)(4.2) =118.73 Ton

Actual a =5.82 cm

Actual x = a / β 5.82/ 0.85 =6.8 cm

ε_s =0.0084 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =26.83 Ton.m >26.7 Ton.m

2-Design For Negative Moment :

Mu =348.66 KN.m As shown in Fig. (6-23)

Mn =348.66 / 0.9 =387.4 KN.m =38.74 Ton.m

$m = 16.47$ Rn =71.63 Kg / cm^2 ρ=0.021 $\rho_{min.} < \rho < \rho_{max.}$

A.s =0.021 (80) (26) =43.68 cm^2

Use 12 Φ 20 mm & 4 Φ 14 mm A.s =43.99 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (43.99)(4.2) =184.8 Ton

Actual a =184.8 /20.4 =9.1 cm

Actual x = a / β, 9.1/ 0.85 =10.7 cm

ε_s =0.0042 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =39.6 Ton.m >38.74 Ton.m ∴ OK.

3-Design For Shear :

V_u = 30.6 Ton

V_u @ critical point = 28.6 Ton

ΦV_c =12.2 Ton

min ΦV_s =4.4 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =24.2 Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ =28.6-12.2=16.4 Ton

Use 2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 17.78 \text{ cm}$$

Use 2Φ 10 (4 leg) , S 17 = cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

$L_d = 76.68$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

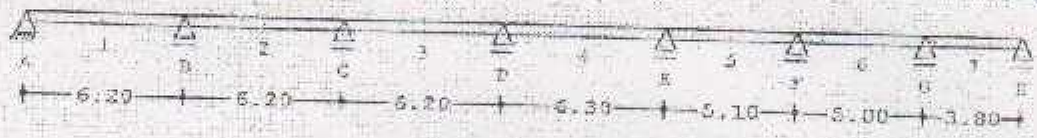
117.8 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (7) :



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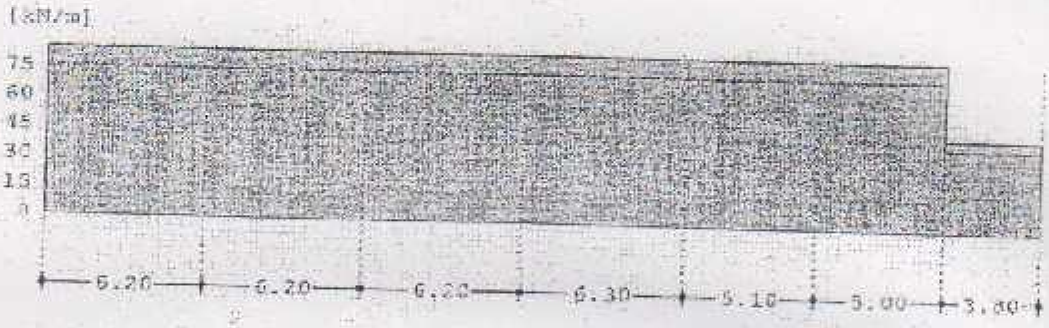

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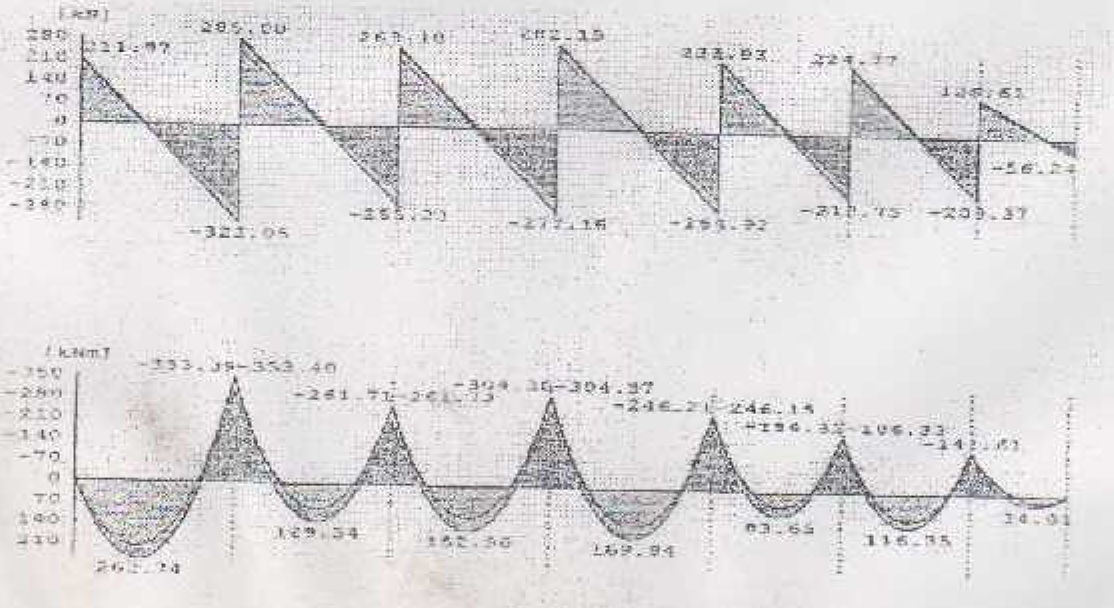


Shear & Moment Envelope Diagram

Fig. (6-24)

B = 80 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 262.74 KN.m As shown in Fig. (6-24)

Mn = 262.74 / 0.9 = 291.93 KN.m = 29.19 Ton.m

As max. = 0.023 (80) (26) = 47.84 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

T max. = $As\ fy$ = (47.84)(4.2) = 200.93 Ton

Mn max. = 200.93(26 − 0.5(9.75)) /100 = 42.45 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 53.98 Kg/cm^2 $\rho = 0.01461$ $\rho_{min} < \rho < \rho_{max}$

As = 0.01461(80) (26) = 30.39 cm^2

Use 10 Φ 20 mm As = 31.42 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(80)a = 20.4 a

Actual T = (31.42)(4.2) = 131.96Ton

Actual a = 6.47 cm

Actual x = a / β = 6.47 / 0.85 = 7.61 cm

$\varepsilon_s = 0.00725 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn 30.04 = 29.19 Ton.m

2 - Design For Negative Moment :

Mu = 353.39 KN.m As shown in Fig. (6-24)

Mn = 353.39/ 0.9 = 392.66 KN.m = 39.27Ton.m

$m = 16.47$ Rn = 72.61 Kg / cm^2 $\rho = 0.02088$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.02088(80)(26) = 43.42 cm^2$

Use 12 Φ 20 mm & 4 Φ 14 mm $As = 43.98 \ cm^2$

Check : tension steel is yield ?

C = 0.85(0.3)(80)a = 20.4 a

Actual T = (43.98)(4.2) = 184.72 Ton

Actual a = 184.72 / 20.4 = 9.05 cm

Actual x = a / β = 9.05 / 0.85 = 10.65 cm

$\varepsilon_s = 0.00432 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 39.67 Ton.m > 39.27 Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 32.21$ Ton

V_u @ critical point = 30.11 Ton

$\Phi V_c = 14.12$ Ton

min $\Phi V_s = 5.16$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 28.24$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 30.11 - 14.12 = 15.99$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 18.23 \text{ cm}$$

Use Φ 10 (4 leg) , S = 18 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

116.03 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

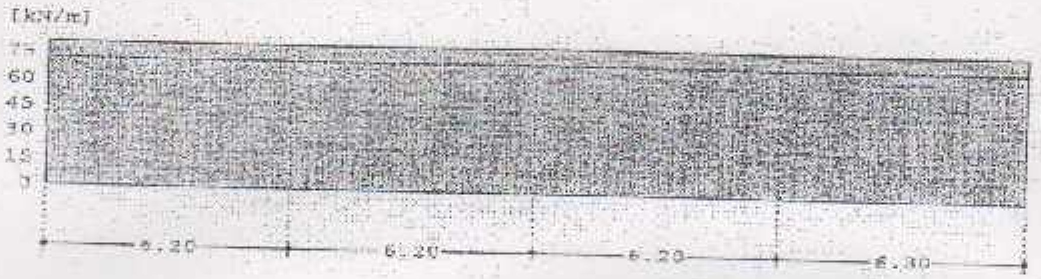
0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m

Use L = 4.2 m

Beam (8)

System



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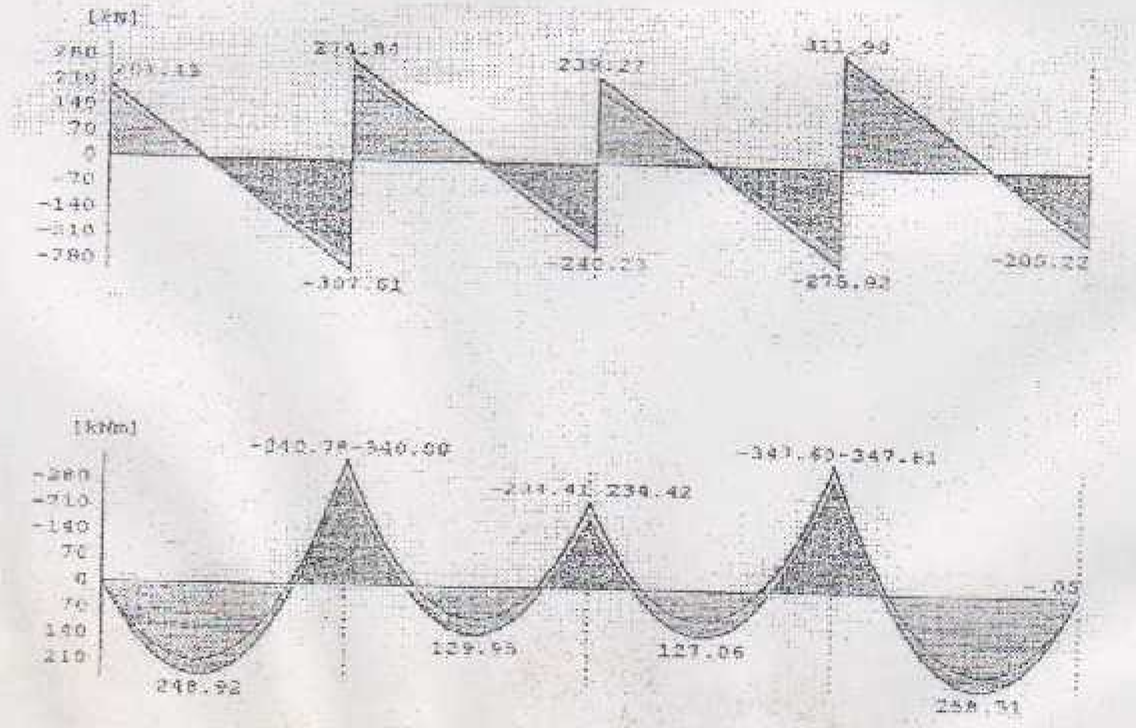


Shear & Moment Envelope Diagram

Fig. (6-25)

B = 80 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 258.31 KN.m As shown in Fig. (6-25)

Mn = 258.31 / 0.9 = 287.01KN.m = 28.7 Ton.m

As max. =0.023 (80) (26) = 47.84 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

Tmax. = As fy = (47.84)(4.2) = 200.93 Ton

Mn max. = 200.93(26 – 0.5(9.75)) /100 = 42.45 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 53.07 Kg / cm^2 ρ = 0.01433 $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01433(80) (26) = 29.8 cm^2

Use 10 Φ 20 mm As = 31.42 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a = 20.4 a

Actual T = (31.42)(4.2) = 131.96 Ton

Actual a = 6.47 cm

Actual x = a / β = 6.47 / 0.85 = 7.05 cm

ε_s = 0.00725 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mn = 30.04Ton.m

2 - Design For Negative Moment :

Mu = 347.61 KN.m   As shown in Fig. (6-25)

Mn = 347.61 / 0.9 = 386.23 KN.m = 38.62 Ton.m

$m = 16.47$ $Rn = 71.41\ Kg/cm^2$ $\rho = 0.02044$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.02044\ (80)\ (26) = 42.52\ cm^2$

Use 12 Φ 20 mm & 4 Φ 14 mm $As = 43.98\ cm^2$

Check : tension steel is yield ?

C = 0.85(0.3)(80)a = 20.4 a

Actual T = (43.98)(4.2) = 184.73 Ton
Actual a = 184.73 / 20.4 = 9.06 cm

Actual x = a / β = 9.06 / 0.85 = 10.65 cm

$\varepsilon_s = 0.00432 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 39.66 Ton.m > 38.62 Ton.m   ∴ OK.

3 - Design For Shear :

$V_u = 31.19$ Ton

V_u @ critical point = 29.15 Ton

$\Phi V_c = 12.1$ Ton

$\min\ \Phi V_s = 4.42$Ton

$0.85\left(\dfrac{\sqrt{fc'}}{3}\right)bd = 24.21$ Ton   complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 31.19 - 12.1 = 17.05$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 17.09 \text{ cm}$$

Use Φ 10 (4 leg) , S = 17 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

116.03 cm ≥ 76.68 cm ∴ available development > Req.

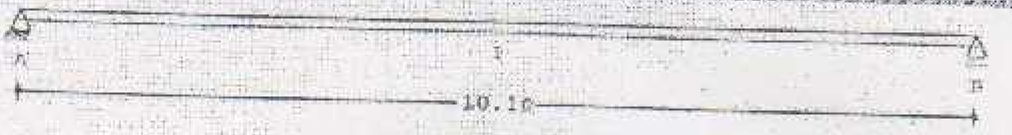
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.2 – 2(0.15)) = 1.77 m

The total length of the top bars = 2 (1.77) + 0.6 = 4.14 m

Use L = 4.2 m

Beam (9)



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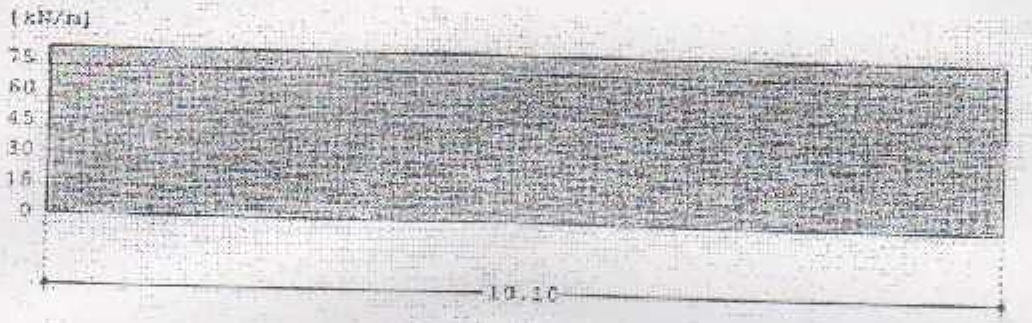

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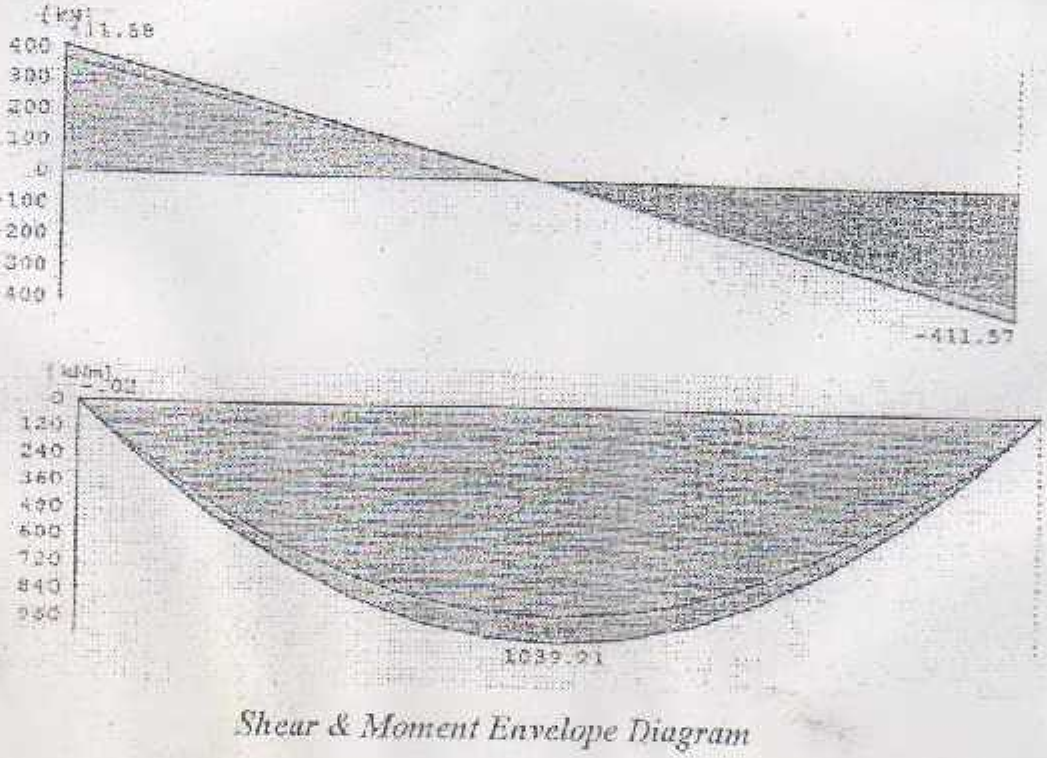


Shear & Moment Envelope Diagram

Fig. (6-26)

B = 40 cm H = 80 cm d = 76 cm

1 - Design For Positive Moment :

Mu = 1039.2 KN.m As shown in Fig. (6-26)

Mn = 1039.2 / 0.9 = 1154.68 KN.m = 115.47 Ton.m

As max. = 0.023 (40) (73.5) = 67.62 cm^2

X_b = 43.24 cm X max. = 0.75 X_b = 32.43 cm

a max. = β X_b = 0.85 (11.47) = 27.56 cm

T max. = As fy = (67.62)(4.2) = 284.0 Ton

Mn max. = 284 (73.5 – 0.5(27.56)) /100 = 169.6 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 49.98 Kg/cm^2 ρ = 0.01337 $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01337 (40) (73.5) = 40.65 cm^2

Use 14 Φ 20 mm in two layers As = 43.98 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (43.98)(4.2) = 184.72 Ton

Actual a = 18.11 cm

Actual x = a / β = 18.11 / 0.85 = 21.31 cm

ε_s = 0.00213 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn = 119.04 Ton.m

2 - Design For Shear :

$V_u = 41.16$ Ton

V_u @ critical point = 39.12 Ton

$\Phi V_c = 23.59$ Ton

min $\Phi V_s = 8.61$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 47.18$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 39.12 - 23.59 = 15.53$ Ton

Use Φ 10 for stirrups

$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 27.43$ cm

Use Φ 10 (2 leg) , S = 25 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

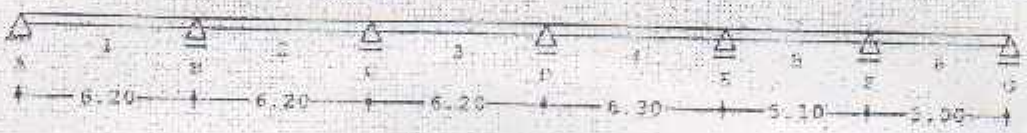
$L_d = 84.35$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 73.5 cm .... control

362.7 cm ≥ 76.68 cm ∴ available development > Req.

Beam (10)



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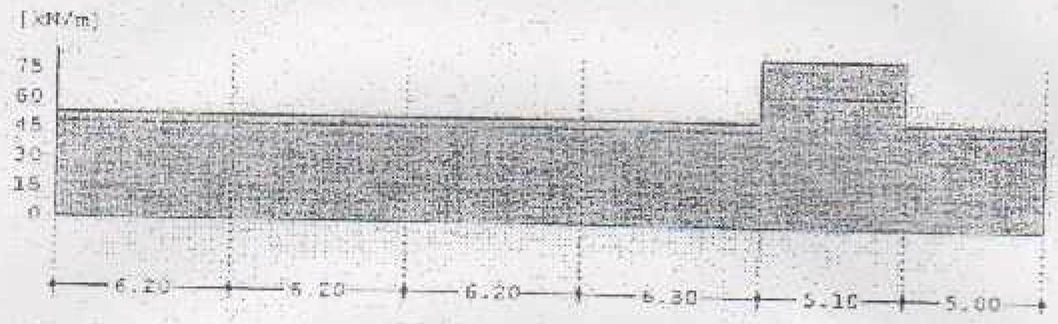

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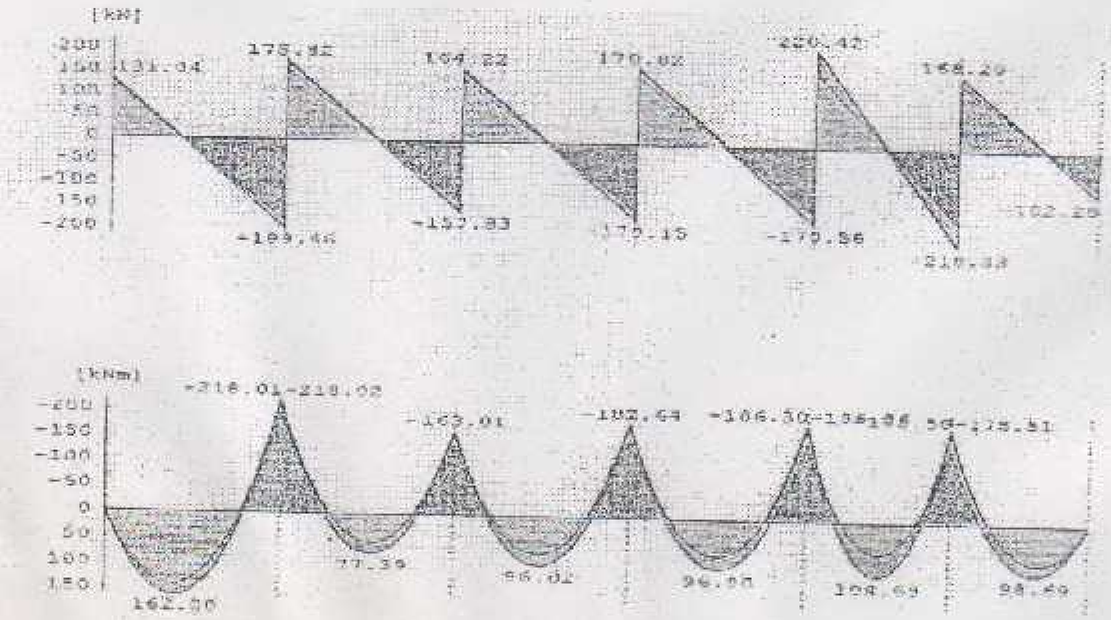


Shear & Moment Envelope Diagram

Fig. (6-27)

B = 50 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 162 KN.m As shown in Fig. (6-27)

Mn = 162 / 0.9 = KN.m = 180 Ton.m

As max. =0.023 (50) (26) = 35.88 cm^2

X_b = 15.29 cm X max. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

T max. = As fy = (35.88)(4.2) = 150.7 Ton

Mn max. = 150.7(26 – 0.5(9.75)) /100 = 31.84 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn =53.25 Kg/cm^2 ρ = 0.01438 $\rho_{min.} < \rho < \rho_{max.}$

As =0.01438 (50) (26) = 18.7 cm^2

Use 6 Φ 20 mm As = 18.85 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50)a = 12.75 a

Actual T = (18.85)(4.2) = 79.17 Ton

Actual a = 6.21 cm

Actual x = a / β = 6.21 / 0.85 = 7.3 cm

ε_s = 0.00767 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn = 18.13 Ton.m

2 - Design For Negative Moment :

Mu = 218.02 KN.m As shown in Fig. (6-27)

Mn = 218.02 / 0.9 = 242.24 KN.m = 24.22 Ton.m

$m = 16.47$ $Rn = 71.67 \ Kg/cm^2$ $\rho = 0.01609$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.01609\,(50)\,(26) = 20.91 \ cm^2$

Use 6 Φ 20 mm & 2 Φ 14 mm $As = 21.99 \ cm^2$

Check : tension steel is yield ?

C = 0.85(0.3)()a = a

Actual T = (21.99)(4.2) = 92.36 Ton

Actual a = 92.36 / 12.75 = 7.24 cm

Actual x = a / β = 7.24 / 0.85 = 8.52 cm

$\varepsilon_s = 0.00615 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 25.36 Ton.m > 24.22 Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 22.04$ Ton

V_u @ critical point = 19.92 Ton

$\Phi V_c = 10.1$ Ton

min $\Phi V_s = 3.68$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 20.17$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 19.92 - 10.1 = 9.82$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 29.68 \text{ cm}$$

Use Φ 10 (4 leg) , S = 25 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

109.04 cm ≥ 76.68 cm ∴ available development > Req.

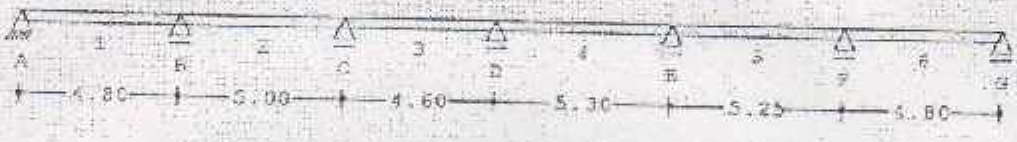
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m

Use L = 4.2 m

Beam (11) :



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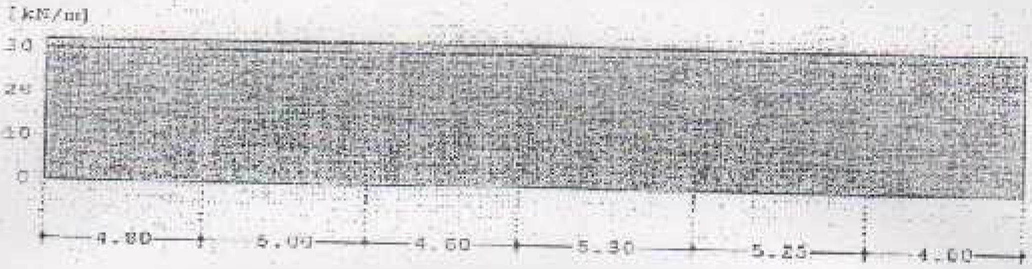

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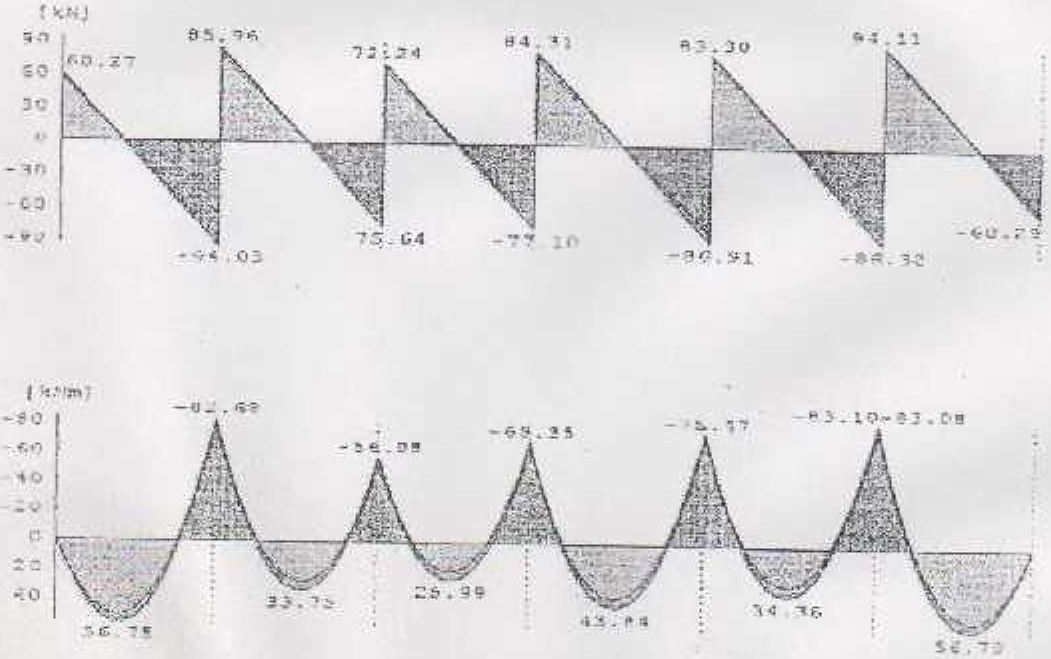


Shear & Moment Envelope Diagram

Fig. (6-28)

B = 40 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 56.79 KN.m As shown in Fig. (6-28)

Mn = 56.79 / 0.9 = 63.1 KN.m = 6.31 Ton.m

As max. = 0.023 (40) (26) = 23.92 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

Tmax. = As fy = (23.92)(4.2) = 100.46 Ton

Mn max. = 100.46 (26 – 0.5(9.75)) /100 = 21.22 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 23.34 Kg/cm^2 $\rho = 0.00584$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.00584 (40) (26) = 6.07 cm^2

Use 4 Φ 14 mm As = 6.16 cm^2

2 Φ 14 straight & 2 Φ 14 bent

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (6.16)(4.2) = 25.87 Ton

Actual a = 2.54 cm

Actual x = a / β = 2.54 / 0.85 = 2.98 cm

ε_s = 0.00.02317 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mu = 6.4 Ton.m

2 - Design For Negative Moment :

Mu = 83.1 KN.m   As shown in Fig. (6-28)

Mn = 83.1 / 0.9 = 92.33 KN.m = 9.23 Ton.m

$m = 16.47$ Rn = 34.13 Kg/cm^2 $\rho = 0.00876$ $\rho_{min} < \rho < \rho_{max}$

As = 0.00876 (40) (26) = 9.11 cm^2

Use 6 Φ 14 mm As = 9.24 cm^2

4 Φ 14 from positive reinforcement & 2 Φ 14 straight

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (9.24)(4.2) = 38.81 Ton

Actual a = 38.81 / 10.2 = 3.8 cm

Actual x = a / β = 3.8 / 0.85 = 4.48 cm

$\varepsilon_s = 0.01441 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 9.35 Ton.m > 9.23 Ton.m   ∴ OK.

3 - Design For Shear :

V_u = 9.41 Ton

V_u @ critical point = 8.13 Ton

ΦV_c = 8.07 Ton

min ΦV_s = 2.95 Ton   complies with category (2)

Use Φ 10 for stirrups

$S = \dfrac{3A_v f_y}{b_w}$ = 98.91 cm

Use Φ 10 (2 leg) , S = 25 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 14) :

$L_d = 53.68$ cm

$L_a = 12\ d_b = 12(1.4) = 16.8$ cm

$= d = 26$ cm .... control

94 cm ≥ 53.68 cm ∴ available development > Req.

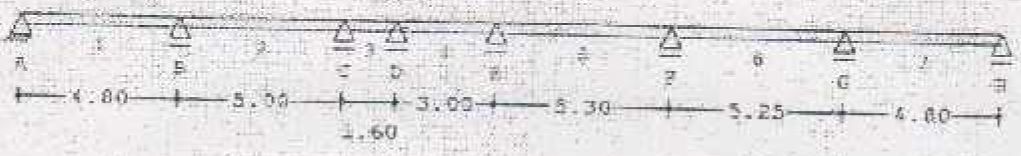
L_d for Top bars (Φ 14) :

0.3 Ln from face of support $= 0.3(5.3 - 2(0.15)) = 1.5$ m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m

Use L = 3.6 m

Beam (12)



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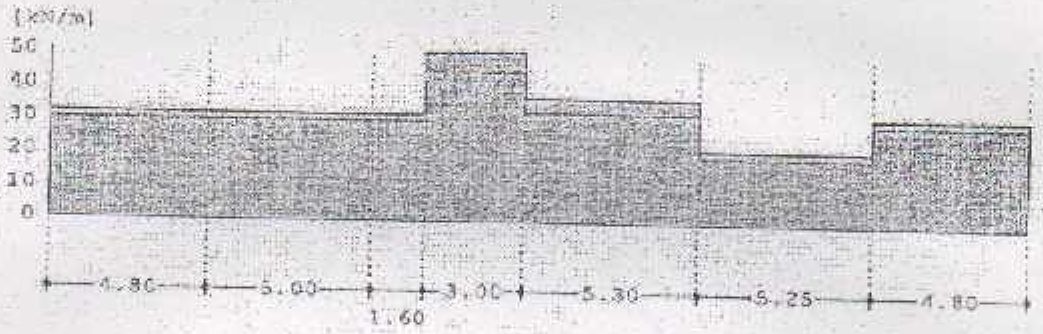

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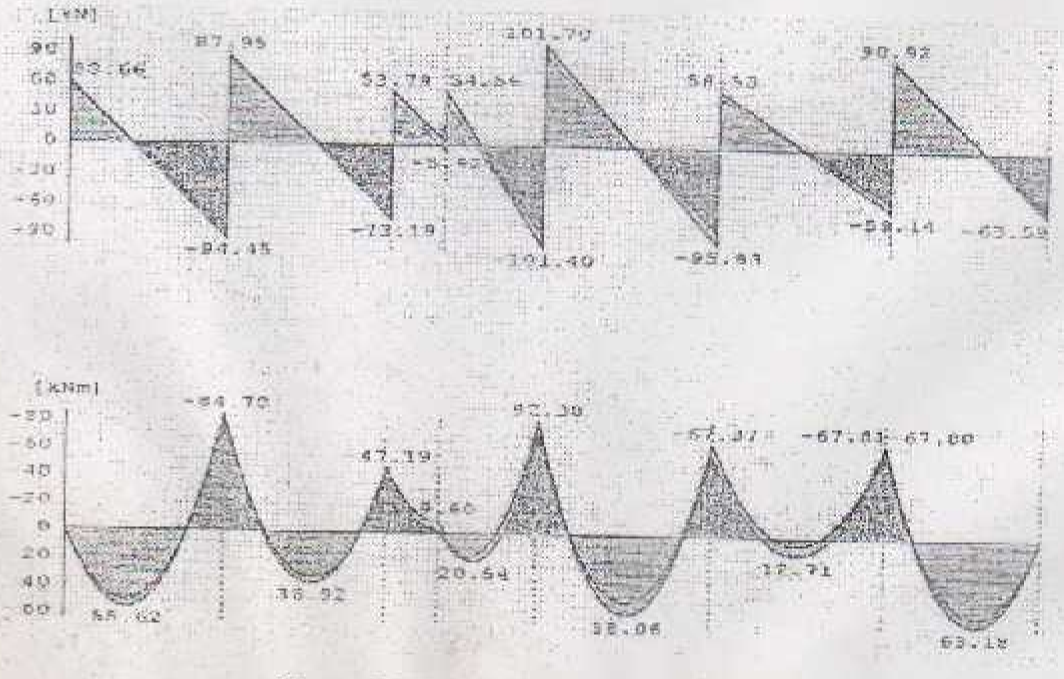


Shear & Moment Envelope Diagram

Fig. (6-29)

B = 40 cm H = 30 cm d = 26 cm

I - Design For Positive Moment :

Mu = 63.18 KN.m As shown in Fig. (6-29)

Mn = 63.18 / 0.9 = 70.2 KN.m = 7.02 Ton.m

As max. = 0.023 (40) (26) = 23.92 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

Tmax. = As fy = (23.92)(4.2) = 100.46 Ton

Mn max. = 100.46(26 – 0.5(9.75)) /100 = 21.22 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 25.96 Kg/cm^2 $\rho = 0.00653$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.00653 (40) (26) = 6.79 cm^2

Use 2Φ 14 mm straight , 3Φ 14 bent As = 7.7 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (7.7)(4.2) = 32.34 Ton

Actual a = 3.17 cm

Actual x = a / β = 3.17 / 0.85 = 3.73 cm

$\varepsilon_s = 0.011791 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 7.9 Ton.m

2 - Design For Negative Moment :

Mu = 84.7 KN.m   As shown in Fig. (6-29)

Mn = 84.7 / 0.9 = 94.11 KN.m = 9.41 Ton.m

$m = 16.47$ Rn = 34.8 Kg/cm^2 $\rho = 0.00895$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.00895(40) (26) = 9.31 cm^2

Use 6 Φ 14 mm from positive steel, 2 Φ 14 mm straight As = 12.31 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(40)a = 10.2 a

Actual T = (10.78)(4.2) = 45.28 Ton

Actual a = 45.28 / 10.2 = 4.44 cm

Actual x = a / β = 4.44 / 0.85 = 5.22 cm

$\varepsilon_s = 0.01194 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 10.77 Ton.m > 9.41 Ton.m   ∴ OK.

3 - Design For Shear :

V_u = 10.17 Ton

V_u @ critical point = 9.25 Ton

ΦV_c = 8.07 Ton

min ΦV_s = 2.95 Ton   complies with category (2)

Use Φ 10 for stirrups

$$S = \frac{3A_v f_y}{b_w} = 98.91 \text{ cm}$$

Use Φ 10 (2 leg) , S = 25 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 14) :

L_d = 53.68 cm

$L_a = 12\ d_b = 12(1.4) = 16.8$ cm

= d = 26 cm ……………… control

103.64 cm > 53.68 cm ∴ available development > Req.

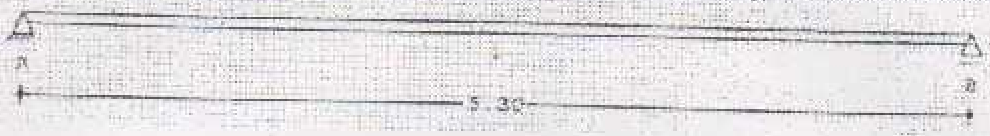
L_d for Top bars (Φ 14) :

0.3 Ln from face of support = 0.3(5.3 – 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m

Use L = 3.6 m

Beam (13):



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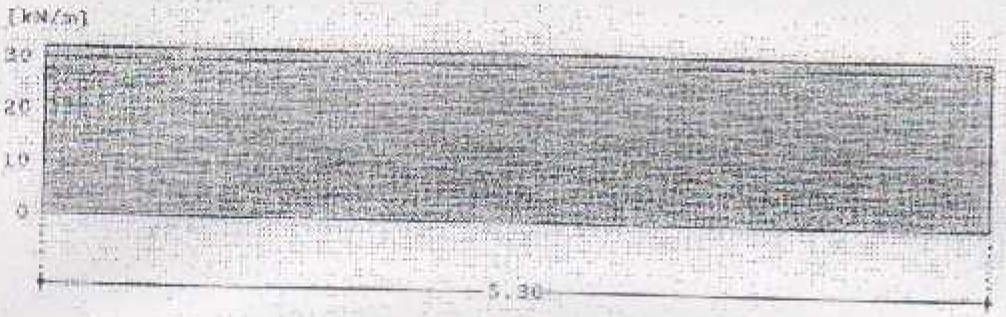

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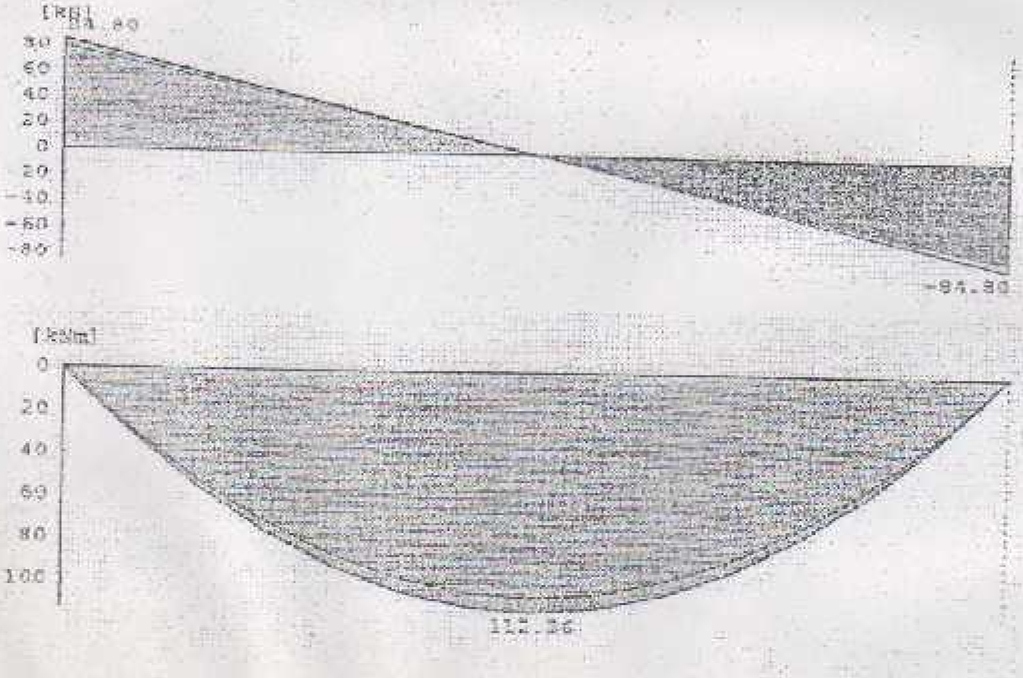


Shear & Moment Envelope Diagram

Fig. (6-30)

B = 40 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu =112.36 KN.m As shown in Fig. (6-30)

Mn = 112.36 / 0.9 = 124.84 KN.m = 12.48 Ton.m

As max. =0.023 (40) (26) = 23.92 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

Tmax. = $As\ fy$ = (23.92)(4.2) = 100.46 Ton

Mn max. = 100.46(26 – 0.5(9.75)) /100 = 21.22 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 46.17 Kg/cm^2 ρ = 0.01222 $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01222(40) (26) = 12.71 cm^2

Use 9 Φ 14 mm As = 13.85 cm^2

3 Φ 14 straight & 6 Φ 14 bent

Check : tension steel is yield ?

C =0.85(0.3)()a = a

Actual T = (13.85)(4.2) = 58.17Ton

Actual a = 5.7 cm

Actual x = a / β = 5.7 / 0.85 = 6.71 cm

ε_s = 0.00862 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn = 13.47 Ton.m

2 - Design For Shear :

$V_u = 8.48$ Ton

V_u @ critical point = 7.17 Ton

$\Phi V_c = 8.07$ Ton

min $\Phi V_s = 2.95$ Ton complies with category (2)

$S = \frac{3A_v f_y}{b_w} = 49.45$ cm

Use Φ 10 (2 leg) , S = 25 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 14) :

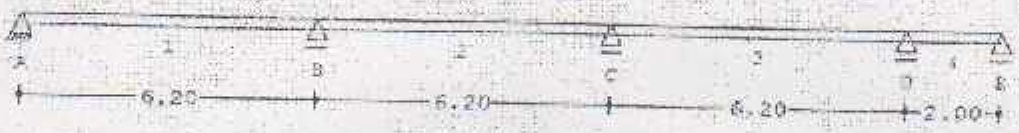
$L_d = 53.68$ cm

$L_a = 12\ d_b = 12(1.4) = 16.8$ cm

$= d = 26$ cm ……………… control

184.84 cm ≥ 76.68 cm ∴ available development > Req.

Beam (14)



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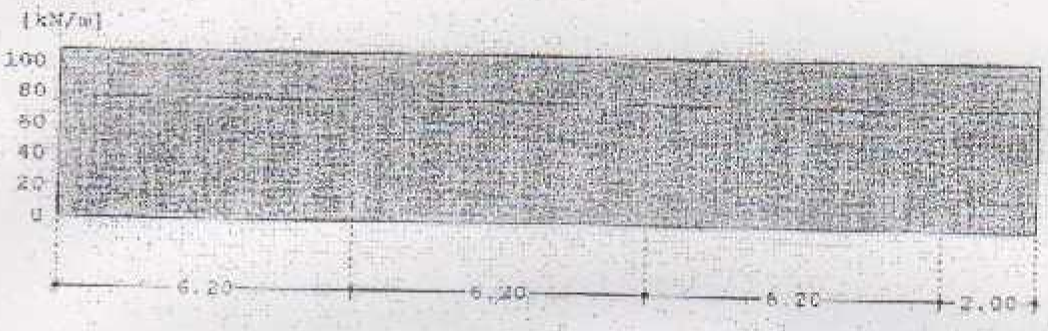

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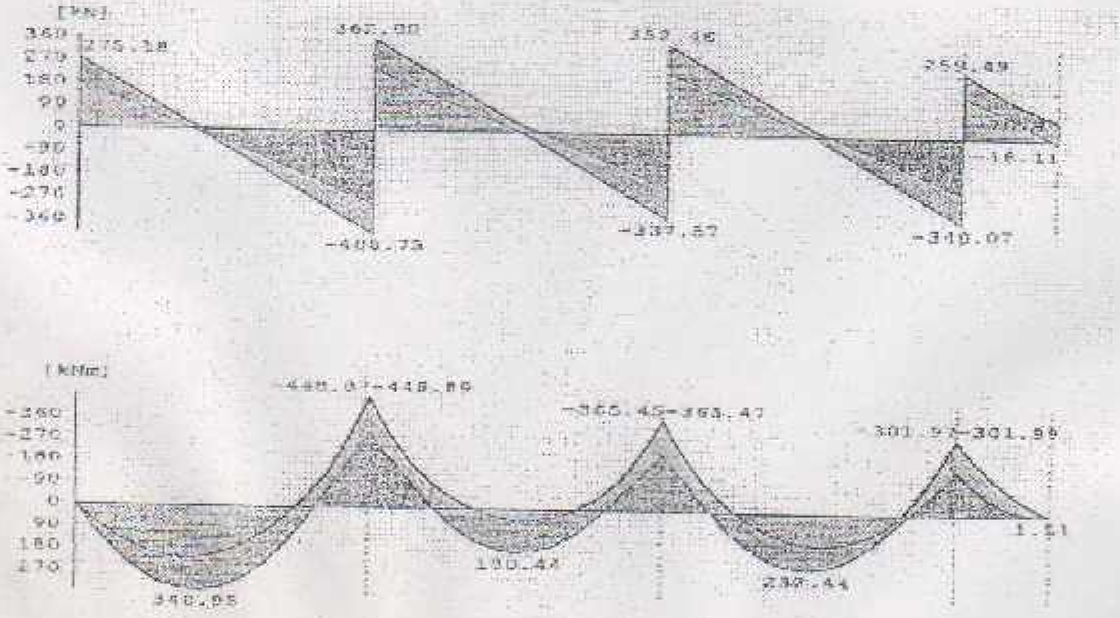


Shear & Moment Envelope Diagram

Fig. (6-31)

B =100 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =348.95 KN.m As shown in Fig. (6-31)

Mn =348.95 / 0.9 =387.7 KN.m =38.77 Ton.m

$As_{max.}$ =0.023 (100) (26) =54.6 cm^2

X_b = 15.29 cm $X_{max.}$ = 0.75 X_b = 11.47 cm

$a_{max.}$ = βX_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(100) =248.62 Ton

Mn max. =248.62 (26 − 0.5(9.75)) /100 = 52.5 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =57.35 Kg/cm^2 ρ =0.0156 $\rho_{min} < \rho < \rho_{max}$

As =0.0156 (100) (26) =40.76 cm^2

Use 13Φ 20 mm As =40.84 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(100)a =25.5 a

Actual T = (40.84)(4.2) =171.52 Ton

Actual a =6.73 cm

Actual x = a / β, 6.73 / 0.85 = cm

ε_s =0.00687 > ε_y = .0021

∴ Tension steel is yielding ∴ OK

Actual Mn =38.83 Ton.m >38.77 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =448.87 KN.m As shown in Fig. (6-31)

Mn =448.87 / 0.9 =498.74 KN.m =49.87 Ton.m

$m = 16.47$ Rn =73.77 Kg/cm^2 $\rho = 0.0213$ $\rho_{min} < \rho < \rho_{max.}$

As =0.0213 (100) (26) =55.4 cm^2

Use 13 Φ 20 mm & 4 Φ 14 mm As =57.02 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(100)a =25.5 a

Actual T = (57.02)(4.2) =239.48 Ton

Actual a =239.48 /25.5 =9.39 cm

Actual x = a / β, 9.39 / 0.85 =11.1 cm

$\varepsilon_s = 0.00402 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =51 Ton.m >49.87 Ton.m ∴ OK.

3-Design For Shear :

V_u = 40.87 Ton

V_u @ critical point = 38.167 Ton

ΦV_c =20.2 Ton

min ΦV_s =7.4 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =40.34 Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ =38.167-20.2 Ton

Use 2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 16.2 \text{ cm}$$

Use 2Φ 10 (4 leg) , S =16 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_b =12(2)=24 cm

= d = 26 cm .... control

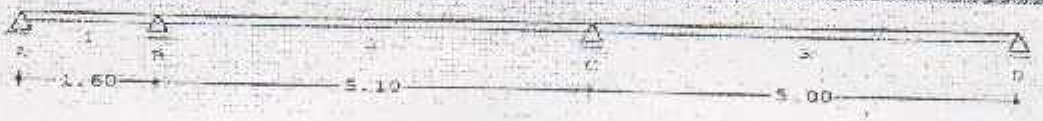
121.2 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.2– 2(0.15)) =1.77 m

The total length of the top bars = 2 (1.77) + 0.6 =4.14 m Use L =4.2 m

Beam (15)



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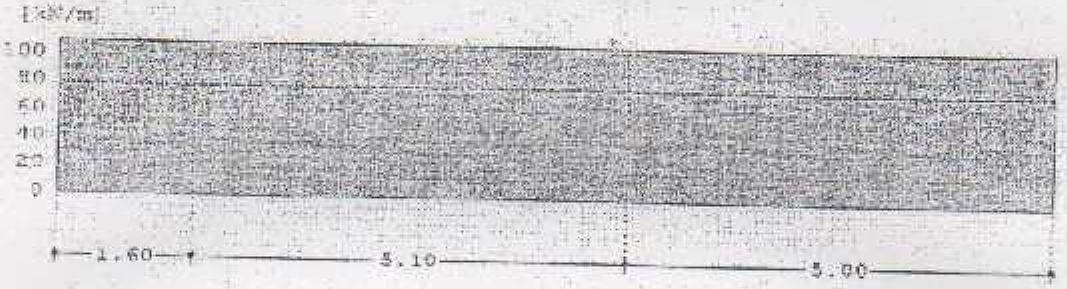

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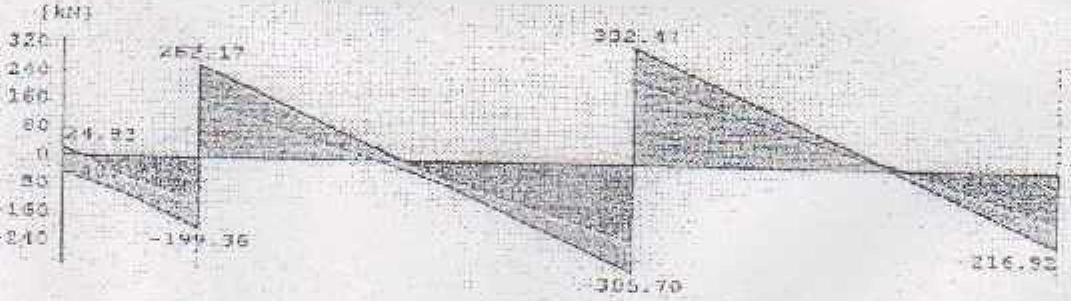
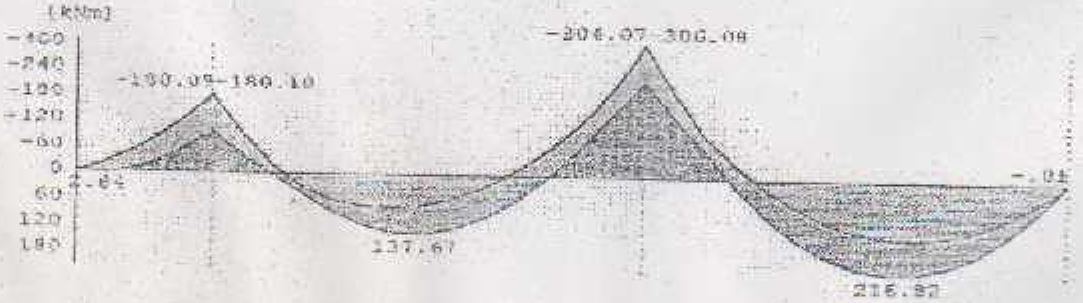




Shear & Moment Envelope Diagram

Fig. (6-32)

B =70 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =216.82 KN.m As shown in Fig. (6-32)

Mn =216.82 / 0.9 =240.9 KN.m =24.1 Ton.m

$As\ max. = 0.023\ (70)\ (26) = cm^2$

$X_b = 15.29$ cm $X max. = 0.75\ X_b = 11.47$ cm

$a\ max. = \beta\ X_b = 0.85\ (11.47) = 9.75$ cm

C =0.85(0.3)(70)(9.75)=174 Ton

Mn max. = 174(26 – 0.5(9.75)) /100 = 36.75 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ $Rn = 50.9\ Kg/cm^2$ $\rho = 0.0136$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.0136\ (70)\ (26) = 24.75\ cm^2$

Use 8Φ 20 mm $As = 25.13\ cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(70)a =17.85 a

Actual T = (25.13)(4.2) =105.54 Ton

Actual a =5.9 cm

Actual $x = a / \beta_1 = 5.9/ 0.85 = 6.95$ cm

$\varepsilon_s = 0.0082 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK

Actual Mn =24.32 Ton.m > 24.1 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =306.1 KN.m   As shown in Fig. (6-32)

Mn =306.1 / 0.9 =340.1 KN.m   =34 Ton.m

m = 16.47   Rn =71.85 Kg/cm^2 ρ=0.021 $\rho_{min} < \rho < \rho_{max}$

As =0.021 (70) (26) =38.22 cm^2

Use 11 Φ 20 mm & 4 Φ 14 mm As =41 cm^2

Check : tension steel is yield ?
C =0.85(0.3)(70)a =17.85 a

Actual T = (41)(4.2) =172.2 Ton

Actual a = 172.2 / 17.85 = 9.647 cm

Actual x = a / β_1 = 9.647 / 0.85 = 11.35 cm

ε_s =0.00387 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =36.46 Ton.m > 34 Ton.m   ∴ OK.

3-Design For Shear :

V_u 33.24 Ton

V_u @ critical point = 30.5 Ton

ΦV_c =14.12 Ton

min ΦV_s =5.2 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =28.24 Ton   complies with category (4)

Req. ΦV_s = V_u - ΦV_c =33.24-14.12 =19.12 Ton

Use 2Φ 10 for stirraps

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 15 \text{ cm}$$

Use 2Φ 10 (4 leg) , S = 15 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

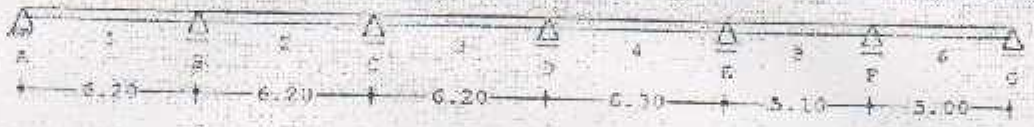
99.2 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(5.1 − 2(0.15)) = 1.44 m

The total length of the top bars = 2 (1.44) − 0.6 = 3.48 m Use L = 3.5 m

Beam (16)



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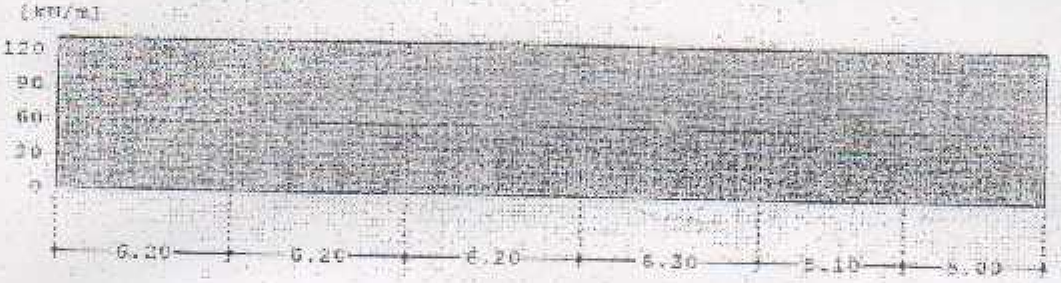

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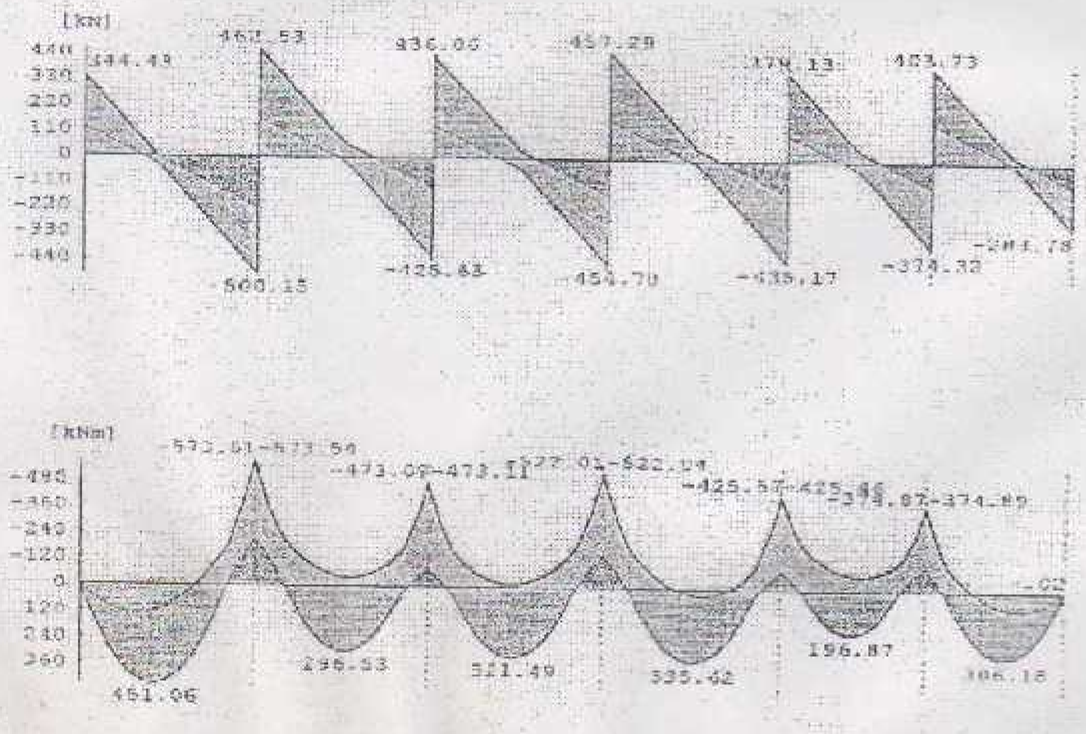


Shear & Moment Envelope Diagram

Fig. (6-33)

B =120 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =451.06 KN.m As shown in Fig. (6-33)

Mn =451.06 / 0.9 =501.2 KN.m =50.1 Ton.m

As max. =0.023 (120) (26) =71.76 cm^2

X_b = 15.29 cm X max. = 0.75 X_b = 11.47 cm

a max. = βX_a = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(120) =298.35 Ton

Mn max = 298.35(26 – 0.5(9.75)) /100 = 63.1 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn =61.8 Kg/cm^2 ρ=0.0171 $\rho_{min.} < \rho < \rho_{max.}$

As =0.0171 (120) (26) =53.5 cm^2

Use 17Φ 20 mm As =53.5 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(120)a =30.6 a

Actual T = (53.5)(4.2) =224.7 Ton

Actual a =7.34 cm

Actual x = a / β = 7.34 / 0.85 = 8.64 cm

ε_s = 0.00602 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =50.2 Ton.m > 50.1 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =573.51 KN.m As shown in Fig. (6-33)

Mn =573.51 / 0.9 =637.233 KN.m =63.7 Ton.m

$m = 16.47$ Rn = 78.5 Kg/cm^2 $\rho = 0.023$ $\rho_{min} < \rho < \rho_{max}$

As =0.023 (120) (26) =71.8 cm^2

Use 21Φ 20 mm & 4 Φ 14 mm As 72.26 = cm^2

Check : tension steel is yield ?

C =0.85(0.3)(120)a =30.6 a

Actual T = (72.26)(4.2) =303.49 Ton

Actual a = 303.49 / 30.6 = 9.9 cm

Actual x = a / β = 9.9 / 0.85 = 11.6 cm

ε_s =0.00367 > ε_y = .0021

∴ Tension steel is yielding ∴ OK

Actual Mn =64.5 Ton.m > 63.1 Ton.m ∴ OK

3-Design For Shear :

V_u = 50 Ton

V_u @ critical point = 46.7 Ton

ΦV_c =24.2 Ton

min ΦV_s =8.84 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =48.4 Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ =46.7-24.2 = 22.2 Ton

Use 2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 13.12 \text{ cm}$$

Use 2Φ 10 (4 leg) , S = 13 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_o =12(2) =24 cm

= d = 26 cm ………………… control

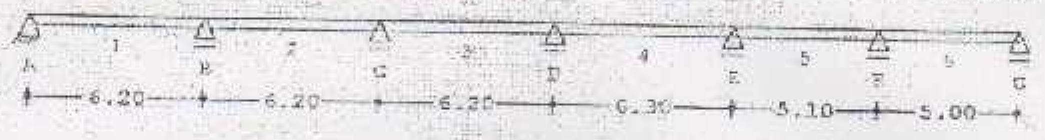
126.4 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (17)



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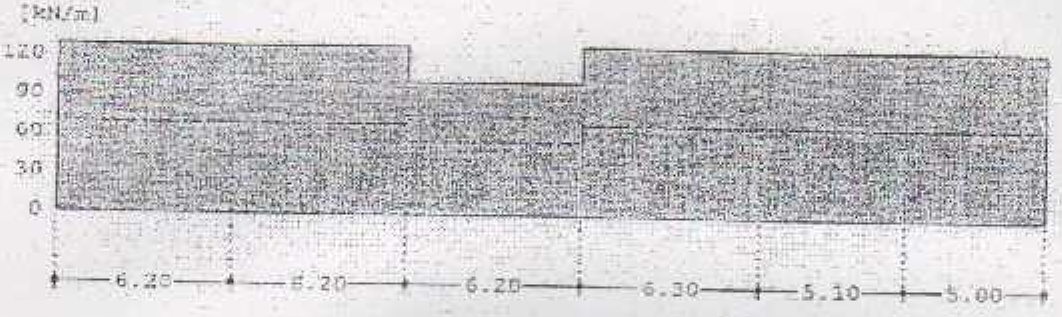
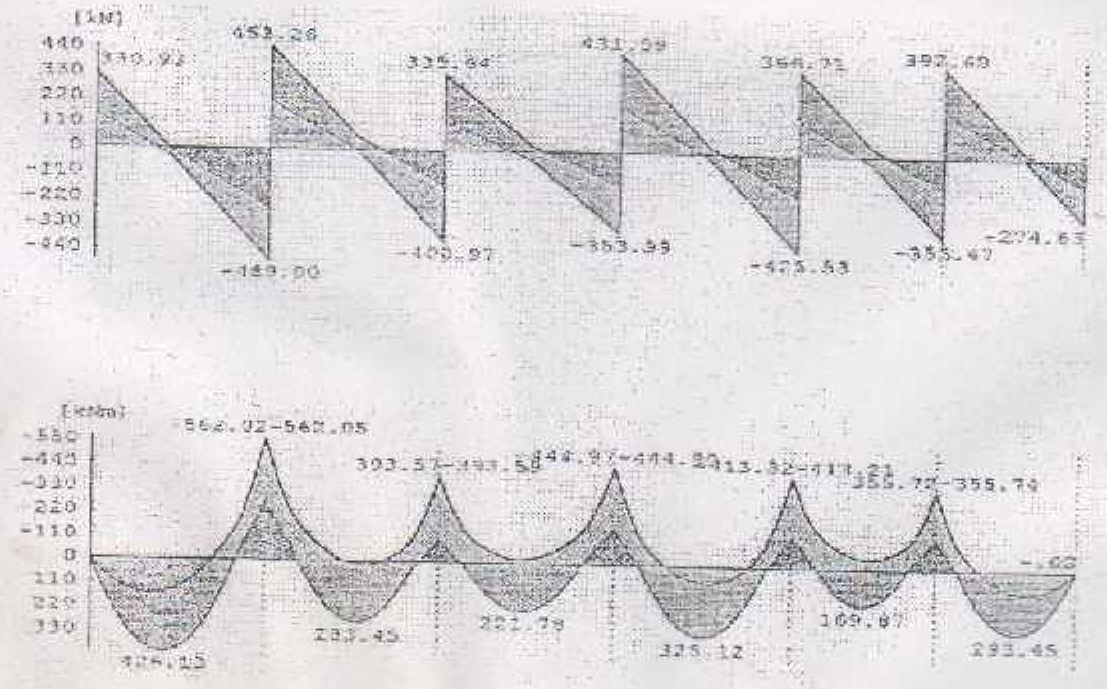


Loads



Shear & Moment Envelope Diagram

Fig. (6-34)

B =120 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =426.13 KN.m As shown in Fig. (6-34)

Mn =426.13 / 0.9 =473.4 KN.m = 47.34 Ton.m

$As\ max. = 0.023\ (120)\ (26) = 71.76\ cm^2$

$X_b = 15.29$ cm $X\max. = 0.75\ X_b = 11.47$ cm

$a\max. = \beta\ X_b = 0.85\ (11.47) = 9.75$ cm

C=0.85(0.3)(9.75)(120) = 298.35 Ton

Mn max.= 298.35(26 – 0.5(9.75)) /100 = 63.1 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ $Rn = 58.4\ Kg/cm^2$ $\rho = 0.016$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.016\ (120)\ (26) = 49.9\ cm^2$

Use 16 Φ 20 mm $As = 50.26\ cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(120)a =30.6 a

Actual T = (50.26)(4.2) =211.1 Ton

Actual a =6.9 cm

Actual $x = a/\beta = 6.9 / 0.85 = 8.1$ cm

$\varepsilon_s = 0.0066 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =47.6 Ton.m > 47.34 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =562.03 KN.m As shown in Fig. (6-34)

Mn =562.03 / 0.9 =624.5 KN.m = Ton.m

$m = 16.47$ Rn = 76.92 Kg/cm^2 $\rho = 0.022$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.022 (120) (26) =70.1 cm^2

Use 21Φ 20 mm & 4 Φ 14 mm As =72.26 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(120)a =30.6 a

Actual T = (72.26)(4.2) =303.49 Ton

Actual a = 303.49 / 30.6 = 9.9 cm

Actual x = a / β = 9.9 / 0.85 = 11.6 cm

$\varepsilon_s = 0.0037 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mu =64.1 Ton.m >62.4 Ton.m ∴ OK.

3-Design For Shear :

$V_u = 48.9$ Ton

V_u @ critical point = 45.6 Ton

$\Phi V_c = 24.2$ Ton

min $\Phi V_s = 8.84$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 48.4$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 45.6 - 24.2 = 21.1$ Ton

Use 2Φ 10 for stirrups

$S = \dfrac{0.85 A_v f_y d}{\Phi V_s}$ =13.6 cm

Use 2Φ 10 (4 leg) , S =13 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_b =12(2)=24 cm

= d = 26 cm .... control

123.3 cm ≥ 76.68 cm ∴ available development > Req.

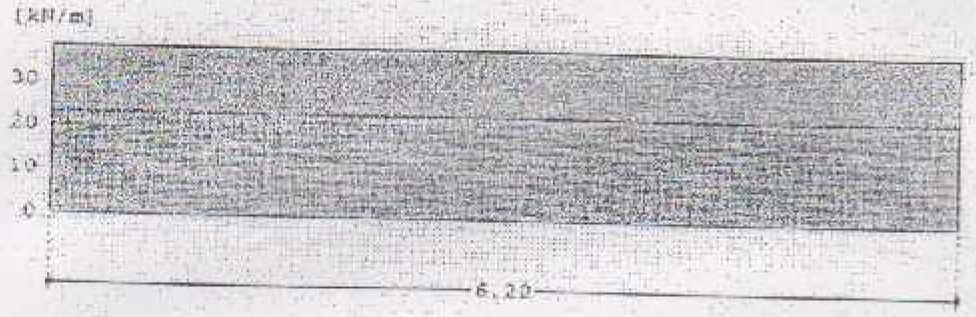
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 cm

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (18) :

System



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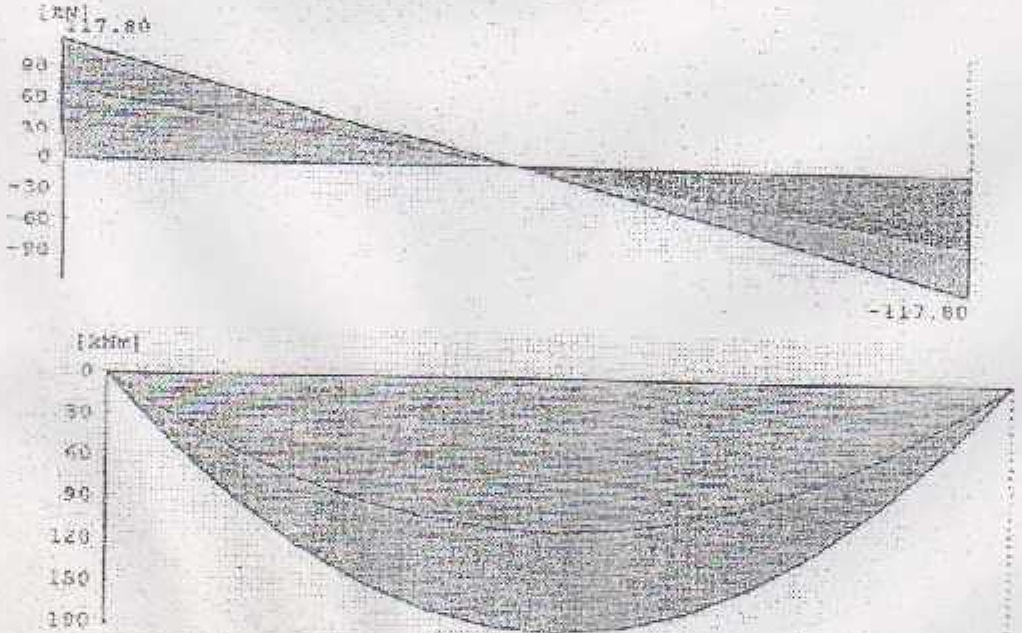


Shear & Moment Envelope Diagram

Fig. (6-35)

B =40 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =182.59 KN.m As shown in Fig. (6-35)

Mn =182.59 / 0.9 =202.8 KN.m = 20.3 Ton.m

As max. =0.023 (40) (26) =23.92 cm^2

X_b = 15.29 cm X max. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(40) =99.45 Ton

Mn max. = 99.45(26 – 0.5(9.75)) /100 = 21 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =75.1 Kg/cm^2 ρ=0.021 $\rho_{min.} < \rho < \rho_{max.}$

As =0.021 (40) (26) =22.65 cm^2

Use 6 Φ 20 mm As =22.81 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(40)a =10.2 a

Actual T = (22.81)(4.2) =95.8 Ton

Actual a =9.4 cm

Actual x = a / β = 9.4 / 0.85 = 11.1 cm

ε_s = 0.00402 > ε_y =0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =20.4 Ton.m >20.3 Ton.m ∴ OK.

2-Design For Shear :

V_u = 11.7 Ton

V_u @ critical point = 10.9 Ton

ΦV_c =8.1 Ton

min ΦV_s =3 Ton

complies with category (3)

Req. $\Phi V_s = V_u - \Phi V_c$ = 10.9-8.1 = 2.8 Ton

Use Φ 8 for stirrups

S =31.8 cm

Use Φ 8 (2 leg) , S =30 cm

3-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

174.33 cm ≥ 76.68 cm ∴ available development > Req.

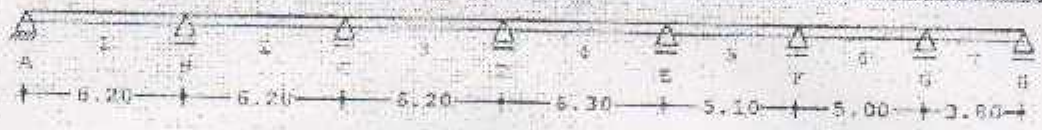
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.2– 2(0.15)) = 1.77 m

The total length of the top bars = 2 (1.77) + 0.6 = 4.14 m

Use L = 4.2 m

Beam (19)



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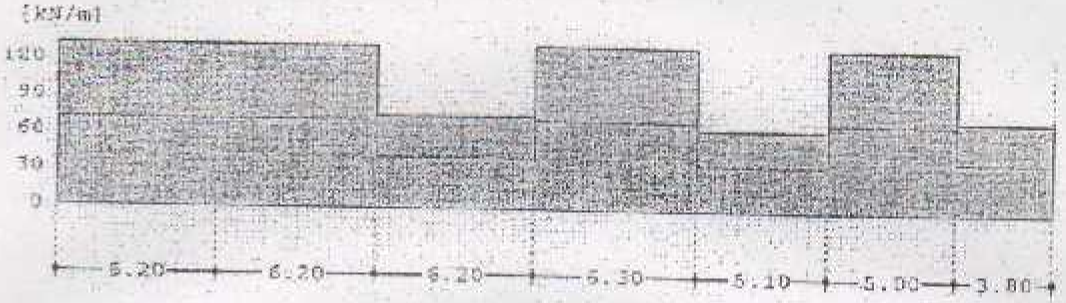

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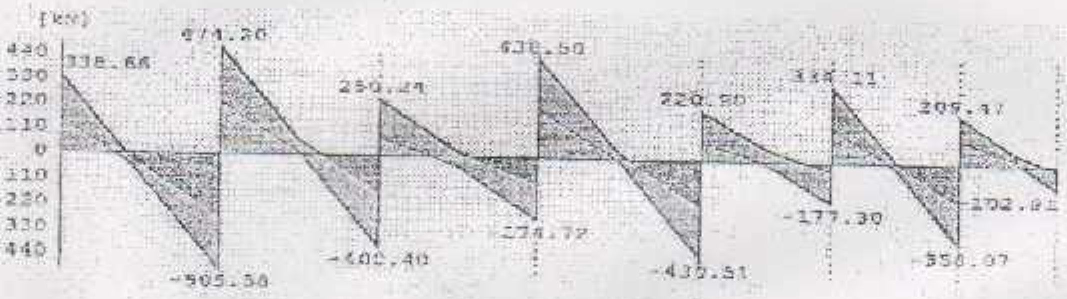
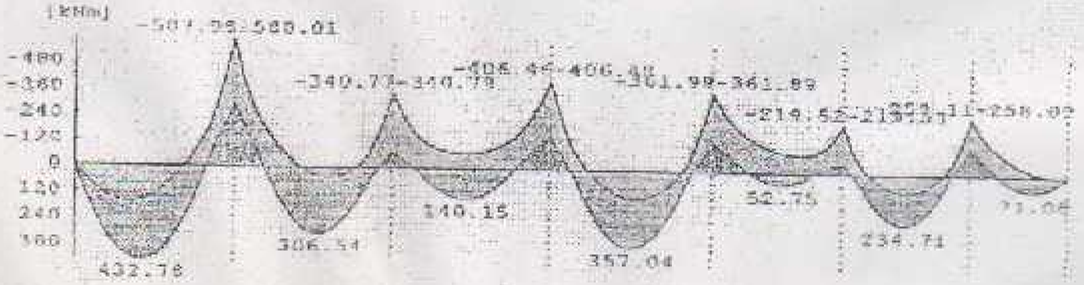




Shear & Moment Envelope Diagram

Fig. (6-36)

B =130 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =432.9 KN.m As shown in Fig. (6-36)

Mn =432.9 / 0.9 =480.9 KN.m =48.1 Ton.m

As max. =0.023 (130) (26) =50.18 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = β X_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(130) =324.5 Ton

Mn max. = 324.5(26 – 0.5(9.75)) /100 = 68.56 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =54.73 Kg/cm^2 $\rho = .0.0148$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.0148 (130) (26) =50.18 cm^2

Use 16Φ 20 mm As =50.27 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(130)a =33.15 a

Actual T = (50.27)(4.2) = 211.134 Ton

Actual a =6.4 cm

Actual x = a / β =6.4 / 0.85 = 7.5 cm

ε_s = 0.0074 > ε_y = .0021

∴ Tension steel is yielding ∴ OK

Actual Mn =48.13Ton.m >48.1 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =588KN.m As shown in Fig. (6-36)

Mn =588 / 0.9 =653.3 KN.m =65.3 Ton.m

$m = 16.47$ Rn = 74.3 Kg / cm^2 $\rho = 0.021$ $\rho_{min.} < \rho < \rho_{max.}$

A s =0.021 (130) (26) =72.65 cm^2

Use 22Φ 20 mm & 4 Φ 14 mm A s =75.41 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(130)a =33.15a
Actual T = (75.4)(4.2) =316.722 Ton

Actual a = 316.72/ 33.15 = 9.55 cm

Actual x = a / β = 9.55 / 0.85 = 11.24 cm

$\varepsilon_s = 0.0039 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =67.2 Ton.m >65.3 Ton.m ∴ OK.

3-Design For Shear :

$V_u = 50.5$ Ton

V_u @ critical point = 47.2 Ton

$\Phi V_c = 20.2$ Ton

min $\Phi V_s = 7.4$Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =40.34 Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 47.2 - 20.2 = 27$ Ton

Use 2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 10.78 \text{ cm}$$

Use 2Φ 10 (4 leg) , S = 10 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

$L_d = 76.68$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

122.37 cm ≥ 76.68 cm ∴ available development > Req.

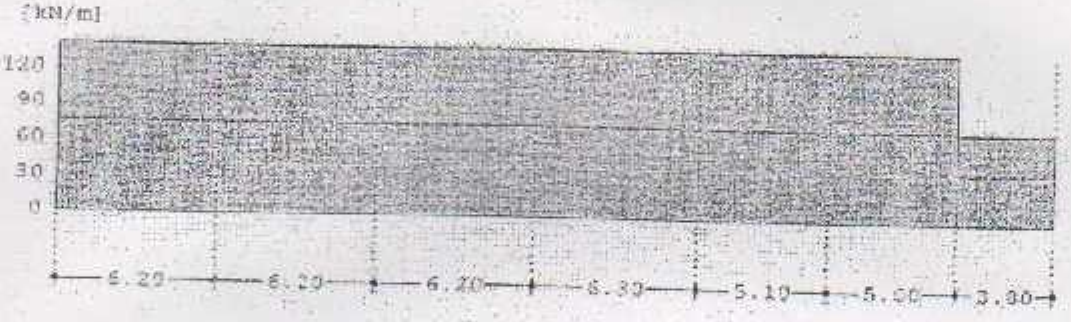
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (20, 30)

System



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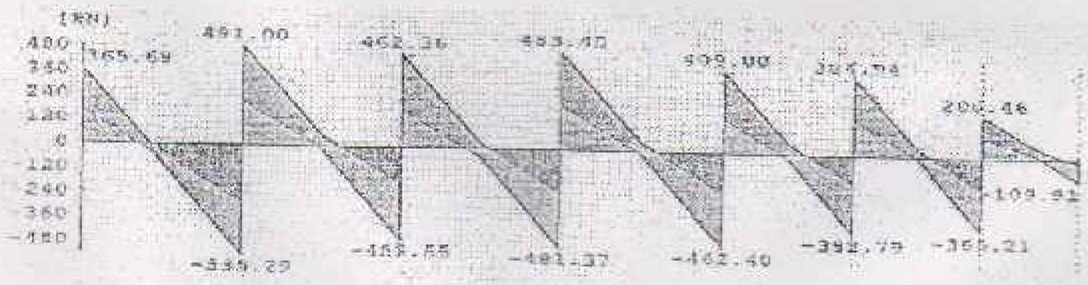
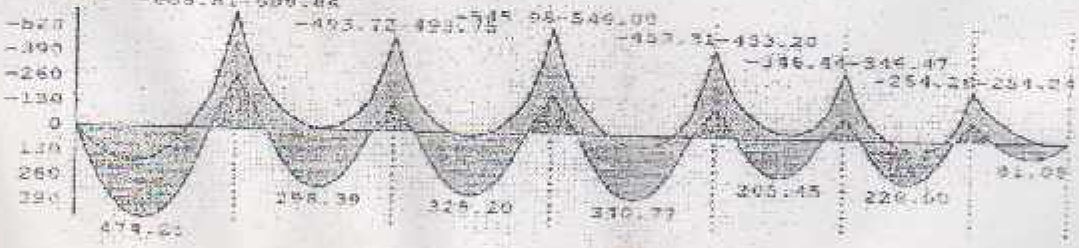




Shear & Moment Envelope Diagram

Fig. (6-37)

B =130 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =474.21 KN.m As shown in Fig. (6-37)

Mn =474.21 / 0.9 =526.9 KN.m = 52.7 Ton.m

As max.=0.023 (130) (26) =77.74 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max.= β X_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(130) =323.21 Ton

Mn max.= 323.21(26 – 0.5(9.75)) /100 = 68.3 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn =59.9 Kg / cm^2 ρ=0.0165 $\rho_{min.} < \rho < \rho_{max}$

As =0.0165 (130) (26) =55.8 cm^2

Use 18Φ 20 mm As =56.54 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(130)a =33.15 a

Actual T = (56.55)(4.2) =237.51 Ton

Actual a = 7.2 cm

Actual x = a / β = 7.2 / 0.85 = 8.843 cm

ε_s = 0.00462 > ε_y =0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =53.2 Ton.m > 52.7 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =608.8 KN.m As shown in Fig. (6-37)

Mn =608.8 / 0.9 =676.4 KN.m =67.64 Ton.m

$m = 16.47$ Rn =77.1 Kg/cm^2 ρ =0.022 $\rho_{min.} < \rho < \rho_{max.}$

A_s =0.022 (130) (26) = 76.17cm^2

Use 23Φ 20 mm & 4 Φ 14 mm A_s =78.55 cm^2

Check : tension steel is yield ?
C =0.85(0.3)(130)a =33.3 a

Actual T = (78.5)(4.2) =329.6 Ton

Actual a = 329.6 / 33.3 = 9.8 cm

Actual x = a / β = 9.8 / 0.85 = 11.64 cm

ε_s =0.0037 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =69.3 Ton.m > 67.4 Ton.m ∴OK.

3-Design For Shear :

V_u = 53.5 Ton

V_u @ critical point = 50 Ton

ΦV_c =24.2 Ton

min ΦV_s =8.84 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd$ =48.4 Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ =50-24.2 =25.8 Ton

Use2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 11.3 \text{ cm}$$

Use2Φ 10 (4 leg) , S =11 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d =76.68 cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

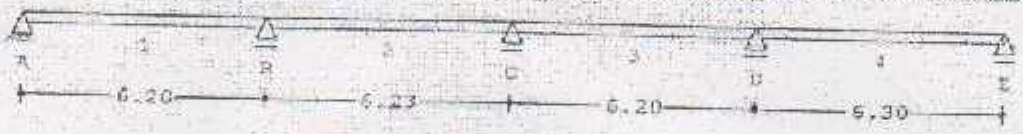
124.9 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (21,31)



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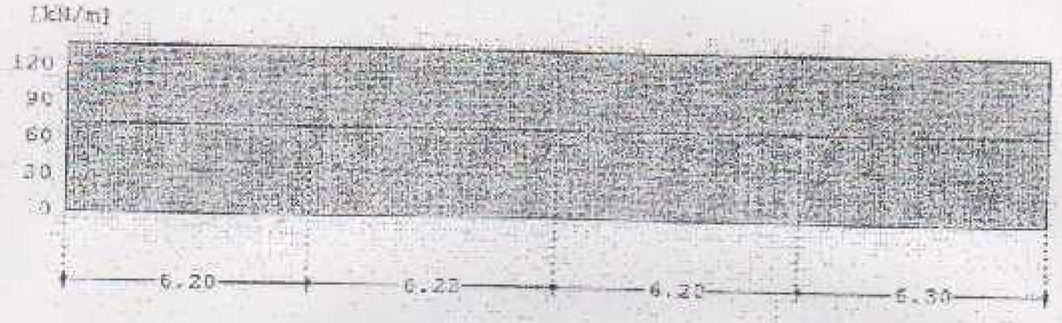

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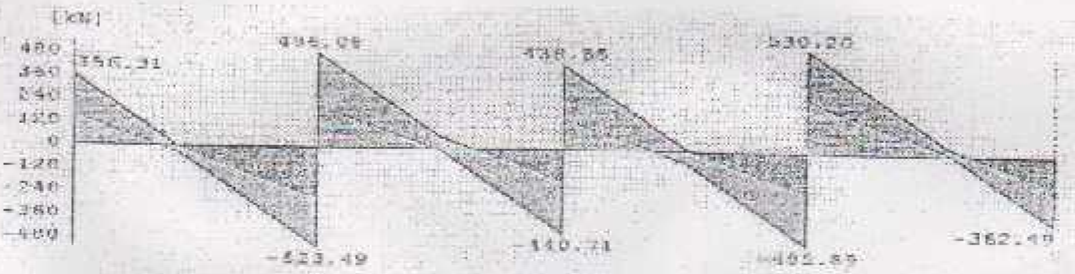
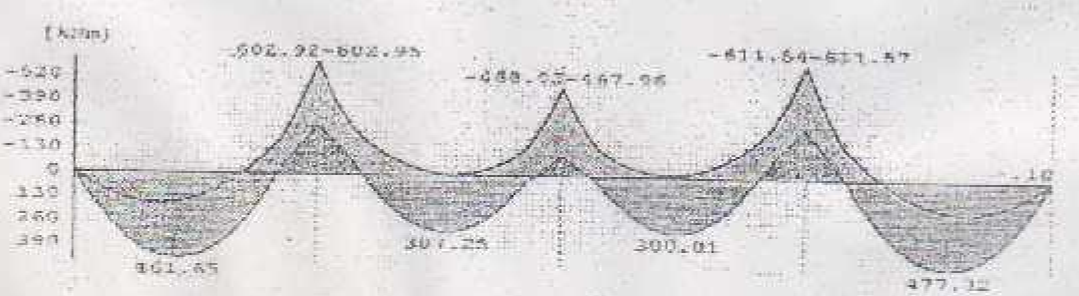




Shear & Moment Envelope Diagram

Fig. (6-38)

B = 40 cm H = 70 cm d = 66 cm

1 - Design For Positive Moment :

Mu = 477.72 KN.m As shown in Fig. (6-38)

Mn = 477.72 / 0.9 = 530.8 KN.m = 53.08 Ton.m

As max. = 0.023 (40) (66) = 60.72 cm^2

X_b = 38.82 cm Xmax. = 0.75 X_b = 29.11 cm

a max. = β X_b = 0.85 (29.11) = 24.75 cm

T max. = As fy = (60.72)(4.2) = 255.02 Ton

Mn max. = 255.02 (66 - 0.5(24.75)) /100 = 136.75 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 30.46 Kg / cm^2 $\rho = 0.00775$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.00775 (40) (66) = 20.45 cm^2

Use 7 Φ 20 mm As = 21.99 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (21.99)(4.2) = 92.36 Ton

Actual a = 9.05 cm

Actual x = a / β = 9.05 / 0.85 = 10.65 cm

$\varepsilon_s = 0.01557 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 56.78Ton.m

2 - Design For Negative Moment :

Mu = 611.57 KN.m As shown in Fig. (6-38)

Mn = 611.57 / 0.9 = 679.5 KN.m = 67.95 Ton.m

$m = 16.47$ Rn = 45.65 Kg/cm^2 $\rho = 0.01207$ $\rho_{min} < \rho < \rho_{max}$

$As = 0.01207\,(40)\,(61) = 29.45\ cm^2$

Use 8 Φ 20 mm & 4 Φ 14 mm (in two layers) $As = 31.42\ cm^2$

Check : tension steel is yield ?
C =0.85(0.3)(40)a = 10.2 a

Actual T = (31.42)(4.2) = 131.96 Ton

Actual a = 131.96 / 10.2 = 12.94 cm

Actual x = a / β = 12.94 / 0.85 = 15.22 cm

$\varepsilon_s = 0.00902 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 71.96 Ton.m > 67.95 Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 53.02$ Ton

V_u @ critical point = 49.58 Ton

$\Phi V_c = 20.48$ Ton

min $\Phi V_s = 7.48$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 40.97$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 53.02 - 20.48 = 32.54$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 22.74 \text{ cm}$$

Use Φ 10 (4 leg) , S = 20 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b =12(2)=24 cm

= d = 66 cm ……………… control

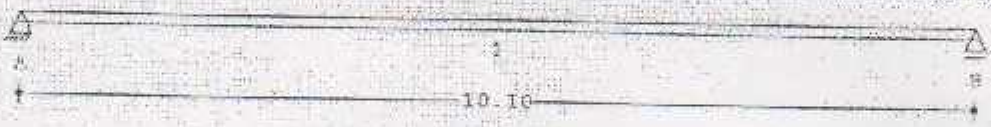
166.11 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (22,32)



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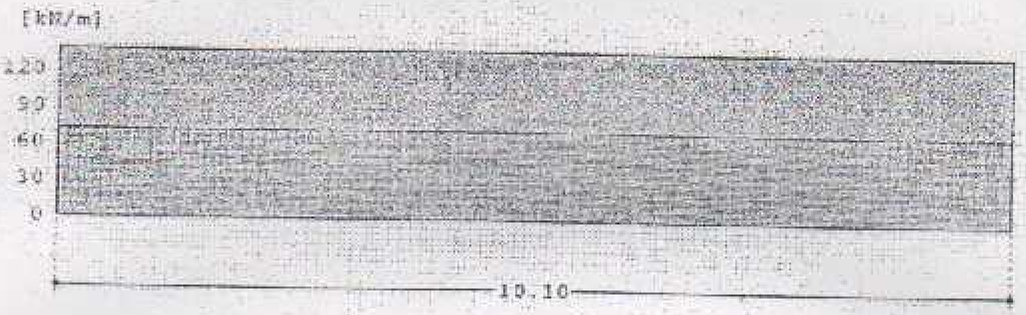

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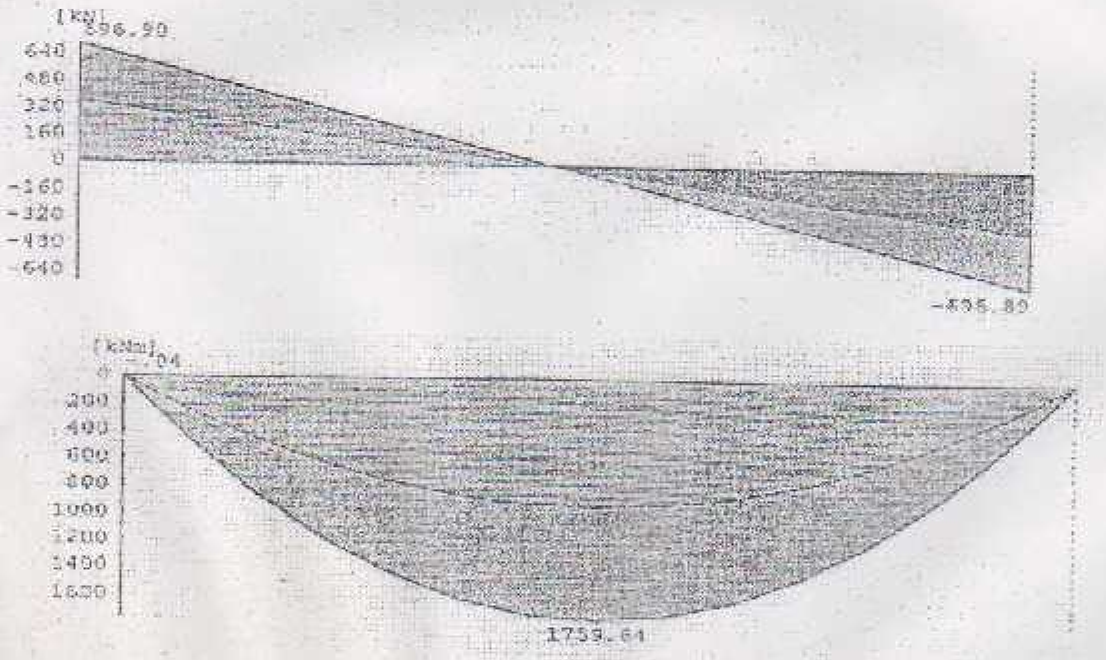


Shear & Moment Envelope Diagram

Fig. (6-39)

B = 50 cm H = 80 cm d = 76 cm

1 - Design For Positive Moment :

Mu = KN.m As shown in Fig. (6-39)

Mn =1759.64 / 0.9 = 1955.15 KN.m = 195.52 Ton.m

As max. =0.023 (50) (76) = 87.4 cm^2

X_b = 44.71 cm Xmax. = 0.75 X_b = 33.53 cm

a max. = β X_b = 0.85 (33.53) = 28.5 cm

Tmax. = As fy = (87.4)(4.2) = 367.08 Ton

Mn max. = 367.08 (76 – 0.5(28.5)) /100 = 226.67 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 67.7 Kg/cm^2 $\rho = 0.01913$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.01913 (50) (76) = 72.71 cm^2

Use 21 Φ 20 mm in three layers As = 79.83 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50) a = 12.75 a

Actual T = (79.83)(4.2) = 335.27 Ton

Actual a = 26.3 cm

Actual x = a / β = 26.3 / 0.85 = 30.94 cm

ε_s = 0.00398 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn = 197.31 Ton.m

2 - Design For Shear :

$V_u = 69.69$ Ton

V_u @ critical point = 66.24 Ton

$\Phi V_c = 29.49$ Ton

min $\Phi V_s = 10.77$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 58.97$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 66.24 - 29.49 = 36.75$ Ton

Use Φ 10 for stirrups

$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 23.18$ cm

Use Φ 10 (4 leg) , S = 23 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

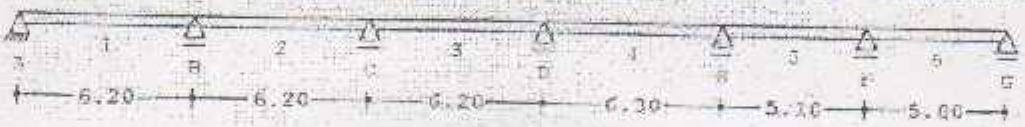
$L_d = 76.68$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

$= d = 72$ cm .... control

373.87 cm ≥ 76.68 cm ∴ available development > Req.

Beam (23,33)



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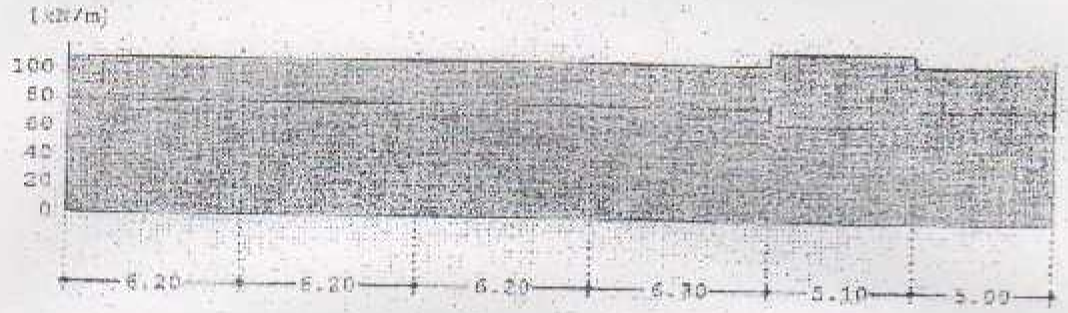

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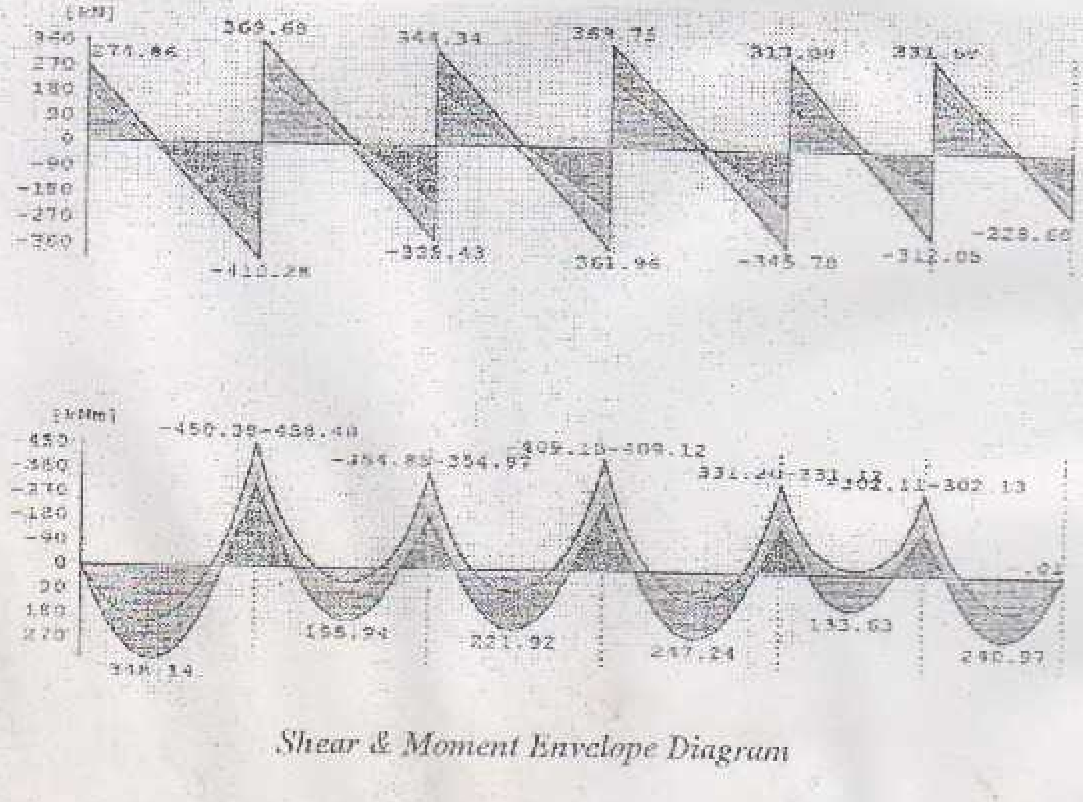


Shear & Moment Envelope Diagram

Fig. (6-40)

B =100 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu =348.14 KN.m As shown in Fig. (6-40)

Mn =348.14 / 0.9 =386.8 KN.m =38.7 Ton.m

As max. =0.023 (100) (26) =59.8 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C =0.85(0.3)(100)(9.75) = 248.625 Ton

Mn max. = 248.625(26 - 0.5(9.75)) /100 = 52.5 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =57.24 Kg/cm^2 ρ =0.0156 $\rho_{min.} < \rho < \rho_{max.}$

As =0.0156 (100) (26) =40.68 cm^2

Use 13Φ 20 mm As =40.84 cm^2

Check : tension steel is yield ?

C =0.85(0.3)()a = a

Actual T = (40.84)(4.2) =171.53 Ton

Actual a =6.73 cm

Actual x = a / β = 6.73 / 0.85 = 7.9 cm

ε_s = 0.00687 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =38.8 Ton.m > 38.7 Ton ∴ OK.

2 - Design For Negative Moment :

Mu =458.38 KN.m As shown in Fig. (6-40)

Mn =458.38 / 0.9 =509.3 KN.m =50.93 Ton.m

$m = 16.47$ Rn = 75.34 Kg/cm^2 $\rho = 0.0218$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.0218\,(100)\,(26) = 56.9\ cm^2$

Use 17Φ 20 mm & 4 Φ 14 mm $As = 59.69\ cm^2$

heck : tension steel is yield ?

C =0.85(0.3)(100)a =25.5 a

Actual T = (56.9)(4.2) =250.7 Ton

Actual a = 250.7/ 25.5 = 9.83 cm

Actual x = a / β = 9.83 / 0.85 = 11.56 cm

$\varepsilon_s = 0.00374 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =52.86 Ton.m > 50.93 Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 41.02$ Ton

V_u @ critical point = 38.3 Ton

$\Phi V_c = 20.17$ Ton

min $\Phi V_s = 7.4$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 40.34$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 38.3 - 20.17 = 18.13$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 16.1 \text{ cm}$$

Use Φ 10 (2 leg) , S = 16 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 78.68 cm

L_a = 12 d_b = 12(2) = 24 cm

= d = 26 cm ……………… control

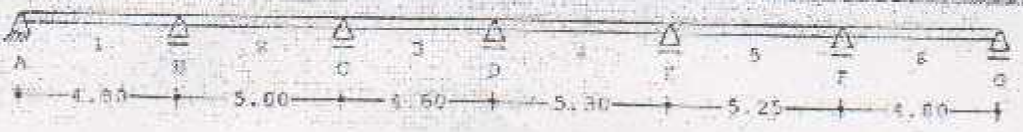
120.6 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(6.3 − 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m Use L = 4.2 m

Beam (24) :



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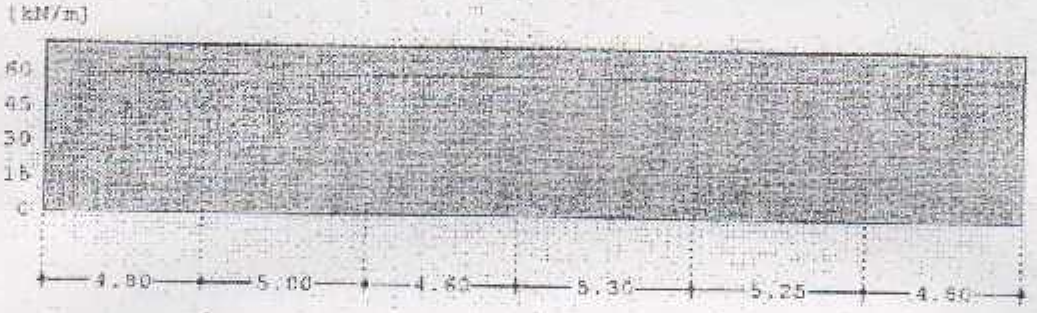

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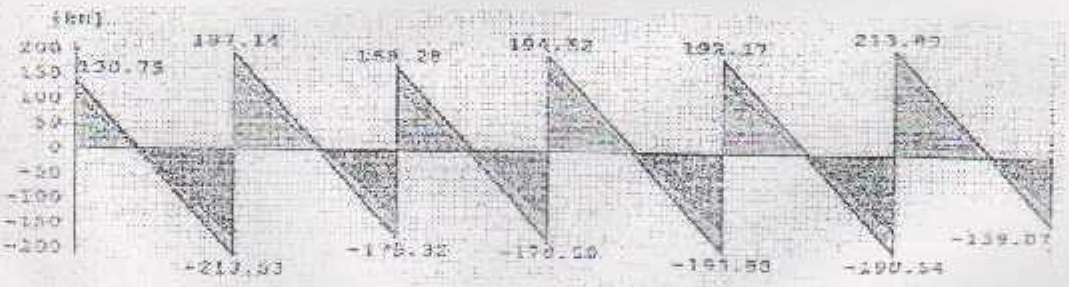
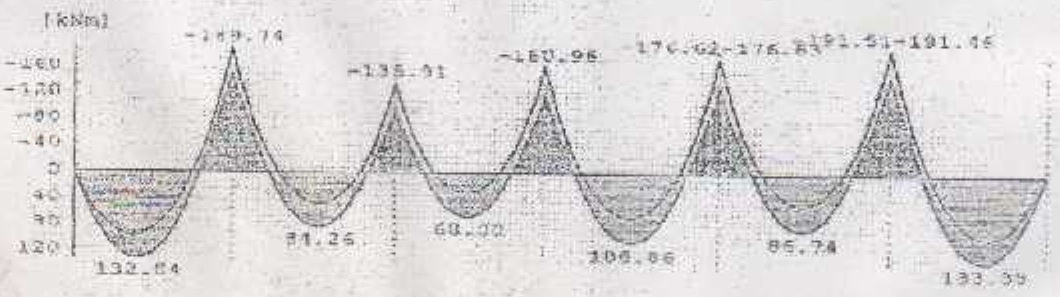




Shear & Moment Envelope Diagram

Fig. (6-41)

B = 50 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 133.38 KN.m As shown in Fig. (6-41)

Mn =133.38 / 0.9 = 148.2 KN.m = 14.82 Ton.m

As max.=0.023 (50) (26) =29.9 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

T max. = $As\ fy$ = (29.9)(4.2) = 125.58 Ton

Mn max. = 125.58(26 − 0.5(9.75)) /100 = 26.53 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 43.85 Kg/cm^2 $\rho = 0.01154$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.01154 (50) (26) = 15 cm^2

Use 5 Φ 20 mm As = 15.71 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(50)a = 12.5 a

Actual T = (15.71)(4.2) = 65.98 Ton

Actual a = 5.18 cm

Actual x = a / β = 5.18 / 0.85 = 6.09 cm

ε_s = 0.00981 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK

Actual Mn = 15.45 Ton.m

2 - Design For Negative Moment :

Mu = 191.51 KN.m As shown in Fig. (6-41)

Mn = 191.51 / 0.9 = 222.11 KN.m = 22.21 Ton.m

$m = 16.47$ Rn = 65.71 Kg/cm^2 $\rho = 0.01845$ $\rho_{min.} < \rho < \rho_{max.}$

A.s = 0.01845 (50) (26) = cm^2

Use 7 Φ 20 mm & 2 Φ 14 mm A.s = 25.13 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(50)a = 12.75 a

Actual T = (25.13)(4.2) = 105.55 Ton

Actual a = 105.55 / 12.75 = 8.28 cm

Actual x = a / β = 8.28 / 0.85 = 9.74 cm

$\varepsilon_s = 0.0501 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 23.07 Ton.m > 22.21Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 21.39$ Ton

V_u @ critical point = 18.38 Ton

$\Phi V_c = 10.09$ Ton

min $\Phi V_s = 3.68$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 20.17$Ton complies with category (4)

Req $\Phi V_s = V_u - \Phi V_c = 21.39 - 10.09 = 8.29$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 17.78 \text{ cm}$$

Use Φ 10 (2 leg) , S = 17 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b = 12(2) = 24 cm

= d = 26 cm .... control

98.23 cm ≥ 76.68 cm ∴ available development > Req.

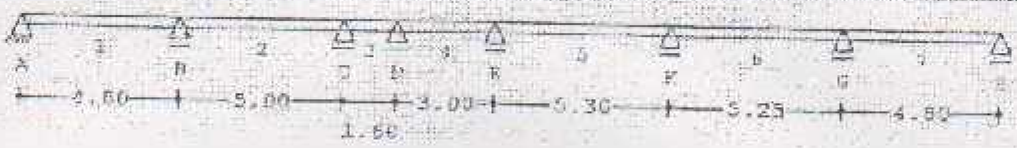
L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(5.3 – 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m

Use L = 4.2 m

Beam (25)



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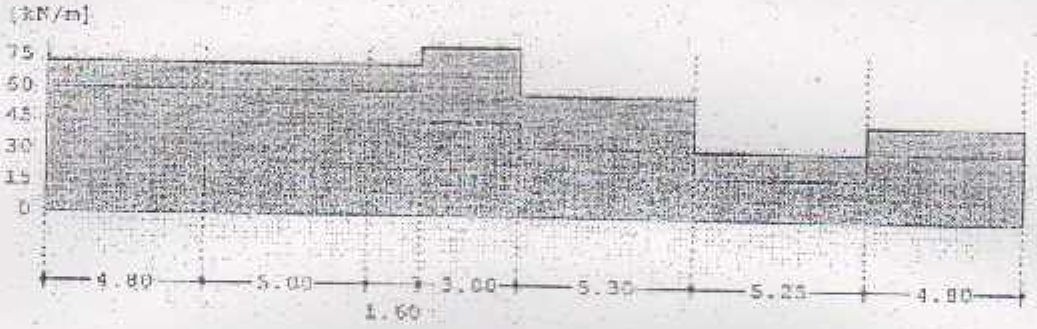

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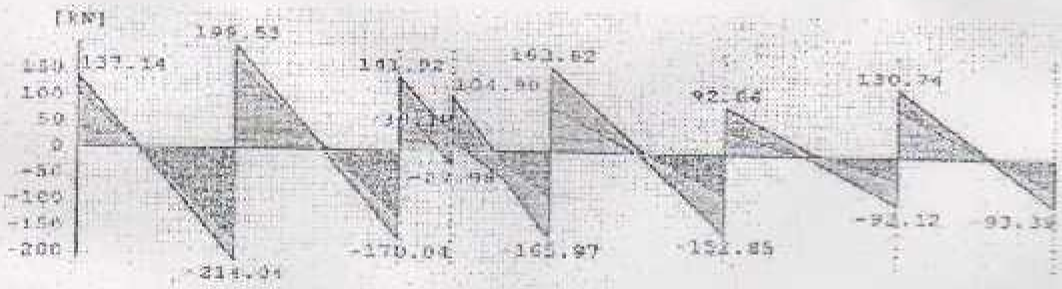
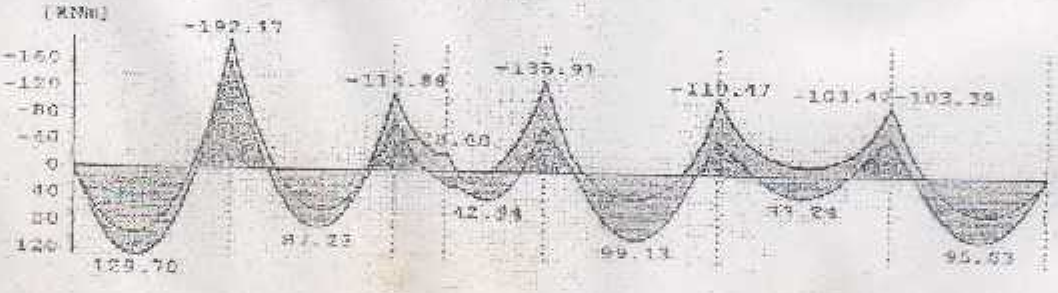




Shear & Moment Envelope Diagram

Fig. (6-42)

B = 50 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 129.7 KN.m As shown in Fig. (6-42)

Mn =129.7 / 0.9 = 144.11 KN.m = 14.41Ton.m

$As\ max. = 0.023\ (50)\ (26) = 29.9\ cm^2$

$X_b = 15.29$ cm $X max. = 0.75\ X_b = 11.47$ cm

$a max. = \beta X_b = 0.85\ (11.47) = 9.75$ cm

$T max. = As\ fy = (29.9)(4.2) = 125.58$ Ton

Mn max. = 125.58(26 – 0.5(9.75)) /100 = 26.53 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 42.64 Kg/cm^2 $\rho = 0.01118$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.01118\ (50)\ (26) =\ cm^2$

Use 5 Φ 20 mm $As = 15.71\ cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(50)a =12.75 a

Actual T = (15.71)(4.2) = 65.98 Ton

Actual a = 5.18 cm

Actual $x = a / \beta = 5.18 / 0.85 = 6.09$ cm

$\varepsilon_s = 0.00981 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 15.45 Ton.m

2 - Design For Negative Moment :

Mu = 192.17 KN.m   As shown in Fig. (6-42)

Mn = 192.17 / 0.9 = 213.5 KN.m = 21.35 Ton.m

$m = 16.47$ Rn = 63.17 Kg/cm^2 $\rho = 0.01759$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.01759\ (50)\ (26) = 22.86\ cm^2$

Use 7 Φ 20 mm & 2 Φ 14 mm $As = 25.13\ cm^2$

Check : tension steel is yield ?

C = 0.85(0.3)(50) a = 12.75 a

Actual T = (25.13)(4.2) = 105.55Ton

Actual a = 105.55 / 12.75 = 8.28 cm

Actual x = a / β = 9.3 / 0.85 = 10.9 cm

$\varepsilon_s = 0.00501 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 23.07 Ton.m > 21.35 Ton.m   ∴ OK

3 - Design For Shear :

$V_u = 21.4$ Ton

V_u @ critical point = 19.59 Ton

$\Phi V_c = 10.09$ Ton

min $\Phi V_s = 3.68$ Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 20.17$ Ton   complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 19.59 - 10.09 = 9.5$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 15.34 \text{ cm}$$

Use Φ 10 (2 leg) , S = 15 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

$L_d = 76.68$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

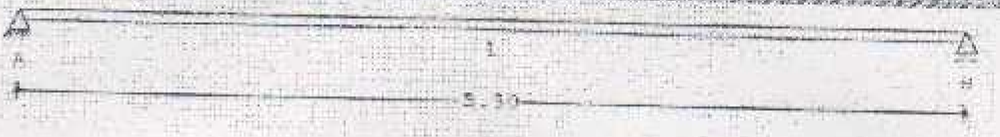
98.2 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(5.3 – 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m Use L = 3.6 m

Beam (26,36)



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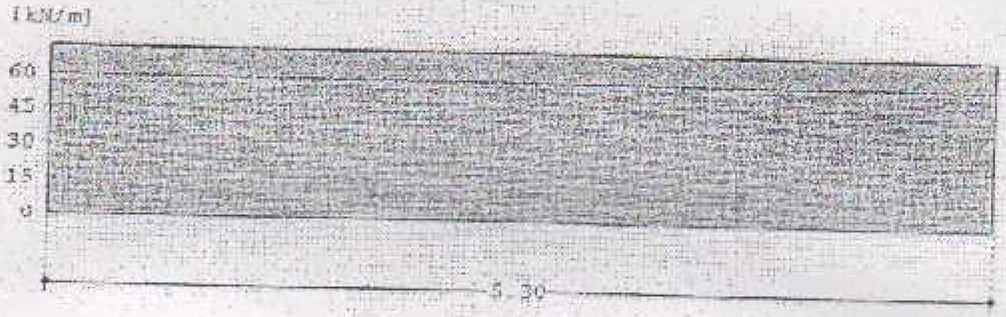

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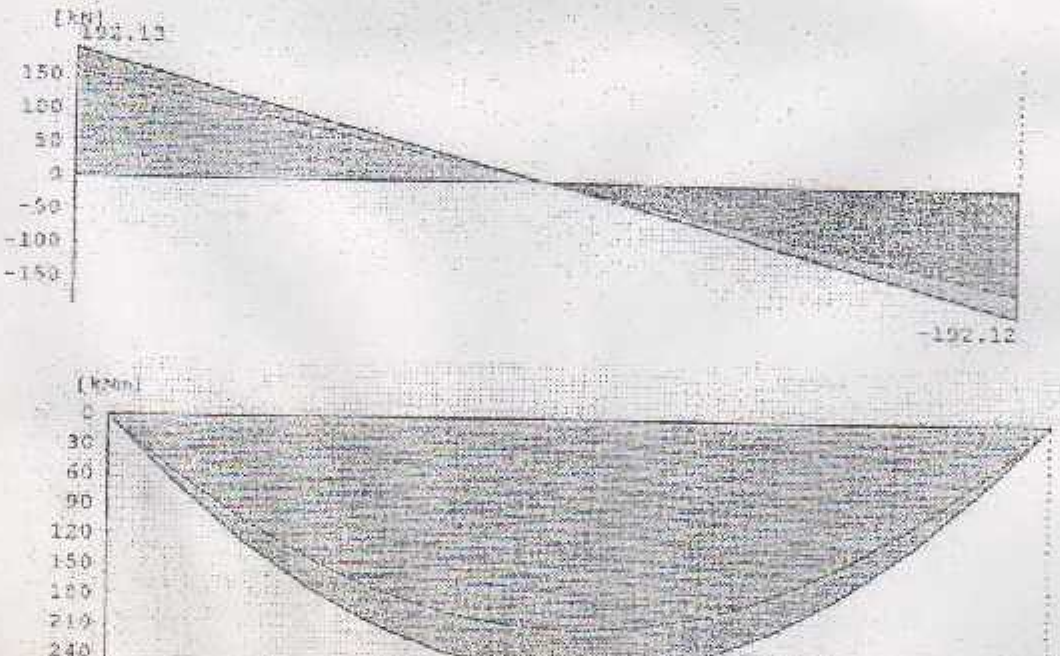


Shear & Moment Envelope Diagram

Fig. (6-43)

B = 60 cm H = 30 cm d = 26 cm

1 - Design For **Positive** Moment :

Mu = 254.57 KN.m As shown in Fig. (6-43)

Mn = 254.57 / 0.9 = 282.86 KN.m = 28.29 Ton.m

As max. = 0.023 (60) (26) = 35.88cm^2

X_b = 15.29 cm X max. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

T max. = $As\ fy$ = (35.88)(4.2) = 150.7 Ton

Mn max. = 150.7 (26 - 0.5(9.75)) /100 = 31.83 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 69.75 Kg/cm^2 $\rho = 0.01985$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01985(60) (26) = 30.97 cm^2

Use 10 Φ 20 mm As = 31.42 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(60) a = 15.3 a

Actual T = (31.42)(4.2) = 131.96Ton

Actual a = 8.63 cm

Actual x = a / β = 8.63 / 0.85 = 10.15 cm

$\varepsilon_s = 0.00468 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 28.61Ton.m

2 - Design For Shear :

V_u = 19.21 Ton

V_u @ critical point = 16.24 Ton

ΦV_c = 13.11 Ton

min ΦV_s = 4.79 Ton complies with category (3)

Use Φ 10 for stirrups

$S = \frac{3 A_v f_y}{b_w} = 30.43$ cm

Use Φ 10 (2 leg) , S = 25 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

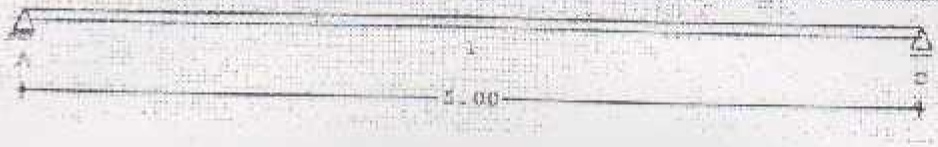
L_d = cm

L_c = 12 d_b =12(2) =24 cm

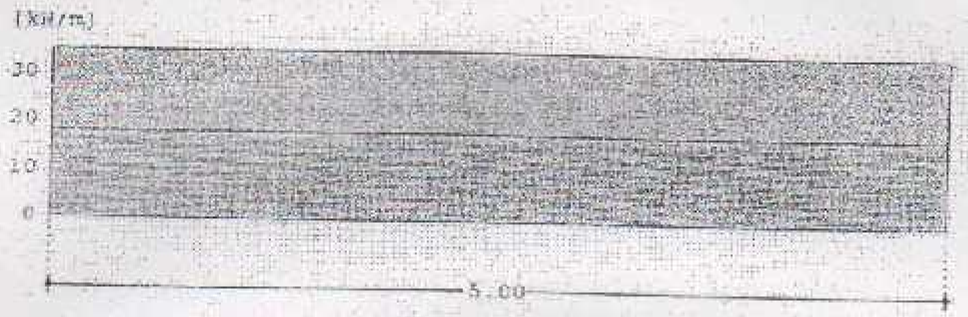
= d = 26 cm ……………… control

177.28 cm ≥ 76.68 cm ∴ available development > Req.

Beam (27)



System



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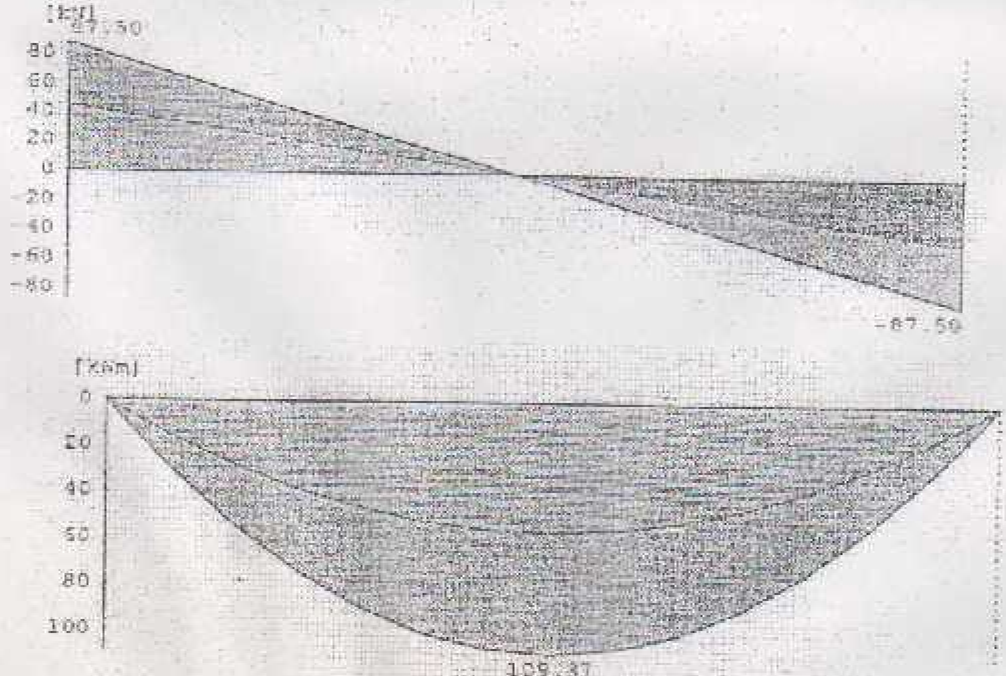


Shear & Moment Envelope Diagram

Fig. (6-44)

B = 40 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 109.37 KN.m As shown in Fig (6-44)

Mn = 109.37 / 0.9 = 121.52 KN.m = 12.15 Ton.m

As max .=0.023 (40) (26) = 23.92 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

Tmax. = As fy = (23.92)(4.2) = 100.46 Ton

Mn max. = 100.46(26 – 0.5(9.75)) /100 = 21.22 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

m = 16.47 Rn = 44.93 Kg/cm^2 ρ = 0.01186 $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01186 (40) (26) = 12.33 cm^2

Use 4 Φ 20 mm As = 12.57 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(40) a = 10.2 a

Actual T = (12.57)(4.2) = Ton

Actual a = 5.18 cm

Actual x = a / β = 5.18 / 0.85 = 6.09 cm

ε_s = 0.00981 > ε_y = 0.0021

∴ Tension steel is yielding ∴ OK.

Actual Mn = 12.36 Ton.m

2 - Design For Shear :

$V_u = 8.75$ Ton

V_u @ critical point = 7.32 Ton

$\Phi V_c = 8.07$ Ton complies with category (4)

Use Φ 10 for stirrups

$$S = \frac{3A_v f_y}{b_w} = 49.45 \text{ cm}$$

Use Φ 10 (2 leg) , S = 40 cm

3 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

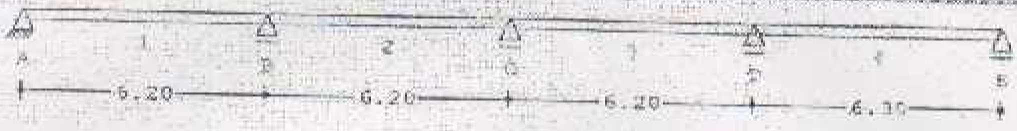
$L_d = 76.68$ cm

$L_a = 12\ d_b = 12(2) = 24$ cm

$= d = 26$ cm .... control

167.26 cm $\geq$ 76.68 cm ∴ available development > Req.

Beam (28)



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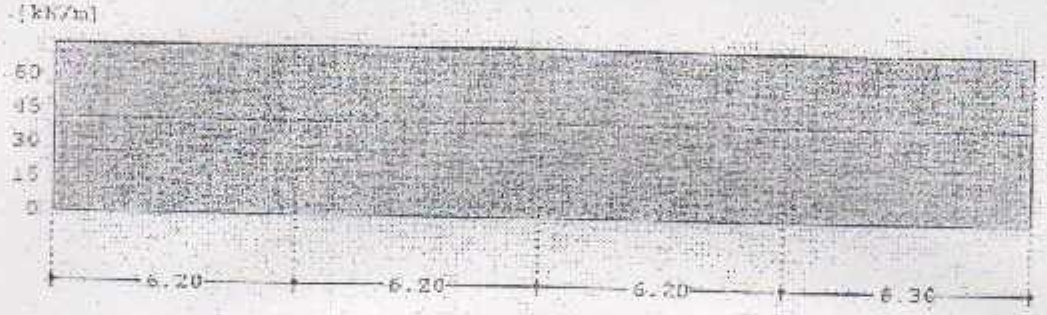
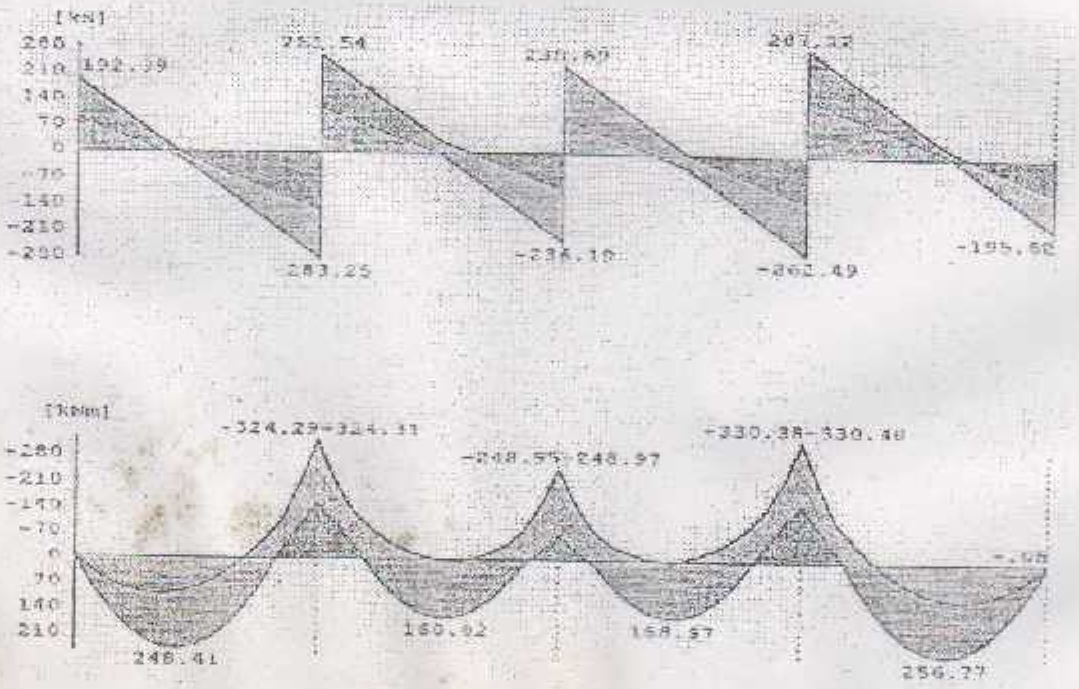


Loads



Shear & Moment Envelope Diagram

Fig. (6-45)

B =80 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =256.77 KN.m As shown in Fig. (6-45)

Mn =265.77 / 0.9 =285.3 KN.m =28.53 Ton.m

As max. =0.023 (80) (26) =54.6 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(80) =198.9 Ton

Mn max. = 198.9(26 – 0.5(9.75)) /100 = 42 Ton.m

Mn max. > Req Mn ∴ singly reinforcement

$m = 16.47$ Rn =52.8 Kg / cm^2 $\rho = 0.0142$ $\rho_{min.} < \rho < \rho_{max.}$

As =0.0142 (80) (26) =29.6 cm^2

Use 10Φ 20 mm As =31.42 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (31.42)(4.2) =131.96 Ton

Actual a =6.46 cm

Actual x = a / β = 6.46 / 0.85 = 7.6 cm

$\varepsilon_s = 0.00726 > \varepsilon_y = .0021$

∴ Tension steel is yielding ∴ OK

Actual Mn =30.1 Ton.m > 28.53 Ton.m ∴ OK.

2-Design For Negative Moment :

Mu =330.4 KN.m As shown in Fig. (6-45)

Mn =330.4 / 0.9 =367.11 KN.m =36.7 Ton.m

$m = 16.47$ Rn =67.86 Kg/cm^2 $\rho = 0.0192$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.0192\,(80)\,(26) = 39.9\ cm^2$

Use 12 Φ 20 mm & 2 Φ 14 mm $As = 40.84\ cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (40.84)(4.2) =171.52 Ton

Actual a = 171.52 / 20.4 = 8.4 cm

Actual x = a / β = 8.4 / 0.85 = 9.89 cm

$\varepsilon_t = 0.0048 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn =37.39 Ton.m >36.7 Ton.m ∴ OK.

3-Design For Shear :

$V_u = 28.7$ Ton

V_u @ critical point = 26.8 Ton

$\Phi V_c = 16.14$ Ton

min $\Phi V_s = 5.89$ Ton

$0.85\left(\dfrac{\sqrt{fc'}}{3}\right)bd = 32.3$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 26.8 - 16.14 = 10.66$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 13.67 \text{ cm}$$

Use Φ 10 (2 leg) , S = 13 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

$L_a = 12\ d_b = 12(2) = 24$ cm

= d = 26 cm .... control

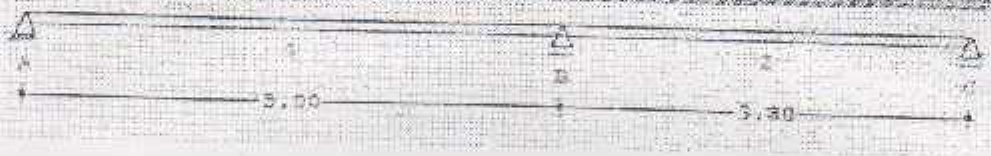
130.8 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

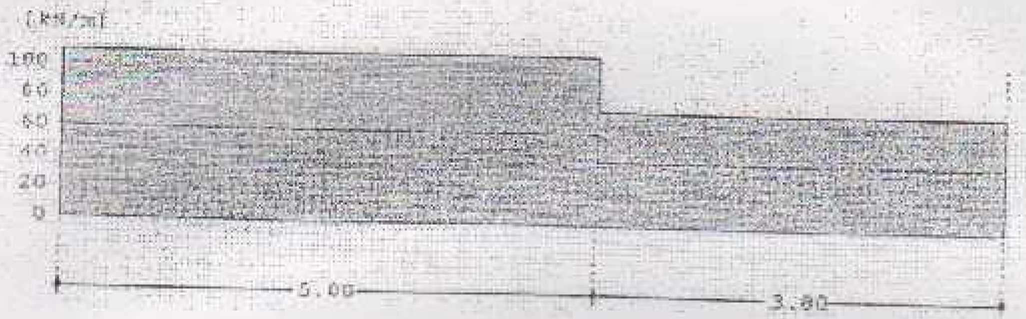
0.3 Ln from face of support = 0.3(6.3 – 2(0.15)) = 1.8 m

The total length of the top bars = 2 (1.8) + 0.6 = 4.2 m <u>Use L = 4.2 m</u>

Beam (29) :



System



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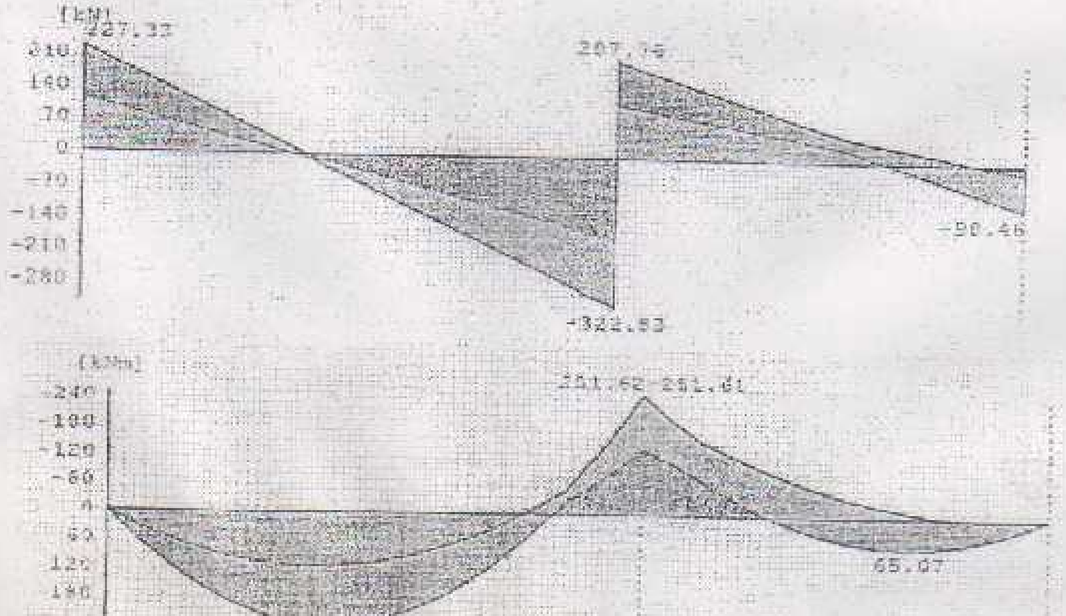


Shear & Moment Envelope Diagram

Fig. (6-46)

B =80 cm H = 30 cm d = 26 cm

1-Design For Positive Moment :

Mu =237.05 KN.m As shown in Fig. (6-46)

Mn =237.05/ 0.9 =263.39 KN.m =26.34 Ton.m

As max. =0.023 (80) (26) =47.84 cm^2

X_b = 15.29 cm Xmax. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

C=0.85(0.3)(9.75)(80) =198.90 Ton

Mn max. = 198.90 (26 – 0.5(9.75)) /100 = 42.02 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn =48.71 Kg/cm^2 ρ =0.01299 $\rho_{min.} < \rho < \rho_{max.}$

As =0.01299 (80) (26) = 27.01 cm^2

Use 9 Φ 20 mm As =28.27 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (28.27)(4.2) =118.73 Ton

Actual a =5.82 cm

Actual x = a / β_1 = 5.82 / 0.85 = 6.85 cm

ε_s =0.00839 > ε_y = .0021

∴ Tension steel is yielding ∴ OK

Actual Mn =27.41 Ton.m >26.34 Ton.m ∴ OK

2-Design For Negative Moment :

Mu =251.62 KN.m As shown in Fig. (6-46)

Mn =251.62 / 0.9 =279.58 KN.m =27.96 Ton.m

$m = 16.47$ Rn = 51.70 Kg/cm^2 ρ=0.0139 $\rho_{min} < \rho < \rho_{max}$

As =0.0139 (80) (26) =28.91 cm^2

Use 8 Φ 20 mm & 4 Φ 14 mm As =31.42 cm^2

Check : tension steel is yield ?

C =0.85(0.3)(80)a =20.4 a

Actual T = (31.42)(4.2) =131.96 Ton
Actual a =131.96 /20.4 =6.47 cm

Actual $x = a/\beta_1 =$ 6.47 / 0.85 =7.61 cm

ε_s =0.00725 > ε_y = .0021

∴ Tension steel is yielding ∴ OK.

Actual Mn =30.04 Ton.m >27.96 Ton.m ∴ OK

3-Design For Shear :

V_u = 32.28 Ton

V_u @ critical point = 29.56 Ton

ΦV_c =16.14 Ton

min ΦV_s =7.4 Ton

$$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 32.28 \text{ Ton}$$ complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ =29.56 – 16.14 = 13.42 Ton

Use 2Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 21.71 \text{ cm}$$

Use 2Φ 10 (4 leg) , S = 20 cm

4-Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68 cm

L_a = 12 d_b = 12(2) = 24 cm

= d = 26 cm ………………… control

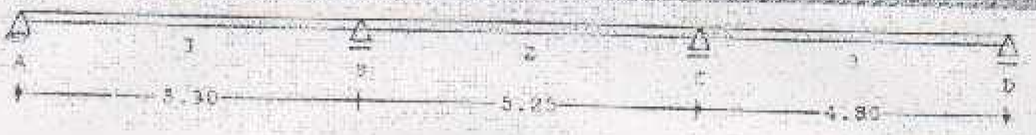
110.91 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

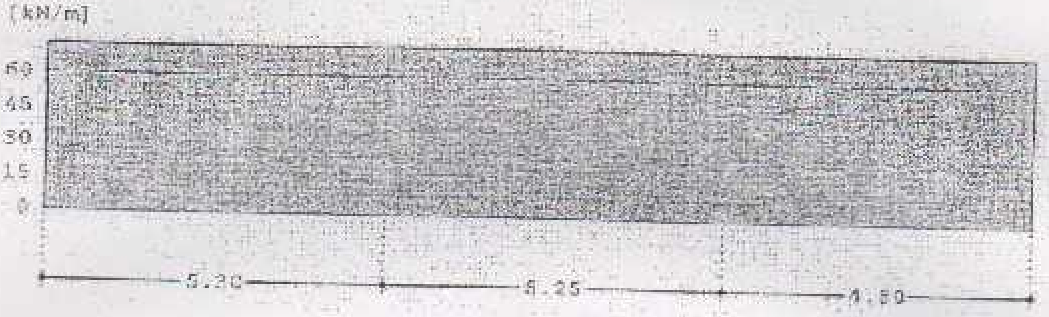
0.3 Ln from face of support = 0.3(5− 2(0.15)) = 1.41 m

The total length of the top bars = 2 (1.41) + 0.6 = 3.42 m Use L = 3.5 m

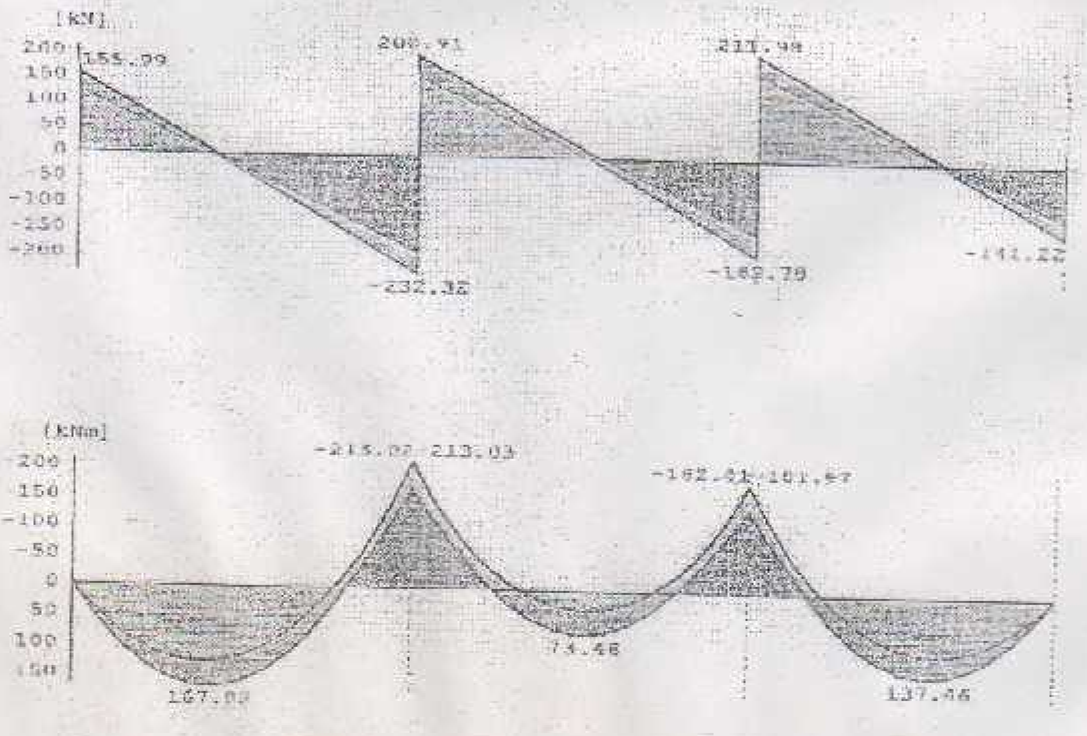
Beam (34) :



System



Loads



Shear & Moment Envelope Diagram

Fig. (6-47)

B = 60 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 167.82 KN.m As shown in Fig. (6-47)

Mn = 167.82 / 0.9 = 186.47 KN.m = 18.65 Ton.m

As max. = 0.023 (60) (26) = 35.88 cm^2

X_b = 15.29 cm X max. = 0.75 X_b = 11.47 cm

a max. = βX_b = 0.85 (11.47) = 9.75 cm

T max. = $As\ fy$ = (35.88)(4.2) = 150.7 Ton

Mn max. = 150.7(26 – 0.5(9.75)) /100 = 31.83 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ Rn = 45.98 Kg/cm^2 $\rho = 0.01217$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.01217 (60) (26) = 18.98 cm^2

Use 7 Φ 20 mm As = 21.99 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(60)a = 15.3 a

Actual T = (21.99)(4.2) = 92.36 Ton

Actual a = 6.04 cm

Actual x = a / β = 6.04 / 0.85 = 7.1 cm

$\varepsilon_s = 0.00799 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 21.22 Ton.m

2 - Design For Negative Moment :

Mu = 213.03 KN.m   As shown in Fig. (6-47)

Mn = 213.03 / 0.9 = 236.7 KN.m = 23.67 Ton.m

$m = 16.47$ Rn = 58.36 Kg/cm^2 $\rho = 0.016$ $\rho_{min.} < \rho < \rho_{max.}$

As = 0.016(60) (26) = 24.97cm^2

Use 7 Φ 20 mm & 2 Φ 14 mm As = 25.13 cm^2

Check : tension steel is yield ?

C = 0.85(0.3)(60)a = 15.3 a

Actual T = (25.13)(4.2) = 105.55Ton

Actual a = 105.55 / 15.3 = 6.9 cm

Actual x = a / β = 6.9 / 0.85 = 8.12 cm

$\varepsilon_s = 0.00661 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 23.8 Ton.m > 23.67 Ton.m   ∴ OK.

3 - Design For Shear :

$V_u = 23.23$ Ton

V_u @ critical point = 18.76 Ton

ΦV_c = 12.1Ton

min ΦV_s = 4.42 Ton

$0.85\left(\frac{\sqrt{fc'}}{3}\right)bd = 24.21$ Ton   complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c$ = 18.76 − 12.1 = 6.66 Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 21.88 \text{ cm}$$

Use Φ 10 (2 leg) , S = 20 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

L_d = 76.68cm

L_a = 12 d_b =12(2) =24 cm

= d = 26 cm .... control

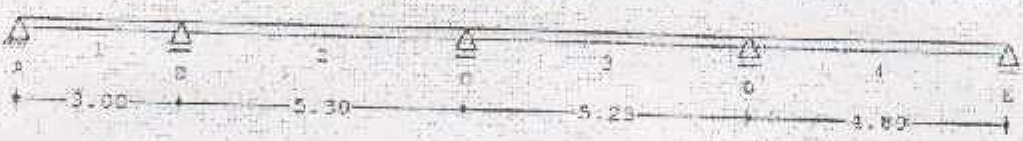
117.15 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

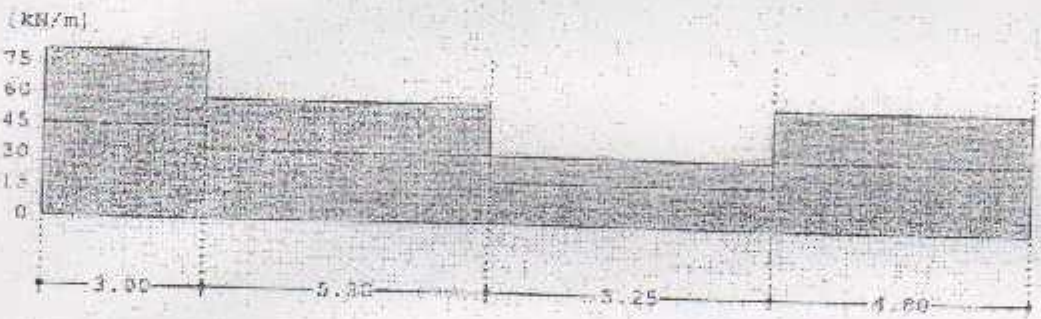
0.3 Ln from face of support = 0.3(5.3 – 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m Use L = 3.6 m

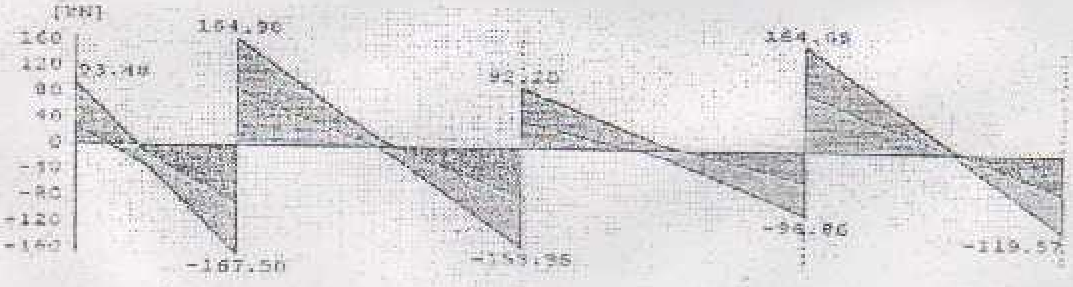
Beam (35)

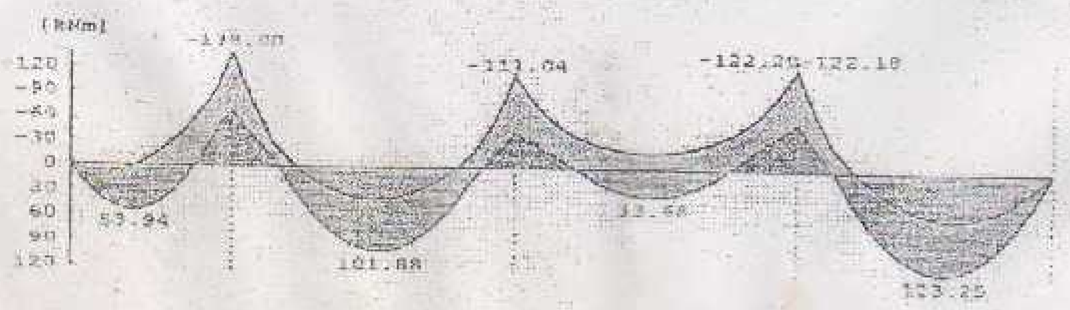


System



Loads





Shear & Moment Envelope Diagram

Fig. (6-48)

B = 40 cm H = 30 cm d = 26 cm

1 - Design For Positive Moment :

Mu = 123.25 KN.m As shown in Fig. (6-48)

Mn = 123.25 / 0.9 = 136.94 KN.m = 13.69 Ton.m

$As\ max. = 0.023\ (40)\ (26) = 23.92 cm^2$

$X_b = 15.29$ cm $X max. = 0.75\ X_b = 11.47$ cm

$a\ max. = \beta X_b = 0.85\ (11.47) = 9.75$ cm

$T max. = As\ fy = (23.92)(4.2) = 100.46$ Ton

Mn max. = 100.46(26 – 0.5(9.75)) /100 = 21.22 Ton.m

Mn max. > Req. Mn ∴ singly reinforcement

$m = 16.47$ $Rn = 50.63\ Kg/cm^2$ $\rho = 0.01357$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.01357\ (40)\ (26) = 14.11\ cm^2$

Use 5 Φ 20 mm $As = 15.27\ cm^2$

Check : tension steel is yield ?

C = 0.85(0.3)(40)a = 10.2 a

Actual T = (15.27)(4.2) = 64.13 Ton

Actual a = 6.29 cm

Actual $x = a / \beta = 6.29 / 0.85 = 7.4$ cm

$\varepsilon_s = 0.00754 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK

Actual Mn = 14.66 Ton.m

2 - Design For Negative Moment :

Mu = 138.01 KN.m As shown in Fig. (6-48)

Mn = 138.01 / 0.9 = 153.34 KN.m = 15.33Ton.m

$m = 16.47$ Rn = 56.69 Kg/cm^2 $\rho = 0.01547$ $\rho_{min.} < \rho < \rho_{max.}$

$As = 0.01547\,(40)\,(26) = 16.09cm^2$

Use 5 Φ 20 mm & 2 Φ 14 mm $As = 18.85cm^2$

Check : tension steel is yield ?

C =0.85(0.3)(40)a = 10.2 a

Actual T = (18.85)(4.2) = 79.17 Ton

Actual a = 79.17 / 10.2 = 7.76 cm

Actual x = a / β = 7.76 / 0.85 = 9.13 cm

$\varepsilon_s = 0.00554 > \varepsilon_y = 0.0021$

∴ Tension steel is yielding ∴ OK.

Actual Mn = 17.51 Ton.m > 15.33Ton.m ∴ OK.

3 - Design For Shear :

$V_u = 16.75$ Ton

V_u @ critical point = 13.19 Ton

$\Phi V_c = 8.07$ Ton

min $\Phi V_s = 3.47$ Ton

$0.85\left(\dfrac{\sqrt{fc'}}{3}\right)bd = 16.14$ Ton complies with category (4)

Req. $\Phi V_s = V_u - \Phi V_c = 13.19 - 8.07 = 5.12$ Ton

Use Φ 10 for stirrups

$$S = \frac{0.85 A_v f_y d}{\Phi V_s} = 28.48 \text{ cm}$$

Use Φ 10 (2 leg) , S = 25 cm

4 - Development length (L_d) :

L_d for bottom bars (Φ 20) :

$L_d = 76.68$ cm

$L_e = 12\ d_b = 12(2) = 24$ cm

$= d = 26$ cm ……………… control

107.73 cm ≥ 76.68 cm ∴ available development > Req.

L_d for Top bars (Φ 20) :

0.3 Ln from face of support = 0.3(5.3 − 2(0.15)) = 1.5 m

The total length of the top bars = 2 (1.5) + 0.6 = 3.6 m <u>Use L = 3.6 m</u>

6-3-3 Beams Summary Tables

Table 6.1 *Beams summry Tables*

Beam N°	Width B(cm)	Dipth H(cm)	Positive As(cm²)	Negative As(cm²)	Stirrups
1	50	30	6Φ20	8Φ20 , 2Φ 14	1Φ10/17cm
2	50	30	4Φ20	5Φ20 , 2Φ 14	1Φ10/25cm
3	70	30	12Φ20	12Φ20 , 4Φ 14	2Φ10/20cm
4	70	30	11Φ20	12Φ20 , 4Φ 14	1Φ10/10cm
5	50	30	5Φ20		1Φ10/30cm
6	80	30	9Φ20	12Φ20 , 4Φ 14	2Φ10/17cm
7	80	30	10Φ20	12Φ20 , 4Φ 14	2Φ10/18cm
8	80	30	10Φ20	12Φ20 , 4Φ 14	2Φ10/17cm
9	40	80	14Φ20		1Φ10/25cm
10	50	30	6Φ20	6Φ20 , 2Φ 14	1Φ10/25cm
11	40	30	2Φ14 strai 2Φ14 bent	6Φ14	1Φ10/25cm
12	40	30	2Φ14 strai 3Φ14 ben	8Φ14	1Φ10/25cm
13	40	30	3Φ14 strai 6Φ14 ben		1Φ10/25cm
14	100	30	13Φ20	13Φ20 , 4Φ 14	2Φ10/16cm
15	70	30	8Φ20	11Φ20 , 4Φ 14	2Φ10/15cm
16	120	30	17Φ20	21Φ20 , 4Φ 14	2Φ10/13cm
17	120	30	16Φ20	21Φ20 , 4Φ 14	2Φ10/13cm
18	40	30	6Φ20		1Φ8/30cm

19	130	30	16Φ20	22Φ20 , 4Φ 14	2Φ10/10cm
20,30	130	30	18Φ20	23Φ20 , 4Φ 14	2Φ10/11cm
21,31	40	70	7Φ20	8Φ20 , 4Φ 14	2Φ10/20cm
22,32	50	80	21Φ20		2Φ10/23cm
23,33	100	30	13Φ20	17Φ20 , 4Φ 14	2Φ10/16cm
24	50	30	5Φ20	7Φ20 , 2Φ 14	1Φ10/17cm
25	50	30	5Φ20	7Φ20 , 4Φ 14	1Φ10/15cm
26,36	60	30	10Φ20		2Φ10/25cm
27	40	30	4Φ20		1Φ10/40cm
28	80	30	100Φ20	12Φ20 , 2Φ 14	1Φ10/13cm
29	80	30	9Φ20	8Φ20 , 4Φ 14	2Φ10/20cm
34	60	30	7Φ20	7Φ20 , 2Φ 14	1Φ10/20cm
35	40	30	5Φ20	5Φ20 , 2Φ 14	1Φ10/25cm

6-4 Columns Design

6-4-1 Load Calculation

The load calculation for all column loads & the support reactions are shown in the tables (6.2) to (6.5).

6-4-2 Design Form for the columns

Pu = (   ) KN

Pn = Pu / 0.56 = (   ) KN

Pn = (   ) Ton

$A_g = (a)(b) = 1250 \text{ cm}^2$

$A_{st,min} = 0.01\ A_g = (\quad) \text{ cm}^2$

$A_{st,max} = 0.08\ A_g = (\quad) \text{ cm}^2$

$Pn = 0.85 f_c' (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = (\quad) \text{ cm}^2$

Tie Design

Tie Φ 10 , if Φ bar ≤ 30 mm   ACI . (7.10.5)

Tie Φ 14 , if Φ bar > 32 mm   ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt}$

$16\ d_b$

Least dimension of columb.

Table (6-2) : *Columns of Fourth Roof*

Column	Beam No.	Dead load (KN)	Live load (KN)	Column Weight (KN)	Pu (KN) From Above	Total (Pu) (KN)	Ag (Sq.cm)	As (Sq.cm)	Type
C 4-1	B 1 B 11	118.95 55.97	12.93 14.30	28	0.0	220.15	25*25	12 Φ 18	L
C 4-2	B 1	338.78	33.35	28	0.0	400.13	50*25	8 Φ 14	A
C 4-3	B 1	298.73	30.98	28	0.0	357.71	50*25	8 Φ 14	A
C 4-4	B 1	256.22	27.73	28	0.0	303.95	50*25	8 Φ 14	A
C 4-5	B 2	194.63	21.98	28	0.0	244.61	50*25	8 Φ 14	A
C 4-6	B 2	284.93	26.55	28	0.0	331.48	50*25	8 Φ 14	A
C 4-7	B 2 B 12	93.94 55.49	9.73 4.17	28	0.0	191.33	25*25	12 Φ 18	L
C 4-8	B 3 B 11	173.42 168.05	24.93 11.94	28	0.0	406.33	50*25	8 Φ 14	A
C 4-9	B 3	500.20	68.00	28	0.0	596.20	50*25	8 Φ 14	A
C 4-10	B 3	420.31	64.31	28	0.0	512.62	50*25	8 Φ 14	A
C 4-11	B 3	459.55	65.98	28	0.0	553.53	50*25	8 Φ 14	A

C 4-12	B 3	397.64	60.27	28	9.0	485.91	50*25	8 Φ 14	A
C 4-13	B 3	392.10	56.27	28	0.0	476.37	50*25	8 Φ 14	A
C 4-14	B 3 B 12	141.79 170.83	20.66 11.56	28	0.0	372.84	50*25	8 Φ 14	A
C 4-15	B 4 B 11	168.61 136.62	24.77 11.26	28	0.0	369.26	50*25	8 Φ 14	A
C 4-16	B 4	496.48	68.00	28	0.0	592.48	50*25	8 Φ 14	A
C 4-17	B 4	362.35	57.30	28	0.0	447.65	50*25	8 Φ 14	A
C 4-18	B 4	400.58	58.94	28	0.0	487.52	50*25	8 Φ 14	A
C 4-19	B 4	397.12	60.27	28	0.0	485.39	50*25	8 Φ 14	A
C 4-20	B 4	381.35	55.92	28	0.0	465.27	50*25	8 Φ 14	A
C 4-21	B 4 B 12	139.22 115.07	20.65 11.90	28	0.0	302.94	50*25	8 Φ 14	A
C 4-22	B 6 B 11	172.09 149.58	24.59 11.38	28	0.0	385.64	50*25	8 Φ 14	A
C 4-23	B 6	518.99	67.99	28	0.0	614.98	50*25	8 Φ 14	A
C 4-24	B 7 B 11	185.65 158.07	26.32 12.14	28	0.0	410.18	50*25	8 Φ 14	A
C 4-25	B 7	535.29	71.76	28	0.0	635.05	50*25	8 Φ 14	A

C 4-26	B 7	450.45	67.88	28	0.0	546.33	50*25	8 Φ 14	A
C 4-27	B 7	489.92	69.40	28	0.0	587.32	50*25	8 Φ 14	A
C 4-28	B 8 B 11	176.52 168.28	24.91 12.14	28	0.0	437.85	50*25	8 Φ 14	A
C 4-29	B 8	514.14	68.31	28	0.0	610.45	50*25	8 Φ 14	A
C 4-30	B 8	415.71	63.80	28	0.0	507.51	50*25	8 Φ 14	A
C 4-31	B 8	518.95	68.88	28	0.0	615.83	50*25	8 Φ 14	A
C 4-32	B 8 B 9	179.94 366.13	25.26 45.45	28	0.0	644.78	50*25	8 Φ 14	A
C 4-33	B 10 B 11	118.53 55.93	12.51 4.36	28	0.0	219.33	25*25	12 Φ 18	L
C 4-34	B 10	341.28	34.00	28	0.0	403.28	50*25	8 Φ 14	A
C 4-35	B 10	288.72	33.34	28	0.0	368.97	50*25	8 Φ 14	A
C 4-36	B 10	307.98	32.99	28	0.0	350.06	50*25	8 Φ 14	A
C 4-37	B 10	323.2	67.76	28	0.0	418.96	50*25	8 Φ 14	A
C 4-38	B 10	320.19	66.34	28	0.0	414.53	50*25	8 Φ 14	A
C 4-39	B 19 B 12	91.95 59.11	10.33 4.48	28	0.0	193.87	25*25	12 Φ 18	L

Table (6-3) : columns of third roof

Column	Beam No.	Dead load (KN)	Live load (KN)	Column Weight (KN)	Pu (KN) From Above	Total (Pu) (KN)	Ag (Sq.cm)	As (Sq.cm)	Type
C 3-1	B 14 B 24	192.52 111.95	82.66 26.84	28	220.15	670.12	25*25	12 Φ 18	L
C 3-2	B 14	548.34	222.42	28	400.13	1198.89	50*25	8 Φ 14	A
C 3-3	B 14	483.51	206.52	28	357.71	1075.74	50*25	8 Φ 14	A
C 3-4	B 14	414.71	184.86	28	303.95	931.52	50*25	8 Φ 14	A
C 3-5	B 15	315.03	146.50	28	244.61	734.14	50*25	8 Φ 14	A
C 3-6	B 15	461.18	177.0	28	331.48	997.66	50*25	8 Φ 14	A
C 3-7	B 15 B 25	152.05 111.05	64.86 26.09	28	191.33	573.38	25*25	12 Φ 18	L
C 3-8	B16 B 24	196.66 336.11	147.77 74.57	28	406.33	1198.44	50*25	8 Φ 14	A
C 3-9	B 16	536.46	426.23	28	596.20	1586.89	50*25	8 Φ 14	A
C 3-10	B 16	507.35	358.15	28	512.62	1406.12	50*25	8 Φ 14	A

C 3-11	B 16	520.49	391.59	28	553.53	1493.61	50*25	8 Φ 14	A
C 3-12	B 16	475.44	338.83	28	485.91	1328.18	50*25	8 Φ 14	A
C 3-13	B 16	434.94	334.11	28	476.37	1282.42	50*25	8 Φ 14	A
C 3-14	B 16 B 25	162.95 341.25	120.82 72.31	28	372.84	1098.17	50*25	8 Φ 14	A
C 3-15	B 17 B 24	168.61 273.24	162.32 70.36	28	369.26	1071.82	50*25	8 Φ 14	A
C 3-16	B 17	496.48	445.79	28	592.48	1562.75	50*25	8 Φ 14	A
C 3-17	B 17	362.35	374.72	28	447.65	1212.27	50*25	8 Φ 14	A
C 3-18	B 17	400.58	385.01	28	487.52	1273.11	50*25	8 Φ 14	A
C 3-19	B 17	397.12	395.03	28	485.39	1305.59	50*25	8 Φ 14	A
C 3-20	B 17	381.35	366.53	28	465.27	1241.15	50*25	8 Φ 14	A
C 3-21	B 17 B 25	139.22 237.58	135.40 74.38	28	302.94	917.52	50*25	8 Φ 14	A
C 3-22	B 19 B 24	172.09 299.15	166.57 73.95	28	385.64	1125.40	50*25	8 Φ 14	A
C 3-23	B 19	518.99	460.80	28	614.98	1622.77	50*25	8 Φ 14	A
C 3-24	B 20 B 24	185.65 316.13	180.04 75.92	28	410.18	1195.92	50*25	8 Φ 14	A

C 3-25	B 20	535.29	491.01	28	635.05	1689.35	50*25	8 Φ 14	A
C 3-26	B 20	450.45	464.47	28	546.33	1489.25	50*25	8 Φ 14	A
C 3-27	B 20	489.92	474.86	28	587.32	1589.10	50*25	8 Φ 14	A
C 3-28	B 21 B 24	176.33 336.57	179.98 75.84	28	437.85	1234.57	50*25	8 Φ 14	A
C 3-29	B 21	515.29	494.27	28	610.45	1648.01	50*25	8 Φ 14	A
C 3-30	B 21	417.22	461.83	28	507.51	1414.56	50*25	8 Φ 14	A
C 3-31	B 21	518.59	497.51	28	615.83	1659.93	50*25	8 Φ 14	A
C 3-32	B 21 B 22	180.00 366.13	182.46 330.77	28	644.78	1732.14	50*25	8 Φ 14	A
C 3-33	B 23 B 24	191.70 111.87	83.16 27.20	28	219.33	661.26	25*25	12 Φ 18	L
C 3-34	B 23	553.29	226.67	28	403.28	1211.24	50*25	8 Φ 14	A
C 3-35	B 23	463.70	216.10	28	368.97	1057.84	50*25	8 Φ 14	A
C 3-36	B 23	511.79	219.93	28	350.06	1128.69	50*25	8 Φ 14	A
C 3-37	B 23	407.59	255.24	28	418.96	1109.97	50*25	8 Φ 14	A
C 3-38	B 23	400.98	242.76	28	414.53	1086.27	50*25	8 Φ 14	A
C 3-39	B 23 B 25	159.82 65.37	68.85 28.02	28	193.87	543.66	25*25	12 Φ 18	L

Table (6-4) : *columns of Second roof*

Column	Beam No.	Dead load (KN)	Live load (KN)	Column Weight (KN)	Pu (KN) From Above	Total (Pu) (KN)	Ag (Sq.cm)	As (Sq.cm)	Type
C 2-1	B 14 B 24	192.52 111.95	82.66 26.84	28	670.12	1112.09	25*25	12 Φ 18	L
C 2-2	B 14	548.34	222.42	28	1198.89	1997.65	50*25	8 Φ 14	B
C 2-3	B 14	483.51	206.52	28	1075.74	1793.77	50*25	8 Φ 14	A
C 2-4	B 14	414.71	184.86	28	931.52	1559.09	50*25	8 Φ 14	A
C 2-5	B 15	315.03	146.50	28	734.14	1223.67	50*25	8 Φ 14	A
C 2-6	B 15	461.18	177.0	28	997.66	1663.84	50*25	8 Φ 14	A
C 2-7	B 15 B 25	152.05 111.05	64.86 26.09	28	573.38	955.43	25*25	12 Φ 18	L
C 2-8	B16 B 24	196.66 336.11	147.77 74.57	28	1198.44	1981.55	50*25	8 Φ 14	B
C 2-9	B 16	536.46	426.23	28	1586.89	2577.58	50*25	16 Φ 18	E
C 2-10	B 16	507.35	358.15	28	1406.12	2299.64	50*25	12 Φ 18	D
C 2-11	B 16	520.49	391.59	28	1493.61	2368.36	50*25	12 Φ 18	D

C 2-12	B 16	475.44	338.83	28	1328.18	2170.45	50*25	8 Φ 18	C
C 2-13	B 16	434.94	334.11	28	1282.42	2079.47	50*25	8 Φ 18	C
C 2-14	B 16 B 25	162.95 341.25	120.82 72.31	28	1098.17	1823.50	50*25	8 Φ 14	B
C 2-15	B 17 B 24	168.61 273.24	162.32 70.36	28	1071.82	1774.35	50*25	8 Φ 14	A
C 2-16	B 17	496.48	445.79	28	1562.75	2533.02	50*25	16 Φ 18	E
C 2-17	B 17	362.35	374.72	28	1212.27	1977.34	50*25	8 Φ 14	B
C 2-18	B 17	400.58	385.01	28	1273.11	2086.70	50*25	8 Φ 18	C
C 2-19	B 17	397.12	395.03	28	1305.39	2125.79	50*25	8 Φ 18	C
C 2-20	B 17	381.35	366.53	28	1241.15	2017.03	50*25	8 Φ 18	C
C 2-21	B 17 B 25	139.22 237.58	135.40 74.38	28	917.52	1532.10	50*25	8 Φ 14	A
C 2-22	B 19 B 24	172.09 299.15	166.57 73.95	28	1125.40	1865.16	50*25	8 Φ 14	B
C 2-23	B 19	518.99	460.80	28	1622.77	2630.56	50*25	14 Φ 20	F
C 2-24	B 20 B 24	185.65 316.13	180.04 75.92	28	1195.92	1981.66	50*25	8 Φ 14	B
C 2-25	B 20	535.29	491.01	28	1689.35	2743.65	50*25	14 Φ 20	F

C 2-26	B 20	430.45	464.47	28	1489.25	2432.17	50*25	16 Φ 18	E
C 2-27	B 20	489.92	474.86	28	1580.10	2572.88	50*25	16 Φ 18	E
C 2-28	B 21 B 24	176.33 336.57	179.98 75.84	28	1234.57	2031.29	50*25	8 Φ 18	C
C 2-29	B 21	515.29	494.27	28	1648.01	2685.57	50*25	14 Φ 20	F
C 2-30	B 21	417.22	461.83	28	1414.56	2321.61	50*25	12 Φ 18	D
C 2-31	B 21	518.59	497.51	28	1659.93	2704.03	50*25	14 Φ 20	F
C 2-32	B 21 B 22	180.00 366.13	182.46 330.77	28	1732.14	2819.50	50*25	18 Φ 22	G
C 2-33	B 23 B 24	191.70 111.87	83.16 27.20	28	661.36	1103.19	25*25	12 Φ 18	L
C 2-34	B 23	553.29	226.67	28	1211.24	2019.26	50*25	8 Φ 18	C
C 2-35	B 23	463.70	216.10	28	1057.84	1765.62	50*25	8 Φ 14	A
C 2-36	B 23	511.79	219.93	28	1128.69	1888.41	50*25	8 Φ 14	B
C 2-37	B 23	407.59	255.24	28	1109.97	1800.98	50*25	8 Φ 14	B
C 2-38	B 23	460.98	242.76	28	1086.27	1758.01	50*25	8 Φ 14	A
C 2-39	B 23 B 25	159.82 65.37	68.85 28.02	28	543.66	893.52	25*25	12 Φ 18	L

Table (6-5) : columns of First roof

Column	Beam No.	Dead load (KN)	Live load (KN)	Column Weight (KN)	Pu (KN) From Above	Total (Pu) (KN)	Ag (Sq.cm)	As (Sq.cm)	Type
C 1-22	B 28 B 34	101.04 126.45	91.35 29.54	28	1863.16	2241.54	50*25	12 Φ 18	D
C 1-23	B 28	294.3	250.50	28	2630.36	3203.36	50*25	20 Φ 22	H
C 1-24	B 30 B 34	185.65 354.20	207.04 79.03	28	1981.66	2835.58	50*25	18 Φ 22	G
C 1-25	B 30	535.29	491.01	28	2743.65	3797.95	50*25	20 Φ 24	J
C 1-26	B 30	450.45	464.47	28	2432.17	3375.07	50*25	20 Φ 22	H
C 1-27	B 30	489.92	474.86	28	2572.88	3565.66	50*25	18 Φ 24	I
C 1-28	B 31 B 34	176.33 336.57	179.98 75.84	28	2031.29	2817.34	50*25	18 Φ 22	G
C 1-29	B 31	515.29	494.27	28	2685.57	3723.13	50*25	20 Φ 24	J
C 1-30	B 31	417.22	461.83	28	2321.61	3228.66	50*25	20 Φ 22	H
C 1-31	B 31	518.59	497.51	28	2704.03	3748.13	50*25	20 Φ 24	J
C 1-32	B 31 B 32	180.00 366.13	182.46 330.77	28	2819.30	3996.86	50*25	22 Φ 24	K

C 1-33	B 33 B 34	191.70 111.70	83.16 27.30	28	1103.19	1547.26	25*25	12 Φ 18	L
C 1-34	B 33	553.29	226.67	28	2019.20	2827.16	50*25	18 Φ 22	G
C 1-35	B 33	463.70	216.10	28	1765.62	2473.4	50*25	16 Φ 18	E
C 1-36	B 33	511.79	219.93	28	1888.41	2648.13	50*25	14 Φ 20	F
C 1-37	B 33	407.39	255.24	28	1800.98	2491.81	50*25	16 Φ 18	E
C 1-38	B 33	400.98	242.76	28	1758.01	2429.75	50*25	16 Φ 18	E
C 1-39	B 33 B 35	159.82 65.37	68.85 28.02	28	893.52	1269.76	25*25	12 Φ 18	L

6-4-3 Design of Columbs

Column (Type A) :

Columns which carries $Pu < 1800$ KN

$Pu = 1793.77$ KN

$Pn = Pu / 0.56 = 3203.16$ KN

$Pn = 320.32$ Ton

$A_g = (50)(25) = 1250\ cm^2$

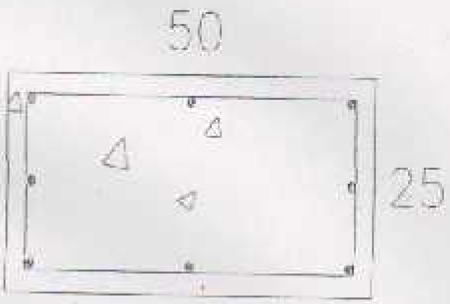
$A_{st.min} = 0.01\,(1250) = 12.5\ cm^2$

$A_{st.max} = 0.08\,(1250) = 100\ cm^2$

$Pn = 0.85 f_c^{'}\,(A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 0.4\ cm^2 < A_{st.min} = 12.5\ cm^2$

Use 8 Φ 14 $A_{st} = 12.32\ cm^2$



$A_{st} = 8\ Φ\ 14$

Tie Φ 10 @ 22 cm

Tie Design :

Tie Φ 10 , if Φ bar < 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{st} = 48\,(1) = 48$ cm

$16\ d_b = 16\,(1.4) = 22.4$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 22 cm

Column (Type B) :

Columns which carries $1800 < Pu < 1999.99$ KN

Pu = 1997.65 KN

Pn = 1997.65 / 0.56 = 3567.23 KN

Pn = 356.72 Ton

$A_g = (50)(25) = 1250 \text{ cm}^2$

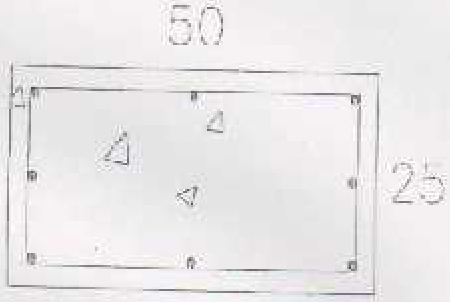
$A_{st\,min} = 0.01\,(1250) = 12.5 \text{ cm}^2$

$A_{st\,max} = 0.08\,(1250) = 100 \text{ cm}^2$

Pn= $0.85 f_c^{'}(A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 9.63 \text{ cm}^2 < A_{st\,min} = 12.5 \text{ cm}^2$

Use 8 Φ 14 $A_{st} = 12.32 \text{ cm}^2$



$A_{st} = 8\,\Phi\,14$

Tie Φ 10 @ 22 cm

Tie Design :

Tie Φ 10 , if Φ bar < 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt} = 48\ (1) = 48$ cm

$16\ d_b = 16\ (1.4) = 22.4$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 22 cm

Column (Type C)

Columns which carries 2000 < Pu < 2199.99 KN

Pu = 2170.45 KN

Pn = 2170.45 / 0.56 = 3875.8 KN

Pn = 387.58 Ton

$A_g = (50)(25) = 1250\ cm^2$

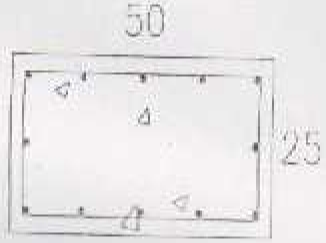
$A_{st.min} = 0.01\ (1250) = 12.5\ cm^2$

$A_{st.max} = 0.08\ (1250) = 100\ cm^2$

$Pn = 0.85 f_c^{'}\ (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 17.45\ cm^2 > A_{st.min} = 12.5\ cm^2$

Use 8 Φ 18 $A_{st} = 20.36\ cm^2$



$A_{st} = 8\ \Phi\ 18$

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{st} = 48\ (1) = 48\ cm$

$16\ d_b = 16\ (1.8) = 28.8\ cm$

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type D) .

Columns which carries 2200 < Pu < 2399.99 KN

Pu = 2375.07 KN

Pn = 2375.07 / 0.56 = 4241.20 KN

Pn = 424.12 Ton

$A_g = (50)(25) = 1250\ cm^2$

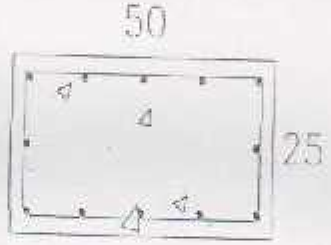
$A_{st.min} = 0.01\ (1250) = 12.5\ cm^2$

$A_{st.max} = 0.08\ (1250) = 100\ cm^2$

$Pn = 0.85 f'_c\ (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 26.71\ cm^2 > A_{st.min} = 12.5\ cm^2$

Use 12 Φ 18 $A_{st} = 30.48\ cm^2$



$A_{st} = 12\ \Phi\ 18$

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar < 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt} = 48\ (1) = 48\ cm$

$16\ d_b = 16\ (1.8) = 28.8\ cm$

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type E) :

Columns which carries $2400 < Pu < 2599.99$ KN

$Pu = 2577.58$ KN

$Pn = 2577.58 / 0.56 = 4602.82$ KN

$Pn = 460.28$ Ton

$A_g = (50)(25) = 1250 \text{ cm}^2$

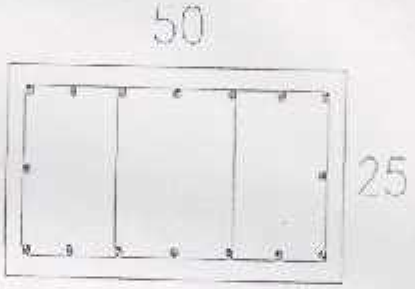
$A_{st,min} = 0.01\,(1250) = 12.5 \text{ cm}^2$

$A_{st,max} = 0.08\,(1250) = 100 \text{ cm}^2$

$Pn = 0.85 f'_c\,(A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 35.88 \text{ cm}^2 > A_{st,min} = 12.5 \text{ cm}^2$

Use 16 Φ 18 $A_{st} = 40.64 \text{ cm}^2$



$A_{st} = 16\ \Phi\ 18$

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{st} = 48\,(1) = 48$ cm

$16\ d_b = 16\,(1.8) = 28.8$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type I) :

Columns which carries $2600 < Pu < 2799.99$ KN

$Pu = 2743.65$ KN

$Pn = 2743.65 / 0.56 = 4899.38$ KN

$Pn = 489.94$ Ton

$A_g = (50)(25) = 1250\ cm^2$

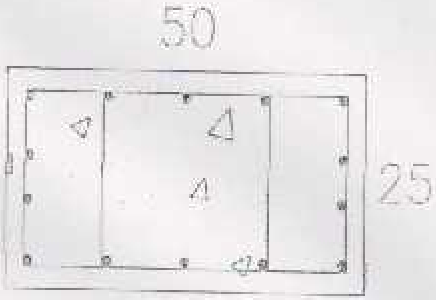
$A_{st,min} = 0.01\,(1250) = 12.5\ cm^2$

$A_{st,max} = 0.08\,(1250) = 100\ cm^2$

$Pn = 0.85 f_c^{'} (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 43.39\ cm^2 > A_{st,min} = 12.5\ cm^2$

Use 14 Φ 20 $A_{st} = 43.98\ cm^2$



$A_{st} = 14\ \Phi\ 20$

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt} = 48\ (1) = 48$ cm

$16\ d_b = 16\ (2) = 32$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type G) :

Columns which carries $3200 < Pu < 3299.99$ KN

$Pu = 3241.54$ KN

$Pn = 3241.54 / 0.56 = 5788.4$ KN

$Pn = 578.85$ Ton

$A_g = (50)(25) = 1250 \text{ cm}^2$

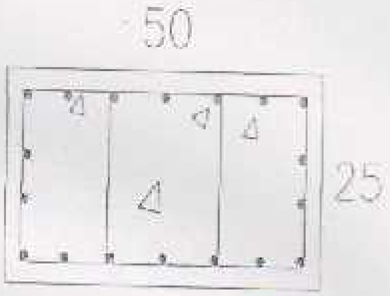
$A_{st.min} = 0.01\ (1250) = 12.5 \text{ cm}^2$

$A_{st.max} = 0.08\ (1250) = 100 \text{ cm}^2$

$Pn = 0.85 f_c' (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 65.93 \text{ cm}^2 > A_{st.min} = 12.5 \text{ cm}^2$

Use 18 Φ 22 $A_{st} = 68.42 \text{ cm}^2$



$A_{st} = 18\ \Phi\ 22$

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{tr} = 48\ (1) = 48$ cm

$16\ d_b = 16\ (2.2) = 35.2$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type H) :

Columns which carries $3300 < Pu < 3399.99$ KN

$Pu = 3375.07$ KN

$Pn = 3375.07 / 0.56 = 6026.91$ KN

$Pn = 602.69$ Ton

$A_g = (50)(25) = 1250 \text{ cm}^2$

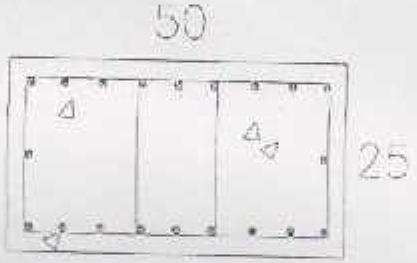
$A_{st.min} = 0.01\,(1250) = 12.5 \text{ cm}^2$

$A_{st.max} = 0.08\,(1250) = 100 \text{ cm}^2$

$Pn = 0.85 f_c' (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 71.97 \text{ cm}^2 > A_{st.min} = 12.5 \text{ cm}^2$

Use 20 Φ 22 $A_{st} = 76.02 \text{ cm}^2$



$A_{st} = 20$ Φ 22

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt} = 48\,(1) = 48$ cm

$16\ d_b = 16\,(2.2) = 35.2$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

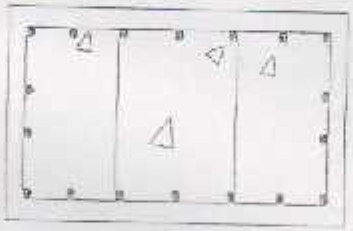
Column (Type I) :

Columns which carries $3500 < Pu < 3599.99$ KN

Pu = 3565.66 KN

Pn = 3565.66 / 0.56 = 6367.25 KN

Pn = 636.73 Ton



$A_g = (50)(25) = 1250 \text{ cm}^2$

$A_{st,min} = 0.01\,(1250) = 12.5 \text{ cm}^2$

$A_{st,max} = 0.08\,(1250) = 100 \text{ cm}^2$

$Pn = 0.85 f_c' (A_g - A_{st}) + (A_{st} f_y)$

$A_{st,min} = 12.5 \text{ cm}^2 < A_{st} = 80.6 \text{ cm}^2 < A_{st,max} = 100 \text{ cm}^2$

Use 18 Φ 24 $A_{st} = 81.42 \text{ cm}^2$

A_{st} = 18 Φ 24

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar > 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{bt} = 48\,(1) = 48$ cm

$16\ d_b = 16\,(2.4) = 38.4$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type J) :

Columns which carries 3700< Pu < 3799.99 KN

Pu = 3723.13 KN

Pn = 3723.13 / 0.56 = 6648.45 KN

Pn = 664.84 Ton

$A_g = (50)(25) = 1250 \text{ cm}^2$

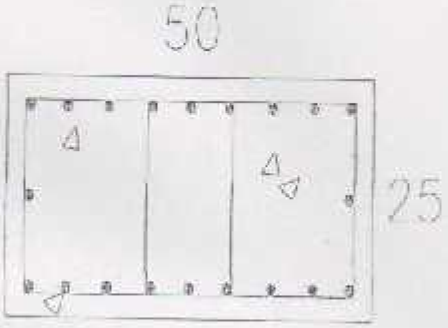
$A_{st.min} = 0.01\,(1250) = 12.5 \text{ cm}^2$

$A_{st.max} = 0.08\,(1250) = 100 \text{ cm}^2$

$Pn = 0.85 f_c' \,(A_g - A_{st}) + (A_{st} f_y)$

$A_{st.min} = 12.5 \text{ cm}^2 < A_{st} = 80.6 \text{ cm}^2 < A_{st.max} = 100 \text{ cm}^2$

Use 20 Φ 24 $A_{st} = 90.48 \text{ cm}^2$



$A_{st} = 20$ Φ 24

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{ti} = 48\,(1) = 48$ cm

$16\ d_s = 16\,(2.4) = 38.4$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

Column (Type K) :

Columns which carries $3900 < Pu < 3999.99$ KN

Pu = 3906.86 KN

Pn = 3906.86 / 0.56 = 6976.54 KN

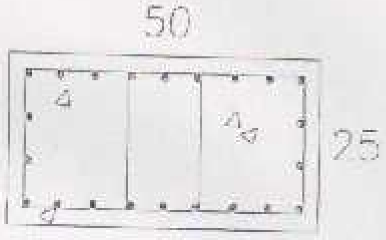
Pn = 697.65 Ton

$A_g = (50)(25) = 1250\ cm^2$

$Pn = 0.85 f_c' (A_g - A_{st}) + (A_{st} f_y)$

$A_{st} = 96.05\ cm^2$

Use 22 Φ 24 $A_{st} = 99.44\ cm^2$



A_{st} = 22 Φ 24

Tie Φ 10 @ 25 cm

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{st} = 48\ (1) = 48$ cm

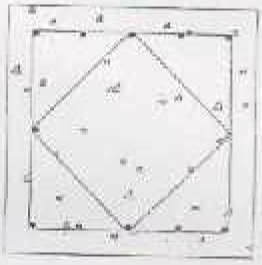
$16\ d_b = 16\ (2.4) = 38.4$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm.

Column (Type I) :

Corner Columns (25cm)(25cm)



Pu = 1547.26 KN

Pn = 1547.26 / 0.56 = 2762.9 KN

Pn = 276.3 Ton

$A_g = (25)(25) = 625 cm^2$

$A_{st\,min} = 0.01\ (625) = 6.25\ cm^2$

$A_{st.max} = 0.08\ (625) = 50\ cm^2$

$A_{st} = 12\ \Phi\ 18$

$Pn = 0.85 f_c'\ (A_g - A_{st}) + (A_{st} f_y)$

Tie Φ 10 @ 25 cm

$A_{st} = 29.63\ cm^2 > A_{st\,min} = 6.25\ cm^2$

Use 12 Φ 18 $A_{st} = 30.54\ cm^2$

Tie Design :

Tie Φ 10 , if Φ bar ≤ 30 mm ACI . (7.10.5)

Tie Φ 14 , if Φ bar ≥ 32 mm ACI . (7.10.5)

Ties Spacing Design :

Minimum of the following spacing is control :

$48\ d_{st} = 48\ (1) = 48$ cm

$16\ d_b = 16\ (1.8) = 28.8$ cm

Least dimension of column = 25 cm

Use Tie Φ 10 @ 25 cm

6-5 Footing Design

6-5-1 Load Calculation

The load calculation for all footing loads & the support reactions are shown in the following table (6.6)

Table (6-5) : Type of Footing

Footing name	Column type	Dead load (Ton)	Live load (Ton)	Factored total load(Ton)	Depth (cm)	Area (Sq.m)	Area of steel
F 1	H,I,J	191.38	146.13	337.51	65	2.7*2.7	15 Φ 18
F 2	K	229.64	161.034	390.7	80	2.9*2.9	16 Φ 18
F 3	F,G	199.913	83.64	283.56	60	2.5*2.5	16 Φ 16
F 4	C,D,E	165.8	83.35	249.2	60	2.4*2.4	13 Φ 16
F 5	B	151.95	47.82	199.77	50	2.2*2.2	11 Φ 16
F 6	A	90.87	31.49	122.4	50	1.8*1.8	8 Φ 16
F 7	L	113.96	41.54	155.5	50	1.9*1.9	8 Φ 18

6-5-2 Footing Design

Foundation Design (F 1) :

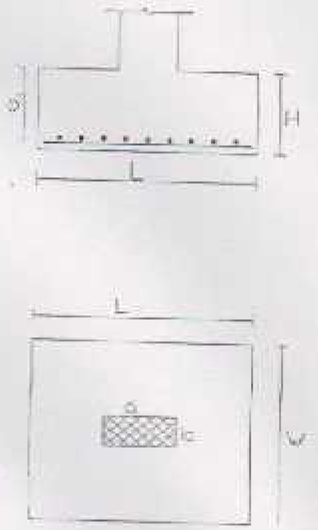


Fig. (6-49) :

Footing Shape

From Column (C 1-26):

Total dead load = 1913.8 KN = 191.38 Ton

Total live load = 1461.29 KN = 146.13 Ton

Factored load = 3375.09 KN = 337.51 Ton

1 – Footing Area :

assume depth of the footing (H) = 65 cm

- Service Load = 191.38 / 1.4 + 146.13 / 1.7 = 222.7 Ton
- Footing Weight = (3)(3)(0.65)(2.4) = 14 Ton
- Earth Pressure = $[(W)(L) - (a)(b)](c)\gamma_s$

  = [(3)(3)-(0.5)(0.25)](0.6)(1.8)

  = 9.6 Ton
- Total Weight = 246.3 Ton

Area (A) = Total Weight / Soil Pressure

= 246.3)(1000)Kg /3.5 Kg/cm^2

= 7.1 m^2

Use L = 2.7 m , W = 2.7 m , A = 7.29 m^2

2 – Shear Strength :

☞ Check this depth for one way action :

$$d = 65 - (8 + \Phi 25) = 54cm$$

$$P_{net} = \frac{P_u}{Area} = \frac{337.51}{7.29} = 46.3 \text{ Ton}/m^2$$

$$V_u = P_{net} \times d \times W = (46.3)(0.54)(2.7) = 70 \text{ Ton}$$

When No shear reinforcement is used :

$$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d = \frac{1}{6}\sqrt{30} \times (270) \times (54) \times \frac{10}{1000} = 133.1 \text{ Ton}$$

$\Phi V_c > V_u$ 113.15 Ton > 70 Ton OK.

☞ Check this depth for two way action *(punching)* :

$$V_u = P_{net} \times \left((W) \times (L) - (a+d)(b+d) \right)$$

$$= 46.3\,[(2.7)(2.7) - (1.04)(0.79)] = 299.5 \text{ Ton}$$

When No shear reinforcement is used :

$$V_c = \frac{1}{6}\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\, b_o d \quad = 360.8 \text{ Ton}$$

$$V_c = \frac{1}{12}\left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f_c'}\, b_o d \quad = 1083.2 \text{ Ton}$$

$$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \quad = 360.8 \text{ Ton}$$

where :

$\beta_c = a / b = 50 / 25 = 2$

b_o = perimeter of critical section taken at (d/2)from the loaded area

$= (1.04)(2) + (0.79)(2) = 3.66$ m

$\alpha_s = 40$ For interior column

$V_c = 360.8$ Ton

$\Phi V_c > V_u$ 306.71 Ton > 299.5 Ton OK

3 – Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$$= \left(46.3 \times 2.7 \times \left(\frac{2.7}{2} \times \frac{0.5}{2}\right)\right) \times 0.5\left(\frac{2.7}{2} \times \frac{0.5}{2}\right) = 75.63 \text{ Ton.m}$$

$$Mn = \frac{Mu}{\Phi} = \frac{75.63}{0.9} = 84 \text{ Ton}$$

$$Rn = \frac{Mn}{bd^2} = \frac{84 \times 10^5}{270 \times 54^2} = 10.67 \ Kg/cm^2$$

$\rho = 0.00259 > \rho_{min} = 0.002$

Req. $A_s = 0.00259\ (270)(54) = 37.85\ cm^2$

Use $15\ \Phi\ 18$ $A_s = 38.17\ cm^2$ (In each way_

4-Development Length (l_d) :

$$L_d = \frac{420}{2\sqrt{30}} \times d_b = 69 \text{ cm}$$

$110 - 8 = 102$ cm > 69 cm

OK

5-Dwell Req.

Minimum Dwells Req. $= 0.005\ (50)(25) = 6.25\ cm^2$

Use $4\ \Phi\ 16$ $A_s = 8.04\ cm^2$

Foundation Design (F 2) :

From Column (C 1-32):

Total dead load = 2296.43 KN = 229.64 Ton

Total live load = 1610.34 KN = 161.034 Ton

Factored load = 3906.8 KN = 390.7 Ton

1 – Footing Area :

assume depth of the footing (H) = 80 cm

- Service Load = 229.64 / 1.4 + 161.034 / 1.7 = 258.76 Ton
- Footing Weight = (3)(3)(0.8)(2.4) = 17.28 Ton
- Earth Pressure =[(3)(3)-(0.5)(0.25)](0.6)(1.8) = 9.6 Ton
- Total Weight = 285.64 Ton

Area (A) =(285.64)(1000)Kg /3.5 Kg/cm^2

= 8.16 m^2

Use L = 2.9 m , W = 2.9 m , A = 8.41 m^2

2 – Shear Strength :

- Check this depth for one way action :

D =0.69

$P_{nu} = \frac{P_u}{Area} = 390.7/8.41 = 46.45$ Ton / m^2

$V_u = P_{nu} \times d \times W = (46.45)(0.69)(2.9) = 92.94$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d = 156.21$ Ton

$\Phi V_c > V_u$ 132.8 Ton > 92.94 Ton OK.

- Check this depth for two way action *(punching)* :

$V_u = P_{nu} \times \left((W) \times (L) - (a + d)(b + d) \right)$

$= 46.45\,[(2.9)(2.9) - (1.12)] = 338.6$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \qquad = 536.65$ Ton

where :

$\beta_c = a / b = 50 / 25 = 2$

$b_o = (1.19)(2) + (0.94)(2) = 4.26$ m

$\Phi V_c > V_u \qquad\qquad 456.2 \text{ Ton} > 338.6 \text{ Ton} \quad \text{OK}$

3 – Bending Moment :

$\text{Mu} = \left(P_{nu} \times W \times \left(\frac{L}{2} \times \frac{a}{2} \right) \right) \times 0.5 \left(\frac{L}{2} \times \frac{a}{2} \right)$

$= \{46.45(2.9)(1.2)\} * (1.2)/2 = 96.98$ Ton.m

Mn = 96.98 / 0.9 = 107.76 Ton

Rn = 7.8 Kg / cm^2

$\rho = 0.00188 < \rho_{min} = 0.002$

Req. $A_s = 0.002\,(290)(69) = 40.02\ cm^2$

Use 16 Φ 18 $\qquad A_s = 40.72\ cm^2$ (In each way)

4-Development Length (L_d) :

$L_d \quad = \frac{420}{2\sqrt{30}} \times d_b = 69$ cm

120 – 8 = 112 cm > 69 cm

OK.

5- Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $\qquad A_s = 8.04\ cm^2$

Foundation Design (F 3) :

From Column (C 1-24):

Total dead load = 1999.13 KN = 199.913 Ton

Total live load = 836.45 KN = 83.64 Ton

Factored load = 2835.58 KN = 283.56 Ton

1 – Footing Area :

assume depth of the footing (H) = 60 cm

- Service Load = 199.91 / 1.4 + 83.64 / 1.7 = 192 Ton
- Footing Weight = (3)(3)(0.6)(2.4) = 12.96 Ton
- Earth Pressure =[(3)(3)-(0.5)(0.25)](0.6)(1.8) = 9.6 Ton
- Total Weight = 214.56 Ton

Area (A) =(214.56)(1000)Kg /3.5 Kg/cm²

= 6.13 m²

Use L = 2.5 m , W = 2.5 m , A = 6.25 m²

2 – Shear Strength :

- Check this depth for one way action :

D =50

$P_{net} = \frac{P_u}{Area} = 283.56/6.25 = 45.37$ Ton / m²

$V_u = P_{net} \times d \times W = (45.37)(0.5)(2.5) = 56.7$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f'_c}\, b_w d = 114.12$ Ton

$\Phi V_c > V_u$ 97 Ton > 56.7 Ton OK.

- Check this depth for two way action *(punching)* :

$V_u = P_{net} \times (\ (W) \times (L) - (a + d)(b + d)\ )$

$= 45.37\ [(2.5)(2.5) - (1)(0.75)] = 249.53$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\ b_o d \qquad = 319.5$ Ton

where :

$\beta_c = a / b = 50 / 25 = 2$

$b_o = (1)\ (2) + (0.75)(2) = 3.5$ m

$\Phi V_c > V_u \qquad$ 271.6 Ton > 249.53 Ton OK

3 – Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$= \{45.37(2.5)(1)\} * (1)/2 = 56.71$ Ton.m

Mn = 56.71 / 0.9 = 63 Ton

Rn = 10.08 Kg / cm^2

$\rho = 0.0024 > \rho_{min} = 0.002$

Req. $A_s = 0.0024\ (250)(50) = 30.617\ cm^2$

Use 16 Φ 16 $\qquad A_s = 32.16\ cm^2$ (In each way)

4-Development Length (L_d) :

$$L_d = \frac{420}{2\sqrt{30}} \times d_b = 61.34 \text{ cm}$$

100 – 8 = 92 cm > 61.34 cm OK.

5-Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $\qquad A_s = 8.04\ cm^2$

Foundation Design - (F 4) :

From Column (C 1-37):

Total dead load = 1657.97 KN = 165.8 Ton

Total live load = 833.48 KN = 83.35 Ton

Factored load = 2491.15 KN = 249.2 Ton

1 – Footing Area :

assume depth of the footing (H) = 60 cm

- Service Load = 165.8 / 1.4 + 83.35 / 1.7 = 167.45 Ton
- Footing Weight = (3)(3)(0.6)(2.4) = 12.96 Ton
- Earth Pressure =[(3)(3)-(0.5)(0.25)](0.6)(1.8) = 9.6 Ton
- Total Weight = 189.95 Ton

Area (A) =(189.95)(1000)Kg /3.5 Kg/cm ^{2}

= 5.42 m ^{2}

Use L = 2.4 m , W = 2.4 m , A = 5.76 m ^{2}

2 – Shear Strength :

- Check this depth for one way action :

D =50

$P_{net} = \frac{P_u}{Area}$ = 249.12/5.76 = 43.23 Ton / m ^{2}

$V_u = P_{net} \times d \times W$ = (43.23)(0.5)(2.4) = 51.87 Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d$ = 109.56 Ton

$\Phi V_c > V_u$ 93.13 Ton > 51.87 Ton OK.

- Check this depth for two way action *(punching)* :

$V_u = P_{net} \times (\ (W) \times (L) - (a+d)(b+d)\ )$

$= 43.23\ [(2.4)(2.4) - (1)(0.75)] = 216.58$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \quad = 319.5$ Ton

where :

$\beta_c = a / b = 50 / 25 = 2$

$b_o = (1)(2) + (0.75)(2) = 3.5$ m

$\Phi V_c > V_u$ 271.6 Ton > 216.58 Ton OK

3 – Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$= \{43.23(2.4)(0.95)\} * (0.95)/2 = 46.82$ Ton.m

Mn = 46.82 / 0.9 = 52 Ton

Rn = 8.67 Kg / cm^2

$\rho = 0.0021 > \rho_{min} = 0.002$

Req. $A_s = 0.0021\ (240)(50) = 25.2\ cm^2$

Use 13 Φ 16 $A_s = 26.14\ cm^2$ (In each way)

4-Development Length (L_d) :

$$L_d \quad = \frac{420}{2\sqrt{30}} \times d_b = 61.34 \text{ cm}$$

95 – 8= 87 cm > 61.34 cm OK

5-Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $A_s = 8.04\ cm^2$

$= 41.3\,[(2.2)(2.2) - (0.9)(0.65)] = 175.73$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\,b_o d \quad = 226.35$ Ton

where :

$\beta_c = a / b = 50 / 25 = 2$

$b_o = (0.9)(2) + (0.65)(2) = 3.1$ m

$\Phi V_c > V_u$ 192.43 Ton > 175.73 Ton OK

3–Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$=\{41.3(2.2)(0.85)\}*(0.85)/2 = 32.82$ Ton.m

Mn = 32.82 / 0.9 = 36.47 Ton

Rn = 10.36 Kg / cm^2

$\rho = 0.0025 > \rho_{min} = 0.002$

Req. $A_s = 0.0025(220)(50) = 22\ cm^2$

Use 11 Φ 16 $A_s = 22.12\ cm^2$ (In each way)

4-Development Length (L_d) :

$$L_d = \frac{420}{2\sqrt{30}} \times d_b = 61.34 \text{ cm}$$

85 – 8 cm = 77 > 61.34 cm OK

5-Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $A_s = 8.04\ cm^2$

Foundation Design (F 5) :

From Column (C 2-2):

Total dead load = 1519.46 KN = 151.95 Ton

Total live load = 478.19 KN = 47.82 Ton

Factored load = 1997.65 KN = 199.77 Ton

1 – Footing Area :

assume depth of the footing (H) = 50 cm

- Service Load = 151.95 / 1.4 + 47.82 / 1.7 = 136.7 Ton
- Footing Weight = (3)(3)(0.5)(2.4) = 10.8 Ton
- Earth Pressure =[(3)(3)-(0.5)(0.25)](0.6)(1.8) = 9.6 Ton
- Total Weight = 157.1 Ton

Area (A) =(157.1)(1000)Kg /3.5 Kg/cm^2 = 4.5 m^2

Use L = 2.2 m , W = 2.2 m , A = 4.84 m^2

2 – Shear Strength :

- Check this depth for one way action :

D =40

$P_{net} = \frac{P_u}{Area}$ = 199.76/4.84 = 41.3 Ton / m^2

$V_u = P_{net} \times d \times W$ = (41.3)(0.4)(2.2) = 36.34 Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d$ = 80.34 Ton

$\Phi V_c > V_u$ 68.3 Ton > 36.34 Ton OK

- Check this depth for two way action *(punching)* :

$V_u = P_{net} \times (\ (W) \times (L) - (a+d)(b+d)\ )$

Foundation Design (F 6):

From Column (C2 - 5):

Total dead load = 908.69 KN = 90.87 Ton

Total live load = 314.98 KN = 31.49 Ton

Factored load = 1223.67 KN = 122.4 Ton

1 – Footing Area :

assume depth of the footing (H) = 50 cm

- Service Load = 90.87 / 1.4 + 31.49 / 1.7 = 83.43 Ton
- Footing Weight = (3)(3)(0.5)(2.4) = 10.8 Ton
- Earth Pressure =[(3)(3)-(0.5)(0.25)](0.6)(1.8) = 9.6 Ton
- Total Weight = 103.83 Ton

Area (A) =(103.83)(1000)Kg /3.5 Kg/cm^2

= 2.96 m^2

Use L = 1.8 m , W = 1.8 m , A = 3.24 m^2

2 – Shear Strength :

- Check this depth for one way action :

D =40

$P_{net} = \dfrac{P_u}{Area} = 122.4/3.24 = 37.8$ Ton / m^2

$V_u = P_{net} \times d \times W = (37.8)(0.4)(1.8) = 27.22$ Ton

When No shear reinforcement is used :

$V_c = \dfrac{1}{6}\sqrt{f_c'}\, b_w d$ 65.75 Ton

$\Phi V_c > V_u$ 55.89 Ton > 27.22 Ton OK.

- Check this depth for two way action *(punching)* :

$V_u = P_{net} \times (\ (W) \times (L) - (a + d)(b + d)\ )$

$= 37.8\ [(1.8)(1.8) - (0.9)(0.65)] = 100.36$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \quad = 226.39$ Ton

where :

$\beta_c = a / b = 50 / 25 = 2$

$b_o = (0.9)(2) + (0.65)(2) = 3.1$ m

$\Phi V_c > V_u$ 192.43 Ton > 100.36 Ton OK

3 – Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$= \{37.8(1.8)(0.65)\} * (0.65)/2 = 14.37$ Ton.m

Mn = 32.82 / 0.9 = 36.47 Ton

Rn = 5.54 Kg / cm^2

$\rho = 0.00133 < \rho_{min} = 0.002$ Use $\rho_{min} = 0.002$

Req. $A_s = 0.002(180)(40) = 14.4\ cm^2$

Use 8 Φ 16 $A_s = 16.1\ cm^2$ (In each way)

4-Development Length (L_d) :

$$L_d = \frac{420}{2\sqrt{30}} \times d_b = 61.34\ cm$$

65 – 8 cm = 57 > 61.34 cm OK

5-Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $A_s = 8.04\ cm^2$

Foundation Design - (F7) :

From Column (C1 - 37): {square column=25cm , 25cm}

Total dead load = 1139.58 KN = 113.96 Ton

Total live load = 415.37 KN = 41.54 Ton

Factored load = 1554.95 KN 155.5 Ton

1 – Footing Area :

assume depth of the footing (H) = 50 cm

- Service Load = 113.96 / 1.4 + 41.54/ 1.7 = 813.98 Ton
- Footing Weight = (3)(3)(0.5)(2.4) = 10.8 Ton
- Earth Pressure =[(3)(3)-(.25)(0.25)](0.5)(1.8) = 8.1 Ton
- Total Weight = 124.73 Ton

Area (A) =(124.73)(1000)Kg /3.5 Kg/cm^2

= 3.563 m^2

Use L = 1.9 m , W = 1.9 m ; A = 3.61 m^2

2 – Shear Strength :

- Check this depth for one way action :

D =40

$P_{net} = \frac{P_u}{Area}$ = 155.5/3.61 = 43.1 Ton / m^2

$V_u = P_{net} \times d \times W$ = (43.1)(0.4)(1.9) = 32.735Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f_c'}\, b_w d$ =69.4 Ton

$\Phi V_c > V_u$ 58.99 Ton > 32.733 Ton OK.

☞ Check this depth for two way action *(punching)* :

$V_u = P_{net} \times \left((W) \times (L) - (a+d)(b+d) \right)$

$= 43.1\,[(1.9)(1.9) \quad (0.65)(0.65)] = 137.4$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{3}\sqrt{f_c'}\, b_o d \quad = 189.87$ Ton

where :

$\beta_c = a / b = 25 / 25 = 1$

$b_o = (0.65)(4) = 2.6$ m

$\Phi V_c > V_u$ 161.39 Ton > 137.4 Ton OK

3 – Bending Moment :

$$Mu = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2} \right) \right) \times 0.5 \left(\frac{L}{2} \times \frac{a}{2} \right)$$

$= \{43.1(1.9)(.825)\} * (.825)/2 = 27.9$ Ton.m

Mn = 27.9 / 0.9 = 31 Ton

Rn = 10.19 Kg / cm^2

$\rho = 0.0024 > \rho_{min} = 0.002$

Req. $A_s = 0.0024(170)(40) = 18.24\ cm^2$

Use 8 Φ 18 $A_s = 20.36\ cm^2$ (In each way)

4-Development Length (L_d) :

$$L_d = \frac{420}{2\sqrt{30}} \times d_b = 69 \text{ cm}$$

82.5 – 8 cm = 74.5 > 69 cm OK

5-Dwell Req.

Minimum Dwells Req. = 0.005 (50)(25) = 6.25 cm^2

Use 4 Φ 16 $A_s = 8.04\ cm^2$

6-6 Stair Design :

6-6-1 Rug Design

Loads:

Dead load :

Plaster = 0.03(22)(1) = 066 KN/m

Concrete = 0.15(24)(1)= 3.6 KN/m

Sand = 0.08(18)(1) = 1.44 KN/m

Tile = 0.02(30)(1) = 0.6 KN/m

Total dead load = 6.3 KN/m

Factored dead load = 1.4(6.3)

=8.82KN/m

live load:

live load = 5KN/m²

factored live load = 1.7(5) = 8.5 KN/m²

Total load = 17.32 KN/m² = 1.73 ton/m²

1- Determine thickness (h) :

Mu = Wl²/8 = 1.73(5.15)²/8 = 5.74 ton/m (for 1m wide strip)

Try ρ = 0.5ρmax =0.0133

m= 16.47

Rn=5.32 Mpa

Required Mn= 5.74/0.9 = 6.4 ton/m

bd ²=Mn/Rn

d = 0.11 m

d=11cm take h=15cm

If use Φ14:

d=15-{1.4/2+3} = 11.3 cm

2-Determine steel area (As):

AS=ρ(b)(d)

=0.0133(100)(11.3)

=15.1 cm^2/m

Use 10Φ14 =15.39 cm^2/m

3-Check shear requirement :

max shear= 0.5583(Wu)(Ln)

= 0.5583(1.73)(4.85)

= 4.7 ton (for 1m wide strip)

d = 11.3

ΦVc = 0.85(√30/6)(100)(11.3)(10)

= 8.77 ton

ΦVc > Vu

8.77ton > 4.7 ton OK

4- development length :

a)Ld for positive steel:

Ld for Φ14 bottoms bars:

Ld=53.67 cm

La=16.8 cm

Mn/Vu+La > Ld

6.4(100)/4.7 +16.8 >53.67

152.96 cm > 53.67 cm OK

b)Ld for negative steel

0.3Ln = 0.3(5.15-2(0.15)) = 1.455 m

Total length =1.455(2)+0.6 = 3.51

Use 3.6 cm

6-6-2 Stair Roof Deisgn :

Loads:

Dead load :

DL=0.15(24)(1)=3.6 KN/m =0.36ton/m

Factored dead load=1.4(0.36)=0.51 ton/m

live load:

live load = 1KN/m²

factored load = 1.7(1) = 1.7 KN/m²

Total load = 0.68 ton/m²

Mu=WL²/8 =0.68(5.5)²/8

= 2.57 ton/m {for 1m stirp)

1- Determine thickness (h) :

Try ρ = 0.5ρmax =0.0133

Use ρ=0.014

m= 16.47

Rn= ρFy(1-0.5 ρm)

Rn=5.2Mpa

Required Mn= 2.57/0.9 = 2.86 ton/m

bd ²=Mn/Rn

d = 0.0742 m

d=7.4cm take h=12cm

If use Φ14.

d=15-{1.4/2+4 cover} = 7.3 cm

2- Determine steel area (As):

As=ρ(b)(d)

= 0.0133(100)(7.3)

= 15.1 cm²/m

Use 7Φ14 =10.78 cm²/m

3- Check shear requirement :

max shear= 0.5583(Wu)(Ln)

=0.5583(0.68)(5.2)

=1.97 ton (for 1m wide strip)

d=7.3

ΦVc=0.85(√30/6)(100)(7.3)(10)

= 5.66 ton

ΦVc > Vu

5.66ton > 1.97 ton OK

4- development length :

a)Ld for positive steel:

Ld for Φ14 buttoms bars:

Ld=53.67 cm

La=16.8 cm

Mn/Vu+La > Ld

2.57(100)/1.97+16.8 >53.67

147.3 cm > 53.67 cm OK

b)Ld for negative steel

0.3Ln = 0.3(5.5-2(0.15)) = 1.56m

Total length =1.56(2)+0.6 = 3.72

Use 3.8cm

6-6-3 Stairs Step Design :

Loads:

Dead load :

plaster =0.03(22)(1)/cos 31 = 0.77 KN/m

Concrete = 0.15(24)(1)/cos31= 4.2 KN/m

Sand =0.08(18)(1) = 1.44 KN/m

Mortar =0.02(22)(1) = 0.44 KN/m

Tile =0.33/0.3{(0.03)(30)(1) = 0.99 KN/m

Steps =0.17(0.5)(24)(1)/cos31 =2.4 KN/m

Total dead load = 10.24 KN/m

Factored dead load = 1.4(10.24) = 14.4 KN/m

live load = 5 KN/m

factored live load = 1.7(5) = 8.5 KN/m

Mu = 46.28 KN.m

Mn = 46.2/0.9 = 51.4 KN.m = 5.14 Ton.m

M = 16.47

$\rho_{max.} = 0.023$

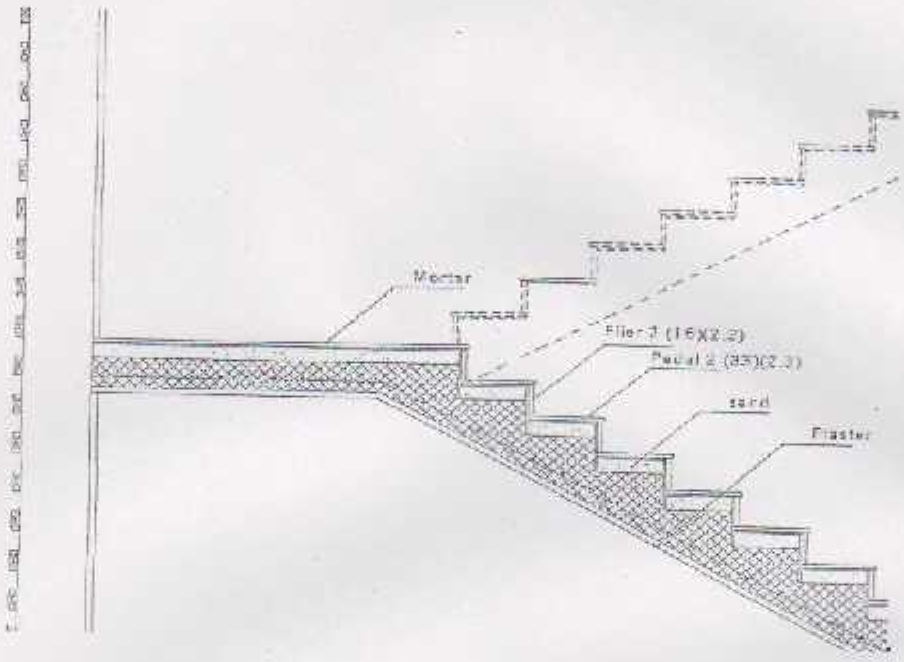
$\rho_{min.} = 0.0033$

$$Rn = \frac{Mn}{bd^2} = \frac{5.14(10)^5}{(100)(12)^2} = 35.7\,Kg/cm^2$$

$\rho = 0.009$ $\qquad 0.0033 \le 0.009 < 0.023$

As = 0.009(100)(12) = 10.8 cm^2/m

Use 7Φ14 = 10.8 cm^2/m



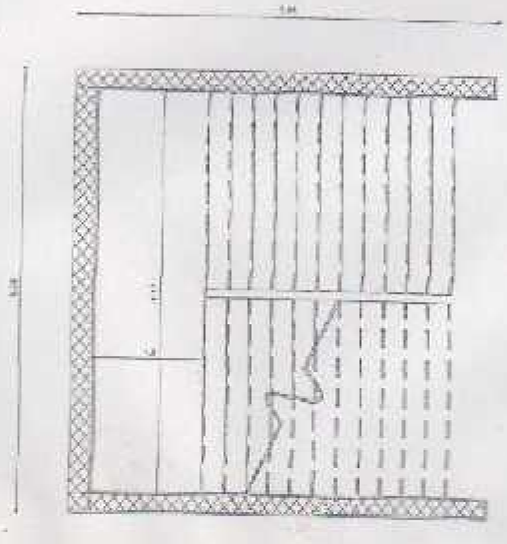


Fig.(6-50): Stair Shape

6-7 Design of Retaining wall & Continuous Footing

6-7-1 Retaining Wall :

Loads:

live load:

live load = 5KN/m²

factored load = 1.7(5) = 8.5 KN/m²

γsoil=18 KN/m³ (Unit weigt of the soil)

Φ=30°

From the software computer (mb program) :

Mu=45.28 KN.m /m

=4.53 ton.m / m

Mn=4.53/0.9 = 5.1 ton.m / m

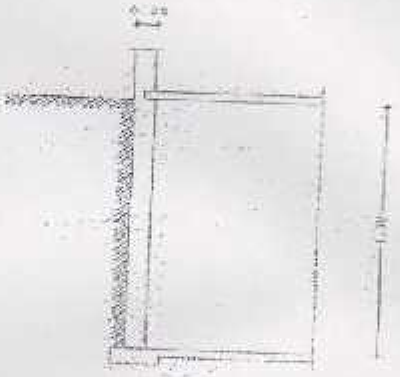


Fig.(6-51): Retaining Wall

I- Determine thickness (h)of retaining wall :

Try ρ = 0.5ρmax =0.0133

Use ρ=0.01

m= 16.47

Rn=3.68 Mpa

bd ²=Mn/Rn

d = 0.1177m

d=11.77 take h=25

If use Φ16:

d=25-(1.6/2+4 cover) = 20.2 cm

2- Determine steel area (As):

$As = \rho(b)(d)$

$= 0.01\ (100)(20.2)$

$= 20.2\ cm^2/m$

Use 10Φ16/m $= 20.2\ cm^2/m$

Vertical steel $= 20.2\ cm^2/m$

S = 10 cm

Horizontal steel = $7.7\ cm^2/m$ (5Φ12/m) S = 20 cm

3- Developement length :

Ld for Φ16:

Ld = 61.37 cm

Ld for Φ12:

Ld = 46 cm

6-7-2 Design of Continuous Footing

From retaining wall :

Total dead load = 2.4(0.25)(4) =24KN/m =2.4 ton/m

Factored dead load =1.4(2.4) =3.36 ton/m

live load = 5KN/ m^2

Factored live load =1.7(5) =8.5 KN/ m^2

1 – Pn on the footing

assume width of the footing (B) = 80 cm

Pu =1.4Dl +1.7LL

Pu =3.36 + 0.85 =4.21 ton/m

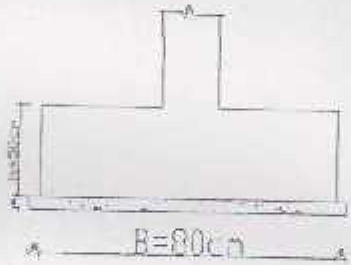


Fig.(6-52).Continuous Footing

$$P_{net} = \frac{P_u}{Area} = Pu/B(1m) = 4.21/0.8(1) = 5.5 \text{ ton/ } m^2$$

2 – Shear Strength :

Use thicknes of footing (h)=50cm d=40cm

$V_u = P_{net} \times d \times W = (5.5)(0.4)(1) = 2.2$ Ton

When No shear reinforcement is used :

$V_c = \frac{1}{6}\sqrt{f_c^{'}}\, b_w d = 36.52$ Ton

$\Phi V_c > V_u$ 31.1 Ton > 2.2 Ton OK.

3 – Bending Moment :

$$Mn = \left(P_{net} \times W \times \left(\frac{L}{2} \times \frac{a}{2}\right)\right) \times 0.5\left(\frac{L}{2} \times \frac{a}{2}\right)$$

$= \{5.5(1)(0275)\}*(0275)/2 = 0.21$ Ton.m

Mn = 0.21 / 0.9 = 0.231 Ton

$Rn = 0.18 \quad Kg/cm^2$

$. m = 16.47$

$\rho = 0.0000428 < \rho_{min} = 0.002$

Req. $A_s = 0.002\,(80)(40) = 6.4\ cm^2$

Use $5\,\Phi\,14$ $\qquad A_s = 7.7\ cm^2$

L_d for $\Phi 14 = 53.7$

Ch

7

Conclusions and Recommendations

CHAPTER 7

Conclusions And Recmmendationns

This final chapter is divided into two important sections in this project. One is to discuss the conclusions of campus and library design . other is for some recommmendations and some suggested ideas to devolp this project .

7.1 Summary and Conclusions

After reviewing a PPU campus and library design from both sides ,architectural and structural side, the following conclusions are recorded:

1. In campus analytical and architectural design ,the current needed areas of PPU campus is amount 46400 m^2 and the required area for the coming 10 years is estimated to be 64400 m^2.

2. This project concentrated to designing a site planning for campus of PPU gathering all requirments faculties's sites and planning architectural and structural design for main library at PPU .

3. Two stages for campus project execusion . Stage one is in the first five years of project time to design and constract four current collages and stage two is the final image of PPU campus .the number of students will reach to 10000 students with required areas equal to 64400 m^2.

5. Also structural requirments were taken in mind such as cracking and fire- resistance requirments to begin a sturactural analyses of members.

6. all of structural members were designed from roof to foundations after strucural analyses of it.

7. the architectural idea in this project is that project of library design is a new kind of architectural design of building in campus of PPU and it is a central building in campus with very considerable requirments .

8. the structural analyses of library is suitable and similar to architectural design ,that the structural members are designed according the architectural design to cover all of needs of university such as distance between columns determined according the spaces needed for readers and books shelfs.

7.2 Recommendations

After ending of this project, the following recommendations must be considered :

- Because the importance of PPU campus design for PPU and UGU and for covers all of society 's needs, it must do another analytical studies for PPU campus site to deal with all expected future needs.

- After designing the library building which is a central part of PPU campus. it is imprtant to design all parts of campus including colleges, sport areas, workshops,and others.

- The library was designed for architecural and structural analyses requirments.So, electrical amd mechnical requirments of librry must complete to satisfy all of its function

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APPENDICES

Appendix (A)

Appendix (B)

Appendix (C)

Appendix (D)

Appendix (E)

APPENDIX (A)

Applied Profissional College

No.	Phase 1	Phase 2
No. of Student	1520	2370
No. of Specialization	26	29
No. of Instructors	103	116

Type of Space	No.	Area	Total Area	No.	Area	Total Area
Headmaster Office	1	30	30	1	30	30
Secretary Office	1	12	12	1	12	12
Committee Room	53	20	1060	60	20	1200
Private Library for College	1	200	200	1	200	200
Lecture Room Type'A'	25	48	1200	29	48	1392
Lecture Room Type'B'	0	80	0	0	80	0
Drawing Room	0	75	0	2	75	150
Computer Lab. Type'A'	2	60	120	2	60	120
Computer Lab. Type'B'	0	0	0	0	0	0
Practice Lab.	9	80	720	9	80	720
Workshops Type'A'	4	100	400	4	100	400
Workshops Type'B'	1	360	360	1	360	360
Wood Workshop	3	60	180	3	60	180
Exhibition Room	1	120	120	1	120	120
Arsheef Room	1	12	12	1	12	12
Reception Room	1	30	30	1	30	30
Stores	2	30	60	3	30	90
Sanitary (Committee)	3wc+4u		45	3+6		45
Sanitary (Male)	40		120	45		140
Sanitary (Female)	20		100	20		100
Corresponder	1	8	8	1	8	8
Photocpier Room	1	9	9	1	9	9
Cafeteria	1	80	80	1	80	80
Hall	1	150	150	1	150	150
Total Area			5016			5548

Applied Science College

No.	Phase 1	Phase 2
No. of Student	755	1100
No. of Specialization	4	6
No. of Instructors	51	72

Type of Space	No.	Area	Total Area	No.	Area	Total Area
Headmaster Office	1	30	30	1	30	30
Secretary Office	1	12	12	1	12	12
Committee Room	26	20	520	36	20	720
Private Library for College	1	200	200	1	200	200
Lecture Room Type'A'	15	48	720	22	48	1056
Lecture Room Type'B'	4	80	320	5	80	400
Drawing Room	0	75	0	0	75	0
Computer Lab. Type'A'	1	60	60	1	60	60
Computer Lab. Type'B'	2	75	150	4	75	300
Practice Lab	2	75	150	2	75	150
Arsheef Room	1	12	12	1	12	12
Reception Room	1	30	30	1	30	30
Stores	1	30	30	3	30	90
Sanitary (Committee)	3wc+6u		50	6wc+10u		100
Sanitary (Male)	24		75	30		110
Sanitary (Female)	14		75	20		110
Corresponder	1	8	8	1	8	8
Dark Room	1	75	75	1	75	75
Photocpier Room	1	9	9	1	9	9
Cafeteria	1	80	80	1	80	80
Hall	1	150	150	1	150	150
Total Area			2756			3702

Management Information College

No.	Phase 1	Phase 2
No. of Student	1260	1320
No. of Specialization	9	11
No. of Instructors	68	83

Type of Space	No.	Area	Total Area	No.	Area	Total Area
Headmaster Office	1	30	30	1	30	30
Secretary Office	1	12	12	1	12	12
Committee Room	34	20	680	42	20	840
Private Library for College	1	200	200	1	200	200
Lecture Room Type'A'	20	48	960	25	48	1200
Lecture Room Type'B'	6	80	480	7	80	560
Drawing Room	0	75	0	0	75	0
Computer Lab. Type'A'	2	90	180	3	90	270
Computer Lab. Type'B'	7	75	525	8	75	600
Arsheef Room	1	12	12	1	12	12
Reception Room	1	30	30	1	30	30
Stores	2	30	60	2	30	60
Sanitary (Committee)	4wc+8u		60	5wc+10u		75
Sanitary (Male)	40		120	50		150
Sanitary (Female)	24		120	30		150
Dark Room	1	75	75	1	75	75
Corresponder	2	8	16	3	8	24
Photocpier Room	1	9	9	1	9	9
Cafeteria	1	80	80	1	80	80
Hall	1	150	150	1	150	150
Total Area			3799			4527

Engineering College

No.	Phase 1	Phase 2
No. of Student	1180	1500
No. of Specialization	10	13
No. of Instructors	78	96

Type of Space	No.	Area	Total Area	No.	Area	Total Area
Headmaster Office	3	30	90	3	30	90
Secretary Office	3	12	36	3	12	36
Committee Room	40	20	800	48	20	960
Private Library for College	3	200	600	3	200	600
Lecture Room Type'A'	27	48	1296	36	48	1728
Lecture Room Type'B'	4	80	320	4	80	320
Drawing Room	6	75	450	8	75	600
Computer Lab. Type'A'	4	60	240	4	60	240
Computer Lab. Type'B'	16	90	1440	19	90	1710
Computer Lab. Type'C'	0	0	0	1	120	120
Computer Lab. Type'D'	1	170	170	1	170	170
Special Computer Lab	0	0	0	1	385	385
Practice Lab Type'A'	9	60	540	9	60	540
Practice Lab Type'B'	2	130	260	2	130	260
Practice Lab Type'C'	3	180	540	3	180	540
Exhibition Type'A'	1	60	60	1	60	60
Exhibition Type'B'	1	120	120	1	120	120
Milestone Workshop	1	75	75	1	75	75
Arsheef Room	3	12	36	3	12	36
Reception Room	3	30	90	3	30	90
Stores	4	30	120	5	30	150
Sanitary (Committee)	4wc+8u		60	6wc+10u		100
Sanitary (Male)	40		120	50		150
Sanitary (Female)	24		120	30		150
Corresponder	3	8	24	3	8	24
Dark Room	1	75	75	1	75	75
Photocopier Room	1	9	9	1	9	9
Cafeteria	1	80	80	1	80	80
Hall	1	150	150	1	150	150
Total Area			7921			9568

عدد الطلبة المتوقع بجامعة بوليتكنيك فلسطين خلال الأعوام القادمة

الطريقة الحسابية البسيطة

تعتبر هذه الطريقة سهلة وبسيطة للتنبؤ بأعداد الطلبة، وحتى يمكن تطبيق هذه الطريقة فإنه يلزم معرفة عدد الطلبة في العام الحالي وعدد الطلبة الذي تحقق في العام الماضي، ويتم الاحتساب كما يلي:

$$\text{عدد الطلبة للعام القادم} = \text{عدد طلبة العام الحالي} \times \frac{\text{عدد طلبة العام الحالي}}{\text{عدد طلبة العام الماضي}}$$

$$\text{عدد طلبة عام } 2002/2001 = \text{عدد طلبة عام } 2001/2000 \times \frac{\text{عدد طلبة عام } 2001/2000}{\text{عدد طلبة عام } 2000/1999}$$

$$= 1928 \times (1928 \div 1826)$$

$$= \underline{2036} \text{ طالب}$$

العـــــام	عدد الطلبة المتوقع	طريقة الاحتساب	ملاحظة رقم
2001/2000	1928	العدد الحالي	(1)
2002/2001	2036	(1826 ÷ 1928) × 1928	
2003/2002	2150	(1928 ÷ 2036) × 2036	
2004/2003	2270	(2036 ÷ 2150) × 2150	
2005/2004	2397	(2150 ÷ 2270) × 2270	
2006/2005	2531	(2270 ÷ 2397) × 2397	
2007/2006	3164	زيادة بنسبة %025	(2)
2008/2007	3417	زيادة بنسبة %08	(3)
2009/2008	3690	=	
2010/2009	3985	=	
2011/2010	4305	=	
2012/2011	4649	=	
2013/2012	5114	زيادة بنسبة %010	(4)
2014/2013	5625	=	
2015/2014	6188	=	
2016/2015	6807	=	
2017/2016	7487	=	
2018/2017	8236	=	
2019/2018	9060	=	
2020/2019	10000	=	

ملاحظة (1): عدد طلبة الجامعة المتوقع للسنوات 2000 – 2006 (حتى اكتمال مشروع البناء):
زيادة سنوية بمعدل 5.6%.

ملاحظة (2): عدد طلبة الجامعة المتوقع لسنة 2006/2007 (عام اكتمال مشروع البناء):
يتوقع أن تطرأ زيادة كبيرة في أعداد الطلبة قد تصل الى 25% وذلك بسبب توفر المرافق الحديثة والمباني ذات الطاقة الاستيعابية الأكبر وما قد يرافق ذلك من فتح لتخصصات جديدة ومتعددة، وهذا في مجمله سيعمل بشكل شبه مؤكد على استقطاب أعدادا أكبر من الطلبة للالتحاق بالجامعة.

ملاحظة (3): عدد طلبة الجامعة المتوقع للسنوات 2008 – 2012 (بعد اكتمال مشروع البناء):
يتوقع أن تطرأ زيادة سنوية في أعداد الطلبة بمعدل 8% وذلك اذا ما تم التخطيط والانفتاح لفتح كليات جديدة تكون قادرة على استقطاب أعدادا إضافية من الطلبة الراغبين في الالتحاق بالجامعة في تلك الكليات.

ملاحظة (4): عدد طلبة الجامعة المتوقع للسنوات 2013-2020 :
يتوقع أن تطرأ زيادة إضافية في أعداد الطلبة لتصل الى 10% وذلك إذا ما تم طرح برامج تؤدي الى الحصول على درجات الماجستير والدكتوراة في بعض التخصصات وما يرافق ذلك من زيادة عامة في أعداد الطلبة حيث أن ذلك من شأنه استقطاب بعض الطلبة الذين يفكرون في الحصول على شهادات عليا للالتحاق من البداية في الجامعة لان ذلك سيسهل عملية قبولهم واستكمالهم للدراسة في الجامعة لنيل الدرجات العلمية العليا التي يطمحون لها.

طريقة السلاسل الزمنية:

العـــــام	التغيـــــر	معـــدل التغيـــر
1994/1993	صفر	صفر
1995/1994	127	%21
1996/1995	142	%19
1997/1996	204	%23
1998/1997	433	%40
1999/1998	195	%13
2000/1999	115	%7
2001/2000	102	%6
المجمـــــوع		%129
معـــدل التغيـــر العـــام		%16

عدد الطلبة المتوقع لعام 2002/2001 =

(معدل التغير العام × عدد طلبة عام 2001/2000) + عدد طلبة عام 2001/2000

= (1928 × %16) + 1928

= 2236 طالبا

أو

بإضافة 100% الى معدل التغير العام وضربه بعدد طلبة عام 2001/2000

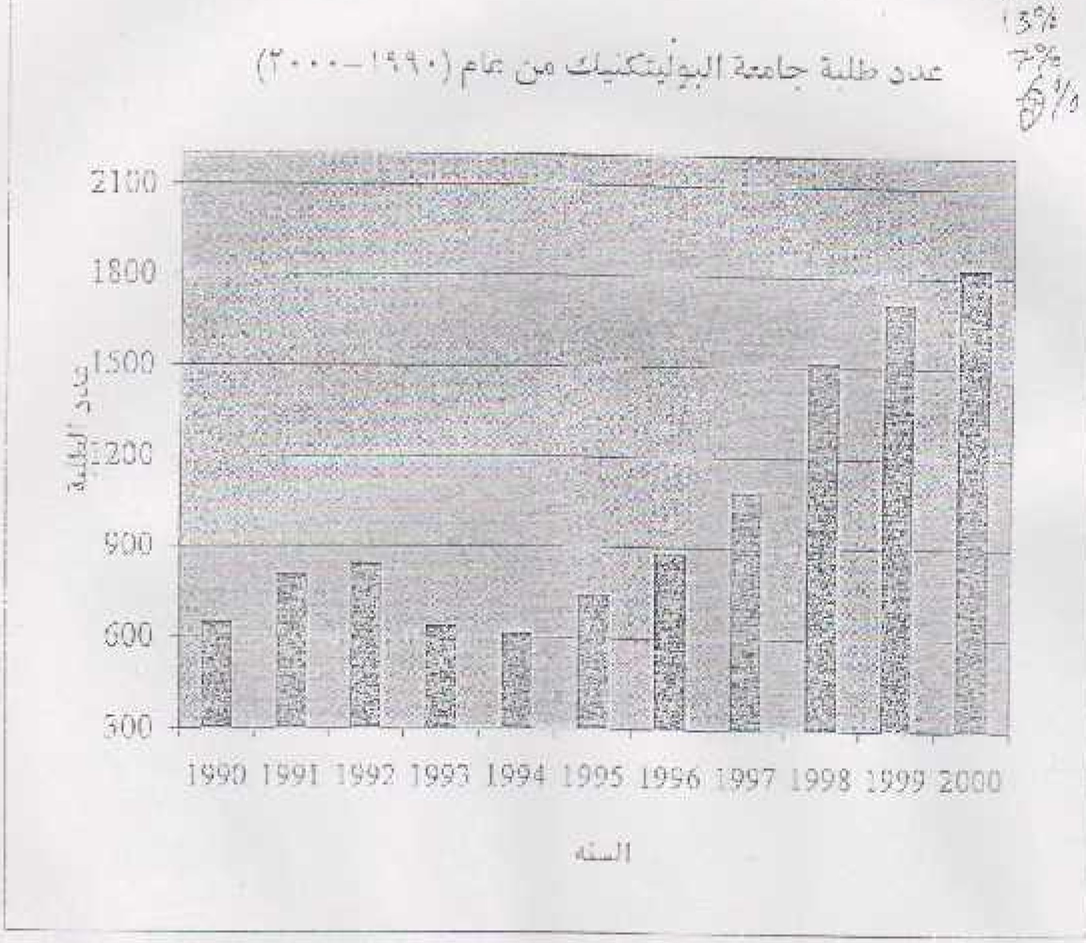
طريقة الاحتساب	عدد الطلبة المتوقع	العام
العدد الحالي	1928	2001/2000
%116 × 1928	2236	2002/2001
%116 × 2236	2594	2003/2002
%116 × 2594	3009	2004/2003
%16 × 3009	3490	2005/2004
%116 × 3490	4048	2006/2005
%116 × 4048	4696	2007/2006
%116 × 4696	5447	2008/2007
%116 × 5447	6319	2009/2008
%116 × 6319	7330	2010/2009
%116 × 7330	8503	2011/2010
%116 × 8503	9863	2012/2011
%116 × 9863	11441	2013/2012
%116 × 11441	13272	2014/2013
%116 × 13272	15396	2015/2014
%116 × 15396	17859	2016/2015
%116 × 17859	20717	2017/2016
%116 × 20717	24032	2018/2017
%116 × 24032	27877	2019/2018
%116 × 27877	32337	2020/2019

24%	651	1990
5%	806	1991
-24%	842	1992
-2.5%	635	1993
+21%	613	1994
+19%	740	1995
23%	882	1996
40%	1086	1997
13%	1516	1998
7%	711	1999
6%	826	2000

-25%

16

340 2001



2255610

MONTHLY CLIMATIC AVERAGES

المعدلات المناخية الشهرية ([illegible])

Station: Hebron

الخليل

Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Max. Temp. (°C)	10.2	11.5	14.6	19.6	23.6	25.9	27.2	27.2	26.0	23.2	17.5	12.1
Mean Min. Temp. (°C)	4.0	4.7	6.5	9.9	13.2	15.8	17.0	17.0	15.9	14.0	9.9	5.6
Absolute Max. Temp. (°C)	21.4	21.0	23.6	32.6	34.0	33.5	38.0	33.4	34.6	31.6	31.6	22.0
Absolute Min. Temp. (°C)	-1.0	-3.0	-0.5	1.0	6.5	10.0	13.0	12.0	12.0	9.0	2.0	-0.4
Mean Temp. (°C)	7.1	8.1	10.5	14.7	18.4	20.8	22.1	22.1	20.9	18.6	13.7	8.8
Mean Wind Speed (Km/h)	12.4	12.8	12.6	11.5	9.3	9.3	9.2	8.7	8.1	8.0	8.8	10.1
Pressure (mbar)	903	902	901	901	901	900	899	899	902	903	904	904
Mean Sunshine Duration (h/day)	4.7	4.8	6.4	8.1	9.0	8.3	9.6	10.9	10.3	9.8	7.0	4.7
Mean RH %	74	72	66	55	48	51	57	60	62	59	64	73
Total Rainfall (mm) *	133.6	141.6	91.7	25.4	4.7	0.5	0.0	0.0	1.6	14.6	66.7	115.5
Total Evaporation (mm) *	65	81	93	139	166	200	221	225	157	112	87	62
Total PET (mm) *	23	25	41	67	97	106	110	106	94	87	54	30
Max. Monthly Rainfall (mm)	351	335	194	235	37	10	0	0	21	65	220	334

* Monthly Total

جدول تقييم المواقع الممكنة:

المعايير	الوزن	موقع وادي الأورية	موقع جحجول ١	موقع طحجول ٢	موقع بئر حرم الرابع	موقع تورا	موقع تل بعير حاجول
كلفة الأرض المتوفرة	١٢	١,٢	١٢	٦	٠,٦	٤	٢,٥
الاستثمار الموجود	٥	٥	٠	٠	٠	٠	٠
الخدمات و توفرها	١٨	١٨	٠	٤,٥	١٨	١٢	١٦
الاستثمار المستقبلي	٥	٠	٥	٥	٠	٣	٥
المجموع	٤٠	٢٤,٢	١٧	١٥,٥	١٨,٦	١٩	٢٣,٥
٢_ المؤشرات الفرعية ٦٠ %							
توفر المساحة المطلوبة	٨	٥	٨	٨	٦,٥	٨	٨
اتصال قطعة الأرض و القدرة على تجميع الحرم الجامعي	١٢	٣	١٢	١٢	١٠	١٢	١٢
سهولة و سيولة المواصلات	١٥	٨	٥	٦	١٤	٧	١٤
الناحية البيئية	٨	٣,٥	٨	٨	٧	٨	٨
شكل و أبعاد قطعة الأرض	١٠	٤	٧	٩	٩,٥	٦	٩
الموقع الجغرافي	٧	٣,٥	٥	٦	٧	٥	٧
المجموع	٦٠	٢٧	٤٥	٤٩	٥٤	٤٦	٥٨

- استناداً الى ما سبق قد رأت اللجنة لاختيارة موقع طحجول (تل بعير) لحصوله على أكبر مجموع للنقاط. يتبعه في المركز الثاني من حيث الأفضلية موقع حرم الرابع.

Appendix (B)

Table IV Parking and loading/unloading requirements

Type of building	Car parking provision	Loading/unloading provision	Cycle parking
Normal housing	Residents: one garage space for each occupancy, (preferably within the curtilage) Visitors: where houses are served directly from a road, driveways provide a minimum of one car space within curtilage of each Where visitors cannot park within curtilage, one off-street space per four dwellings	Refuse collection vehicle within 25 m of each disposal point (dustbin position). Some authorities require vehicle within 12 m. Where communal containers (paladins) are used, maximum distance 9 m Furniture removal vehicle as near as possible, not further than 25 m	
Minimum-cost housing	Space should be provided, if not laid out, to allow for one resident's or visitor's parking space per dwelling, provided public transport is available	As above	
Old people's housing	One garage space per two dwellings	As above	
Sheltered housing	Resident and non-resident staff: one car space per two members present at peak period Visitors: use empty staff places, but provide one additional place per five dwellings	As above, plus provision for special passenger vehicle with tail lift, etc. Minimum provision for daily loading/unloading 50 m^2	
Shops	Staff: one car space (preferably in enclosed yard behind shop) for each 100 m^2 gross floor area or, if known, one space per managerial staff plus one for every four other staff Customers: one space for each 25 m^2 gross floor area. In superstores with gross floor area exceeding 2000 m^2 allow one space per 10 m^2. (Not appropriate when goods sold are obviously bulky, e.g. carpets, boats)	See diagrams of loading bays General minima as follows: Gross floor space not exceeding: 500 m^2 – Minimum space required: 50 m^2 1000 – 100 2000 – 150 each additional 1000 m^2 – 50 m^2	1 per 300 m^2 with minimum of 4
Banks	Staff: one space for each managerial or executive staff, plus one per four others Customers: one space per 15 m^2 of net public floor space in banking hall	Minimum 25 m^2	2
Offices	Staff: one space for each 25 m^2 of gross floor area, or one space for each managerial and executive staff, plus one space per four others Visitors: 10% of staff parking provision	General minima: Gross floor space not exceeding: 100 m^2 – Minimum space required: 50 m^2 500 – 100 1000 – 150 each additional 1000 m^2 – 25 m^2	1 per 200 m^2 with minimum of 4
Production buildings (factories)	Staff: one car space per 50 m^2 of gross floor area Visitors: 10% of staff parking provision	See loading bay diagram. Provision to be commensurate with expected traffic	1 per 500 m^2 with minimum of 4
Storage buildings (warehouses)	Staff: one space per each 200 m^2 of gross floor space	General minima as follows: Gross floor space not exceeding: 100 m^2 – Minimum space required: 70 m^2 250 – 140 500 – 170 1000 – 200 2000 – 300 each additional 1000 m^2 – 50 m^2	1 per 1000 m^2 with minimum of 4
Hotels, motels and public houses	Resident staff: one space per household Non-resident staff: one space for each three staff members employed at peak period Resident guests: one space per bedroom Bar customers: one space for each 4 m^2 of net public space in bars Occasional diners: no additional provision required If conferences are held in the hotel, space required should be assessed separately at one space for each five seats	General minima as follows: Gross floor space not exceeding: 500 m^2 – Minimum space required: 140 m^2 1000 – 170 2000 – 200 each additional 1000 m^2 – 25	1 per 10 beds with minimum of 4
Restaurants and cafés	Resident staff: one space per household Non resident staff: one space per three members employed at peak period Diners: one space for each two seats in dining area. (For transport cafés, the space should be a lorry space of 45 m^2, and the arrangement should be such that vehicles can enter and leave without reversing)	General minima as follows: Dining floor space not exceeding: 100 m^2 – Minimum space required: 50 m^2 250 – 75 500 – 100	1 per 25 m^2 with minimum of 4
Licensed clubs	Resident staff: one space per household Non-resident staff: one space for each three members employed at peak period Performers: one space for each solo performer and/or group expected at peak Patrons: one space per two seats, or one space per 4 m^2 net public floor space	Minimum 50 m^2	1 per 25 m^2 with minimum of 4

Table IV (continued)

Type of building	Car parking provision	Loading/unloading provision	Cycle parking
Dance halls and discotheques	Staff: one space per three members at peak period Performers: three spaces Patrons: one space per 10 m^2 of net public floor space	Minimum 50 m^2	1 per 25 m^2 with minimum of 4
Cinemas	Staff: one space per three members at peak period Patrons: one space per 5 seats	Minimum 50 m^2 Space required within site by main entrance for two cars to pick up and set down patrons	1 per 100 seats with minimum of 4
Theatres	Staff: one space per three members at peak period Performers: one space per 10 m^2 of gross dressing room accommodation Patrons: one space for each three seats	Minimum 100 m^2 Space required within site by main entrance for two cars to pick up and set down patrons	1 per 100 seats with minimum of 4
Swimming baths	Staff: one space for every two members normally present Patrons: one space per 10 m^2 pool area	Minimum 50 m^2	1 per 4 staff
Sports facilities and playing fields	Staff: one space per three members normally present Players: one space for each two players able to use the facility simultaneously, provided public transport is reasonably close. Otherwise two spaces for each three players Spectators: provide only if more than three times the number of players	Minimum 50 m^2	1 per 4 staff
Marinas	Staff: one space per three members normally present Boat-users: two spaces for each three mooring-berths. (If other facilities are included, eg restaurant, shop etc, provide additional spaces at 50% normal provision for each additional facility)	Minimum 50 m^2	1 per 4 staff
Community centres and assembly halls	Staff: one space for each three members normally present Patrons: one space for every five seats for which the building is licensed	Minimum 50 m^2	1 per 4 staff
Places of worship	Worshippers: one space per ten seats in space for worship	Minimum 50 m^2 Space provided within site close to main entrance for two cars to set down and pick up worshippers	1 per 60 seats minimum 4
Museums and public art galleries	Staff: one space per two members normally on duty Visitors: one space per 30 m^2 of public display space	Minimum 50 m^2	1 per 300 m^2 minimum 4
Public libraries	Staff: one space per three members normally on duty Borrowers: one space for each 300 adult ticket holders with a minimum of three spaces. If there are separate reference facilities, provide additional spaces at one for each ten seats	Minimum 50 m^2 If used as a base for a mobile library, provide another 50 m^2 to park this	1 per 100 m^2 minimum 4
Hospitals	Staff: one space for each doctor and surgeon, plus one space for each three others Outpatients and visitors: one space for each three beds	General minima as follows: Gross floor space not exceeding / Minimum space required: 1000 m^2 — 200 m^2 2000 — 300 4000 — 400 5000 — 500 every additional 2000 m^2 — 100 m^2	1 per 12 beds
Health centres, surgeries, clinics	Staff: one space per doctor etc one space per two members of staff other than doctors etc employed at peak period Patients: two spaces per consulting room	Sufficient for requirements specified, including if necessary space for special vehicle for non-ambulant patients	4
Special schools, day-care centres and adult training centres	Staff: one space for each two members normally present Attenders: in many cases these will be transported to the centre. For certain centres for the physically handicapped, allow one space for special or adapted self drive vehicle per four attenders	Minimum 30 m^2 Accommodation for special passenger vehicle Space provided within the site for cars and/or buses to set down and pick up	1 per 6 staff
Nursery and primary schools	Staff: one space per two members normally present Visitors: two spaces Hard surface play area used for parking on open days etc	Minimum 50 m^2	1 per 6 staff
Secondary schools	Staff: one space per two members normally present Visitors: schools with up to 1000 pupils – four spaces, larger schools – eight spaces	Minimum 50 m^2 Space provided within site for school buses to set down and pick up	

Table IV (continued)

Type of building	Car parking provision	Loading/unloading provision	Cycle parking
Sixth form colleges	Staff: one space per two members normally present Visitors: colleges with up to 1000 pupils – five spaces; larger schools – ten Hard surface play area used for parking on occasion	Minimum 50 m²	1 per 6 staff 1 per 3 students
Further education colleges and retraining centres	Staff: one space for each member normally present Students and visitors: one space for each three students normally present	Minimum 50 m²	1 per 6 staff 1 per 3 students

Appendix (C)

Minimum uniformly distributed live loads

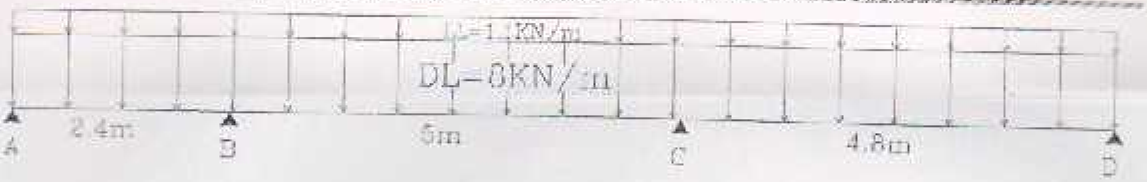
Occupancy or Use	Live Load (KN/m^2)	Occupancy or Use	Live Load (KN/m^2)
Residential :		Hospitals :	
Private apartment	1.9	Operating rooms	2.9
Public rooms	4.8	Private rooms	1.9
Corridors	2.9	Wards	1.9
Armories & Drill rooms	7.2	Balcony	4.8
Assembly halls & other		Garages (passenger cars)	4.8
Fixed seat	2.9	Floors:150%of the max.	
Movable seat	4.8	wheel	
Corridors	4.8	Dining rooms & restaurants	4.8
Dance halls	4.8	Dwellings	1.9
Hotels : Guest rooms	1.9	Penal institutions :	
Public rooms	4.8	Cell blocks	1.9
Corridors	4.8	Corridors	4.8
Libraries : Reading rooms	2.9	Office building : Offices	3.8
Stack rooms	7.2	Lobbies	4.8
Manufacturing	6.0	Marquees	3.6
Schools : Class rooms	1.9	Storage warehouse : Light	6.0
Corridors	4.8	Heavy	12.0

Table 1: Total Areas for Various Numbers of Reinforcing Bars

Diameter (mm)	Weight (kg/m)	Total Area for Shown Number of Bars (cm^2)									
		1	2	3	4	5	6	7	8	9	10
6	0.211	0.28	0.57	0.85	1.13	1.41	1.70	1.98	2.26	2.54	2.83
8	0.377	0.50	1.01	1.51	2.01	2.51	3.02	3.52	4.02	4.52	5.03
10	0.596	0.79	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85
12	0.852	1.13	2.26	3.39	4.52	5.65	6.79	7.92	9.05	10.18	11.31
14	1.160	1.54	3.08	4.62	6.16	7.70	9.24	10.78	12.32	13.85	15.39
16	1.520	2.01	4.02	6.03	8.04	10.05	12.06	14.07	16.08	18.10	20.11
18	1.920	2.54	5.09	7.63	10.18	12.72	15.27	17.81	20.36	22.90	25.45
20	2.370	3.14	6.28	9.42	12.57	15.71	18.85	21.99	25.13	28.27	31.42
22	2.870	3.80	7.60	11.40	15.21	19.01	22.81	26.61	30.41	34.21	38.01
24	3.410	4.52	9.05	13.57	18.10	22.62	27.14	31.67	36.19	40.72	45.24
25	3.700	4.91	9.82	14.73	19.63	24.54	29.45	34.36	39.27	44.18	49.09
26	4.000	5.31	10.62	15.93	21.24	26.55	31.86	37.17	42.47	47.78	53.09
28	4.650	6.16	12.32	18.47	24.63	30.79	36.95	43.10	49.26	55.42	61.58
30	5.330	7.07	14.14	21.21	28.27	35.34	42.41	49.48	56.55	63.62	70.69
32	6.070	8.04	16.08	24.13	32.17	40.21	48.25	56.30	64.34	72.38	80.42
34	6.860	9.08	18.16	27.24	36.32	45.40	54.48	63.55	72.63	81.71	90.79
35	7.260	9.62	19.24	28.86	38.48	48.11	57.73	67.35	76.97	86.59	96.21
36	7.990	10.18	20.36	30.54	40.72	50.89	61.07	71.25	81.43	91.61	101.79
40	9.470	12.57	25.13	37.70	50.27	62.83	75.40	87.96	100.53	113.10	125.66
50	14.800	19.63	39.27	58.90	78.54	98.17	117.81	137.44	157.08	176.71	196.35
60	21.300	28.27	56.55	84.82	113.10	141.37	169.65	197.92	226.19	254.47	282.74

Manual Structural Analysis for rib (2):

Case (1) :



Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=9.1 KN/m LL=2.4m

WR=9.1KN/m LR=5m

$M_L L_L + 2M_C(L_R + L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-31.45 -284.37

14.8Mb+5Mc=-315.83 ....(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=9.1 KN/m LL=5m

WR=9.1KN/m LR=4.8m

5Mb+19.6Mc = -535.97 ....(2)

By solving equation :

Mc=-23.98 KN.m

Mb=-13.24

Ay=5.4 KN

By=37.01 KN

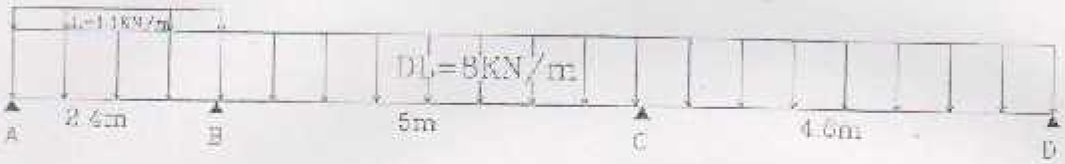
Cy=51.74 KN

Dy=16.84 KN

Table 2: Minimum Beam Width (cm) According to the ACI Code

Diameter (mm)	Number of Bars in Single Layer of Reinforcement								Add for Each Added bar
	2	3	4	5	6	7	8	9	
6	17.10	20.20	23.30	26.40	29.50	32.60	35.70	38.80	3.10
8	17.30	20.60	23.90	27.20	30.50	33.80	37.10	40.40	3.30
10	17.50	21.00	24.50	28.00	31.50	35.00	38.50	42.00	3.50
12	17.70	21.40	25.10	28.80	32.50	36.20	39.90	43.60	3.70
14	17.90	21.80	25.70	29.60	33.50	37.40	41.30	45.20	3.90
16	18.10	22.20	26.30	30.40	34.50	38.60	42.70	46.80	4.10
18	18.30	22.60	26.90	31.20	35.50	39.80	44.10	48.40	4.30
20	18.50	23.00	27.50	32.00	36.50	41.00	45.50	50.00	4.50
22	18.70	23.40	28.10	32.80	37.50	42.20	46.90	51.60	4.70
24	18.90	23.80	28.70	33.60	38.50	43.40	48.30	53.20	4.90
25	19.00	24.00	29.00	34.00	39.00	44.00	49.00	54.00	5.00
26	19.20	24.40	29.60	34.80	40.00	45.20	50.40	55.60	5.20
28	19.60	25.20	30.80	36.40	42.00	47.60	53.20	58.80	5.60
30	20.00	26.00	32.00	38.00	44.00	50.00	56.00	62.00	6.00
32	20.40	26.80	33.20	39.60	46.00	52.40	58.80	65.20	6.40
34	20.80	27.60	34.40	41.20	48.00	54.80	61.60	68.40	6.80
35	21.00	28.00	35.00	42.00	49.00	56.00	63.00	70.00	7.00
36	21.20	28.40	35.60	42.80	50.00	57.20	64.40	71.60	7.20
40	22.00	30.00	38.00	46.00	54.00	62.00	70.00	78.00	8.00
50	25.00	35.00	45.00	55.00	65.00	75.00	85.00	95.00	10.00
60	28.00	40.00	52.00	64.00	76.00	88.00	100.00	112.00	12.00

Case (2) :



Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=9.1 KN/m LL=2.4m

WR=8 KN/m LR=5m

$M_L L_L + 2M_C(L_L + L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-281.45 ....(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=8 KN/m LL=5m

WR=8 KN/m LR=4.8m

5Mb+19.6Mc = -472.1 ....(2)

By solving equation

Mc=-21.5 KN.m

Mb=-11.7

Ay=6.08 KN

By=33.8 KN

Cy=39.6 KN

Dy=17.8 KN

Case (3)

Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=8 KN/m LL=2.4m

WR=8 KN/m LR=5m

$M_L L_L + 2M_c(L_R + L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-277.65 ....(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=8 KN/m LL=5m

WR=9.1 KN/m LR=4.8m

5Mb+19.6Mc = -501.61 ....(2)

By solving equation:

Mc=-22.76 KN.m

Mb=-11.1

Ay=4.97 KN

By=31.9 KN

Cy=48.9 KN

Dy=17.1 KN

Case (4) :

Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=8 KN/m LL=2.4m

WR=9.1 KN/m LR=5m

$M_L L_L + 2M_C(L_R+L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-312.03 ....(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=9.1 KN/m LL=5m

WR=8 KN/m LR=4.8m

5Mb+19.6Mc = -505.6 ....(2)

By solving equation:

Mc=-22.35 KN.m

Mb=-13.5

Ay=3.97 KN

By=36.2 KN

Cy=48.4 KN

Dy=14.54 KN

Case (5) :

Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=9.1 KN/m LL=2.4m

WR=9.1 KN/m LR=5m

$M_L L_L + 2M_C(L_R + L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-315.83 ....(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=9.1 KN/m LL=5m

WR=8 KN/m LR=4.8m

5Mb+19.6Mc = -505.6 ....(2)

By solving equation:

Mc=-22.3 KN.m

Mb=-13.76

Ay=5.2 KN

By=37.7 KN

Cy=48.3 KN

Dy=14.55 KN

Case (6) :

Structural Analysis By (TRE MOMENT EQUASTION):

Span ABC :

ML=0 MC=Mb MR=Mc

WL=8 KN/m LL=2.4m

WR=9.1 KN/m LR=5m

$M_L L_L + 2M_C(L_L + L_R) + M_R L_R = -W_L L_L^3/4 - W_R L_R^3/4$

14.8Mb+5Mc=-312.1 ………(1)

Span BCD :

ML=Mb MC=Mc MR=0

WL=9.1 KN/m LL=5m

WR=9.1 KN/m LR=4.8m

5Mb+19.6Mc = -535.97 ………(2)

By solving equation:

Mc=-24.1 KN.m

Mb=-12.91

Ay=4.22 KN

By=35.5 KN

Cy=51.85 KN

Dy=16.82 KN

General Notes

Shear and moment diagram

Palestinian Polytechnic University

Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Shear and moment diagram

Supervisor Dr. Ghassan Al-Dwiek

Dr. Maher Amro

Desined by :

Ayman Kraleid

Munzer AL- Sa'id

Zaher Mahmoud

Yousef AL-Jabary

SCALE	no.of DRAW	DATE
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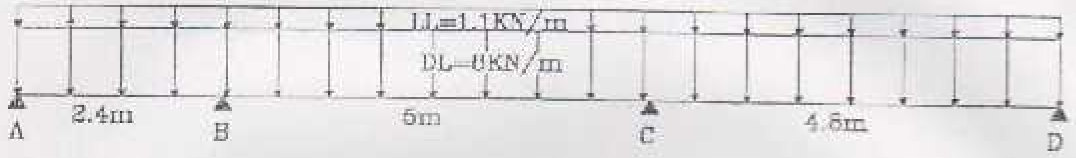

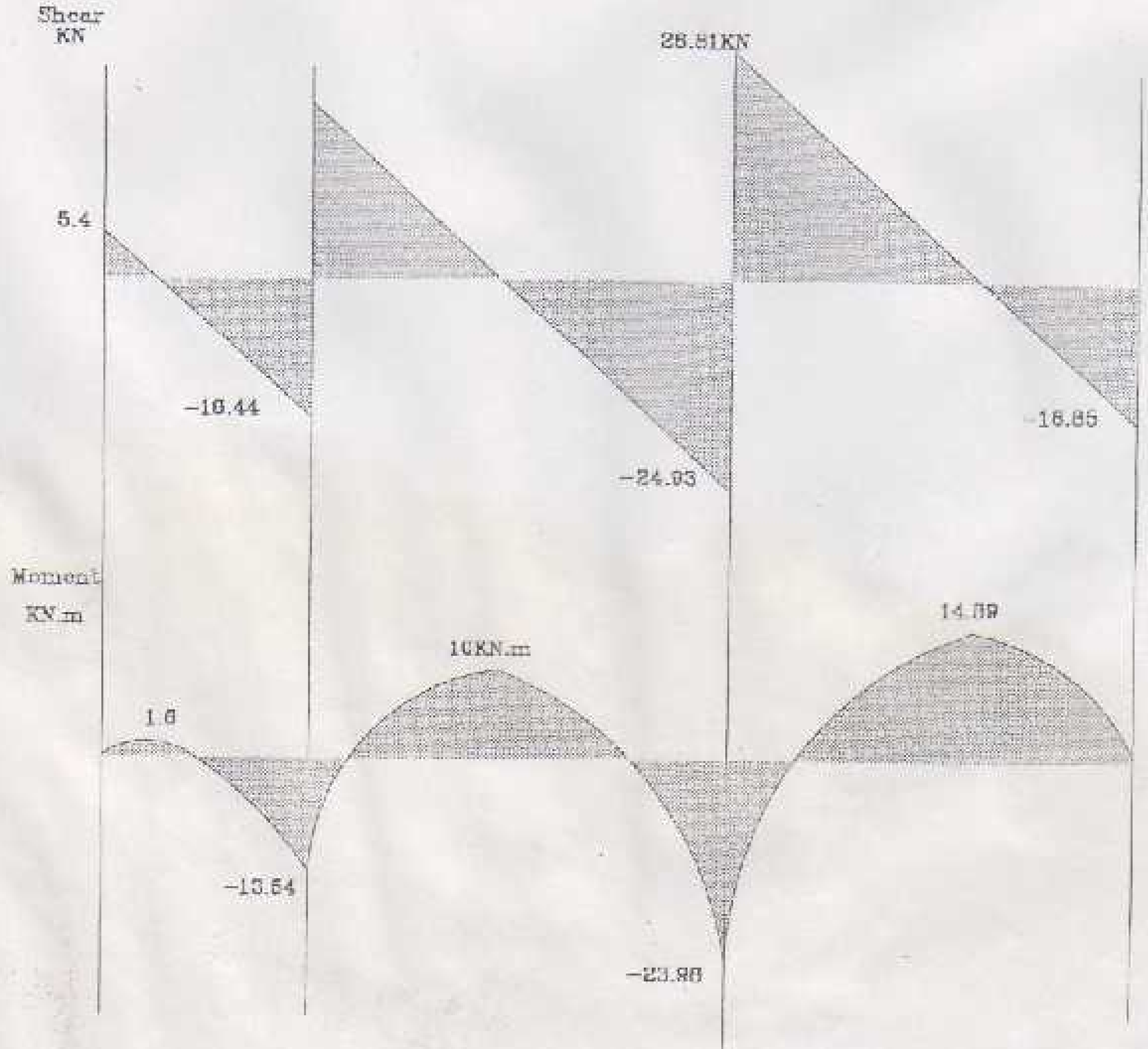

Shear & Moment Diagram

Case (1)

General Notes

Shear and moment diagram

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Supervisor Dr. Ghassan Al-dwaik, Dr. Maher Amro

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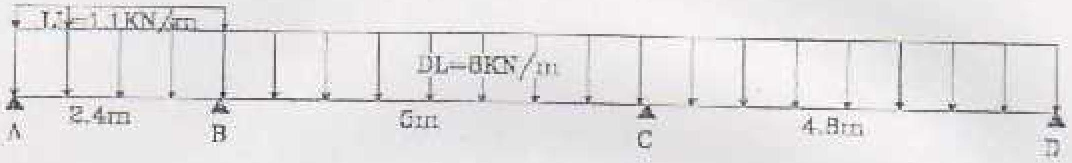

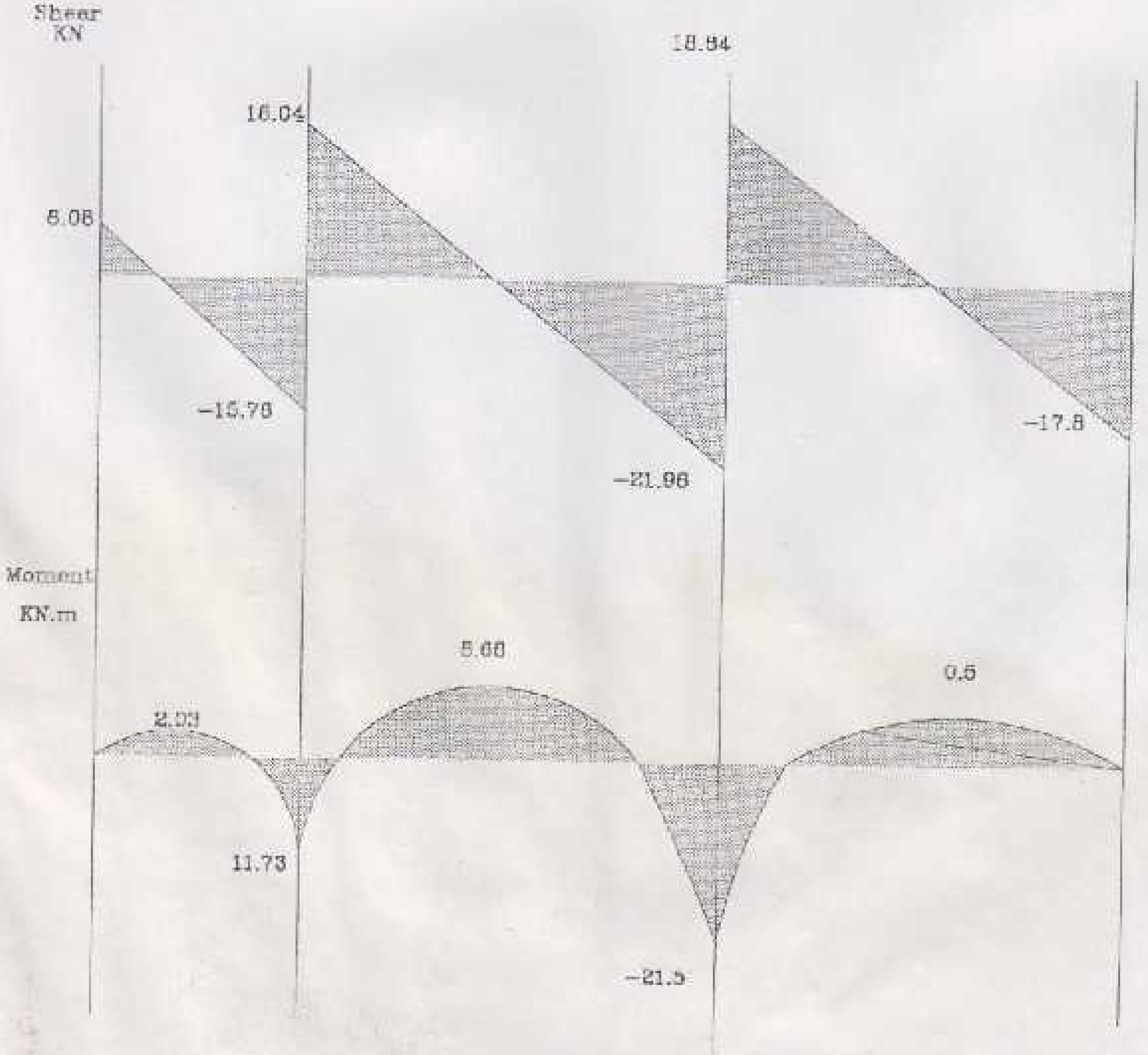

Shear & Moment Diagram
Case (2)

(383)

General Notes

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Campus design

DRAWING TITLE :
Shear and moment diagram

Supervisor Dr. Ghassan Al-dwaik
Dr. Maher Amro

Desined by :
Ayman Sweiti
Mosour Al- Sa'di
Maher Mahmud
Yousif Al-Jabery

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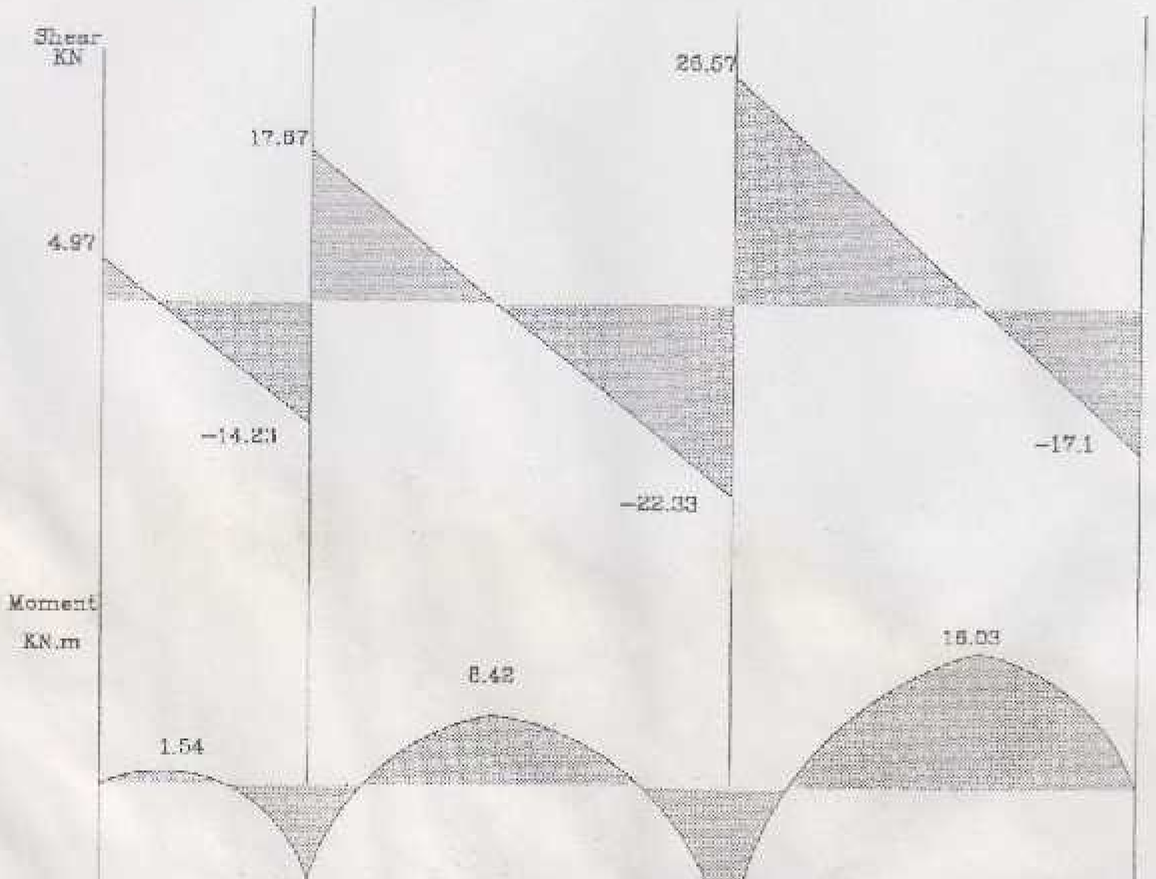
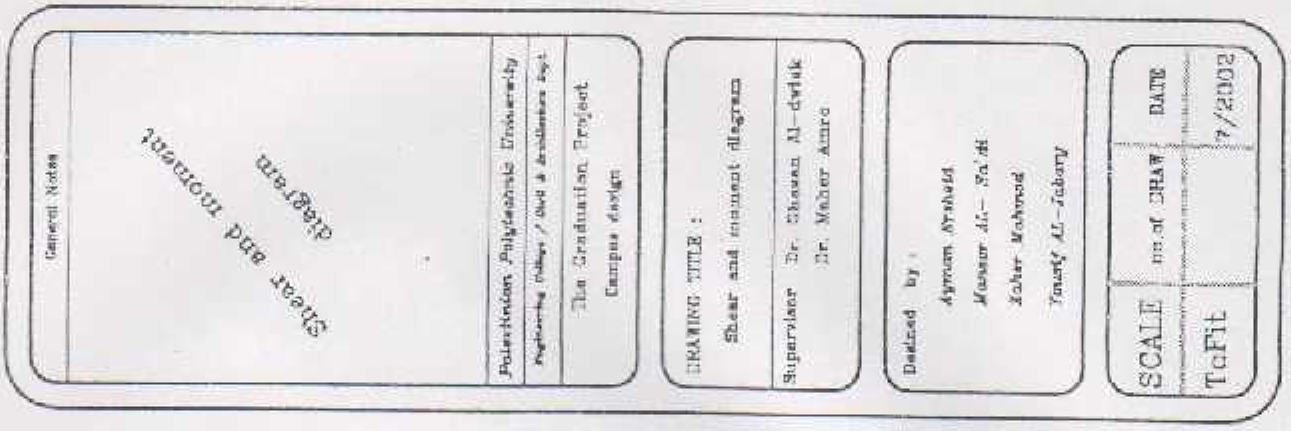
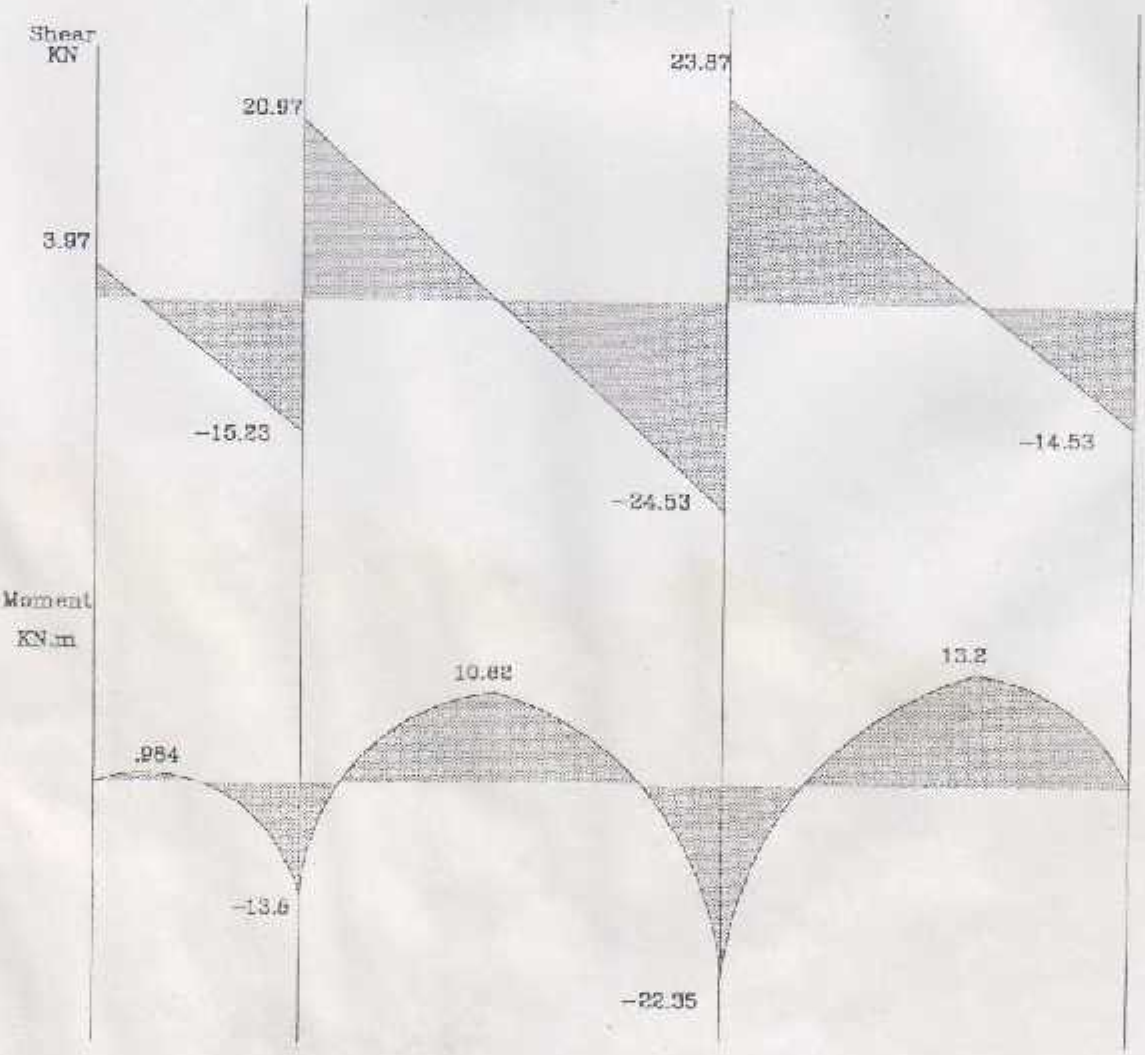


Shear & Moment Diagram
Case (3)





Shear & Moment Diagram
Case (4)

(385)

General Notes

Shear and moment diagram

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Campus design

DRAWING TITLE :

Shear and moment diagram

Supervisor Dr. Ghassan Al-dwiek

Dr. Maher Amro

Desined by :

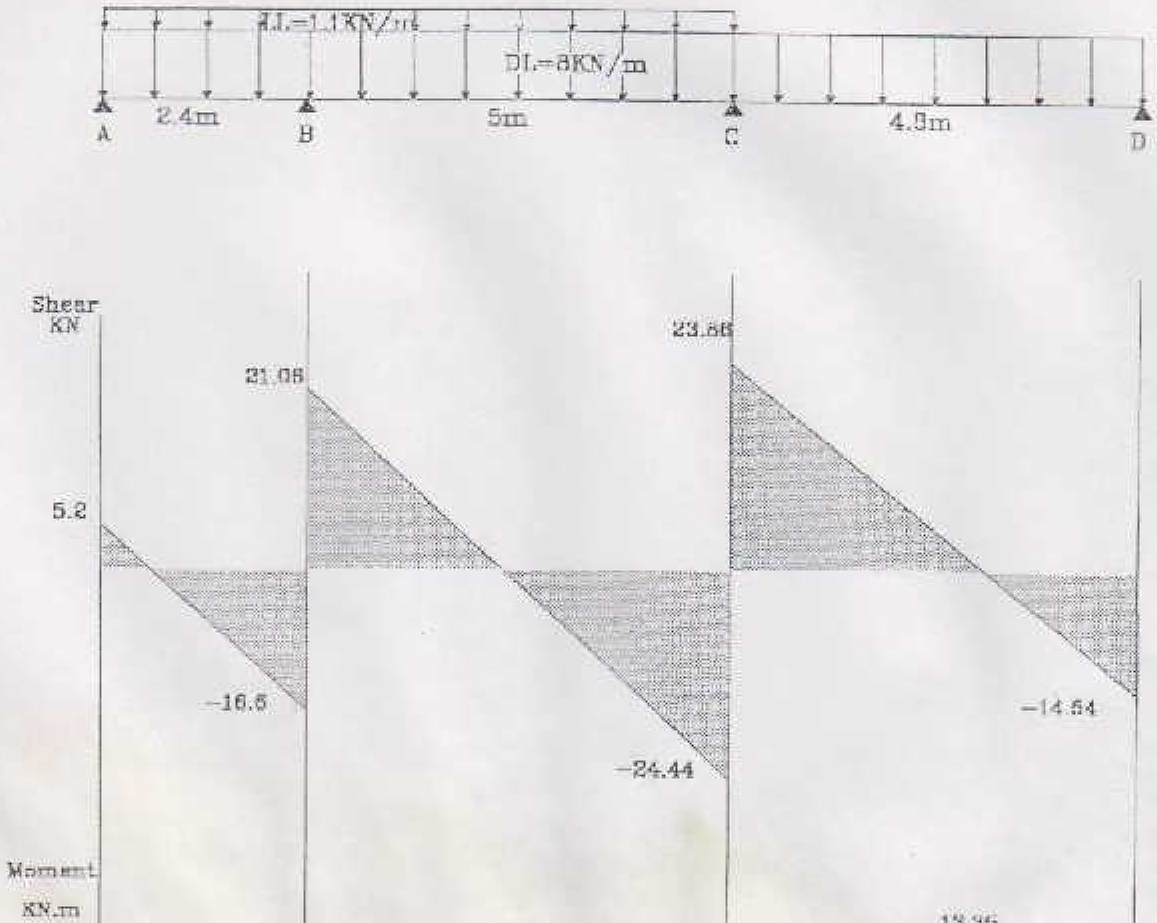
Ayman Srahaid

Monzer AL- Sa'di

Nader Mahmoud

Yousif AL-Jabary

SCALE	DRAW	DATE
ToFit		7/2002



Shcar & Moment Diagram

Case (5)

(386)

General Notes

Shear and moment diagram

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Campus design

DRAWING TITLE :
Shear and moment diagram

Supervisor Dr. Ghassan Al-dalou
Dr. Maher Amro

Designed by :
Ayman Rashaid
Munzer AL- Sa'di
Maher Mahmoud
Yousef AL-Jabary

SCALE	no.of DRAW	DATE
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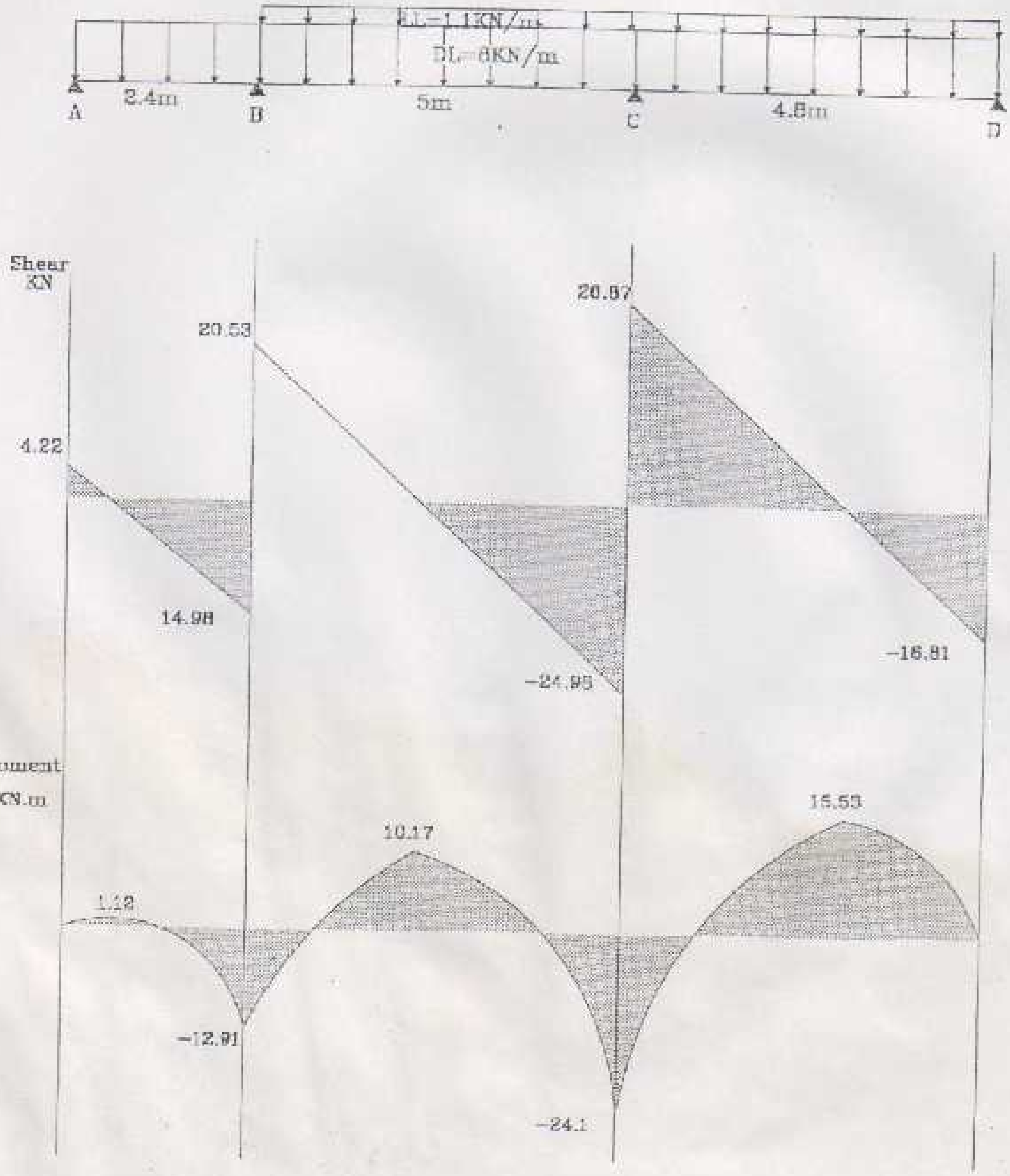

Shear & Moment Diagram
Case (6)

General Notes

Shear and moment Envelope

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Engineering College / Civil & Architecture Dept.

The Graduation Project

Campus design

DRAWING TITLE :

Shear and moment diagram

Supervisor Dr. Ghassan Al-dwiek

Dr. Maher Amro

Drafted by :

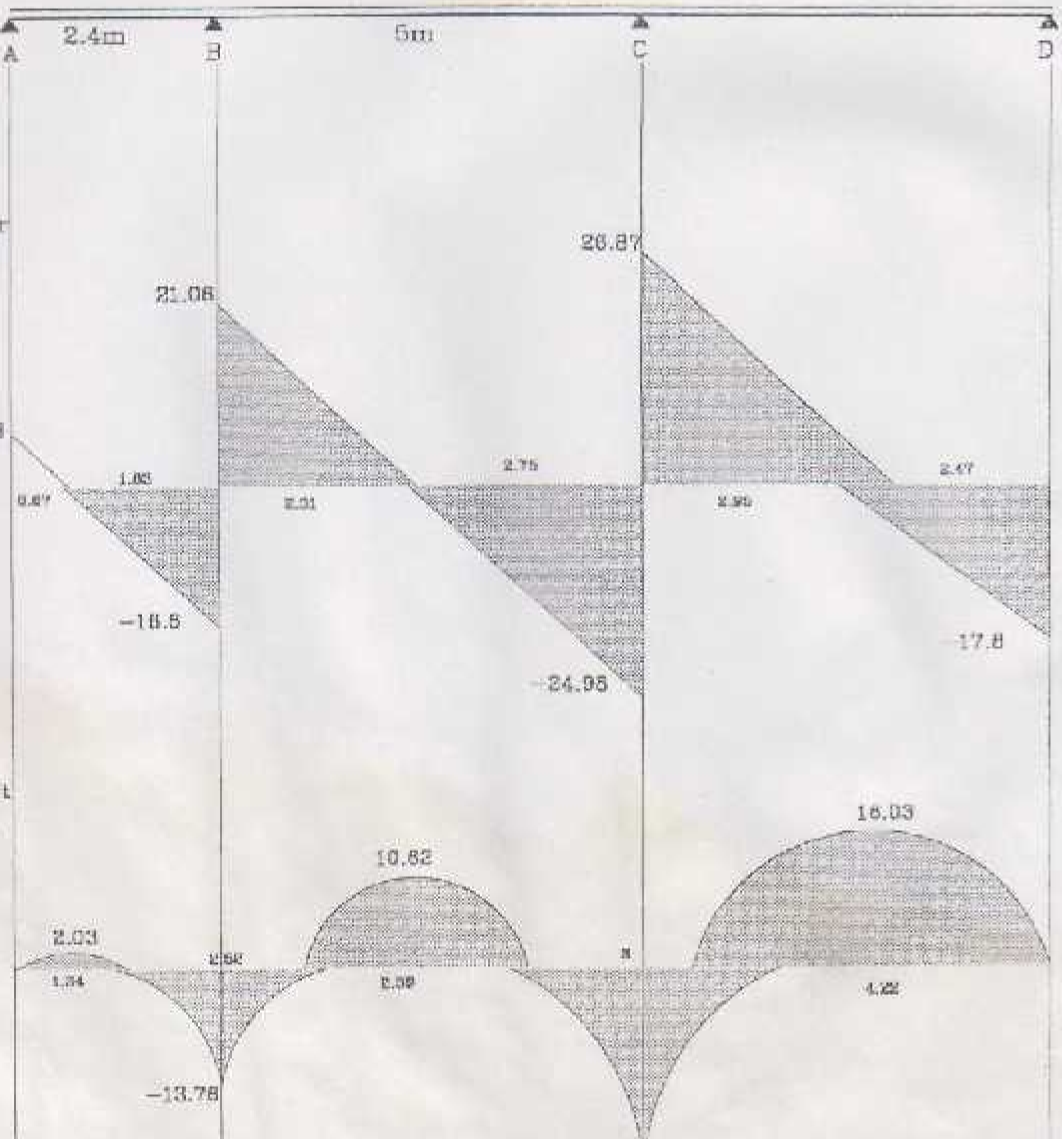
Ayman Ershaid

Munser AL- Sa'di

Saher Mahmod

Yousif AL-Jabary

SCALE	no.of DRAW	DATE
ToFit		7/2002



Envelope Shear & Moment

Appendix (D)

Architectural Drawings

This appendix is an attachment with this project

Appendix (E)

Structural Drawings

This appendix is an attachment with this project