DARBHANGA COLLEGE OF ENGINEERING

COURSE FILE

OF

DESIGN OF STEEL STRUCTURE (011620)

Faculty Name:

AHSAN RABBANI

Assistant Professor, DEPARTMENT OF CIVIL ENGINEERING

विज्ञान एवं प्रावैधिकी विभाग

Department of Science and Technology

Government of Bihar

CONTENTS

DEPARTMENT OF CIVIL ENGINEERING

DARBHANGA COLLEGE OF ENGINEERING, DARBHANGA

VISION

Department of Civil Engineering is striving to become a premier academic centre for quality Education, Entrepreneurship and Research in different areas of civil engineering with a strong social commitment.

MISSION

1. To produce highly competent and technologically capable professionals by collaboration with relevant industries.

2. To motivate graduates towards innovation and research in the field of civil engineering.

3. To provide quality education in undergraduate levels with strong emphasis on professional’s ethics and social commitment.

CIVIL ENGINEERING PROGRAM EDUCATIONAL OBJECTIVES

PROGRAM EDUCATIONAL OBJECTIVES (PEOs)

PEO1	To prepare our graduates to have successful careers in design and analysis of various Civil Engineering structures and also motivate them to pursue higher studies and research in the relevant fields.
PEO2	To prepare our graduates as a good cognizance of Societal, Environmental and Ethical issues and have effective communication skills.
PEO3	To develop awareness of contemporary professionals issues and encourage them to support the Engineering profession through contribution in professional's societies and/or Educational Institutions.

PROGRAM SPECIFIC OUTCOMES (PSOs)

The PSOs of Civil engineering programme supported by the curriculum are given below.

PSO1	To function as design consultants in the relevant industry for the design of civil engineering structures using modern software tool.
PSO2	To develop knowledge in some specific technical areas of civil engineering; Structural, Geotechnical, Transportation, Earthquake and Environmental engineering.

PROGRAMME OUTCOMES (PO)

PO1	**Engineering knowledge**: An ability to apply the knowledge of mathematics, science, engineering fundamentals, and an engineering specialization to get the solution of the engineering problems.
PO2	**Problem analysis:** Ability to Identify, formulates, review research literature, and analyze complex engineering problems.
PO3	**Design/development of solutions:** Ability to design solutions for complex engineering problems by considering social, economical and environmental aspects.
PO4	**Conduct investigations of complex problems:** Use research-based knowledge to design, conduct analyse experiments to get valid conclusion.
PO5	**Modern tool usage:** ability to create, select, and apply appropriate techniques, and to model complex engineering activities with an understanding of the limitations.
PO6	**The engineer and society:** Ability to apply knowledge by considering social health, safety, legal and cultural issues.
PO7	**Environment and sustainability:** Understanding of the impact of the adopted engineering solutions in social and environmental contexts.
PO8	**Ethics**: Understanding of the ethical issues of the civil engineering and applying ethical principles in engineering practices.
PO9	**Individual and teamwork:** Ability to work effectively as an individual or in team, as a member or as a leader.
PO10	**Communication:** An ability to communicate clearly and effectively through different modes of communication.
PO11	**Project management and finance:** Ability to handle project and to manage finance related issue
PO12	**Life-long learning:** Recognize the need for, and have the preparation and ability to engage in independent and life-long learning.

Course Description

Many civil engineering structures are made up of steel. Knowledge of designing and detailing of steel structures is very important for civil engineers in order to make structures safe and serviceable during its life span. Limit State design philosophy is currently used worldwide for design of steel structures and its various components. Also precise and correct detailing of structural drawing is necessary in order to get the correct behavior of structures and leads to smooth construction of structures. This course will provide detailed knowledge of design and detailing of steel structures as per Indian standards.
The concepts of this course are applicable in all civil engineering structures. The Design of Steel Structures curriculum is designed to prepare interested students for a future career in the field of Structural Engineering, Earthquake and Wind Engineering. The course deals with design of steel structures using "Limit State Design Method". The design methodology is based on the latest Indian Standard Code of Practice for general construction (IS 800:2007). The subject covers all the necessary components such as material specifications, connections and elementary design of structural members for designing industrial steel structures. The course provides material specifications and design considerations. It provides relevant material properties of different types of steel. It deals with two types of connections namely welded and bolted connections.

Outcomes

- The students would have knowledge on the design of structural steel members subjected to compressive, tensile and bending forces, as per current code and also know to design structural systems such as steel industrial sheds, plate girders, eccentric connection, moment resistant connection etc.

Course Objectives

- To introduce the students to limit state design of structural steel members subjected to compressive, tensile and bending loads, including connections.
- Design of structural systems such as steel industrial sheds, column base, moment resistant connection etc. as per provisions of current code (IS 800 - 2007) of practice.

Pre-requisites

- Student should know about the basic knowledge about mechanics of solids
- Student should know about the basic knowledge about strength of materials

Guidelines

- IS 800: 2007, General Construction in Steel - Code of Practice, Bureau of Indian Standards, New Delhi

Instructional Objective

- The student should familiar to limit state design of structural steel members and they should be made to
- **IO1**: To Design the connections (Bolted, Welded).
- **IO2**: To design the steel members under tensile load.
- **IO3**: To design the steel members under compression.
- **IO4**: To design the beam under laterally supported and unsupported.
- **IO5**: To design the column bases, plate girder, steel industrial sheds etc.

Course Outcome (5)

At the end of this course, the students will be able to

CO1: Understand the knowledge of different connections used in steel structures

CO2: Evaluate how to determine the design strength of tensile members

CO3: Evaluate how to determine the design strength of compression members

CO4: Understand about laterally supported, laterally un-supported beam, plate girder and design of column bases.

CO5: Analyze the plastic theory on steel structures.

CO-PO MAPPING

Sl No.	Course Outcome	PO
1	**CO1:** Understand the knowledge of different connections used in steel structures	PO1, PO2, PO3, PO4, PO5, PO6, PO7, PO8, PO9, PO10, PO11, PO12
2	**CO2:** Evaluate how to determine the design strength of tensile members	PO1, PO2, PO3, PO4, PO6, PO7, PO8, PO9, PO12
3	**CO3:** Evaluate how to determine the design strength of compression members	PO1, PO2, PO3, PO5, PO6, PO7, PO8, PO9, PO12
4	**CO4:** Understand about laterally supported, laterally un-supported beam, plate girder and design of column bases	PO1, PO2, PO3, PO6, PO8, PO9, PO10, PO11, PO12
5	**CO5:** Analyze the plastic theory on steel structures	PO1, PO2, PO4, PO5, PO6, PO8, PO9, PO10, PO11, PO12

Course Outcomes	PO1	PO2	PO3	PO4	PO5	PO6	PO7	PO8	PO9	PO10	PO11	PO12	PSO	PSO
CO1: Understand the knowledge of different connections used in steel structures	3	3	3	1	1	1	1	1	2	1	1	2		
CO2: Evaluate how to determine the design strength of tensile members	3	3	3	1	0	1	1	1	2	0	0	1		
CO3: Evaluate how to determine the design strength of compression members	3	3	3	0	1	1	1	1	2	0	0	1		
CO4: Understand about laterally supported, laterally un-supported beam, plate girder and design of column bases	3	2	3	0	0	1	0	1	2	1	1	2		
CO5: Analyze the plastic theory on steel structures	3	3	0	2	1	1	0	1	1	1	1	3		

Correlation Level: 1- Slight (Low) 2- moderate (Medium) 3 – Substantial (High)

B. Tech. VI Semester (Civil)
CE- 620 Design of Steel Structure

L/P T P/D Total		Max Marks:	100
2 -0 - 2 4	Final Exam:	70 Marks	
	Sessional:	20 Marks	
	Internals:	10 Marks.	

Introduction to Design: Design Loads and Load combinations, Working Stress Design, Plastic Design, LRFD Method, Introduction to steel and steel structures. **Lecture: 4**

Design of structural Fasteners: rivets, bolts and welds. **Lecture: 6**

Design of tension members **Lecture: 4**

Design of compression member: laced and battened columns. **Lecture: 6**

Design of flexure members: Beams- rolled sections, built up section, plate Girders- riveted/ bolted and welded, Design of eccentric connections: riveted/ bolted and welded. **Lecture: 8**

Design of beam: Columns and columns based welded and riveted column bases- moment resistant connection - semi rigid connection- design of supports. **Lecture: 5**

Design of steel industrial sheds. Wind Design. **Lecture: 8**

Introduction inelastic action and plastic hinges application of PD and LRFD **Lecture: 3**

Total: 44 lecture

Textbooks:

1. Bhavikatti.S.S, "Design of Steel Structures" By Limit State Method as per IS: 800–2007, IK International Publishing House Pvt. Ltd., 2009.
2. Duggal. S.K, "Limit State Design of Steel Structures", Tata McGraw Hill Publishing Company.
3. Chandak, N.R., "Design of Steel Structures", Katson Publication.
4. Subramanian.N, "Design of Steel Structures", Oxford University Press, New Delhi, 2013.
5. Shah.V.L. and Veena Gore, "Limit State Design of Steel Structures", IS 800–2007 Structures Publications, 2009.
6. Negi, L. S., "Design of Steel Structures", Tata McGraw Hill.

References:

1. IS 800: 2007, General Construction in Steel - Code of Practice, (Third Revision), Bureau of Indian Standards, New Delhi, 2007.
2. IS 875 (Part 1): Indian Standard Code of Practice for Dead Loads, Bureau of Indian Standards, New Delhi.
3. IS 875 (Part 2): Indian Standard Code of Practice for Imposed Loads, Bureau of Indian Standards, New Delhi.
4. IS 875 (Part 3): Indian Standard Code of Practice for Wind Loads, Bureau of Indian Standards, New Delhi.

Gate Syllabus:

Working stress and Limit state design concepts; Design of tension and compression members, beams and beam- columns, column bases; Connections – simple and eccentric, beam-column connections, plate girders and trusses; Plastic analysis of beams and frames.

Time Table: 6th Semester (Jan-June 2019)

Darbhanga College of Engineering, Darbhanga wef: 07/01/2019

6th Semester

DAY	Dept.	09:10-10:00 (1)	10:00-10:50 (2)	10:50-11:40 (3)	11:40-12:30 (4)	12:30-1:00 (5)	01:00-01:50(6)	01:50-02:40 (7)	02:40-03:30 (8)	03:30-04:20 (9)
MONDAY	EE (S8)	UEP	ICS	PE	MP			ICS-Lab/MP-Lab		
	CSE (S7)	CD Lab			PPL			CD	FLAT	
	ME (S6)	DME (PS)		HMT (MR)	NCM (MK)		DME LAB (M1)/ HMT LAB (M2)			
	CE (S3)	DCS-1	TE-1	SA-II			EE-I	TE LAB (G-2)		
TUESDAY	EE (S8)	ICS	UEP	EMI	MP			PE	Student Counseling	
	CSE (S7)	WA Lab			PPL			CD		FLAT(T)
	ME (S6)	HMT (MR)	DME (PS)	I&M (EEED)	CMS (RG)	L	IEA (T)	NCM (T)	English	
	CE (S3)	TE-1	DCS-1 LAB (G-2)				EE-I	S & R LAB (G-1)		
WEDNESDAY	EE (S8)	MP-Lab/ICS-Lab					MP	EMI	English	
	CSE (S7)	OOAD	WA	PPL				CD(T)	FLAT	
	ME (S6)	NCM (MK)	HMT (MR)	CMS (RG)	I&M (EEED)	U	IEA (T)		CMS (T)	
	CE (S3)	SA-II		EE-I	S & R		TE-1	DCS-1 LAB (G-1)		
THRUSDAY	EE (S8)	PE	ICS	UEP			EMI	PE-Lab/EMI-Lab		
	CSE (S7)	IEA	SE	WA	OOAD			CD		FLAT
	ME (S6)	I&M (T)	NCM (MK)	HMT (MR)	IEA (PK)	N	DME LAB (M2)/ HMT LAB (M1)			
	CE (S3)	SS	DCS-1	SS LAB (G-1)/EE LAB (G-2)			S & R	S & R LAB (G-2)		
FRIDAY	EE (S8)	MP	EMI	UEP			PE	PE-Lab		
	CSE (S7)	IEA	SE	WA	OOAD			SE-LAB		
	ME (S6)	CMS (T)	DME (PS)	IEA (PK)	I&M (EEED)	C	I&M (T)	SEMINAR (CPS)		
	CE (S3)	SS	DCS-1	SS LAB (G-2)/EE LAB (G-1			S & R	TE LAB (G-1)/		
SATURDAY	EE (S8)	EMI	PE	UEP	ICS		MP	EMI-Lab		
	CSE (S7)	IEA	SE	WA	OOAD	H	English		Student Counseling	
	ME (S6)		NCM (T)	IEA (PK)	CMS (RG)			SEMINAR (CPS)		
	CE (S3)	Class test		Industry Awareness Program			Industrial Visit Related Work			

Sl.No.	Electrical and Electronics Enginering		Sl.No.	Computer Science and Engineering	
1	UEP	Ms. Sweta Kumari	1	SE	Mr. Sunil Kumar
2		Mr. Abhisek Sharma	2	WA	Mr. Akhilesh Kumar
3	ICS	Dr. Ravi Ranjan	3	CD&PPL	Mr. Zoheb Hasan
4	PE	Mr. Deepak Singh	4	FLAT	Ms. Ajeet Kumar Gupta
5	Microproc	Mr. Amit kumar	5	OOAD	Mrs. Divya
6	EIM	Mr. Diwakar verma	6	English	Ms. Ratnakshi Roy
7	English	Ms. Ratnakshi Roy			
Sl.No.		**Mechanical Enginering**	**Sl.No.**		**Civil Engineering**
1	IEA	Mr. Prabhakar Kumar	1	S & R	Dr. A.K. Rai/Mr. Loknath kumar
2	DME	Mr. Prashant Kumar Singh	2	DCS	Mr. S.S. Choudhary
3	HMT	Mr. Madhav Ram	3	EE	Mr. Jitendra Kumar
4	I&M	Mr. Akhil Mohamad KK	4	TEI	Mr. Prashant kumar
5	CMS	Mr. Rajat Gupta	5	SS	Mr. RAVI RANJAN
6	NCM	Mr. Mukesh Kumar	6	English	Mr. Ritwik Balo
7	English	Mr. Ritwik Balo			

05/01/19

(Mr. Ravi kumar)
Co-Cordinator Time Table

(Dr. A. K. Choudhary)
Coordinator Time Table

07/1/19

(Dr. A. K. Rai)
PRINCIPAL..

Time Table: 6th Semester (Jan-June 2020)

Darbhanga College of Engineering, Darbhanga WEF:13/01/2020

6th Semester

DAY	Dept.	09:00-10:00 (1)	10:00-11:00 (2)	11:00-12:00 (3)	12:00-1:00 (4)	01:00-02:00	2:00-3:00 (5)	3:00-4:00 (6)	4:00-5:00 (7)
MONDAY	EE (S8)	ICS	ICS-Lab/PE-Lab				CA		PE
	CSE (S5)	CD Lab			PPL		CD	FLAT	
	ME (S6)	DME		HMT	NCM		I&M (T)	SEMINAR (CPS)	
	CE (S7)	DSS	DCS-I		TE-I		ENG. ENV LAB (G-1)/DCS-I LAB (G-2)		
TUESDAY	EE (S8)	EMI	UEP	ICS	MP		PE	CA	
	CSE (S5)	WA Lab			PPL		CD		FLAT(T)
	ME (S6)	HMT	DME (PS)	I&M	CMS	L	IEA (T)	NCM (T)	R.C.
	CE (S7)		S &R	DSS	ENG. ENV-I		DSS LAB		
WEDNESDAY	EE (S8)	PE-Lab/ICS-Lab			EMI		MP	Remedial Class	
	CSE (S5)	OOAD	WA	PPL		U	CD(T)	FLAT	
	ME (S6)	NCM	HMT	CMS	I&M		IEA (T)	ENGLISH	CMS (T)
	CE (S7)		SA-II		TE-I		DCS-I LAB (G-1)/ENG. ENV LAB (G-2)		
THRUSDAY	EE (S8)	PE	ICS	UEP	EMI		MP-Lab/EMI-Lab		
	CSE (S5)	IEA	SE	WA	OOAD	N	CD		FLAT
	ME (S6)	I&M(T)	NCM	HMT	IEA		DME LAB (M2)/ HMT LAB (M1)		
	CE (S7)	S &R	ENG. ENV-I		DCS-I		TE LAB (G-1)/S &R LAB (G-2)		
FRIDAY	EE (S8)	MP	EMI	UEP	PE		EMI-Lab/MP-Lab		
	CSE (S5)	IEA	SE	WA	OOAD	C	SE-LAB		
	ME (S6)	CMS (T)	I&M	DME	IEA		DME LAB (M1)/ HMT LAB (M2)		
	CE (S7)	SA-II		TE-I	S &R		TE LAB (G-2)/S &R LAB (G-1)		
SATURDAY	EE (S8)	PE(T)	EMI(T)	UEP(T)	ICS(T)		MP(T)	ICS	ENGLISH
	CSE (S5)	IEA	SE	WA	OOAD	H	ENGLISH	Remedial Class	
	ME (S6)		NCM (T)	CMS	IEA		SEMINAR (CPS)		
	CE (S7)		DSS LAB				Remedial Class		

Sl.No.	Electrical and Electronics Enginering	
1	UEP	Mr. Abheshek Sharma
2	CA	Mr. Dhirendra Kumar
3	ICS	Dr. Ravi Ranjan
4	PE	Ms. Deepak Singh
5	Microproc	Mr. Amit kumar
6	EIM	Mr. Diwakar verma
7	English	Ms. Ratnakshi Roy

Sl.No.	Computer Science and Engineering	
1	SE	Mr. Sunil Kumar
2	WA	Mr. Akhilesh Kumar
3	CD&PPL	Mr. Zoheb Hasan
4	FLAT	Ms. Ajeet Kumar Gupta
5	OOAD	Mr. Dhirendra Kumar
6	English	Ms. Ratnakshi Roy

Sl.No.	Mechanical Enginering	
1	IEA	Mr. Prabhakar Kumar
2	DME	Mr. Prashant Kumar Singh
3	HMT	Mr. C. P Singh, Mr. Madhav Ram
4	I&M	Mr. Tabish Shanu
5	CMS	Mr. Rajat Gupta
6	NCM	Mr. Rajat Gupta
7	English	Mr. Ritwik Balo

Sl.No.	Civil Engineering	
1	S & RM	Mr. Loknath kumar
2	DCS-I	Mr. S.S. Choudhary
3	EE-I	Mr. Jitendra Kumar
4	TE-I	Mr. Aditya Kumar
5	DSS	Mr. Ahsan Rabbani
6	SA-II	Mr. S.S. Choudhary/Mr. Ahsan Rab

10/01/2020 HOD EEE 10/1/20 10/01/20 10/01/2020

10/01/2020
Co-Ordinator
(Mr. Ravi Kumar)

10/1/2020
Time Table Incharge
(Dr. A K Choudhary)

10/01/2020
PRINCIPAL
(Dr. Achintya)

List of Student: B Tech Civil Engineering (2016-2020)			
Sl. No.	Registration No.	Roll No.	Name
1	16101111001	16-C-68	KANHAIYA KUMAR YADOV
2	16101111002	16-C-52	VISHAL RAJ
3	16101111003	16-C-12	VINEET KUMAR
4	16101111004	16-C-66	RISHI KUMAR
5	16101111005	16-C-03	KIRTHI
6	16101111006	16-C-24	MITESH KUMAR MITESH
7	16101111007	16-C-37	ANKESH KUMAR
8	16101111008	16-C-16	SHUDHANSHU SHEKHAR JHA
9	16101111009	16-C-05	SHIKHA
10	16101111010	16-C-56	KUMARI PRIYANSHU
11	16101111011	16-C-41	MOTI LAL MANJHI
12	16101111012	16-C-43	KESHAV KUMAR
13	16101111013	16-C-64	CHANDAN KUMAR
14	16101111014	16-C-50	PREMRANJAN KUMAR
15	16101111015	16-C-54	RAJNISH KUMAR
16	16101111016	16-C-10	AMAR KUMAR
17	16101111017	16-C-28	SAURAV KUMAR SHANU
18	16101111018	16-C-27	RAHUL KUMAR
19	16101111019	16-C-33	ABHISHEK KUMAR SHUKLA
20	16101111020	16-C-58	NARENDRA KUMAR
21	16101111021	16-C-31	RUPAK RAJ
22	16101111022	16-C-63	RAHUL RAVI
23	16101111023	16-C-36	SANTOSH KUMAR
24	16101111024	16-C-12	PRINCE KUMAR
25	16101111025	16-C-32	NEERAJ KUMAR
26	16101111026	16-C-47	PRABHAT RANJAN
27	16101111027	16-C-51	MD ZAKI AHMAD
28	16101111028	16-C-30	HEMANT KUMAR
29	16101111029	16-C-11	AMIT RAJ
30	16101111030	16-C-15	RAKESH KUMAR
31	16101111031	16-C-62	MUSAFIR KUMAR
32	16101111032	16-C-09	AJAZ AHMAD
33	16101111033	16-C-04	POOJA KUMARI
34	16101111034	16-C-48	SHIVAMVEER KUMAR
35	16101111035	16-C-26	SUNIL KUMAR
36	16101111036	16-C-65	ATISH DEEPANKAR
37	16101111037	16-C-20	VIKRAM BHARTI

38	16101111038	16-C-46	DIPESH KUMAR
39	16101111039	16-C-34	CHANDRAMANI KUMAR
40	16101111040	16-C-40	AMIT KUMAR
41	16101111041	16-C-19	RAJEEV RANJAN
42	16101111042	16-C-07	SOPHIA KHATOON
43	16101111043	16-C-01	ADITI
44	16101111044	16-C-69	PRIYADARSHI KUMAR
45	16101111045	16-C-22	RAJVANSHI KUMAR SINGH
46	16101111046	16-C-57	BHUDEV BHASKAR
47	16101111047	16-C-44	SUDHIR KUMAR
48	16101111048	16-C-67	CHANDRESH KUMAR
49	16101111049	16-C-18	DILIP KUMAR
50	16101111050	16-C-21	RAMESH KUMAR SAH
51	16101111051	16-C-29	UMANG BHARDWAJ
52	16101111053	16-C-49	MD SALIK ANWAR
53	16101111054	16-C-61	RAUSHAN KUMAR
54	16101111055	16-C-02	SAIMA FIRDAUS
55	16101111056	16-C-60	DURGESH KUMAR
56	16101111058	16-C-38	RAM RATAN KUMAR
57	16101111059	16-C-59	SHANKAR RAM
58	17101111901	17-LE-C-05	PANKAJ KUMAR SAH
59	17101111902	17-LE-C-04	RAHUL KUMAR
60	17101111903	17-LE-C-06	ANKESH KUMAR
61	17101111904	17-LE-C-01	ADARSH ANAND
62	17101111905	17-LE-C-12	PRATEEK KUMAR
63	17101111906	17-LE-C-03	SANATAN KUMAR JHA
64	17101111907	17-LE-C-02	SACHIN KUMAR
65	17101111908	17-LE-C-08	MRITYUNJAY KUMAR
66	17101111909	17-LE-C-07	BIBEKANAND KUMAR
67	17101111910	17-LE-C-11	KUMAR SUMAN SAURABH
68	17101111911	17-LE-C-10	PINKEE KUMARI
69	17101111912	17-LE-C-09	JAI KUMAR

List of Student: B Tech Civil Engineering (2017-2021)			
Sl. No.	**Registration No.**	**Roll No.**	**Name**
1	17101111001	17-C-66	ARUN KUMAR
2	17101111002	17-C-29	PRADUMN KUMAR
3	17101111003	17-C-69	KETAN SINHA
4	17101111004	17-C-10	MD MATLUB NEYAZ
5	17101111005	17-C-03	DEEPAK KUMAR
6	17101111006	17-C-24	MANIKANT KUMAR
7	17101111007	17-C-15	SURAJ KUMAR
8	17101111008	17-C-32	AKASH RAMAN
9	17101111009	17-C-47	SURENDRA KUMAR SAHU
10	17101111010	17-C-13	ASHUTOSH KUMAR
11	17101111011	17-C-14	NITISH KUMAR
12	17101111012	17-C-33	BAUSKI KUMAR
13	17101111013	17-C-02	SANJEEV KUMAR SINGH
14	17101111014	17-C-36	SANDEEP KUMAR
15	17101111015	17-C-52	SHUBHAM KUMAR
16	17101111016	17-C-59	DEEPAK KUMAR
17	17101111017	17-C-40	REKESH KUMAR MAHTO
18	17101111018	17-C-18	UMA SHANKAR
19	17101111020	17-C-20	AMIT KUMAR
20	17101111021	17-C-67	RADHESH JHA SUMAN
21	17101111022	17-C-17	ANISH ANAND
22	17101111023	17-C-60	RAM BILASH YADAV
23	17101111024	17-C-51	SACHIN KUMAR
24	17101111025	17-C-26	ANSHU KUMAR
25	17101111026	17-C-01	UDAY LAL DAS
26	17101111028	17-C-55	CHANDAN KUMAR
27	17101111029	17-C-04	SANJEET KR PRABHAKAR
28	17101111030	17-C-07	NAVEEN KUMAR
29	17101111031	17-C-08	RAKESH KUMAR
30	17101111032	17-C-11	RANJEET KUMAR PATHAK
31	17101111033	17-C-16	MANISH KUMAR
32	17101111034	17-C-19	PRAVEEN KUMAR
33	17101111035	17-C-21	NAVIN KUMAR
34	17101111036	17-C-22	VISHAL KUMAR
35	17101111037	17-C-25	ISAAC ARYA
36	17101111038	17-C-28	VIKASH KUMAR
37	17101111039	17-C-49	MEERA KUMARI

38	17101111040	17-C-31	NEHAL RAJ
39	17101111041	17-C-50	SNEHLATA
40	17101111042	17-C-57	ASHISH KUMAR
41	17101111043	17-C-34	OM PRAKASH
42	17101111044	17-C-61	DARPAN KUMAR
43	17101111045	17-C-41	VIKASH KUMAR
44	17101111046	17-C-05	PRINCE KUMAR
45	17101111047	17-C-63	LALU KUMAR
46	17101111048	17-C-65	SANTOSH ANAND
47	17101111049	17-C-06	MD ESTEKHAR ALAM
48	17101111050	17-C-44	TARUN KUMAR
49	17101111052	17-C-09	MASOOM AKHTAR
50	17101111054	17-C-42	MUKUL KUMAR
51	17101111055	17-C-35	SHUBHAM RAJ
52	17101111056	17-C-54	KRISHNA ARYA
53	17101111057	17-C-43	PRATIK KUMAR
54	17101111058	17-C-53	AJIT KUMAR YADAV
55	17101111059	17-C-68	ABHISHEK RAJ
56	17101111060	17-C-58	RAHUL KUMAR
57	17101111061	17-C-56	VIKRAM KUMAR
58	17101111062	17-C-38	RAKESH KUMAR
59	17101111063	17-C-46	NIRANJAN KUMAR
60	17101111064	17-C-64	NITISH KUMAR
61	17101111065	17-C-39	MANISH KUMAR SINGH
62	17101111066	17-C-62	VIVEK MANI
63	18101111001	18-LE-C-05	AMIT KUMAR
64	18101111901	18-LE-C-03	SITA RAM YADAV
65	18101111902	18-LE-C-04	RANVEER KUMAR YADAV
66	18101111903	18-LE-C-01	NITISH KUMAR
67	18101111904	18-LE-C-02	SONIKA NIRANJAN
68	18101111905	18-LE-C-05	INDRA KUMAR
69	18101111906	18-LE-C-07	SUMIT KUMAR
70	18101111908	18-LE-C-06	NAVEEN KUMAR
71	18101111909	18-LE-C-09	DEEPAK KUMAR

Institute/college Name	Darbhanga College of Engineering, Darbhanga
Corse/Branch	B.Tech./Civil Engineering
Year/Semester	III/VI
Course Code/Choice	011620/ Core
Course credits	4
Course Name	Design of Steel Structure
Lecture/ Sessional (per week)	4/0
Course Teacher name	Ahsan Rabbani
Deptt./Designation	Civil Engineering/Assistant Professor

Lecture Plan:

Lecture on topic	Lecture Number	Proposed Date of Lecture
Introduction		
Importance of steel structure, Type of steel structure and its properties	**1**	
Type of load acting on the steel structures, calculation of the loads and load combination	**2**	
Design Philosophies: Limit State Method (LSM), Working Stress Design, Ultimate load design	**3**	
Plastic design and LRFD method	**4**	
Design of structural fasteners		
Type of connection, advantages and disadvantages of riveted connection	**5**	
Introduction to bolted connection, types of bolts, advantages and disadvantages of bolted connection	**6**	
Terminology used in bolted connection, Numerical question related to design of bolted connection.	**7**	
Numerical question related to design of bolted connection	**8**	
Numerical question related to design of bolted connection	**9**	
Numerical question related to design of bolted connection	**10**	
Introduction to welded connection, advantages and disadvantages of welded connection, Types of welded joints, weld size for butt weld.	**11**	
Introduction to fillet weld, specification of fillet weld, Design stresses in weld	**12**	
Numerical on design of welded connection	**13**	
Numerical on design of welded connection	**14**	
Numerical on design of welded connection, design of eccentric connection	**15**	
Design of eccentric connection	**16**	
Design of tension members		
Different types of tension member, modes of failure of tension member, Design steps as per IS code	**17**	
Factors affecting the strength of tension member, Angles Under Tension	**18**	
Numerical questions on design of tension member	**19**	
Numerical questions on design of tension member	**20**	
Numerical questions on design of tension member	**21**	
Introduction to Lug Angle, Gussets and Other Sections and its designing steps	**22**	
Design of Compression members		
Introduction to compression member, Modes of failure of compression member, buckling of column, Buckling class of cross section	**23**	
Elastic Buckling of Slender Compression Member, Sections used for Compression Members	**24**	
Effective Length of Compression Member, Design steps of compression member as per IS 800 (2007)	**25**	
Numerical question on design of compression member using formula	**26**	
Numerical question on design of compression member using formula	**27**	

Numerical question on design of compression member using table	**28**	
Designing of built-up member, numerical question on built-up section	**29**	
Introduction to lacing and batten, Design steps of lacing and batten as per IS 800 (2007)	**30**	
Numerical question on design of single & double lacing column	**31**	
Numerical question on design of batten	**32**	
Design of flexural member		
Design of laterally supported and laterally unsupported beam as per IS Code	**33**	
Numerical questions on design of laterally supported beam	**34**	
Numerical questions on design of laterally supported beam	**35**	
Numerical questions on design of laterally unsupported beam	**36**	
Numerical questions on design of laterally unsupported beam	**37**	
Design of built-up section, Introduction to plate girder: Riveted/bolted and welded	**38**	
Design of eccentric connection: Riveted/bolted and welded	**39**	
Design of Beams		
Introduction, Beam Types, Section Classification, Behavior of Beam in Bending	**40**	
Design of column and slab bases, Numerical question on slab bases	**41**	
Introduction to moment resistant connection, semi rigid connection, design of supports	**42**	
Design of steel industrial shed		
Introduction to design of steel industrial shed	**43**	
Design of structural member due to wind load	**44**	
Plastic analysis		
Introduction to inelastic action and plastic hinges	**45**	
Determination of plastic section modulus, moment resistance and shape factor	**46**	
Theory of plastic analysis, numerical question on plastic theory	**47**	
Numerical question on plastic theory	**48**	
Numerical question on plastic theory, concept of LRFD	**49**	

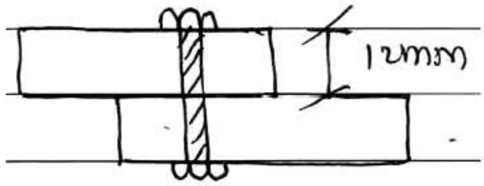
DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

Assignment No. 02

1. **A lap joint consists of two plates 200 x 12 mm connected by means of 20 mm diameter bolts of grade 4.6. All bolts are in one line. Calculate strength of single bolt and no. of bolts to be provided in the joint.**



2. **Design the Lap joint for the plates of sizes 100 × 16 mm and 100 × 10 mm thick connected so as to transmit a factored load of 100 kN using single row of 16 mm diameter bolts of grade 4.6 and plate of 410 grade.**

3. **A discontinuous compression member consists of 2 ISA 90 × 90 × 10 mm connected back to back on opposite sides of 12 mm thick gusset plate and connected by welding. The length of strut is 3 m. It is welded on either side. Calculate design compressive strength of strut.**
   For ISA 90 × 90 × 10, Cxx = Cyy = 25.9 mm Ixx = Iyy = 126.7 × 104 mm4, rzz = 27.3 mm values of f_{cd} are

KL/r	90	100	110	120
f_{cd} (N/mm^2)	121	107	94.6	83.7

4. **State with a sketch the effective length for a compression member as per IS 800 – 2007 having end conditions as**
   (i) Translation restrained at both ends and rotation free at both ends
   (ii) Translation and rotation restrained at both ends

5. **Design a tension member consisting of single unequal angle section to carry a tensile load of 340 kN. Assume single row 20 mm bolted connection. The length of member is 2.4 m. Take fu = 410 MPa, α = 0.80**

Section available (mm)	Area (mm^2)
ISA 100 × 75 × 8	1336
ISA 125 × 75 × 8	1538
ISA 150 × 75 × 8	1748

6. **Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.**

7. **The shear lag width for ISA 75X75X10 is (Assume gauge distance = 40 mm).**

8. **A single angle section 90X60X10 is connected with gusset plate with 7 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is**

the design tensile strength of the section for rupture of net section? (Assume the section is connected with longer leg and gauge distance = 50 mm)

9. **A single ISA 75 × 50 × 8 is connected (longer leg) with gusset plate using use 4 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the Design tensile strength due to block shear failure? (Assume gauge distance = 35 mm)**

10. **An ISA 90 x 90 x 8 used as tension member is connected to a 10 mm gusset plate by fillet weld of size 5 mm. The design strength of the member is 300 kN. Calculate the length of the weld.**

DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

Assignment No. 03

1. **Check whether ISMB250@37.4 kg/m is suitable or not as a simply supported beam over an effective span of 6 m. The compression flange of beam is laterally supported throughout the span. It carries udl of 15 kN/m (including self wt.). Properties of ISMB 250 are bf = 125 mm, tf = 12.5 mm, tw = 6.9 mm, Ixx = 5131.6×104 mm4, Zxx = 410×103 mm3, r1 = 13.0 mm, Z_{px} = 465.71 × 10^3 mm^3, γ_{m0} = 1.1, β_b = 1 and fy = 250 MPa.**

2. **Limiting width to thickness ratio for single beam section of plastic class is 9.4 and d/tw = 84. State whether ISMB 500 @ 852 N/m is of plastic class or not. For ISMB 500; h = 500 mm, bf = 180 mm, tf = 17.2 mm, tw = 10.2 mm, r1 = 17.0 mm, fy = 250 MPa.**

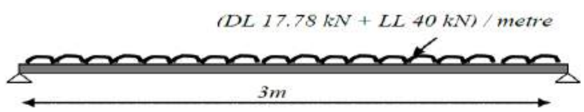
3. **A hall of size 12m x 18m is provided with Fink type trusses at 3 m c/c. Calculate panel point load in case of Dead load and live load from following data.**
   a. **Unit weight of roofing = 150 N/m^2**
   b. **Self-weight of purlin = 220 N/m^2**
   c. **Weight of bracing = 80 N/m^2**
   d. **Rise to span ratio = 1/5**
   e. **No. of panels = 6**

4. **An industrial building has trusses for 14 m span. Trusses are spaced at 4m c/c and rise of truss in 3.6m. Calculate panel point load in case of live load and wind load using following data :**
   a. **Coefficient of external wind pressure = - 0.7**
   b. **Coefficient of internal wind pressure = ± 0.2**
   c. **Design wind pressure = 1.5 kPa**
   d. **Number of panels = 08**

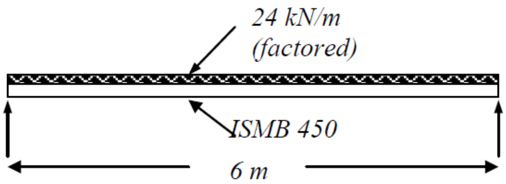
5. **Design a slab base for column ISHB 400 @ 82.2 kg/m to carry factored axial compressive load of 2000 kN. The base rests on concrete pedestal of grade M20. For ISHB 400, bf = 250 mm, fy = 250 MPa, fu = 410 MPa, γ_{mo} = 1.1, tf = 12.7 mm.**

6. **Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base?**

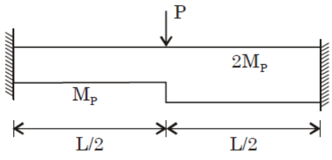
7. **Design a suitable 'I' beam for a simply supported span of 3 m and carrying a dead or permanent load of 17.78 kN/m and an imposed load of 40 kN/m. Assume full lateral restraint and stiff support bearing of 100 mm.**



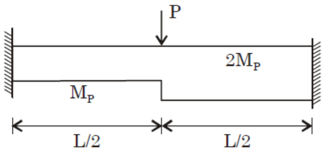
8. **Check the adequacy of ISMB 450 to carry a uniformly distributed load of 24 kN / m over a span of 6 m. Both ends of the beam are attached to the flanges of columns by double web cleat.**



9. **A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to**



10. **A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to**



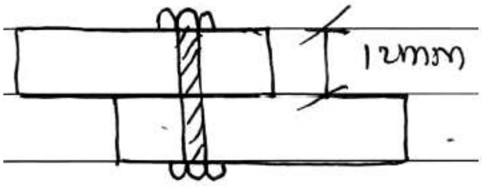
DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

Assignment No. 02

1. **A lap joint consists of two plates 200 x 12 mm connected by means of 20 mm diameter bolts of grade 4.6. All bolts are in one line. Calculate strength of single bolt and no. of bolts to be provided in the joint.**



Solution:
Given

Nominal diameter of bolt = 20 mm

$\therefore$ Net area of bolt at thread (A_{nb}) $= 0.78 \times \frac{\pi}{4} \times d^2$

$= 0.78 \times \frac{\pi}{4} \times 20^2$

$A_{nb} = 245.04\ mm^2$

For fe 410 grade steel plate (assumed)
Ultimate stress for plate $f_y = 410\ N/mm^2$
For 4.6 grade of bolt
Ultimate stress for bolt (f_{ub}) $= 4 \times 100 = 400\ N/mm^2$
Yield stress for bolt (f_{yb}) $= 400 \times 0.6 = 240\ N/mm^2$

Now find design shearing strength of bolt (Vdsb)
$\therefore$ we know that

$$\therefore \quad Vdsb = \frac{fub}{\sqrt{3} \times Y_{mb}} [n_n \times A_{nb} + n_s + A_{ns}]$$

Here number of shear plane with threat intercepting the shear plane $n_n = 1$
Number of shear plane without thread intercepting the shear plane $n_s = 0$

$$\therefore \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 243.04 + 0]$$

γ_{mb} = partial factor of safety for bolt material = 1.25

$V_{dsb} = 45.27 \times 10^3 N$

Now find design bearing strength of bolt (V_{dsb})

$$V_{dph} = 25 \times kb \times (d \times t) \times \frac{fy}{\gamma_{mb}}$$

Here coefficient k_b is minimum of

(1) $\left[\frac{e}{3dh}, \frac{p}{3dh} - 0.25, \frac{f_{ub}}{f_u}, 1\right]$

(a) Diameter of hole (dh) = Nominal diameter + 2
= 20 + 2 = 22 mm

(b) End distance (e) = 2d = 2×20 = 40 mm

(c) Pitch (p) = 2.5 d
= 2.5×20 = 50 mm

(i) $\frac{e}{3dh} = \frac{40}{3 \times 22} = 0.606$

(ii) $\frac{p}{3dh} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{fub}{fy} = \frac{400}{410} = 0.975$ &

(iv) 1

Hence Kb = 0.507 mm … take minimum value

Now find design bearing strength of bolt (V_{dpb})

$$= 2.5 \times Kb \times (d \times t) \times \frac{f_y}{r_{mb}}$$

$$= 2.5 \times 0.507 \times (20 \times 12) \times \frac{410}{1.25}$$

$$V_{dph} = 99.77 \times 10^3 \text{ N}$$

Now find bolt value i.e. strength of bolt

$\therefore$ Bolt value = minimum strength between shearing & bearing strength of bolt i.e. minimum between V_{dsb} & V_{dpb}

$= 45.27 \times 10^3$ N

Full strength of member $= 0.9 \times \frac{fu}{r_m} \times$ Area of plan

$$= \frac{0.9 \times 410}{1.25}(250 - 1 \times 22) \times 12$$

$$= 630.54 \times 10^3 \text{ N}$$

Full strength of plan

$$\therefore \text{ No of bolts} = \frac{\text{full strongth of plate}}{\text{Bolt value}}$$

$$= \frac{630.54 \times 10^3}{45.27 \times 10^3}$$

= 13.92 Say 14 Nos

2. **Design the Lap joint for the plates of sizes 100 × 16 mm and 100 × 10 mm thick connected so as to transmit a factored load of 100 kN using single row of 16 mm diameter bolts of grade 4.6 and plate of 410 grade.**

Solution: Given

$f_u = 410$ N/mm^2 $f_{ub} = 400$ N/mm^2

d = 16 mm do = 18mm

$v_{mb} = 1.25$ $P_u = 100$ kN

Strength of bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$$

$$= 0.78 \times \frac{\pi}{4} \times 16^2 = 156.82$$

Design

strength of bolt in shear

$$\text{i.e. } Vdsb = \frac{Fub}{\sqrt{3}} \frac{(n_n\, Anb + n_s\, A_{sb})}{rmb}$$

$$= \frac{400}{\sqrt{3}} \frac{1 \times 157}{1.25} = 29.006 \times 10^3 \text{ N}$$

$\therefore$ Vdsh = 29 N

$$\therefore \text{ No. of bolts required} = \frac{Pu}{Vdsb} = \frac{100}{29}$$

$= 3.4 \cong 4$ No.

No. of bolts required = 4 no.

Arranging bolts in single rows

Equating tensile capacity per pitch length

$$Tdn = 0.9 \frac{F_u}{rm_1}(P - do) \cdot t$$

$$29 \times 10^3 = 0.9 \times \frac{410}{125}(P-18) \times 10$$

$$P = \left(\frac{29 \times 10^3 \times 125}{0.9 \times 410 \times 10}\right) + 18$$

$$= 27.82 < 2.5 \times d = 2.5 \times 16$$

$$= 40$$

$\because$ Provide pitch P = 40 mm
and edge distance = 17 × do [for rough edge]
= 17 × 18
= 30.6 ≅ 30

kb is smallest of

i) $\frac{e}{3do} = \frac{30}{3 \times 18} = 0.56$

ii) $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25$
$= 0.49$

iii) $\frac{Fub}{Fu} = \frac{400}{410} = 0975$

(iv) 1

Min. Value = 0.49

Hence Kb = 0.49
$\therefore$ Design bearing strength

$$Vdsh = \frac{Vnpb}{rmb} = \frac{2.5 \times kb.d.t\ F_u}{rmb}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25}$$

$$= 64288 \text{ N} = 64.29 \text{ kN}$$

Vdsb = 64.29 kN > 29 kN
$\because$ Ok no revision is required

Check for the strength of plate

$$Tdn = \frac{0.9\,An \cdot Fu}{rm} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

$$= 188.93 \text{ kN} > 110\text{kN}$$

safe

Provide 4–16 mm ϕ bolts of 40mm
Pitch with edge distance of 30 mm as shown in fig.

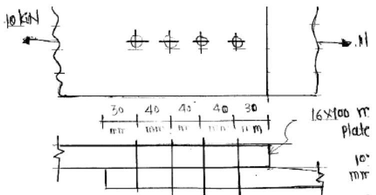


Fig: showing connection of Lap joint

3. **A discontinuous compression member consists of 2 ISA 90 × 90 × 10 mm connected back to back on opposite sides of 12 mm thick gusset plate and connected by welding. The length of strut is 3 m. It is welded on either side. Calculate design compressive strength of strut.**
For ISA 90 × 90 × 10, Cxx = Cyy = 25.9 mm Ixx = Iyy = 126.7 × 104 mm4, rzz = 27.3 mm values of f_{cd} are

KL/r	**90**	**100**	**110**	**120**
f_{cd} (N/mm^2)	**121**	**107**	**94.6**	**83.7**

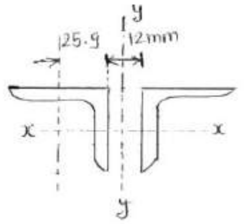
Solution:

(i) r_{zz} = 27.3 mm (Due to symmetry @ zz axis)
(ii) $I_{yy} = 2[I_y + A \cdot h^2]$
$= 2[126.7 \times 10^4 + 1703\,(25.9 + 12/2)^2]$

(A is calculated by calculating Area of both leg separately and then adding them)
$\therefore I_{yy} = 5999979\ mm^4$

(iii) $\therefore r_{yy} = \sqrt{\dfrac{I_{yy}}{A_g}} = \sqrt{\dfrac{5999979}{2 \times 1703}} = 41.97$ mm

r_{min} = minimum of r_{zz} and r_{yy}
r_{min} = 27.3 mm



(iv) For discontinuous double angle, effective length
KL = 0.85L = 0.85 × 3 = 2.10 m = 2100mm

$S.R. = \dfrac{KL}{r_{min}} = \dfrac{2100}{27.3} = 76.92$

KL/r (SR)	fcd
70	152
80	136

Hence,

$$fcd = fcd_1 - \frac{fcd_1 - fcd_2}{SR_2 - SR_1}$$

$$fcd = fs_2 - \frac{152 - 136}{80 - 70}(76.92 - 70)$$

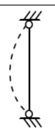
fcd =140.928 N/mm^2

(v) Design compressive Strength
Pd = fcd × Ag
Pd = 140.928 × (2 × 1703)
Pd = 480 × 10^3 N
Pd = 480 kN

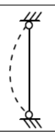
4. **State with a sketch the effective length for a compression member as per IS 800 – 2007 having end conditions as**
(i) Translation restrained at both ends and rotation free at both ends
(ii) Translation and rotation restrained at both ends

Answer:

(i) Translation restrained at both ends and rotation free at both ends

Restrained	Free	Restrained	Free		1.0L

(ii) Translation and rotation restrained at both ends

Restrained	Restrained	Restrained	Restrained		0.65 L

5. **Design a tension member consisting of single unequal angle section to carry a tensile load of 340 kN. Assume single row 20 mm bolted connection. The length of member is 2.4 m. Take fu = 410 MPa, α = 0.80**

Section available (mm)	Area (mm^2)
ISA 100 × 75 × 8	1336
ISA 125 × 75 × 8	1538
ISA 150 × 75 × 8	1748

Solution:

(A) Appropriate gross area required

$$\text{Reqd } Ag = \frac{1.1 \times Tag}{fy}$$

$$= \frac{1.1 \times 340 \times 10^3}{250}$$

$$= 1496 \text{ mm}^2$$

Try 15A 125 × 75 × 8 mm giving Ag = 1538 mm^2 r_{min} = 16.1 mm. Assuming longer leg connected, check the strength of the section

i) Design strength due to yielding of gross section

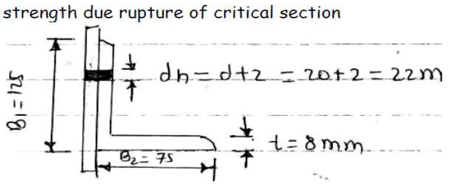
$$Tag = \frac{Ag \times fy}{r_{mo}}$$

$$= \frac{1538 \times 250}{1.10}$$

= 349545.4N

Tag = 349.54 kN

ii) Design strength due rupture of critical section



$$Tdn = \alpha An \frac{lu}{r_{m1}}$$

$An = Anc + Ago$

$Anc = (B_1 - d_h - t/2) \times t$

$= (125 - 22 - 8/2) \times 8$

Anc = 792 mm^2

$Ago = (B_2 - t/2)t$

$= (75 - 8/2)8$

= 568mm^2

An = Anc + Ago

An = 792 + 568

An = 13602 mm^2

Considering more than four bolt's in a raw α = 0.8

$$Tdn = \frac{0.8 \times 1360 \times 410}{1.25}$$

Tdn = 356.864 kN

Design of bolts

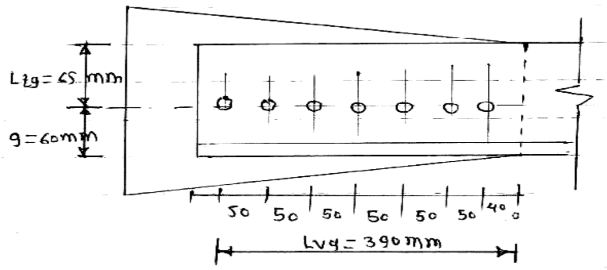
Capacity of bolts in single shear = 45. 3 KN

Capacity of bolt in bearing = 20 × 8 × 410 × 10^{-3}

= 65.6 kN

least bolt value = 45.3 kN (min of two above)

Number of bolts required = $\frac{340}{45.3}$ = 75 say 8



Assuming edge dist^n = 40mm
g = 60mm
Spacing of bolts = 50mm

$Avg = Lvg \times t = 390 \times 8 = 3120mm^2$

$Avg = 3120\ mm^2$

$Avn = \{Lug - [No. of bolts - 0.5\,dh]\} \times t$

$Avn = \{390 - [(8-0.5)22]\} \times 8 = 1800mm^2$

$Avn = 1800mm^2$

$Atg = Ltg \times t = 65 \times 8 = 500\ mm^2$

$Atg = 520\ mm^2$

$Atn = (65 - (0.5 \times 22)) \times 8$

$Atn = 432\ mm^2$

$Tdb_1 = Avg\,fy / (\sqrt{3} \times \gamma_{mc}) + 0.9 Atn\,fu / \gamma_{m1}$

$= 3120 \times 250 / (\sqrt{3} \times 1.10) + 0.9 \times 432 \times \dfrac{410}{1.25}$

$= 409393.8 + 127526.4$

$= 5369202\ N$

$Tdb_1 = 536.92\ kN$

$Tdb_2 = 424.962\ kN$

Tdb = lesser than Tdb_1 and Tdb_2 = 424.96 kN

$\therefore$ The tensile strength of angle = lesser of Tag, Tdn and Tdb
(349.54, 356.86 and 426.96)
= 349.54 kN

This is greater than required 340 kN

Check for slenderness ratio $\lambda = \dfrac{L}{r_{min}} = \dfrac{2400}{16.1}$

$149.06 < 250$

6. **Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.**

Solution:

1) Bolt force:

$P_x = 0;\ P_y = 100\ kN;$

Total eccentricity $x' = 200+250/2 = 325$ *mm*

$M = P_y x' = 100x325 = 32500$ *kN-mm*

Try the arrangement shown in Fig. E1
Note: minimum pitch = 60 mm and
minimum edge dist. = 60 mm

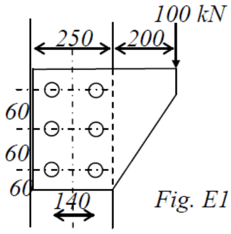


Fig. E1

$n = 6$

$\Sigma r_i^2 = \Sigma x_i^2 + \Sigma y_i^2 = 6(70)^2 + 4(60)^2 = 43800\ mm^2$

Shear force on the farthest bolts (corner bolts)

$$R_1 = \sqrt{\left\{\left[\frac{32500 \times 60}{43800}\right]^2 + \left[\frac{100}{6} + \frac{32500 \times 70}{43800}\right]^2\right\}} = 81.79\ kN$$

2) Bolt capacity
Try M20 HSFG bolts

Bolt capacity in single shear $= 1.1\ K\ \mu\ P_o = 1.1 \times 0.45 \times 177 = 87.6\ kN$

ISHB 400 flange is thicker than the bracket plate and so bearing on the bracket plate will govern.
Bolt capacity in bearing $= d\ t\ p_{bg} = 20 \times 8 \times 650 \times 10^{-3} = 104\ kN$

$\therefore$ *Bolt value* $= 87.6\ kN > 81.79$ *safe.*

7. **The shear lag width for ISA 75X75X10 is (Assume gauge distance = 40 mm).**

Solution:

The length of outstanding leg will be w = 75 mm and w1 = 40 mm.

So the shear lag width, bs = w + w1 – t = 75 + 40 – 10 = 105 mm.

8. **A single angle section 90X60X10 is connected with gusset plate with 7 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the design tensile strength of the section for rupture of net section? (Assume the section is connected with longer leg and gauge distance = 50 mm)**

Solution:

Anc = (90 - 10/2 - 22) × 10 = 630 mm2
Ago = (60 - 10/2) × 10 = 550 mm2
An = 630 + 550 = 1180 mm2

The length of outstanding leg will be w = 60 mm and w1 = 50 mm. So the shear lag width, bs = w + w1 – t = 60 + 50 – 10 = 100 mm.

Distance between end bolts , Lc = 6 × 50 = 300 mm.

$\beta = 1.4 - 0.076 \frac{b_s}{L_c} \times \frac{w}{t} \times \frac{f_y}{f_u} = 1.4 - 0.076 \times \frac{100}{300} \times \frac{60}{10} \times \frac{250}{410} = 1.307$

Thus, $T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta f_y A_{go}}{\gamma_{mo}} = \frac{0.9 \times 410 \times 630}{1.25} + \frac{1.307 \times 250 \times 550}{1.1}$

$= 349.35 \times 10^3$ N = 349.35 kN.

9. **A single ISA 75 × 50 × 8 is connected (longer leg) with gusset plate using use 4 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the Design tensile strength due to block shear failure? (Assume gauge distance = 35 mm)**

Solution:
Avg = 8 × (3 × 50 + 30) = 1440 mm2
Avn = 8 × (3 × 50 + 30 – 3.5 × 22) = 824 mm2
Atg = 8 × 40 = 320 mm2 [assuming gauge *g* = 35 for 75 mm leg]
Atn = 8 × (40 – 0.5 × 22) = 232 mm2

$$T_{db1} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{mo}} = \frac{0.9 \times 410 \times 824}{\sqrt{3} \times 1.25} + \frac{250 \times 320}{1.1} = 213.16 \times 10^3 \text{ N} = 213.16 \text{ kN}$$

$$T_{db2} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 f_u A_{tn}}{\gamma_{m1}} = \frac{1440 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410 \times 232}{1.25} = 257.44 \times 10^3 \text{ N} = 257.44 \text{ kN}$$

So, T_{db} = 213.16 kN.

10. **An ISA 90 x 90 x 8 used as tension member is connected to a 10 mm gusset plate by fillet weld of size 5 mm. The design strength of the member is 300 kN. Calculate the length of the weld.**

Solution: Force resisted by weld at lower side of angle $P_1 = 300 \times \frac{90-25.1}{90} = 216.33$ kN

Force resisted by weld at upper side of angle $P_2 = 300 \times \frac{25.1}{90} = 83.67$ kN

Assuming size of weld as 5mm, the throat thickness t_e will be 0.707 × 5 = 3.535 mm

Length required at lower side $L_{w1} = \frac{P_1}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{216.33 \times 10^3}{\frac{3.535 \times 410}{\sqrt{3} \times 1.25}} = 323.15$ mm ≈ 324 mm

Length required at upper side $L_{w2} = \frac{P_2}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{83.67 \times 10^3}{\frac{3.535 \times 410}{\sqrt{3} \times 1.25}} = 124.9$ mm ≈ 125 mm

DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

Assignment No. 03

1. **Check whether ISMB250@37.4 kg/m is suitable or not as a simply supported beam over an effective span of 6 m. The compression flange of beam is laterally supported throughout the span. It carries udl of 15 kN/m (including self wt.). Properties of ISMB 250 are bf = 125 mm, tf = 12.5 mm, tw = 6.9 mm, Ixx = 5131.6×104 mm4, Zxx = 410×103 mm3, r1 = 13.0 mm, Z_{px} = 465.71 × 10^3 mm^3, y_{m0} = 1.1, β_b = 1 and fy = 250 MPa.**

Solution:

(i) Loads and factored BMS

w = 15kN/m

Factored udl, wd = 15 × 1.5 = 22.5 kN/m

Factored BM, Md = $\frac{wd.le^2}{8} = \frac{22.5 \times 6^2}{8}$

=101.25 kN/m

Factored S.F. Vd = $\frac{wd.le}{2} = \frac{22.5 \times 6}{2}$ =67.5kN

(ii) Plastic modulus of section required

Z_p reqd. = $\frac{Md.\gamma_{mo}}{fy} = \frac{101.25 \times 10^6 \times 1.1}{250}$

= 445.5 × 10^3 mm^3

Z_p reqd. < Z_P avil (=465.71 × 10^3 mm^3)

(iii) Classification of beam section

d = h -2(f_t + γ_1) = 250 - 2(12.5 + 13)

= 199 mm

$\frac{bh}{tf} = \frac{\frac{125}{2}}{12.5} = 5.0 < 9.4$

$\frac{d}{tw} = \frac{199}{6.9} = 28.84 < 67$

As $\frac{bh}{tf} < 9.4$ and $\frac{d}{tw} < 67$ ∴ Section classification is plastic

(iv) Check for shear

$$Vdr = \frac{f_y \times tw \times h}{\gamma_{mo}\sqrt{3}} \quad \text{OR} \quad 0.525 \; fy.tw.h$$

$= \frac{250 \times 6.9 \times 250}{1.1 \times \sqrt{3}} = 226348$ N

= 226.35 KN > Vd (=67.5kN)

Also, $\frac{Vd}{Vdr} = \frac{67.5}{226.35} = 0.298 < 0.6$

∴ Check for shear is satisfied.

(v) Check for deflection

$\delta_{allowable} = \frac{L}{300}$

$= \frac{6000}{300}$

= 20 mm

$d_{max} = \frac{5}{384} \frac{wL^4}{FI}$

$= \frac{5}{384} \times \frac{15 \times 6000^4}{2 \times 10^5 \times 5131.6 \times 10^4}$

As $\delta_{max} > \delta_{allowable}$

∴ Deflection check is not O.K.

Hence, ISMB 250 is not a suitable section for given loading and span

2. **Limiting width to thickness ratio for single beam section of plastic class is 9.4 and d/tw = 84. State whether ISMB 500 @ 852 N/m is of plastic class or not. For ISMB 500; h = 500 mm, bf = 180 mm, tf = 17.2 mm, tw = 10.2 mm, r1 = 17.0 mm, fy = 250 MPa.**

Solution:

$\frac{h}{bf} < 8.4\,\varepsilon$ ... For class –1 (plastic)

Given section is ISMB500

$\therefore$ h = 500

& bf = 180

$$\frac{h}{bf} = \frac{500}{180} = 2.78 < 8.4\,\varepsilon$$

$$\text{but } \varepsilon = \sqrt{\frac{250}{f_y}}$$

$$\therefore \quad \varepsilon = \sqrt{\frac{250}{250}}$$

$$\varepsilon = 1$$

$$\therefore \frac{h}{bf} = 2.78 < 8.4 \times 1$$

$= 2.78 < 8.4$ ... hence the class is plastic

3. A hall of size 12m x 18m is provided with Fink type trusses at 3 m c/c. Calculate panel point load in case of Dead load and live load from following data.

a. **Unit weight of roofing = 150 N/m^2**
b. **Self-weight of purlin = 220 N/m^2**
c. **Weight of bracing = 80 N/m^2**
d. **Rise to span ratio = 1/5**
e. **No. of panels = 6**

Solution:

(A) Span of truss = 12 m

Spacing = 3m /c/c

Types of truss = sink

No. of panel point = 6

$$\text{Rise} = \frac{\text{span}}{5}$$

$$= \frac{12}{5} = 24\text{m}$$

$$\theta = \tan^{-1}\left(\frac{\text{Risc}}{L/2}\right) = \tan^{-1}\left(\frac{2.4}{6}\right) = 21.80^\circ$$

Calculation of dead load

(i) Weight of roofing = 150 N/m^2

(ii) Weight of Purlin = 220 N/m^2

(iii) Weight of truss = $\left(\frac{L}{3} + 5\right) \times 10$

$= 90$ N/m^2

(iv) Weight of bracing = 80 N/m^2

Total dead load = 540 N/m^2

Total dead load on one truss = 540 N/m^2

$= 540 \times 12 \times 3$

$= 19.44$ kN

$$\text{Dead load on each panel point} = \frac{19.44}{2} = 3.2\text{H kN}$$

D on end panel point = $\frac{324}{2}$

= 1.62 kN

Live load calculation

L.L. on purlin = 750 – (0 – 10) × 20)

= 750 – [2180 – 10) × 20]

= 514 N/m^2 > 400 N/m^2

L.L of truss = 2/3 × 514 = 342.67 N/m^2

∴ Total L. L. = L.L. of truss × span × spacing

= 342.67 × 12 × 3

= 12336 N

L.L. m each panel = $\frac{12336}{6}$ = 2056 N

L.L. m end panel = $\frac{2056}{2}$ = 1028 N

4. An industrial building has trusses for 14 m span. Trusses are spaced at 4m c/c and rise of truss in 3.6m. Calculate panel point load in case of live load and wind load using following data :

a. Coefficient of external wind pressure = - 0.7

b. Coefficient of internal wind pressure = ± 0.2

c. Design wind pressure = 1.5 kPa

d. Number of panels = 08

(A) Span of trus = 14m

Spacing of truss = 3.6 m

No. of panels = 8

Design wind pressure = 1.5 kpa

= 1.5 × 10^3 N/m^2

$$\theta = \tan^{-1}\left(\frac{\text{Rise}}{\text{Span}/2}\right) = \frac{3.6}{14/2} = 27.22^\circ$$

∴ θ = 27.22°

Wind load calculation

Coefficient of external wind pressure

Cpe = -0.7

Coefficient of internal wind pressure

Cpi = ±02

Total wind press = [Cpe – Cpi] × P_z

Wind load combination

i) w.c = [–0.7 – (0.2)] × 1500 = 750 N/ m^2

ii) w.c = [–0.7 – (+0.2)] × 1500 = 1350 N/m^2

Max. intensity = –1350 N/m^2

Length of principle dafter = $\frac{L/2}{\cos\theta}$

= $\frac{[1412]}{\cos 27.22}$

Length of principle dafter = 7.87 m

∵ Sloping area = 2 × 7.87 × 4

= 62.96 m^2

∵ Total wind load = Max. intensity × sloping area

= 1350 × 62.96

= 84996 N

Wind

∵ load an each panel = $\frac{84996}{8}$

∴ wind load on end panel = -10624.5 N

∴ wind load on end panel = $\frac{-10624.5}{2}$

= 5312.25 N

Live load calculation

Live load on purlin = 750 - [(θ - 10) × 20]

= 750 - [(27.22 - 10) × 20]

= 405.6 × 4v N/m^2

Hence ok

L.L. on truss

$= 2/3 \times 405.6$

$= 270.4 \ N/m^2$

$\because$ Total L.L $=$ L.L. intensity $\times$ Span $\times$ spacing

$= 270.4 \times 14 \times 4$

$= 15142.4$ N

$$\because \text{ load on each panel} = \frac{T.L}{\text{No. of Panel}}$$

$$= \frac{15142.4}{8}$$

$= 1892.8$ N $= 1.892$ kN

$$\text{and load on end panel} = \frac{1892.8}{2}$$

$= 946.4$ N $= 0.926$ kN

5. Design a slab base for column ISHB 400 @ 82.2 kg/m to carry factored axial compressive load of 2000 kN. The base rests on concrete pedestal of grade M20. For ISHB 400, bf = 250 mm, fy = 250 MPa, fu = 410 MPa, γ_{mo} = 1.1, tf = 12.7 mm.

Solution:

(A) Given

Factored load pu $= 2000$ kN

$= 2000 \times 10^3$ N

Fck = 20

D = 400

B = 250 i.e bf

$v_{mo} = 1.1$

tf = 12.7

fy = 250 N/mm^2

Bearing Strength of conc

$= 0.6$ fek

$= 0.6 \times 20 = 12 \ N/m^2m$

Bearing area of base plate

$$A = \frac{Pu}{\text{Bearing strength of conc}}$$

$$A = \frac{2000 \times 10^3}{12} = 166.67 \times 16^3$$

Size of base plate

length of plate

$$Lp = \frac{D-B}{2} + \sqrt{\left(\frac{D-B}{2}\right)^2 + A}$$

$$= \frac{400-250}{2} + \sqrt{\left(\frac{400-250}{2}\right)^2 + 1666.67 \times 10^3}$$

$= 490.08 \simeq 500$

$$Bp = \frac{A}{Lp} = \frac{166.67 \times 10^3}{500} = 333.34 \simeq 350$$

Larger Projection

$$a = \left(\frac{Lp-p}{2}\right) = \frac{500-400}{2} = 50 \text{ mm}$$

Smaller Projection

$$b = \left(\frac{Bp-B}{2}\right) = \frac{350-250}{2} = 50 \text{ mm}$$

Area of base plate

$Ap = 500 \times 350 = 175 \times 10^3$

Ultimate Pressure from below m the Slab base

$$w = \frac{Pu}{A} = \frac{2000 \times 10^3}{175 \times 10^3} = 11.42 \ N/mm^2$$

Thickness of slab base

$$f_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)vm_o}{fy}}$$

$$= \sqrt{\frac{2.5 \times 11.42(50^2 - 0.3 \times 50^2) \times 1.10}{250}}$$

$= 14.82$ mm $>$ tf i.e. k - 7

$\simeq 15$ mm

Hence provide slob base plate having dimension 500 × 350 × 15

6. Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base?

Answer:

Design steps to find thickness

1) To calculate area (A) of base plate

   A = Column load/Bearing strength

   Bearing strength of concrete = $0.6\ f_{ck}$

2) Select the size of base plate.

   L_p & B_p be the sizes of plate

   D = length or longer length

   B = width or shorter side of the column

   Consider square plate

   $$L_P = \frac{(D-B)}{2} + \sqrt{\left[\left\{\frac{(D-B)}{2}\right\}^2 + A\right]}$$

   $$B_P = \frac{A}{L_p}$$

   Large projection $a = \frac{(L_p - D)}{2}$

   Shorter projection $b = \frac{(B_p - B)}{2}$

   Area of base plate provided = $L_P \times B_P = (D + 2a) \times (B + 2b)$

3) Calculate ultimate bearing pressure

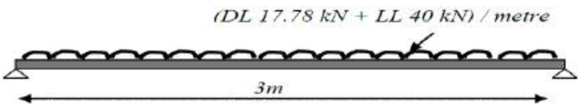
   $$w = \frac{P}{(L_p \times B_p)}$$ [1 mark]

4) Calculate thickness of base plate

   $$t_s = \left[\left(\frac{2.5 \times w(a^2 - 0.3 \times b^2) r_{mo}}{f_y}\right)\right]^{0.5}$$ [1 mark]

Function of anchor bolt : To connect concrete pedestal and base plate anchor bolts are used.

7. Design a suitable 'I' beam for a simply supported span of 3 m and carrying a dead or permanent load of 17.78 kN/m and an imposed load of 40 kN/m. Assume full lateral restraint and stiff support bearing of 100 mm.



Solution:

Design load calculation:

factored load = $\gamma_{LD} \times 17.78 + \gamma_{LL} \times 40$ *kN*

in this example the following load factors are chosen.

γ_{LD} and γ_{LL} *are taken as 1.35 and 1.50 respectively.*

γ_{LD} *– partial safety factor for dead or permanent loads*
γ_{LL} *– partial safety factor for live or imposed loads*

Total factored load = $1.35 \times 17.78 + 1.5 \times 40.0 = 84$ *kN / m*

Factored bending moment = $84 \times 3^2 / 8 = 94.50$ *kN – m*

Z—value required for f_y=250 MPa ; γ_m =1.15

$$Z_{reqd} = \frac{94.5 \times 1000 \times 1000 \times \gamma_m}{250}$$

Z_{reqd} = 434.7 cm ^{3}

Try ISMB 250

$$\varepsilon = \sqrt{\frac{250}{250}} = 1.0 \qquad D = 250 \text{ mm}$$

$B = 125$ mm
$t = 6.9$ mm
$T = 12.5$ mm
$I_{xx} = 5131.6 \text{ cm}^4$
$I_{yy} = 334.5 \text{ cm}^4$

Section classification:

Flange criterion = B/2T = 5.

Web criterion = (D – 2T)/t = 32.61

Since B/2T <8.92 ε & (D-2T)/t < 82.95 ε

The section is classified as ‘ **PLASTIC** ’

Moment of resistance of the cross section:

Since the section considered is ‘PLASTIC’

$$M_C = \frac{S \times f_y}{\gamma_m}$$

Where S is the plastic modulus

‘S’ for ISMB 250 = 459.76 cm^3

$M_c = 459.76 \times 1000 \times 250 / 1.15$

$= 99.95$ kN-m > 94.5 kN-m

Hence ***ISMB-250*** *is adequate.in flexure.*

Shear resistance of the cross section:

This check needs to be considered more importantly in beams where the maximum bending moment and maximum shear force may occur at the same section simultaneously, such as the supports of continuous beams. For the present example this checking is not required. However for completeness this check is presented.

Shear capacity $P_v = \dfrac{0.6 f_y A_v}{\gamma_m}$

$A_v = 250 \times 6.9 = 1725\ mm^2$

$P_v = 0.6 \times 250 \times 1725 / 1.15 = 225\ kN$

F_v = factored max shear = $84 \times 3 / 2 = 126.\ kN$

$F_v / P_v = 126/225.0 = 0.56 < 0.6$

Hence the effect of shear need not be considered in the moment capacity calculation.

Check for Web Buckling:

The slenderness ratio of the web $= L_E/r_y = 2.5\ d/t = 2.5 \times 194.1/6.9$

$= 70.33$

The corresponding design compressive stress f_c is found to be

$f_c = 203\ MPa$ (Design stress for web as fixed ended column)

Stiff bearing length = 100 mm

45^0 dispersion length $n_1 = 125.0$ mm

$P_w\ (100 + 125.0) \times 6.9 \times 203.0$

$= 315.16\ kN$

$315.16 > 126$ Hence web is safe against shear buckling

Check for web crippling at support

Root radius of ISMB 250 = 13 mm

Thickness of flange + root radius = 25.5 mm

Dispersion length (1:2.5) $n_2 = 2.5 \times 25.5 = 63.75$ mm

$P_{crip} = (100+63.75) \times 6.9 \times 250 / 1.15$

$= 245.63\ kN > 126 kN$

Hence ISMB 250 has adequate web crippling resistance

Check for serviceability – Deflection:

Load factors for working loads γ_{LD} and $\gamma_{LL} = 1.0$

design load = 57.78 kN/m.

$$\delta = \frac{5 \times 57.78 \times 3000^4}{384 \times 2.1 \times 10^5 \times 5131.6 \times 10^4}$$

Max deflection $= 5.65\ mm$

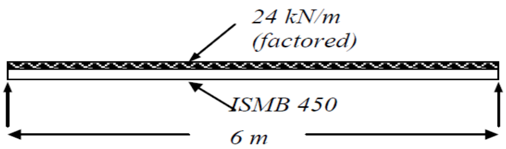
$$= \frac{L}{531}$$

$$\frac{L}{531} < \frac{L}{200}$$

Hence serviceability is satisfied

Result :-- Use ISMB – 250.

8. **Check the adequacy of ISMB 450 to carry a uniformly distributed load of 24 kN / m over a span of 6 m. Both ends of the beam are attached to the flanges of columns by double web cleat.**

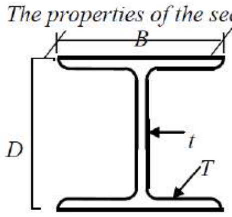


Design check:

For the end conditions given, it is assumed that the beam is simply supported in a vertical plane, and at the ends the beam is fully restrained against lateral deflection and twist with, no rotational restraint in plan at its ends.

Section classification of ISMB 450

The properties of the section are:



Depth, D = 450 mm

Width, B = 150 mm

Web thickness, t = 9.4 mm

Flange thickness, T = 17.4 mm

Depth between fillets, d = 379.2 mm

Radius of gyration about minor axis, $r_y = 30.1$ *mm*

Plastic modulus about major axis, $S_x = 1512.8 * 10^{-3}$ mm^3

Assume $f_y = 250$ N/mm^2, $E = 200000$ N/mm^2, $\gamma_m = 1.15$,

$p_y = f_y / \gamma_m = 250 / 1.15 = 217.4\ N / mm^2$

(I) Type of section

Flange criterion:

$$b = \frac{B}{2} = \frac{150}{2} = 75\ mm$$

$$\frac{b}{T} = \frac{75.0}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon \quad \text{where } \varepsilon = \sqrt{\frac{250}{f_y}}$$

Hence O.K.

Web criterion:

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\,\varepsilon$$

Hence O.K.

Since $\frac{b}{T} < 8.92\varepsilon$ and $\frac{d}{t} < 82.95\varepsilon$, the section is classified as 'plastic'

(II)Check for lateral torsional buckling:

Equivalent slenderness of the beam, $\lambda_{LT} = nuv\lambda$

where, n = slenderness correction factor (assumed value of 1.0)

u = buckling parameter (assumed as 0.9)

λ = slenderness of the beam along minor axis

$$= \frac{6000}{30.1} = 199.33$$

v = slenderness factor (which is dependent on the proportion of the flanges and the torsional index [D / T])

= 0.71 (for equal flanges and λ = 199.33)

Now, $\lambda_{LT} = 1.0 * 0.9 * 0.71 * 199.33$

= 127.37

Bending strength, p_b = 84 Mpa (for λ_{LT} = 127.37) (from Table 11 of BS 5950 Part I)

Buckling resistance moment $M_b = S_x * p_b$

$$= (1512.78 * 84)/1000$$

$$= 127.07 \text{ kN m}$$

For the simply supported beam of 6.0 m span with a factored load of 24.0 KN/m

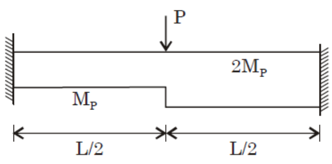
$$M_{max} = \frac{w\ell^2}{8} = \frac{24*6^2}{8}$$

$$= 108.0 \text{ KN m} < 127.07 \text{ kN m}$$

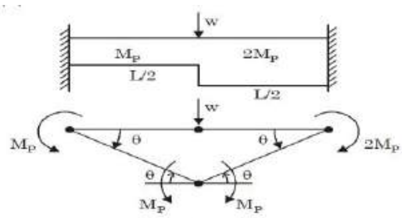
Hence $M_b > M_{max}$

ISMB 450 is adequate against lateral torsional buckling.

9. **A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to**



Solution:

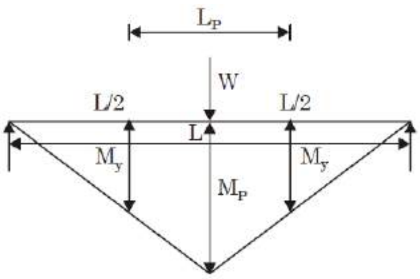


$$5M_P\theta = \frac{WL}{2}\theta$$

$$\Rightarrow \quad W = \frac{10M_P}{l}$$

10. **A simply supported beam of span L supports a concentrated load W at its midspan. If the cross-section of the beam is circular, then the length of elastic-plastic zone of the plastic hinge will be**

Solution:



$$\frac{M_P}{M_y} = \frac{\frac{L}{2}}{\frac{L-L_P}{2}}$$

$$\frac{Mp}{My} = \frac{1}{1-\frac{L_P}{L}}$$

$$1-\frac{Lp}{L} = \frac{1}{\frac{M_P}{M_y}}$$

$$L_P = L\left[1-\frac{1}{\frac{M_P}{M_y}}\right]$$

$$= L\left[1-\frac{1}{\text{Shape factor}}\right]$$

$$= L\left[1-\frac{1}{1.7}\right] = \frac{7L}{17}$$

Darbhanga College of Engineering, Darbhanga

Department of Civil Engineering

Max. Marks – 20 **Duration** – 2 Hrs **Subject**- Design of Steel Structure **Semester** -VI **Code**- 011620

Note: -Attempt all question 5 x 4= 20

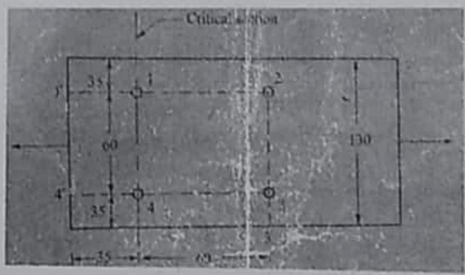
1. Write a brief note on different connections used in steel structures with diagram. {CO1}

OR

Explain different types of bolt with sketch. {CO1}

2. Determine the design tensile strength of the plate 130 mm x 12 mm with the holes for 16 mm diameter bolts as shown in figure. Steel used is of Fe 410 grade quality. {CO2}

$T_{db} = 332.986$ kN



OR

What are the various steps involved to determine the design strength of tensile members as per IS 800 (2007)? Explain briefly. {CO2}

3. Determine the design compressive strength for a column made up of ISHB 350 @ 710.2 N/m and 3.5 m high. The column is restrained in direction and position at both the ends. Use steel of grade Fe 410. {CO3}

$P_d = 2088.06$ kN

4. What do you understand by laterally supported beam? Determine the design bending strength of ISLB 350 @ 486 N/m considering the beam to be laterally supported. The design shear force V is less than the design shear strength. The unsupported length of the beam is 3.0 m. Assume steel of grade Fe 410. {CO4}

$M_d = 193.43$ kN

DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

MCQ: Total 20 **Time: 60 Minute**

Each question carries equal mark

1. **Which of the following is a correct criterion to be considered while designing?**
   a. Structure should be aesthetically pleasing but structurally unsafe
   b. Structure should be cheap in cost even though it may be structurally unsafe
   c. Structure should be structurally safe but less durable
   d. Structure should be adequately safe, should have adequate serviceability

2. **Tacking fasteners are used when**
   a. minimum distance between centre of two adjacent fasteners is exceeded
   b. maximum distance between centre of two adjacent fasteners is exceeded
   c. maximum distance between centre of two adjacent fasteners is not exceeded
   d. for aesthetic appearance

3. **Strength of bolt is**
   a. minimum of shear strength and bearing capacity of bolt
   b. maximum of shear strength and bearing capacity of bolt
   c. shear strength of bolt
   d. bearing capacity of bolt

4. **The types of welded joints does not depend on**
   a. size of members connected at joint
   b. type of loading
   c. area available for welding
   d. size of weld

5. **The design nominal strength of fillet weld is given by**
   a. f_u
   b. $\sqrt{3}\, f_u$
   c. $f_u/\sqrt{3}$
   d. $f_u/(1.1 \times \sqrt{3})$

6. **Which of the following is not true regarding effective throat thickness of weld?**
   a. Effective throat thickness should not be less than 3mm
   b. It should not exceed 0.7t or 1t, where t is thickness of thinner plate of elements being welded
   c. Effective throat thickness = K x size of weld, where K is a constant
   d. Effective throat thickness = K x $(\text{size of weld})^2$, where K is a constant

7. **Which of the following is true about built up section?**
   a. Built up members are less rigid than single rolled section
   b. Single rolled section are formed to meet required area which cannot be provided by built up members
   c. Built up members can be made sufficiently stiff

d. Built up sections are not desirable when stress reversal occurs

8. **What is the maximum effective slenderness ratio for members always in tension?**
   a. 400
   b. 200
   c. 350
   d. 150

9. **The design tensile strength of tensile member is**
   a. minimum of strength due to gross yielding, net section rupture, block shear
   b. maximum of strength due to gross yielding, net section rupture, block shear
   c. strength due to gross yielding
   d. strength due to block shear

10. **What is the net section area of steel plate 40cm wide and 10mm thick with one bolt if diameter of bolt hole is 18mm?**
    a. 38.2 cm^2
    b. 20 cm^2
    c. 240 mm^2
    d. 480 mm^2

11. **Which of the following is property of compression member?**
    a. member must be sufficiently rigid to prevent general buckling
    b. member must not be sufficiently rigid to prevent local buckling
    c. elements of member should be thin to prevent local buckling
    d. elements of member need not prevent local buckling

12. **What is slenderness ratio of compression member?**
    a. ratio of effective length to radius of gyration
    b. ratio of radius of gyration to effective length
    c. difference of radius of gyration and effective length
    d. product of radius of gyration and effective length

13. Minimum size of weld for a plate of size less than 10 mm is
    a. 3 mm
    b. 5 mm
    c. 6 mm
    d. 8 mm

14. **The web is susceptible to shear buckling when d/t_w**
    a. $<67\varepsilon$
    b. $< 2\times67\varepsilon$
    c. $>67\varepsilon$
    d. $< 70\varepsilon$

15. **Which of the following condition is true for kinematic theorem?**
    a. load must be greater than collapse load
    b. load must be less than collapse load
    c. load must be not equal to collapse load
    d. load cannot be related to collapse load

16. **Imperfection factor α for buckling class c will be?**
   a. 0.49
   b. 0.34
   c. 0.76
   d. 0.56

17. **A plate of size 200 mm x 15 mm with 16 mm diameter bolts in single row. Total no. of bolts in the plate is 3 and the plate is in tension. What will be the design strength of plate due to yielding of gross section?**
   a. 671.8 kN
   b. 636.5 kN
   c. 646.5 kN
   d. 681.8 kN

18. **A simply supported beam of effective span 1.5 m carrying a factored concentrated load of 360 kN at mid span. What will be the plastic section modulus of the beam?**
   a. 394000 mm^3
   b. 494000 mm^3
   c. 594000 mm^3
   d. 694000 mm^3

19. **A 3 m long column carries a factored load of 4500 kN. The column is effectively held at both the ends and restrained in direction at one of the ends. What will be the value of slenderness ratio of the member? Assume minimum radius of gyration of the member is 150 mm.**
   a. 14
   b. 16
   c. 18
   d. 20

20. **The shear leg width for ISA 100x100x10 is (Assume gauge = 40 mm)**
   a. 150
   b. 140
   c. 130
   d. 120

Darbhanga College of Engineering, Darbhanga

Department of Civil Engineering

B.Tech [VIth Semester (CE)]

Mid. Sem Exam

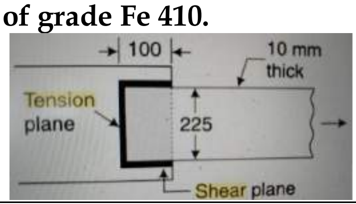
(Session: 2020-21)

Course Code–011620

Design of Steel Structures

Time: 2 Hours ***Max. Marks: 20***

Note: Attempt all questions. CO–Course Outcomes, BL–Bloom Level
Assume any suitable data if required

S. No.	Question	Marks	CO	BL
1.	Explain various types of rolled steel section with suitable diagram or Calculate the strength of a 20 mm diameter bolt of grade 4.6 for the following cases. The main plates to be jointed are 12 mm thick. a) Lap joint b) Single cover butt joint; the cover plate being 10 mm thick c) Double cover butt joint; each of the cover plate being 8 mm thick.	5	CO1	L2
2.	Explain various steps involved to determine the design strength of tensile member as per IS 800 (2007). or Determine the block shear strength of the welded tension member shown in fig. Steel is of grade Fe 410. 	5	CO3	L4
3.	Determine the design axial load capacity of the column ISHB 300 @ 577 N/m if the length of column is 3 m and its both ends pinned.	5	CO4	L5
4.	An I-section beam is fabricated with plates of following dimensions. Flanges: 380 x 20 mm Web: 1600 x 15 mm Classify flanges, web and the section	5	CO5	L5

Code : 011620

B.Tech 6th Semester Exam., 2016

DESIGN OF STEEL STRUCTURES

Time : 3 hours *Full Marks* : 70

Instructions :

(i) The marks are indicated in the right-hand margin.

(ii) There are ***NINE*** *questions in this paper.*

(iii) Attempt ***FIVE*** *questions in all.*

(iv) Question No. 1 is compulsory.

1. Write short answers of the following (any *seven*) : 2×7=14

(a) Define and differentiate between pitch and gauge for riveted joint.

(b) Find the shape factor for square of side a with its diagonal parallel to the zz-axis.

(c) Find the rivet value for 20 mm dia power-driven rivets which are connecting two plates of thickness 14 mm and 16 mm by lap joint.

(d) Draw the bending stress diagram under a column base which is subjected to a point load p at an eccentricity e.

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(e) An I-section beam is fabricated with plates of the following dimensions :

Flanges : 600 mm × 20 mm

Web : 1600 mm × 12 mm

Classify flanges, web and the section. Also determine the plastic moment capacity of the beam about its strong axis, if the grade of steel is Fe 410.

(f) Draw a roof truss and label the following members on it :

(i) Upper chord member

(ii) Lower chord member

(iii) Principal rafter

(g) A continuous beam of constant M_p has three equal span and carries total uniformly distributed load W on each span. Find the value of collapse load for the beam.

(h) For a welded plate girder with vertical stiffeners, what is the maximum depth of web provisionable in design when the thickness of the web plate is 5 mm?

(i) What are laterally supported and unsupported beams?

(j) Why is curtailment of flanges carried out in the design of a plate girder?

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2. (a) A tie member consisting of an ISA 80 mm × 50 mm × 8 mm (Fe 410 graded steel) is welded to a 12 mm thick gusset plate at site. Design welds to transmit load equal to the design strength of the member. 12

(b) Concave shape fillet welds are avoided. Comment. 2

3. Find out the collapse load for a propped cantilever subjected to a uniformly distributed load W/unit length over its entire span. 14

4. (a) Explain the defects in welded connections with appropriate figures. 3

(b) Two plates, 10 mm and 18 mm thick, are to be jointed by double-cover butt joint. Design the joint for the following data : 11

Factored design load	:	750 kN
Bolt diameter	:	20 mm
Grade of steel	:	Fe 410
Grade of bolts	:	4.6
Cover plates 2 (One on each side)	:	8 mm thick

5. A tension member 0·95 m long is to resist a service dead load of 20 kN and a service live load of 60 kN. Design a rectangular bar of standard structural steel of grade Fe 410. Assume that the member is connected by one line of 16 mm diameter bolts of grade 4·6. 14

6. Design a built-up column 10 m long to carry factored axial load of 1080 kN. The column is restrained in position but not in direction at both the ends. Provide single lacing system with bolted connections. Assume that steel is of grade Fe 410 and bolts are of grade 4·6.

(a) Design the column with two channels placed back-to-back.

(b) Design the column with two channels placed toe-to-toe.

(c) Which of the two systems is economical? 14

7. An ISLB 300 @ 369·8 N/m transmits an end reaction of 385 kN, under factored loads, to the web of ISMB 450 @ 710·2 N/m. Design a bolted framed connection. Steel is of grade Fe 410 and bolts are of grade 4·6. 14

8. (a) Determine the design bending strength of ISLB 350 @ 486 N/m considering the beam to be laterally supported. The design shear force V is less than the design shear strength. The unsupported length of the beam is 3·0 m. Assume that steel is of grade Fe 410. 7

(b) A simply supported steel joist of 4·0 m effective span is laterally supported throughout. It carries a total uniformly distributed load of 40 kN (inclusive of self-weight). Design an appropriate section using steel of grade Fe 410. 7

(Continued)

9. Design the principal rafter of a fink-type roof truss for the following data. Design also its connection using 20 mm diameter bolts : 14

Design compressive load—

165 kN (due to D.L. and L.L.)

Design tensile load—

60 kN (due to D.L. and W.L.)

Length of rafter panel—2·235 m

Grade of steel—Fe 410

Grade of bolts—4·6

★★★

Code : 011620

B.Tech 6th Semester Examination, 2017

Design of Steel Structures

Time : 3 hours *Full Marks : 70*

Instructions :

(i) *There are **Nine Questions** in this Paper*

(ii) *Attempt **Five** questions in all.*

(iii) ***Question No. 1 is Compulsory.***

(iv) *The marks are indicated in the right-hand margin*

1. Write short answer (any seven) of the following: 2×7=14

(a) A propped cantilever of span L is subjected to a concentrated load of mid span. If MP is plastic moment capacity of beam then find the value of collapse load.

(b) Give disadvantages of welded joints.

(c) What is slenderness ratio? How does it affect the load carrying capacity of column.

(d) Give the factor by which effective length of battened column is altered

(e) A groove weld is to connect two plates 180 mm × 18 mm each. Determine the design bending strength of the joint, if it is subjected to a moment of 13 KNm. Also, determine the adequacy of the joint, if the shear force at the joint is

P.T.O.

200 kN. Assume the welds to be of double-U shop welded

(f) Give, in detail, various load considered for the design of roof Trusses.

(g) Calculate the value of the least radius of gyration for a compound column consisting of ISHB 250 @ 536.6 N/m with one cover plate 300 mm × 20 mm an each flange.

(h) What is the difference in behaviour of long and intermediate column?

(i) Differentiate between the bending and buckling of beam.

(j) Why is a reduction of live loads done for the columns of multi-storey structures?

2. (a) A 120 mm diameter and 6 mm thick pipe is fillot welded to a 14 mm plate. It is subjected to a vertical factored load of 4.5 kN at 1.0 m from the welded end and a factored twisting moment of 1.8 KNm. Design the joint assuming shop welding and steel of grade Fe 410. 12

(b) In what situation are concave fillet welds recommended? 2

3. Explain the term 'plastic hinge". Explain the theorems of plastic collapse. Find out he collapse load for a fixed beam subjected to a point load 'W' at its mid span. 14

4. (a) Explain various modes of failure (behaviour) of bolted connections, with neat sketches. 3

(b) Two flats (Fe 410 Grade steel), each 210 mm × 8 mm, are to be jointed using 20 mm diameter, 4.6 grade bolts, to form a lap joint. The joint is supposed to transfer a factored load of 250 KN. Design the joint and determine suitable pitch for the bolts. 11

5. Design a bridge truss diagonal subjected to a factored tensile load of 300 KN. The length of the diagonal is 3.0 m. The tension member is connected to a gusset plate 16 mm thick with one line of 20 mm diameter bolts of grade. 8.8. 14

6. Design a double angle discontinuous strut to carry a factored load of 135 KN, resulting from combination with wind load. The length of the strut is 3.0 m between intersections. The two angles are placed back-to-back (with long legs connected) and are tack bolted. Use steel of grade Fe 410.

(i) Angles are placed on opposite sides of 12 mm gusset plate.

(ii) Angles are placed on same side of 12 mm gusset plate.

 14

7. Design a welded plate girder 24 m in span and laterally restrained throughout. It has to support a uniform load of 100 KN/m throught the span exclusive of self-weight. Design the girder without intermediate transverse stiffeners. The steel for the flange and web plates is of grade Fe 410. Design the

cross section, the end load bearing stiffener and connections 14

8. (a) An I-section beam is fabricated with plates of following dimensions

Flanges : 600 × 20 mm

Web : 1600 × 12 mm

Classify flanges, web and the section. Also determine the plastic moment capacity of the beam about its strong axis, if the grade of steel is Fe 410. 7

(b) Determine the design bending strength of ISLB 350 @ 486 N/m considering the beam to be laterally unsupported. The design shear force V is less than the design shear strength. The unsupported length of the beam is 3.0 m. Assume steel of grad Fe 410. 7

9. Design a strut in a roof truss for the following data. 14

Length of strut	: 2.235 m
Factored compressive force	: 50 KN (due to D.L. and L.L.)
Factored tensile force	: 17.80 KN (due to D.L. and W.L.)
Grade of steal	: Fe 410
Grade of bolts	: 4.6
Bolt diameter	: 20 mm

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Code : 011620

B.Tech 6th Semester Exam., 2018

DESIGN OF STEEL STRUCTURE

Time : 3 hours *Full Marks* : 70

Instructions :

(i) *The marks are indicated in the right-hand margin.*

(ii) *There are* ***NINE*** *questions in this paper.*

(iii) *Attempt* ***FIVE*** *questions in all.*

(iv) *Question No.* ***1*** *is compulsory.*

1. Choose the correct answer of the following (any *seven*) : 2×7=14

(a) In plastic analysis, the shape factor for circular sections is

(i) 1·5

(ii) 1·6

(iii) 1·697

(iv) None of the above

(b) A beam is defined as a structural member subjected to

(i) axial loading

(ii) transverse loading

(iii) axial and transverse loading

(iv) None of the above

(c) The failure of a web plate takes place by yielding if the ratio of the clear depth to thickness of the web is less than

(i) 45 (ii) 55

(iii) 60 (iv) 82

(d) The most economical section for a column is

(i) rectangular

(ii) solid round

(iii) flat strip

(iv) tubular section

(e) The distance between e.g. of compression and e.g. of tension flanges of a plate girder is known as

(i) overall depth

(ii) clear depth

(iii) effective depth

(iv) None of the above

(f) The allowable stress, in axial tension for rolled I-sections and channels, is taken as

(i) 1420 kg/cm^2 (ii) 1500 kg/cm^2

(iii) 2125 kg/cm^2 (iv) 1810 kg/cm^2

(g) If d is the distance between the flange angles, the vertical stiffeners in plate girders are spaced not greater than

(i) d

(ii) $1 \cdot 25d$

(iii) $1 \cdot 5d$

(iv) $1 \cdot 75d$

(h) The cross-section of a standard fillet is a triangle whose base angles are

(i) 45° and 45°

(ii) 30° and 60°

(iii) 40° and 50°

(iv) 20° and 70°

(i) A second horizontal stiffener is always placed at the neutral axis of the girder if the thickness of the web is less than

(i) $d/250$ for structural steel

(ii) $d/225$ for high tensile steel

(iii) Both (i) and (ii)

(iv) Neither (i) nor (ii)

(j) The thickness t of a single flat lacing should not be less than

(i) 1/30th length between inner end rivets

(ii) 1/40th length between inner end rivets

(iii) 1/50th length between inner end rivets

(iv) 1/60th length between inner end rivets

2. Explain the following : 14

(a) Local and lateral buckling of beam

(b) Checks required for beam design

3. Calculate the design compressive load for a column made up of ISHB 350 @ 710·2 N/m and 3·5 m high. The column is restrained in direction and position at both the ends. Use steel of grade Fe 410. 14

4. Design a simply supported beam of span 4·2 m carrying reinforce concrete floor in which top compression flange is embedded. Beam is carrying 20 kN/m dead load and 20 kN/m imposed load, resume Fe 410 grade steel. 14

5. Design a suitable angle section to carry tensile force of 250 kN. Use welded connection. 14

6. Discuss the following : 14

(a) Prying action

(b) Advantage of fillet weld over butt weld

(c) Comparison of welded joints with bolted joints

7. *(a)* Explain some of the common defects in the welds. 7

(b) Write the advantage of welded joints over bolted joints. 7

8. Design a tension member to carry a pull of 830 kN. The member is 3·2 m between c/c of intersections. Design the member using channel section. 14

9. A tie member of truss consists of double angle section each 80 mm × 80 mm × 8 mm welded on the opposite side of a 12 mm thick gusset plate. Design a fillet weld for making connection in the workshop. The factored tensile force in the member is 300 kN. 14

★★★

Code : 011620

B.Tech 6th Semester Exam., 2019

DESIGN OF STEEL STRUCTURE

Time : 3 hours *Full Marks* : 70

Instructions :

(i) The marks are indicated in the right-hand margin.

*(ii) There are **NINE** questions in this paper.*

*(iii) Attempt **FIVE** questions in all.*

*(iv) Question No. **1** is compulsory.*

1. Choose the correct answer of the following (any *seven*) : 2×7=14

(a) Generally, the purlins are placed at the panel points so as to avoid

(i) axial force in rafter

(ii) shear force in rafter

(iii) deflection of rafter

(iv) bending moment in rafter

(b) The effective length of a fillet weld should not be less than

(i) two times the weld size

(ii) four times the weld size

(iii) six times the weld size

(iv) weld size

(c) In moment resistant connections, the moment resistance of riveted connection depends upon

(i) shear in rivets

(ii) compression in rivets

(iii) tension in rivets

(iv) strength of rivets in bearing

(d) The thickness of the web of a mild steel plate girder is less than $d/200$. If only one horizontal stiffener is used, it is placed at

(i) the neutral axis of the section

(ii) 2/3rd of the depth of the neutral axis from the compression flange

(iii) 2/5th of the depth of the neutral axis from the compression flange

(iv) 2/5th of the height of the neutral axis from the tension flange

(e) The risk coefficient k depends on

(i) mean probable design life of structures

(ii) basic wind speed

(iii) Both *(i)* and *(ii)*

(iv) None of the above

(f) Stiffeners are used in a plate girder

(i) to reduce the compressive stress

(ii) to reduce the shear stress

(iii) to take the bearing stress

(iv) to avoid bulking of web plate

(g) When the bolts are subjected to reversal of stresses, the most suitable type of bolt is

(i) black bolt

(ii) ordinary unfinished bolt

(iii) turned and fitted bolt

(iv) high strength bolt

(h) The overlap of batten plates with the main members in welded connections should be more than

(i) 3t

(ii) 4t

(iii) 6t

(iv) 8t

where *t* is thickness of batten plate.

(i) The effective length of a compression member of length L held in position at both ends but not restrained in direction, is

(i) L

(ii) $0 \cdot 67L$

(iii) $0 \cdot 85L$

(iv) $1 \cdot 5L$

(j) Pick up the correct statement from the following.

(i) The minimum pitch should not be less than 2·5 times the gross diameter of the rivet.

(ii) The minimum pitch should not be less than 12 times the gross diameter of the rivet.

(iii) The maximum pitch should not exceed 10 times the thickness or 150 mm whichever is less in compression.

(iv) All of the above

2. (a) Give the details of various rolled steel section with suitable diagram. 7

(b) A tension bar 100 mm × 100 mm is to carry a load of 150 kN. A specimen of the same quality steel cross-section area 800 mm^2 was tested in the workshop. The maximum load carried by the specimen was 400 kN. Find the ultimate tensile strength, factor of safety in the design and the gauge length. 7

3. (a) Explain in brief various types of loads to be considered in the design of steel structure. 7

(b) A portal frame consists of two hinge supported column of 4 m height separated by a beam of span 5 m and loaded up to collapse with downward uniformly distributed load of 15 kN/m and lateral point load of 50 kN at left beam column junction. Find the plastic moment of resistance if it is of uniform strength. 7

4. (a) Differentiate between fillet weld and butt weld. Under which conditions fillet weld is preferred? 7

(b) Why is lacing provided in the columns? 7

5. (a) A tie bar 100 mm × 16 mm is welded to another plate. It is subjected to factored pull of 300 kN. Find the minimum overlap required if 8 mm site fillet welds are used. Assume any missing data. 7

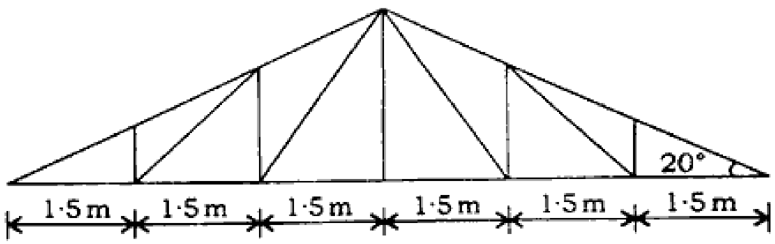
(b) Write down the steps for the design of axially loaded columns. 7

6. A simply supported beam of span 6 m supports a reinforced concrete slab. The compression flange of the beam is restrained due to its connection with the slab. The beam is subjected to a dead load of 25 kN/m and an imposed load of 20 kN/m. Design the beam. 14

7. Design a grillage foundation with I-sections for a column having a load of 5000 kN. Column is provided with a base plate of size 700 mm × 800 mm. Take bearing capacity of the soil as 200 kN/m^2. Assume any missing data. 14

8. Design a built-up tension member to carry a factored force of 340 kN. Use 20 mm diameter black bolts and gusset plate of 8 mm thick. 14

9. The line diagram of a steel truss of 9 m span, angle of slope 20° is shown in the figure below. The roof sheeting is of corrugated GI sheets of unit weight 150 N/sq.m of plan area. The truss supports purlins of unit weight 75 N/sq.m of plan area. The weight of bracing used may be taken as 20 N/sq.m of plan area. The spacing of trusses is 4 m and height of eaves 4·5 m. If the building is of medium permeability, determine *(a)* dead load, *(b)* wind load and *(c)* live load at various panel points of the truss. **Assume design** wind pressure as 1500 N/mm^2 : **14**



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Problem 1

Design a hand operated overhead crane, which is provided in a shed, whose details are:

Capacity of crane = 50 kN

Longitudinal spacing of column = 6m

Center to center distance of gantry girder = 12m

Wheel spacing = 3m

Edge distance = 1m

Weight of crane girder = 40 kN

Weight of trolley car = 10 kN

Design by allowable stress method as per IS: 800 - 1984

To find wheel load (refer fig.1):

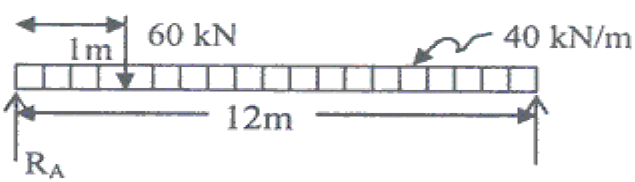


Fig.1

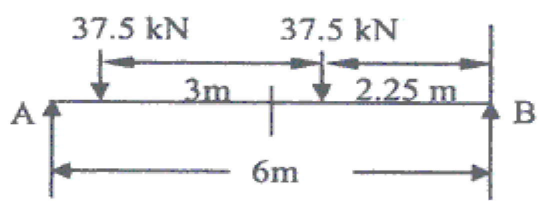


Fig.2

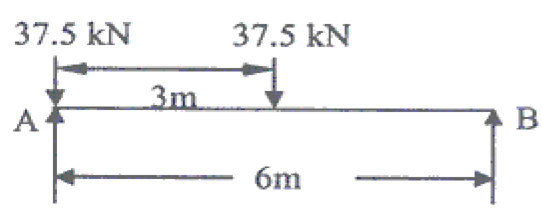


Fig.3

R_A = 20 + 60 (11 / 12) = 75 kN

Wheel load = R_A / 2 = 37.5 kN

To find maximum BM in gantry girder (refer fig.2):

R_A = 46.88 kN

R_B = 28.12 kN

Max. BM = 28.12 x 2.25 = 63.27 kN-m

Adding 10% for impact,

M_1 = 1.1 x 63.27 = 69.60 kN-m

Max. BM due to self - weight of girder and rail taking total weight as 1.2 kN/m

$$M_2 = \frac{wl^2}{8} = 5.4 kNm$$

Therefore Total BM, M = 75 kN-m

To find maximum shear force (refer fig.3):

SF = R_A = 59.85 kN.

To find lateral loads:

This is given by 2.5% of (lateral load / number of wheel = 0.025 x 60 / 2 kN = 0.75 kN

Therefore Max BM due to lateral load by proportion is given by, ML = (63.27 / 37.5) x 0.75 = 1.27 kN-m

Design of section

Approximate section modulus Z_c required, $(M / \sigma_{bc}) = 75 \times 10^6 / 119 = 636 \times 10^3 mm^3$ [for λ = 120, D / T = 25].

Since, the beam is subjected to lateral loads also, higher section is selected.

For, ISMB 450 @ 710.2 N/m,

$Z_x = 1350.7\ cm^3$, T = 17.3 mm,

$t = 9.4mm$, $I_{yy} = 111.2cm^3$

$r_y = 30.1mm$,

$b_f = 150mm$

To find allowable stresses,

$T/t = 17.4 / 9.4 = 1.85 < 2$

$D/T = 450 / 17.4 = 25.86 \sim 26$

$L/r_y = 6000 / 30.1 = 199.3 \sim 200$

Therefore allowable bending compression about major axis is, $s_{bc'\,x} = 77.6$ N/mm^2

Actual stress in compression side, $s_{b'\,x} = M / Z = 55.5\ N / mm^2$.

The bending moment about Y-axis is transmitted only to the top of flange and the flange is treated as rectangular section. The allowable stress is $s_{bc.\,y} =$ 165 MPa (i.e. 0.66 fy).

Z_y of the flange = $111.2 / 2 = 55.6\ cm^3$

Therefore $s_{by} = M_y / Z_y = 1.27 \times 10^6 / 55.6 \times 10^3$

$= 22.84$ MPa

The admissible design criteria is = $(s_{bx} / s_{bcx}) + (s_{by} / s_{bcy}) = (55.5 / 77.6) + (22.84 / 165) = 0.715 + 0.138 = 0.854 < 1$. Hence , the design is safe. So, ISMB 450 is suitable

Check for shear:

Design shear stress, τ_x $V / (Dt) = 59.85 \times 10^3 / 450 \times 9.4 = 14.15$ MPa. This is less than τ_a $(0.4f_y)$. Hence, the design is o.k.

Check for deflection and longitudinal bending can be done as usual.

Design by limit state method as per IS: 800 draft code

For ISMB 450, properties are given below:

T = 17.4mm, t = 9.4mm, b = 150mm, r_y = 30.1mm, Z_p = 1533.33 cm^3, Z_c = 1350.7 cm^3, Shape factor = 1.15, I_{zz} = 30390.8 cm^4, H^1 = d = 379.2mm

Section classification:

Flange criteria: b / T = 75 / 17.4 = 4.31 < 9.4

No local buckling. Therefore OK

Web criteria: d / t_w = 379.2 / 9.4

= 40.34 < 83.9

No local buckling. Therefore OK

Section plastic.

Shear capacity:

$$F_{vd} = (f_{yw} A_v) / (\sqrt{3}\gamma_{mo})$$
$$= (250 \times 450 \times 9.4) / (\sqrt{3} \times 1.1)$$
$$= 555043 \text{ N} \cong 555 \text{ kN}$$

F_v / F_{vd} = (59.85 x 1.5) / 555 = 0.1634 < 0.6

Check for torsional buckling (CI.8.2.2):

t_f / t_w <= 17.4 / 9.4 = 1.85 < 2.

Therefore β_{LT} = 1.2 for plastic section

M_{cr} = Elastic critical moment

$$M_{cr} = \frac{\beta_{LT}\pi^2 h E I_y}{2(Kl)^2}\left[1 + \frac{1}{20}\left\{\frac{\frac{KL}{r_y}}{\frac{h}{t_f}}\right\}^2\right]^{0.5}$$

$$= \frac{1.2\pi^2 \times 450 \times 2 \times 10^5 \times 834 \times 10^4}{2 \times 6000^2} \times \left[1 + \frac{1}{20}\left\{\frac{\frac{6000}{30.1}}{\frac{450}{17.4}}\right\}^2\right]^{0.5}$$

$$= 246 \times 10^6 \ N\text{-}m$$

$$Now,\ \lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} = 1.248$$

(β_b = 1 for plastic section)

Therefore $\phi_{LT} = 0.5\ [1 + \alpha_{LT}\ (\lambda_{LT} - 0.2) + \lambda^2_{LT}$

$= 0.5\ [1 + 0.21\ (1.248 - 0.2) + 1.248^2]$

$= 1.389$

$$X_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}}$$

$$= 1/1/999 = 0.5$$

$$f_{bd} = \frac{X_{LT}\, f_y}{\gamma_{mo}} = (0.5 \times 250)\ /\ 1/1$$

$$= 113.7\ MPa$$

Therefore $M_d = \beta_b.\ z_p.\ f_{bd}$

$= 1 \times 1533.3 \times 10^3 \times 113.7$

$= 1.7434 \times 10^8 = 174.34$ kN-m

Factored longitudinal moment, $M_f = 75 \times 1.5 = 112.5$ kN-m

Factored lateral moment, $M_{fL} = 1.27 \times 1.5 = 1.91$ kN-m

Lateral BM capacity = M_{dL}

$$= \frac{Z_{yy}.f_y}{1.10} = \frac{\frac{Z_{ey}}{2}(Shape Factor) f_y}{1.10}$$

$$= \frac{\left[\frac{111.2 \times 10^3}{2}\right] 1.15 \times 250}{1.10}$$

$$= 14.53 \times 10^6 \ N \ or \ 14.53 \ kN - m$$

For Safety,

$$\frac{M_f}{M_d}\Big]_{longi} + \frac{M_f}{M_d}\Big]_{lateral}$$

= (112.5 / 135.9) + (1.91 / 14.53) = 0.96 < 1

Hence, it is safe

Problem 2:

Design a member (beam - column) of length 5.0^M subjected to direct load 6.0^T (DL) and 5.0^T (LL) and bending moments of M_{zz} {3.6^{TM} (DL) + 2.5^{TM} (LL)} and M_{yy} {0.55^{TM} (DL) + 0.34^{TM} (LL)} at top and M_{zz} {5.0^{TM} (DL) + 3.4^{TM} (LL)} and M_{yy} {0.6^{TM} (DL) + 0.36^{TM} (LL)} at bottom.

LSM {Section 9.0 of draft IS: 800 (LSM version)}

Factored Load,

N = 6.0 x 1.50 + 5.0 x 1.5 = 16.5^T (Refer Table 5.1 for load factors)

Factored Moments:

At top, $M_z = 9.15^{TM}$, $M_y = 1.335^{TM}$

At bottom, $M_z = 12.60^{TM}$, $M_y = -1.44^{TM}$

Section used = MB500.

For MB500:

$A = 110.7\ cm^2$; $r_{yy} = 3.52$ cm,

$I_{zz} = 48161.0\ cm^4$; $Z_{zz} = 1809\ cm^3$

$Z_{yy} = 152.2\ cm^3$; $Z_{pz} = 2074.7\ m^3$;

$Z_{py} = 302.9\ cm^3$

$d = 465.6$ mm, $t_w = 10.29$ mm

$r_{zz} = 20.21$ cm, $b = 180$ mm;

$I_{yy} = 1369.8\ cm^3$

(i) Sectional Classifications (Refer Fig: 3.1 and Table - 3.1 of draft code):

$b / t_f = 90 / 17.2 = 5.23 < 8.92$, therefore the section is plastic.

$d / t_w = 465.6 / 10.2 = 45.65 < 47$, therefore the section is semi-compact for direct load and plastic for bending moment.

(ii) Check for resistance of the section against material failure due to the combined effects of the loading (Clause-9.3.1):

N_d = Axial strength = $A.\ f_y / \gamma_{mo}$

$= 110.7 \times 2500 / 1.10 \times 10^{-3} = 251.68^T$

$N = 16.5^T$

$n = N / N_d = 16.5 / 251.68 = 0.066 < 0.2$

Therefore $M_{ndz} = 1.1 M_{dz} (1-n) < M_{dz}$

Therefore $M_{ndz} = 1.1\{1.0 * 2074.7 * 2500 / 1.10\} (1 - 0.066) * 10^{-5} < M_{dz}$

Therefore $M_{ndz} = 48.44^{T\text{-}m} > M_{dz}\ (=47.15^{T\text{-}m})$

Therefore $M_{ndz} = M_{dz} = 47.15^{T\text{-}m}$

For, $n <= 0.2$ $M_{ndy} = M_{dy}$

$M_{ndy} = (1.0 \times 302.9 \times 2500) / (1.10 \times 100000) = 6.88^{T\text{-}m}$

($\beta_b = 1.0$ for calculation of M_{dz} and M_{dy} as per clause 8.2.1.2)

Therefore $(M_y / M_{ndy})^{\alpha 1} + (M_z / M_{ndz})^{\alpha 2} = (1.44 / 6.88)^1 + (12.60 / 47.15)^2 = 0.281 <= 1.0$

{α_1 = 5n but >= 1, therefore α_1 = 5 x 0.066 = 0.33 = 1

α_2 = 2 (As per Table 9.1)}

Alternatively,

$(N / N_d) + (M_z / M_{dz}) + (M_y / M_{dy})$ = { $(16.5 \times 10^3 \times 1.10) / (110.7 \times 2500) + (12.60 \times 10^5 \times 1.10) / (302.9 \times 2500) = 0.54 <= 1.0$

(iii) Check for resistance of the member for combined effects of buckling (Clause- 9.3.2):

(a) Determination P_{dz}, P_{dy} and P_d (Clause 7.12)

$KL_y = KL_z = 0.85 \times 500 = 425$mm

$(KL_z / r_z) = 425 / 20.21 = 21.03$

$(KL_y / r_y) = 425 / 3.52 = 120.7$

Therefore, non- dimensional slenderness ratios, $\lambda_z = 0.237$ and $\lambda_y = 1.359$.

For major axis buckling curve **'a'** is applicable (Refer Table 7.2).

From Table 7.4a,

f_{cdz} = 225.4 MPa

$P_{dz} = 225.4 \times 10 \times 110.7 \times 10^{-3} = 249.65^T$

For minor axis buckling curve 'b' is applied (Refer Table 7.2)

From Table 7.4b,

f_{cdy} = 90.80 MPa

$P_{dy} = 90.80 \times 10 \times 110.7 \times 10^{-3} = 100.60^T$

Therefore, $P_d = P_{dy} = 100.60^T$

(b) Determination of M_{dz} (Clause 9.3.2.2 and Clause 8.2.2).

Elastic critical moment is given by (clause 8.2.21).

$M_{cr} = [(\beta_{LT}\,\pi^2 EI_y h) / \{2(KL)^2\}]\,[1 + 1/20\,\{(KL / r_y) / (h / t_f)\}^2]^{0.5}$

$=[(1.20 \times \pi^2 \times 2 \times 10^5 \times 1369.8 \times 10^4 \times 500) / (2 \times 4250^2)]\,[1 + 1/20\,\{(120.70) / (500 / 17.2)\}^2]^{0.5}$

$= 6.129 \times 10^8$ N-mm

$$\lambda_{LT} = \sqrt{(\beta_b Z_p f_y / M_{cr})}$$
$$= \sqrt{(1.0 \times 2074.70 \times 10^3 \times 250 / 6.129 \times 10^8}$$
$$= 0.92$$
$$\phi_{LT} = 0.5\,[1 + \alpha_{LT}\,(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$
$$= 0.5\,[1 + 0.21(0.92 - 0.2) + 0.92^2]$$
$$= 0.999$$
$$X_{LT} = 1 / [\phi_{LT} + \{\phi_{LT}^2 - \lambda_{LT}^2\}^{0.5}]$$
$$= 0.72 \leq 1.0$$
$$f_{bd} = X_{LT} f_y / \gamma_m = 0.72 x\ 250 / 1.1 = 168.3\ MPa$$
$$M_{dz} = \beta_b . Z_{pz} . f_{bd}$$
$$= 1.0 \times 2074.7 \times 1638 \times 10^{-5}$$
$$= 33.98^{T\text{-}m}$$
$$\left[\beta_{LT} = 1.20,\ \beta_b = 1.0, \alpha_{LT} = 0.21\right]$$

(c) Determination of M_{dy} (Clause 9.3.2.2)

$M_{dy} = \beta_b\ Z_{py}.\ f_y / \gamma_m$

$= 1.0 \times 302.90 \times 2500 / 1.1 \times 10^{-5} = 6.88^{T\text{-}m}$

(d) Determination of C_z (Clause 9.3.2.2) From Table - 9.2,

$$\psi_z = 9.15 / 12.6 = 0.726$$
$$\therefore \beta_{mz} = 1.8 - 0.7 \times 0.726 = 1.292$$
$$\therefore \mu_{fz} = \lambda_z (2\beta_{mz} - 4) + (Z_{pz} - Z_{ez}) / Z_{ez} \leq 0.9$$

= 0.237 (2 x 1.292 - 4) + 0.1469

= -0.188

For torsional buckling,

$$\mu_{LT} = 0.15\lambda_y\beta_{MLT} - 0.15 \le 0.90$$

Since, $\beta_{MZ} = \beta_{MLT}$,

$$\therefore \mu_{LT} = 0.15 \times 1.292 \times 1.359 - 0.15 = 0.113$$

Since μ_z is larger of μ_{lz} and μ_{LT}, $\mu_z = 0.113$

$$\therefore C_z = 1 - (\mu_z.P)/P_{dz}$$

$$= 1 - 0.113 \times 16.5/249.65 = 0.993 \le 1.50$$

(e) Determination of C_y (Clause 9.3.2.2)

$$\psi_y = 1.335/(-1.44) = -0.927$$

$$\therefore \beta_{my} = 1.8 - 0.7 \times (-0.927) = 2.449$$

$$\therefore \mu_y = \lambda_y(2\beta_{MY} - 4) + (Z_{py} - Z_{ey})/Z_{ey} \le 0.9$$

$$= 1.359(2 \times 2.449 - 4) + 0.99 = 2.22 \approx 0.90$$

$$\therefore C_y = 1 - (\mu_y.P)/P_{dy}$$

$$= 1 - 0.90 \times 16.5/100.6 = 0.85$$

$$\therefore P/P_d + C_Z M_z/M_{dz} + C_y M_y/M_{dy}$$

$$= 16.50/100.60 + 0.993 \times 12.60/33.98 + 0.85 \times 1.44/6.88$$

$$= 0.164 + 0.368 + 0.178 = 0.71 \le 1.0$$

Therefore, section is safe. Interaction value is less in LSM than WSM.

Design of typical beam

Example-1

The beam (ISMB 400) in Fig.1 is designed considering it is fully restrained laterally.

1. WSM (clause 6.2 of IS:800 - 1984):

Bending moment, $M=18.75^{Tm}$

For ISMB 450, $Z=1350.7\ cm^3$

Therefore

$s_{bc(cal)} = 18.75 * 10^5 / 1350.7$

$= 1388.17\ kg/cm^2 < 1650\ Kg/cm^2$

Therefore percentage strength attained is 0.8413 or 84.13%s

2. LSM (clause 8.2 of draft IS:800):

Factored load = $1.5 * 2.0 + 1.5 * 4.0 = 9.0^T$

Factored bending moment = $9.0 * 5.0^2 / 8$

$= 28.125^{Tm}$

Factored shear force = $22.50^T = Fv$

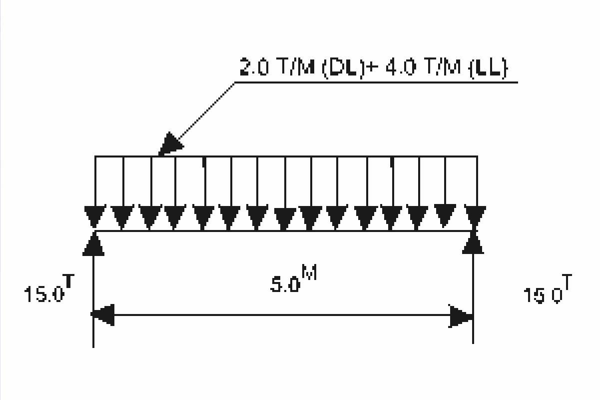


Fig.1

For, ISMB450,

$D = 450$ mm $B = 150$ mm

$t_w = 9.4$ mm $T=17.4$ mm

$I_{xx} = 30390.8\ cm^4$ $Iyy = 834.0$ cm

$r_{yy} = 3.01$ cm $h_1 = 379.2$ cm = d

i) Refering Table 3.1 of the code for section classification, we get :

Flange criterion = (B / 2) / T = (150 / 2) /17.4

= 4.31 < 9.4.

Web criteria = $d / t_w = 379.2 / 9.4$

= 40.34 < 83.9.

Therefore, section is plastic.

ii) Shear capacity (refer clause 8.4 of draft code) :

$F_{vd} = (f_{yw} .A_v)/ (\sqrt{3}\, g_{mo}) = (f_{yw} .ht_w)/(\sqrt{3}\, g_{mo})$

$= (2500 * 45.0 * 0.94)/ (\sqrt{3} * 1.10 * 1000)$

= 55.50T

$F_v / F_{vd} = 22.50 /55.5 = 0.405 < 0.6.$

Therefore , shear force does not govern permissible moment capacity (refer clause 8.2.1.2 of the draft code).

iii) Since the section is ' plastic' ,

$M_d = (Z_p .f_y) / g_{mo}$

where , Z_p = Plastic Modulus = 1533.36 cm^3

(refer Appendix-I of draft code).

Therefore , $M_d = (1533.36*2500) /1.10 * 10^5$

$= 34.85^{Tm}$

Percentage strength attained for LSM is (28.125 / 34.85) = 0.807 or 80.7 % which is less than 84.3 % in the case of WSM .

iv) Now , let us check for <u>web buckling :</u>

K_v = 5.35 (for transverse stiffeners only at supports as per clause 8.4.2.2 of draft code).

The elastic critical shear stress of the web is given by :

$\tau_{cr.e} = (k_v .\pi^2 .E) / \{ 12 (1 - \mu^2)(d / t_w)^2 \}$

$= (5.35\ \pi^2\ E) / \{ 12 (1 - 0.3^2)(379.2 / 9.4)^2 \}$

$= 594.264\ N / mm^2$.

Now as per clause 8.4.2.2 of draft code ,

$\lambda_w = \sqrt{\ } \{ f_{yw} / (\sqrt{3}\, \tau_{cr.e})\}^{0.5}$

$= \{250 / (\sqrt{3} * 594.264)\}^{0.5} = 0.4928 < 0.8$.

where , λ_w is a non-dimensional web slenderness ratio for shear buckling for stress varying from greater than 0.8 to less than 1.25 .

Therefore , τ_b (shear stress corresponding to buckling) is given as :

$\tau_b = f_{yw} / \sqrt{3} = 250 / \sqrt{3} = 144.34\ N / mm^2$.

$V_d = d.t_w.\ \tau_b / \gamma_{mo}$

$= 37.92 * 0.94 * 1443.34 * 10^{-3} / 1.10$

$= 46.77^{Ts}$

As $V_D > F_v$ ($= 22.5^T$), the section is safe in shear.

For checking of deflection :

$s = (5 * 60 * 500^4) / (384 * 2.1 * 10^6 * 30390.8)$

= 0.76 cm = 7.6 mm = L / 658

Hence O.K.

Example - 2

The beam (ISMB 500) shown in Fig2 is carrying point load as shown , is to be designed. The beam is considered to be restrained laterally.

1 . <u>In WSM (clause 6.2 of IS : 800 - 1984) :</u>

Bending Moment , M = (5 + 20) * 3 / 4

= 26.25^{Tm}

For, ISMB 500 : Z = 1808.7 cm^3

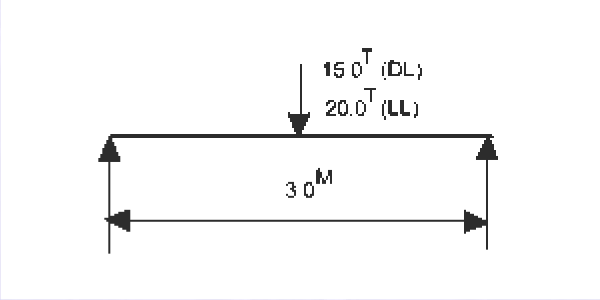


Fig .2

So , $\sigma_{bc(cal)}$ = 26.25 * 10^5 / 1808.7 Kg / cm^2

= 1451.318 Kg / m^2 .

Therefore , percentage strength attained is 0.88 or 88 %.

2 . <u>In LSM (clause 8.2 of draft IS : 800):</u>

Factored load = 1.5 * 15 + 1.5 * 20 = 52.50^T

Therefore,

Factored bending moment = 52.5 * 3.0 / 4

= 39.375^{Tm}

Factored shear force, F_v = 52.50 / 2 = 26.25^T

i) For ,<u>section classification</u> of this ISMB500 , (refer table 3.1 of the code) , we get :

$D = 500$ mm $\quad d = 424.1$ mm

$r_1 = 17$mm $\quad r_y = 3.52$ cm

$Z_p = 20.25.74$ cm^3 $\quad T = 17.2$ mm

$b = 180$ mm $\quad t_w = 10.2$ mm .

Flange criterion , $(b / 2) / T = (180 / 2) / 17.2$

$= 5.23 < 9.4$

Web criterion , $d / t_w = 424.1 /10.2$

$= 41.57 < 83.9$ (O.K)

Therefore the section is plastic.

ii) Shear capacity (clause 8.4):

$F_{vd} = (f_{yw} .A_v)/ (\sqrt{3} \gamma_{mo})$

$= (2500 * 50 * 1.02)/ (\sqrt{3} * 1.10 * 1000)$

$= 66.92^T$

$F_v = 26.25^T$

Since $F_v / F_{vd} < 0.6$. shear force does not govern Bending Strength

iii) For moment check :

Since the section is plastic ,

$M_d = (Z_p .f_y) / \gamma_{mo}$

$= (2025.74 * 2500) /1.10 * 10^5$

$= 46.04^{Tm}$

Therefore , Percentage strength attained is (39.375 / 46.04) = 0.855 or 85.5 % which is less than 88 % in case of WSM

iv) Check for web buckling :

K_v = 5.35 (for transverse stiffeners only at supports as per clause 8.4.2.2 of draft code).

$\tau_{cr.e} = (k_v .\pi^2 .E) / \{ 12 (1 - \mu^2)(d / t_w)^2 \}$

$= (5.35\, \pi^2 E) / \{ 12 (1 - 0.3^2)(424.1 / 10.2)^2 \}$

$= 559.40\ N / mm^2$.

Now as per clause 8.4.2.2 ,

$\lambda_w = \sqrt{\ } \{ f_{yw} / (\sqrt{3}\, \tau_{cr.e})\}^{0.5}$

$= \{250 / (\sqrt{3} * 559.40)\}^{0.5} = 0.508 < 0.8$.

Therefore ,

$\tau_b = f_{yw} / \sqrt{3} = 250 / \sqrt{3} = 144.34\ N / mm^2$.

$V_D = d.t_w. \tau_b / \gamma_{mo}$

$= 42.41 * 1.02 * 1443.4 * 10^{-3} / 1.10$

$= 56.76^T$

Since $V_D > F_v$ ($= 26.25^T$), the section is safe against shear.

For checking of deflection :

$\sigma = (35000 * 300^3) / (48 * 2.1 * 10^6 * 45218.3)$

= 0.207 cm = 2.07 mm = L / 1446

Hence O.K.

Example - 3

The beam , ISMB500 as shown in Fig.3 is to be designed considering no restraint along the span against lateral buckling .

i) In <u>WSM (clause 6.2, 6.2.2, 6.2.3 and 6.2.4 and 6.2.4.1 of IS : 800 - 1984) :</u>

Bending moment, $M = 2.1 * 6^2 / 8 = 9.45^{Tm}$

For, ISMB500 :

$L = 600$ cm $\quad r_y = 3.52$ cm

$T = 17.2$ mm $\quad Z = 1808.7\ cm^3$

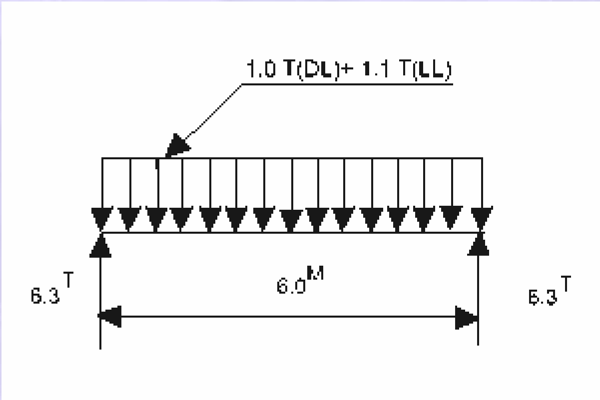


Fig.3

Therefore,

$Y = 26.5 * 10^5 / (600 / 3.52)^2 = 91.2$

$X = 91.21 \sqrt{[1 + (1 / 20)\{(600 * 1.72) / (3.52 * 50)\}^2]}$

$= 150.40$

and $f_{cb} = k_1 (X + k_2y) C_2 / C_1 = X$

(since $C_2 = C_1$, $k_1 = 1$ and $k_2 = 0$)

Now ,

$\sigma_{bc(perm)} = (0.66\ f_{cb}.f_y) / \{f_{cb}^n + f_y^n\}^{1/n}$

$= 746$ Kg / cm^2.

and $\sigma_{bc(cal)} = 9.45 * 10^5 / 1808.7$

$= 522.5$ Kg / cm^2.

Therefore , percentage strength attained is (522.5 / 746) = 0.7 or 70 %.

ii) In <u>LSM (clause 8.2 of draft IS : 800) :</u>

For MB 500 :

D = 500 mm T = 17.2 mm

B = 180 mm t = 10.2 mm

$Z_p = 2025.74$ cm^3 $r_{yy} = 3.52$ cm

$H_1 = d = 424.1$ mm

$I_{zz} = 45218.3$ cm^4 , $I_{yy} = 1369.8$ cm^4

iii) <u>Classification of section (ref . Table 3.1 of the code):</u>

b / T = 90 / 17.2 = 5.2 < 9.4 , hence O.K.

d / t = 424.1 / 10.2 = 41.6 < 83.90 (O.K)

Therefore , the section is Plastic .

iv) Check for <u>torsional buckling (clause 8.2.2) :</u>

t_f / t_w for ISMB 500 = 17.2 / 10.2 = 1.69 $\leq$ 2.0

Therefore , $\beta_{LT} = 1.20$,for plastic and compact aestions.

M_{cr} = Elastic critical moment given as :

$M_{cr} = \{ (\beta_{LT} \pi^2 .EI_y) / (KL)^2 \}[1 + (1 / 20) \{KL / r_y) / (h / t_f) \}^2]^{0.5} (h/ 2)$

(ref . clause 8.2.2.1)

$= (1.20 \pi^2 * 2 * 10^6 * 1369.8 / 600^2)[1 + (/ 20)\{ (600 / 3.52) / (50 / 1.72)\}^2]^{0.5} (50 / 2)$

= 371556.3 Kg-cm .

Now , $\lambda_{LT} = \sqrt{(\beta_b .Z_p .f_y / M_{cr})} = 1.1675$

(since $\beta_b = 1.0$ for plastic and compact sections)

Therefore ,

$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$= 0.5 [1 + 0.2 (1.1675 - 0.2) + 1.1675^2]$

$= 1.283$ ($\alpha_{LT} = 0.21$ for rolled section).

Therefore $X_{LT} = 1 / [\phi_{LT} + \{ \phi_{LT}^2 - \lambda_{LT}^2 \}^{0.5}]$

$= 1 / [1.283 + \{ 1.283^2 - 1.1675^2 \}^{0.5}]$

$= 0.55$

$M_d = 1.0 * 0.55 * 2025.74 * 2500 / 1.10 * 10^{-5}$

$= 25.32^{Tm}$

Now actual moment is obtained as follows:

Factored load = 1.0 * 1.5 + 1.1 * 1.5 = 3.15^{Tm}

Factored moment = $14.175^{Tm} < M_d$

Percentage strength attained is (14.75 / 25.32) = 0.56 or 56 % which is less than 70 % in case of WSM.

Hence , ***the beam is safe both in LSM and WSM design but percentage strength attained is comparatively less in LSM for the same section***

Problem:

The girder showed in Fig. E1 is fuly restrained against lateral buckling throughtout its sapan. The span is 36 m and carries two concentrated loads as show in Fig. E1. Design a plate girder.

Yield stress of steel, f_y *= 250 N/mm²*

Material factor for steel, γ_m *= 1.15*

Dead Load factor, γfd *= 1.50*

Imposed load factor, γfl *= 1.50*

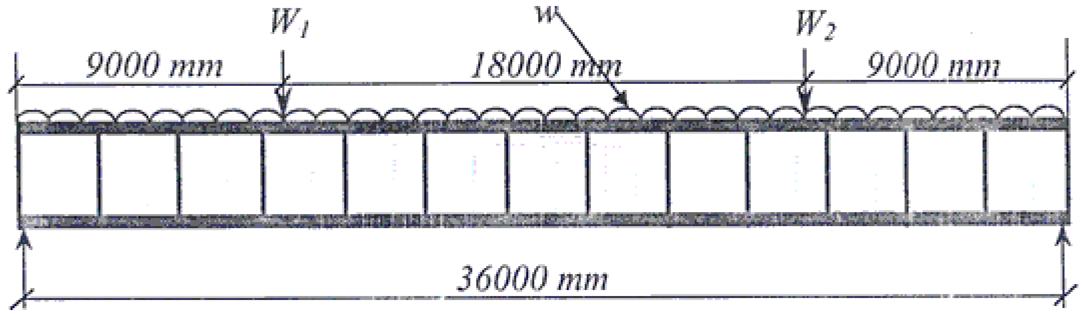


Fig. E1 Example plate girder

1.0 Loading

Dead load:

Uniformly distributed load, wd = 18 kN/m

Concentrated load, W_{1d} = 180 KN

Concentrated load, W_{2d} = 180 KN

Live load:

Uniformly distributed load, W = 35 kN/m

Concentrated load, W_{1l} = 400 kN

Concentrated load, W_{2l} = 400 kN

Factored Loads

$w' = w_d * \gamma_{fd} + w_l * \gamma f_l = 18 * 1.5 + 35 * 1.5 = 79.5$ kN/m

$W'_1 = W_{1d} * \gamma fd + W_{1l} * \gamma f_l = 180 * 1.5 + 400 * 1.5 = 870$ kN

$W'_2 = W_{2d} * \gamma fd + W_{2l} * \gamma f_l = 180 * 1.5 + 400 * 1.5 = 870$ kN

2.0 Bending moment and shear force

	Bending moment (kN-m)	Shear force (kN)
UDL effect	$\frac{w\ell^2}{8} = \frac{79.5*36*36}{8} = 12879$	$\frac{w\ell}{2} = 1431$
Concentrated load effect	$\frac{w\ell}{4} = 870*9 = 7830$	w=870
Total	20709	2301

The desigh shear forces and bending moments are shown inf Fig. E2.

3.0 Initial sizing of plate girder

Depth of the plate girder:

The recommended span/depth ratio for simply supported girder varies between 12 for short span and wo for long span girder. Let us consider depth of the girder as 2600 mm.

$$\frac{\ell}{d} = \frac{36000}{2600} = 1\ 38$$

Depth of 2600 mm is acceptable.

(For drawing the bending moment and shear force diagrams, factored loads are considered)

Fig.E2 Bending moment and shear force diagrams

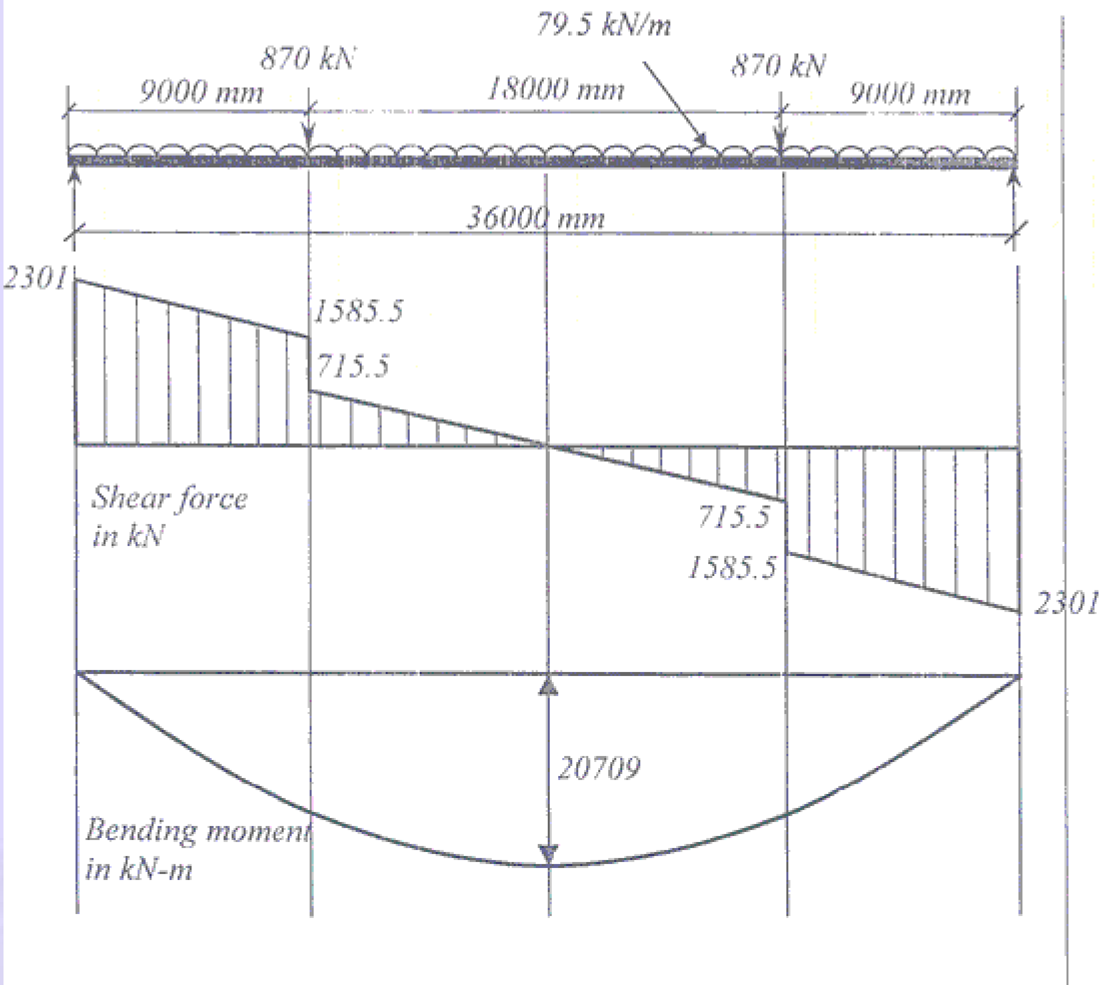


Fig. E2 Bending moment and shear force diagrams

Flange:

P_y = 250/1.15 = 217.4 N/mm ^{2}

Single flange area,

$$A_f = \frac{M_{max}}{d_{Py}} = \frac{20709*10^6}{2600*217.4} = 366375.5 mm^2$$

By thumb rule, the flange width is assumed as 0.3 times the depth of the section.

Try 780 X 50 mm, giving an area = 39000 mm$^{2.}$

Web:

Minimum web thickness for plate girder in builidings usually varies between 10 mm to 20 mm. Here, thickness is assumed as 16mm.

Hence, web size is 2600 X 16 mm

4.0 Section classification

Flange:

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{\frac{1}{2}} = \left\{\frac{250}{250}\right\}^{\frac{1}{2}} = 1.0$$

$$b = \frac{B - t}{2} = \frac{780 - 16}{2} = 382$$

$$\frac{b}{T} = \frac{382}{50} = 7.64 < 9\varepsilon$$

Hence, Flange is COMPACT SECTION.

Web:

$$\frac{d}{t} = \frac{2600}{250} = 162.5 > 67\varepsilon$$

Hence, the web is checked for shear buckling.

5.0 Checks

Check for serviceability:

$$\frac{d}{250} = \frac{2600}{250} = 10.4\text{mm} < t$$

$$\text{Since}, t > \frac{d}{250}$$

Web is adequate for serviceability.

Check for flange buckling in to web:

Assuming stiffener spacing, a > 1.5 d

$$t \geq \frac{d}{294}\left(\frac{P_{yf}}{250}\right)^{\frac{1}{2}} = \frac{2600}{294} x \left(\frac{217.4}{250}\right)^{\frac{1}{2}} = 8.2 \text{ mm}$$

Since, t (=16 mm) > 8.2 mm, the web is adequate to avoid flange buckling into the web.

Check for moment carrying capacity of the flanges:

The moment is assumed to be resisted by flanges alone and the web resists shear only.

Distance between centroid fo flanges, $h_s = d + T = 2600 + 50 = 2650$ mm

$A_f = B * T = 780 * 50 = 39000$ mm^2

$M_c = P_{yf} * A_f * h_s = 217.4 * 39000 * 2650 * 10\text{-}6 = 222468.3$ kN-m

> 20709 kN-m

Hence, the section in adequate for carrying moment and web is designed for shear.

6.0 Web design

The stiffeners are spaced as shown in Fig.E5. Three different spacing values 2500, 3250 and 3600 mm are adopted for trail as shown in fig. E5.

End panel (AB) design:

d=2600 mm

t = 16 mm

Maximum shear stress in the panel is

$$f_v = \frac{F_{VA}}{dt} = \frac{2301*10^3}{2600*16} = 55.3\ \text{N/mm}^2$$

$$\frac{a}{d} = \frac{2500}{2600} = 0.96$$

$$\frac{d}{t} = \frac{2600}{16} = 162\ 5$$

Calcualtion of critical stress,

$$q_e(\text{when } a/d \le 1) = \left[0.75 + 1/\ (a/d)^2\right]\left[1000/(d/t)\right]^2$$
$$= \left[0.75 + 1/(0.96)^2\right]\left[1000/(162.5)\right]^2$$
$$= 69.5\ \text{N/mm}^2$$

Slenderness parameter,

$$\lambda_w = \left[0.6\left(f_{yw}/\gamma_m\right)/\ qe\right]^{\frac{1}{2}}$$
$$= \left[0.6(250/1.15)/69.5\right]^{\frac{1}{2}}$$
$$= 1.37 > 1.25$$

Hence, Critical shear strength ($q_{cr} = q_e$) = 69.5 N/mm^2

Since, $f_v < q_{cr}$ (55.3 < 69.5)

Tension field action need not be utilised for design.

Checks for the end panel AB:

End panel AB should also be checked as a beam (Spanning between the flanges of the girder) capable of resisting a shear force R_{tf} and a moment M_{tf}due to anochor forces.

(In the following calculations boundary stiffeners are omitted for simplicity)

Check for shear capacity of the end panel:

$$H_q = 0.75dt\ P_y\left[1-\frac{q_{cr}}{0.6P_y}\right]^{\frac{1}{2}}$$

$$q_{cr} = 69\ 5\ N/mm^2$$

$$H_q = 0\ 75*2600*16*250/1.15\left[1-\frac{69.5}{0.6*(250/1.15)}\right]^{\frac{1}{2}} = 4636\ kN.$$

$$R_{tf} = \frac{H_q}{2} = \frac{4636}{2} = 2318\ kN$$

$$A_v = t.a = 16*2500 = 4000\ mm^2$$

$$P_v = 0.6P_{yw}A_{\frac{1}{2}} = 0\ 6*(250/1.15)*40000/1000 = 5217\ kN$$

Since, $R_{tf} < P_v$, the end panel can carry the shear force.

Check for moment capacity of end panel AB:

$$M_{tf} = \frac{H_q d}{10}$$

$$= \frac{4636*2600}{10}*10^{-3} = 1205\ kN\text{-}m$$

$$y = \frac{a}{2} = \frac{2500}{2} = 1250$$

$$I = \frac{1}{12}ta^3 = \frac{1}{12}*16*2500^3 = 2083*10^7\ mm^4$$

$$M_q = \frac{1}{y}py = \frac{2083*10^7}{1250}*(250/1.15)*10^{-6} = 3623\ kN\text{-}m$$

Since, $M_{tf} < M_q$ $(1205 < 3623)$

The end panel can carry the bending moment.

Design of panel BC:

Panel BC will be designed using tension field action

d = 2600 mm ; t = 16 mm

$$f_v = \frac{F_{VB}}{dt} = \frac{2102.3*10^3}{2600*16} = 50.5\ N/mm^2$$

$$\frac{a}{d} = \frac{3250}{2600} = 1.25$$

$$\frac{d}{t} = \frac{2600}{16} = 162.5$$

$$P_y = \frac{250}{1.15} = 217.4 N/mm^2$$

Calculation of basic shear strength, q_b:

$$\text{Elastic critical stress, } q_e \text{ (when } a/d > 1) = \left[1.0 + 0.75/(a/d)^2\right]\left[1000/(d/t)\right]^2$$

$$= \left[1.0 + 0.75/(1.25)^2\right]\left[1000/(162.5)\right]^2$$

$$= 56.0\ N/mm^2$$

$$\text{Slenderness parameter, } \lambda_n = \left[0.6\left(f_{yw}/\gamma_m\right)/q_e\right]^{\frac{1}{2}}$$

$$= \left[0.6(250/1.15)/56.0\right]^{\frac{1}{2}}$$

$$= 2.33 > 1.25$$

$$\text{Hence, Critical shear stength } = 56.0\ N/mm^2\ (q_{cr} = q_e)$$

$$\phi_t = \frac{1.5q_{cr}}{\sqrt{1+\left(\frac{a}{d}\right)^2}} = \frac{1.5*56.0}{\sqrt{1+(1.25)^2}} = 52.5$$

$$y_b = \left(P_{yw}^2 - 3q_{cr}^2 - \phi_t^2\right)^{\frac{1}{2}} - \phi_t = \left(217.4^2 - 3*56.0^2 + 52.5^2\right)^{\frac{1}{2}} - 52.5 = 149$$

$$q_b = q_{cr} + \frac{y_b}{2\left[\frac{a}{c} + \sqrt{1+\left(\frac{a}{d}\right)^2}\right]} = 56.0 + \frac{149}{2\left[1.25 + \sqrt{1+(1.25)^2}\right]} = 82.1 N/mm^2$$

$$\text{Since, } q_b > f_v \qquad (82.1 > 50.5)$$

Panel BC is safe against shear buckling.

7.0 Design of stiffeners

Load bearing stiffener at A:

Design should be made for compression force due to bearing and moment.

Design force due to bearing, F_b = 2301 kN

Force (F_m) due to moment M_{tf}, is

$$F_m = \frac{M_{tf}}{a} = \frac{1205}{2500} * 10^3 = 482 \text{ kN}$$

Total compression = $F_c = F_b + F_m$ = 2301 + 482 = 2783 kN

Area of Stiffener in contact with the flange, A:

Area (A) should be greater than

$$\frac{0.8\, F_c}{P_{ys}}$$

$$\frac{0.8\, F_c}{P_{ys}} = \frac{0.8*2783}{217.4} * 10^3 = 10241 \text{ mm}^2$$

Try stiffener of 2 flats of size 270 x 25 mm thick

Allow 15 mm to cope for web/flange weld

A = 255 * 25 * 2 = 12750 mm^2 > 10241 mm^2

Bearing check is ok.

Check for outstand:

Outstand from face of web should not be greater than

$$20\, t_s \varepsilon$$

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{\frac{1}{2}} = \left\{\frac{250}{250}\right\}^{\frac{1}{2}} = 1.0$$

$$\text{Outstand } b_s = 250 \text{ mm} < 20\, t_s \varepsilon (= 20*25*1.0 = 500)$$

$$b_s = 250 \text{ mm} < 13.7\, t_s \varepsilon (= 13.7*25*1.0 = 342.5)$$

Hence, outstand criteria is satisfied.

Check stiffener for buckling:

The effective stiffener section is shown in Fig. E3

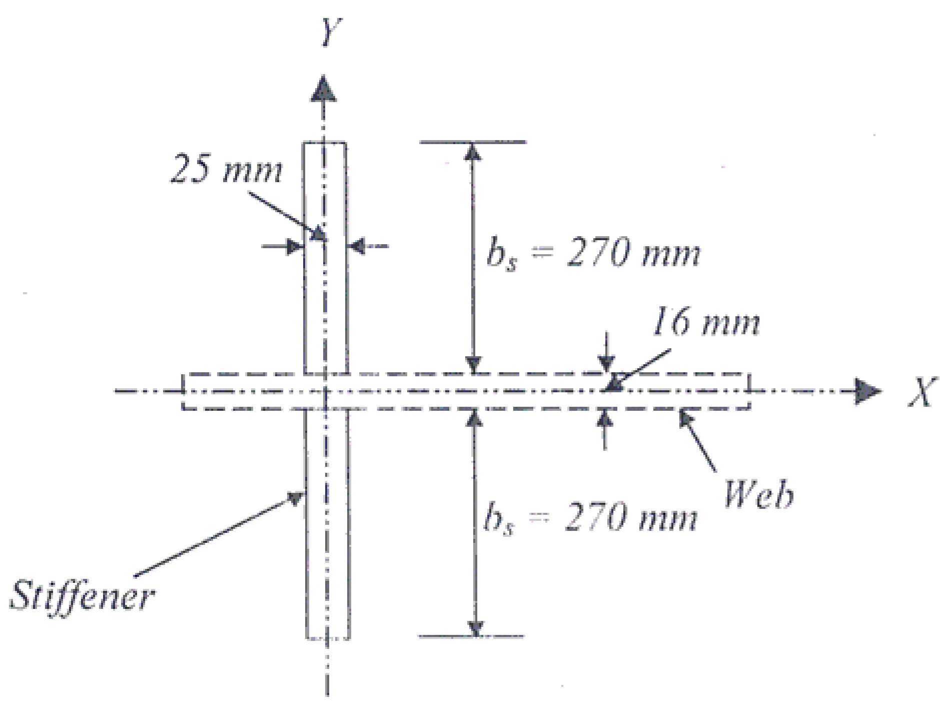


Fig. E3 End bearing stiffener

The buckling resistance due to web is neglected here for the sake of simplicity.

$$I_x = \frac{25*556^3}{12} - \frac{1}{12}*25*16^3 = 35807*10^4 \text{ mm}^4$$

$$A_e = \text{Effective area} = 270*25*2 = 13500 \text{ mm}^2$$

$$r_x = \left[\frac{I_x}{A_e}\right]^{\frac{1}{2}} = \left[\frac{35807*10^4}{13500}\right]^{\frac{1}{2}} = 162.8 \text{ mm}$$

Flange is restrained against rotation in the plane of stiffener, then

I_e = 0.71 = 0.7 * 2600 = 1820

$$\lambda = \frac{l_e}{r_x} = \frac{1820}{162.8} = 11.2$$

For $f_y = 250$ N/mm^2 and $\lambda = 11.2$

$$\sigma_c = 250 \text{N/mm}^2$$

from table(30 of chapter on axially compressed columns

Buckling resistance of stiffener is

$$P_c = \sigma_c A_e / \gamma_m = 250 * 1350 / 1.15 = 2935 \text{ kN}$$

$$\text{Since } F_c < P_c \ (2783 < 2935)$$

Therefore, stiffener provided is safe against buckling.

Check stiffener A as a bearing stiffener:

Local capacity of the web:

Assume, stiff bearing length b_1=0

n_2 = 2.5 * 50 * 2 = 250 BS 5950 : Part -1, Clause 4.5.3

$P_{crip} = (b_1 + n_2)\, tP_{yw}$

= (0 + 250) * 16 * (250/1.15) * 10^{-3} = 870 kN

Bearing stiffener is designed for F_A

$F_A = F_x = P_{crip}$ = 2783 - 870 = 1931 kN

Bearning capacity of stiffener alone

$P_A = P_{ys} * A$ = (50/1.15) * 13500/1000 = 2935 kN

Since, $F_A < P_A$ (1931 < 2935)

The designed stiffener is OK in bearing.

Stiffener A - Adopt 2 flats 270 mm X 25 mm thick

Design of intermediate stiffener at B:

Stiffener at B is the most critical one and will be chosen for the design.

Minimum Stiffness

$$I_s \geq 0.75dt^3 \text{ for } a \geq d\sqrt{2}$$
$$I_s \geq \frac{0.75dt^3}{a^3} \text{ for } a < d\sqrt{2}$$
$$d\sqrt{2} = \sqrt{2} * 2600 = 3677 \text{ mm}$$
$$a < d\sqrt{2} \qquad (3250 < 3677)$$

Conservatively ' t ' is taken as actual web thickness and minimum ' a ' is used.

$$\frac{1.5d^3t^3}{a^2} = \frac{1\ 5*2600^3*16^3}{3250^2} = 1022*10^4 \text{ mm}^4$$

Try intermediate stiffener of 2 flats 120 mm X 14 mm

$$(I_s)_{Provided} = \frac{14*256^3}{12} - \frac{14*16^3}{12} = 1957*10^4 \text{ mm}^4$$
$$I_s > \frac{1.5d^3t^3}{a^2}, \text{the section statisfied minimum stiffeness requirement}$$

Check for outstand:

$$\text{Outstand of the stiffener } \leq 13.7\, t_s \varepsilon$$
$$13.7 t_s \varepsilon = 13.7*14*1.0 = 192 \text{ mm}$$

Outstand = 120 mm (120 < 192)

Hence, outstand criteria is satisfied.

Buckling check:

Stiffener force, $F_q = V - V_s$

Where V = Total shear force

$V_s = V_{cr}$ of the web

a / d = 3600 / 2600 = 1.38

d / t = 2600 / 16 = 162.5

$$\text{Elastic critical stress, } q_e \text{ (when } a/d > 1) = \left[1.0 + 0.75/(a/d)^2\right]\left[1000/(d/t)\right]^2$$
$$= \left[1.0 + 0.75/(1.38)^2\right]\left[1000/(162.5)\right]^2$$
$$= 52.8 \text{ N/mm}^2$$

$$\text{Slenderness parameter, } \lambda_w = \left[0.6(f_{yw}/\gamma m)/qe\right]^{\frac{1}{2}}$$
$$= \left[0.6(250/1.15)/52.8\right]^{\frac{1}{2}}$$
$$= 2.47 > 1.25$$

Hence, Critical shear strength = 52.8 N/mm^2 ($q_{cr} = q_e$)

$V_{cr} = q_{cr}dt = 52.8 * 2600 * 16 * 10^{-3} = 2196$ kN

Buckling resistance of intermediate stiffener at B:

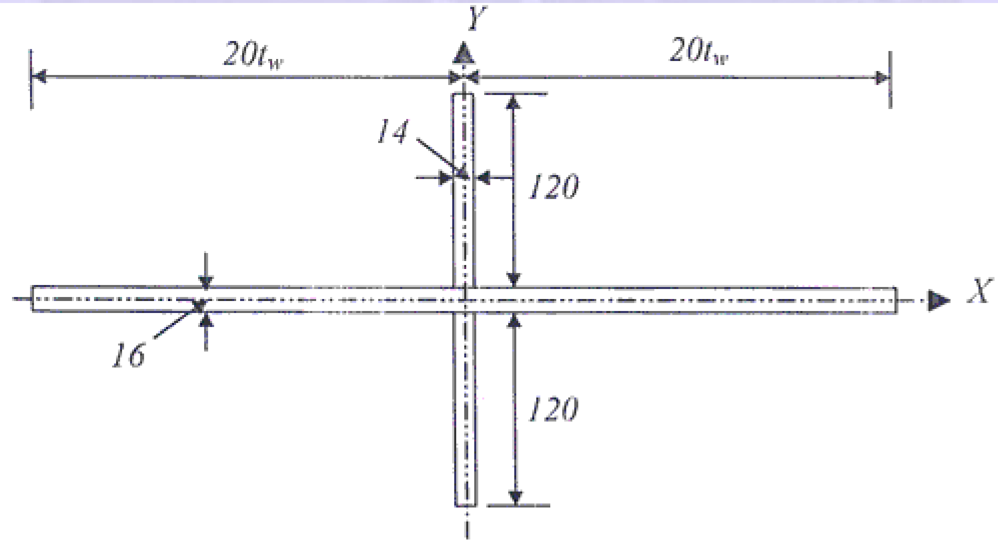


Fig.E4 Effective section

$$20\ t_w = 20 * 16 = 3230\ \text{mm}$$

$$I_x = \frac{1}{12} * 14 * 256^3 + \frac{640 * 16^3}{12} - \frac{14 * 16^3}{12} = 1979 * 10^4\ \text{mm}^4$$

$$A = 240*14+640*16 = 13600\ \text{mm}^2$$

$$r_x = \left[\frac{1979 * 10^4}{13600}\right]^{\frac{1}{2}} = 38.1$$

$$l_e = 0.7 * 2600 = 1820$$

$$\lambda = \frac{l_e}{r_x} = \frac{1820}{38.1} = 48.0$$

For $f_y = 250\ \text{N/mm}^2$ and $\lambda = 48.0$

From table3 of chapter on axially compressed columns,

$$\sigma_c = 213.2\ \text{N/mm}^2$$

Buckling resistance = (213.2/1.15) * 13600 * 10^{-3} = 2521 kN

Shear force at B, V_B = 2301-{(2301 - 1585.5)*(2500/9000)] = 2102 kN

Stiffener force, Fq = [21201 - 2196] < 0

and

Fq < Buckling resistance

Hence, intermediate stiffener is adequate

Intermediate stiffener at B - Adopt 2 flats 120 mm X 14 mm

Intermediate stiffener at E (Stiffener subjected to external load):

Minimum stiffness calculation:

$$a = 3600 \qquad d\sqrt{2} = 3677$$

$$a < d\sqrt{2} \qquad (3600 < 3677)$$

$$I_s \geq \frac{1.5 d^3 t^3}{a^2} = \frac{1.5 * 2600^3 * 16^3}{3600^2} = 833 * 10^4$$

Try intermediate stiffener 2 flats 100 mm X 12 mm thick

$$(I_s)_{Provided} = 1007 * 10^4 \text{ mm}^4$$

$$(I_s)_{Provided} > I_s \qquad [1007 * 10^4 > 833 * 10^4]$$

Hence, OK

Buckling Check:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \le 1$$

$$F_q = V - V_s \qquad V = 1585\ 5 \text{ kN}$$

$$V_s = V_{cr} = q_{cr} dt = 52.8 * 2600 * 16 * 10^{-3} = 2196 \text{ kN}$$

$$F_q \text{ is negative and } F_q - F_x = 0$$

$$M_s = 0$$

$$F_x = 870 \text{ kN}$$

Buckling resistance of load carrying stiffener at D:

(Calculation is similar to stiffener at B)

$$20\, t_w = 20 * 16 = 320 \text{ mm}$$

$$I_x = \frac{1}{12} * 12 * 216^3 + \frac{640 * 16^3}{12} - \frac{12 * 16^3}{12} = 1029 * 10^4 \text{ mm}^4$$

$$A = 200 * 12 + 640 * 16 = 12640 \text{ mm}^2$$

$$r_x = \left[\frac{1029 * 10^4}{12640}\right]^{\frac{1}{2}} = 28\ 5$$

$$l_e = 0.7 * 2600 = 1820$$

$$\lambda = \frac{l_e}{r_x} = \frac{1820}{28.5} = 63.9$$

$$\text{For } f_y = 250 \text{ N/mm}^2 \text{ and } \lambda = 63\ 9$$

From table3 of chapter on axially comperssed columns,

$$\sigma_c = 180 \text{ N/mm}^2$$

Buckling resistance, P_x (180/1.15) * 2640 * 10^{-3} = 1978 kN.

F_x/P_x = 870/1978 = 0.44 <1.0

Hence, Stiffener at D is OK against buckling

Stiffener at D - Adopt flats 100 mm X 12 mm thick

Web check between stiffeners:

$f_{ed} \le P_{ed}$

$f_{ed} = w^1 / t = 79.5 / 16 = 4.97$ N/mm^2

When compression flange is restrained against rotation relative to the web

$$P_{ed} = \left[2.75 + \frac{2}{(a/d)^2}\right]\frac{E}{(d/t)^2} = \left[2.75 + \frac{2}{\left(\frac{3600}{2600}\right)^2}\right]\frac{200000}{\left(\frac{2600}{16}\right)^2}$$

$$= \frac{3.79*20000}{26406} = 28.7 \text{ N/mm}^2$$

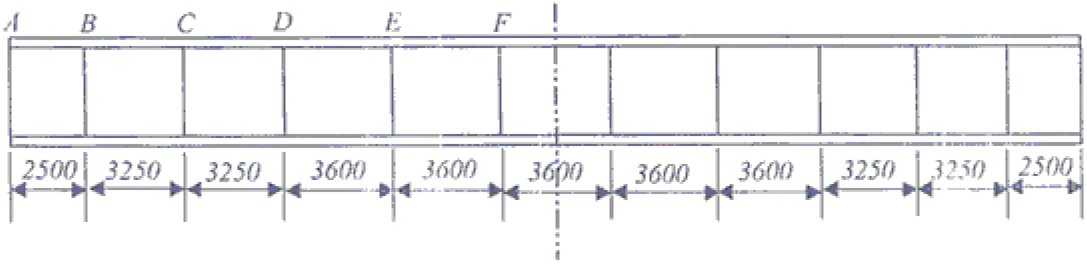
Since,

$f_{ed} \le P_{ed}$

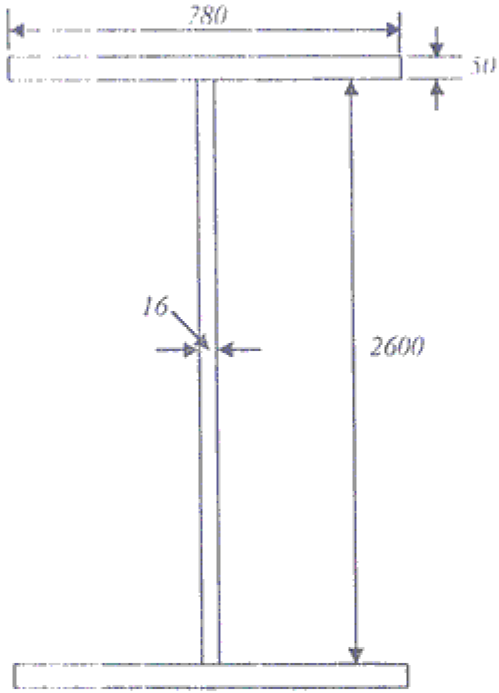
[4.97 <28.7], the web is OK for all panels.

8.0 FINAL GIRDER

(All dimensions are in mm)



(a) Longitudinal section of plate girder



(b) Flange and web proportions

Fig. E5 Final girder

Examples

Problem: 1

A non – sway intermediate column in a building frame with flexible joints is 4.0 m high and it is ISHB 300 @ 588 N/m steel section. Check the adequacy of the section when the column is subjected to following load:

Factored axial load = 500 kN

Factored moments:

	M_x	M_y
Bottom	+ 7.0 kN –m	- 1.0 kN - m
Top	+ 15.0 kN – m	+ 0.75 kN – m

[f_y = 250 N/mm^2 ; E = 2* 10^5 N/mm^2]

Assume effective length of the column as 3.4 m along both the axes.

Cross-section properties:

Flange thickness	=	T	=	10.6 mm
Clear depth between flanges	=	d	=	300 – (10.6 * 2)
			=	278.8 mm
Thickness of web	=	t	=	7.6 mm
Flange width	=	2b	=	250 mm
		b	=	125 mm
Area of cross-section	=	A_g	=	7485 mm^2

τ_x = 129.5 mm

τ_y = 54.1 mm

I_x = $12545.2*10^4$ mm^4

I_y = $2193.6*10^4$ mm^4

Z_x = $836.3*10^3$ mm^3

Z_y = $175.5*10^3$ mm^3

Z_{px} = $953.4*10^3$ mm^3

Z_{py} = $200.1*10^3$ mm^3

(i) Type of section:

$$\frac{b}{T} = \frac{125}{10.6} = 11.8 < 13.65 \in$$

$$\frac{d}{t} = \frac{278.8}{7.6} = 36.7 < 40.95 \in$$

$$\text{where, } \in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Hence, cross- section is " SEMI-COMPACT" (Class 3)

(ii) Check for resistance of cross-section to the combined effects for yielding:

f_{yd} = f_y/γ_a = 250/1.15

= 217.4 N/mm^2

A_g = 7485 mm^2

Z_x = $836.3*10^3$ mm^3

Z_y = $175.5*10^3$ mm^3

F_c = 500 kN

M_x = 15 kN-m

M_y = 1.0 kN-m

The interaction equation is:

$$\frac{F_c}{A_g\, f_y\, d} + \frac{M_x}{Z_x\, f_y\, d} + \frac{M_y}{Z_y\, f_y\, d} \le 1$$

$$\frac{500\,x\,10^3}{7845\,x\,217.4} + \frac{15\,x\,10^6}{836.3\,x\,217.4} + \frac{1\,x\,10^3}{175.5\,x\,10^3\,x\,217.4}$$

$$= 0.307 + 0.083 + 0.026 = 0.416 < 1.0$$

Hence, section is O.K. against combined effects

(iii) Check for resistance of cross-section to the combined effects for buckling:

Slenderness ratios:

Effective length of the column = 3.4 m

λ_x = 3400/129.5 = 26.3

λ_y = 3400/54.1 = 62.8

λ_1 = $\pi(E/f_y)^{1/2}$ = $\pi(200000/250)^{1/2}$

= 88.9

Non-dimensional slenderness ratios:

$$\bar{\lambda} = \frac{\lambda}{\lambda_1}$$

$$\bar{\lambda}_x = \frac{26.3}{88.9} = 0.296$$

$$\bar{\lambda}_y = \frac{62.8}{88.9} = 0.706$$

Calculation of χ :

Imperfection factors:

$\alpha_x = 0.21$; $\alpha_y = 0.34$

ϕ - values:

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$\phi_x = 0.5[1 + 0.21(0.296 - 0.2) + (0.296)^2] = 0.554$

$\phi_y = 0.5[1 + 0.34(0.706 - 0.2) + (0.706)^2] = 1.006$

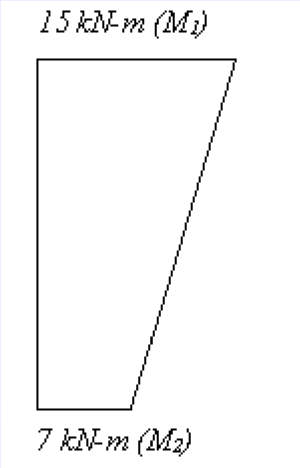
χ - values: $$\chi = \frac{\chi_1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{\frac{1}{2}}} \le 1.0$$

$\chi_x = 1/[0.554 + (0.554^2 - 0.296^2)^{1/2}] = 0.978$

$\chi_y = 1/[1.006 + (1.006^2 - 0.706^2)^{1/2}] = 0.580$

The interaction equation is

$$\frac{F_c}{f_d} + \frac{k_x M_x}{M_{ux}} + \frac{k_y M_y}{M_{uy}} \le 1$$



$\psi_x = M_2/M_1 = 7/15 = 0.467$

$\beta_{Mx} = 1.8 - 0.7\psi = 1.8 - 0.7\times 0.467$

$= 1.473$

$\mu_x = \bar{\lambda}_x(2\beta_{Mx} - 4) = 0.296\ (2\times 1.473 - 4)$

$= -0.312$

$$k_x = 1 - \frac{\mu_x F_c}{P_{cx}} = 1 - \frac{\mu_x F_c}{\chi_x A f_y} = 1 - \frac{(-0.312)\,x\,500\,x\,10^3}{0.978\,x\,7485\,x\,250} = 1.085$$

$\psi_y = 0.75/(-1.0) = -0.75$

$\beta_{My} = 1.8 - 0.7\psi$

$= 1.8 + 0.7\times 0.75 = 2.325$

$\mu_y = \bar{\lambda}_y(2\beta_{My} - 4)$

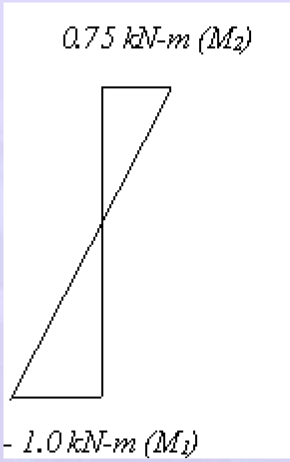
$= 0.706\ (2\times 2.325 - 4) = 0.459$

$$k_y = 1 - \frac{\mu_y F_c}{P_{cy}} = 1 - \frac{\mu_y F_c}{\chi_y A f_y} = 1 - \frac{0.459\,x\,500\,x\,10^3}{0.58\ x\,7485\,x\,250} = 0.788$$

Note: $F_{cl} = \chi_{min} A_g f_{yd}$

$M_{ux} = Z_x f_{yd}$

$M_{uy} = Z_y f_{yd}$



Substituting the interaction equation,

$$\frac{500 \times 10^3}{7845 \times 217.4 \times 0.58} + \frac{15 \times 10^6 \times 1.085}{836.3 \times 10^3 \times 217.4} + \frac{1 \times 10^6 \times 0.788}{175.5 \times 10^3 \times 217.4}$$

$$= 0.530 + 0.089 + 0.021 = 0.640 < 1.0$$

Hence, section is O.K. against combined effects for buckling.

Structural steel design project

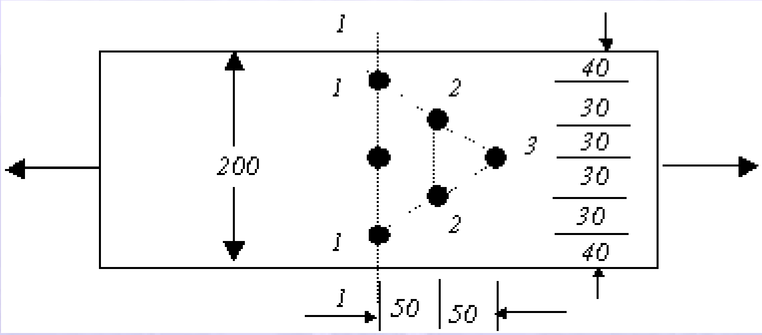
Worked example 1

Problem 1:

Determine the desig tensile strength of the plate (200 X 10 mm) with the holes as shown below, if the yield strength and the ultimate strength of the steel used are 250 MPa and 420 MPa and 20 mm diameter bolts are used.

f_y = 250 MPa

f_u = 420 MPa



Calculation of net area, A_{net}:

A_n Results you need, click here

P_t is lesser of

$$(i)\ A_g f_y / \gamma M_0 = \frac{200*10*250/1.15}{1000} = 434.8 kN$$

$$(ii)\ 0.9 A_n f_u / \gamma M_1 = \frac{0.9*1342*420/1.25}{1000} = 405.8 kN$$

$$P_t = 405.8\ kN$$

Efficiency of the plate with holes = $\frac{P_t}{A_g f_y / \gamma M_0} = \frac{409.8}{434.8} = 0.93$

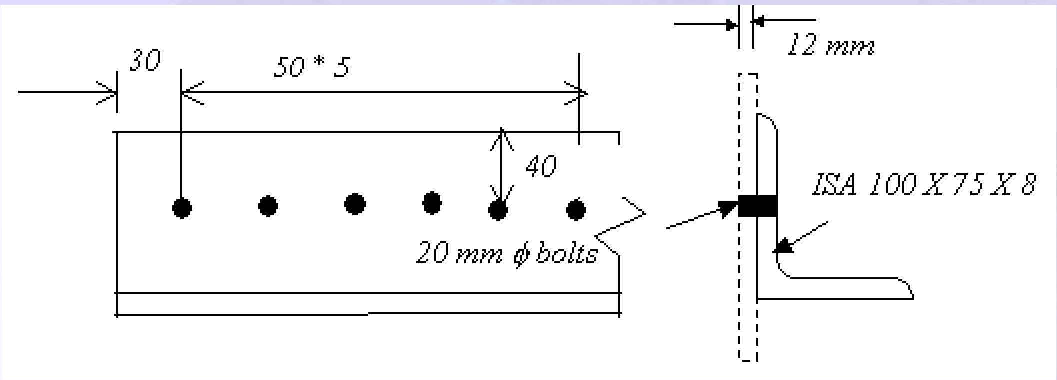
Structural steel design project

Worked example 2

Problem 2:

Analysis of single angle tension members

A single unequal angle 100 X 75 X 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 nos. 20 mm diameter bolts to transfer tension. Determine the design tensile strength of the angle. (a) if the gusset is connected to the 100 mm leg, (b) if the gusset is connected to the 75 mm leg, (c) if two such angles are connected to the same side of the gusset through the 100 mm leg. (d) if two such angles are connected to the opposite sides of the gusset through 100 mm leg.



a) The 100mm leg bolted to the gusset :

A_{nc} = (100 - 8/2 - 21.5) *8 = 596 mm^2.

A_o = (75 - 8/2) * 8 = 568.mm^2

A_g = ((100-8/2) + (75 – 8/2)) * 8 =1336 mm^2

Strength as governed by tearing of net section:

Since the number of bolts = 4; $\beta = 1.0$

$$P_t = A_{nc} f_u / \gamma_{m1} + \beta A_0 f_y / \gamma_{m0}$$

= 596 * 420/1.25 + 1.0 * 568 * 250 / 1.15

= 323734 N (or) 323.7 kN

Strength as governed by yielding of gross section:

$$P_t = A_g f_y / \gamma_{m0}$$

$$= 1336 * 250 / 1.15 = 290435 \text{ N (or) } 290.4 \text{ kN}$$

Block shear strength

V_g – Grass "shearing"

t_n – Tearing net

$$P_v = \left(0.62\, A_{vg}\, f_y / \gamma_{m0} + A_{tn}\, f_u / \gamma_{m1}\right)$$ (Shear yield + tensile fracture)

= 0.62 * (5 *50 +30)* 8 * 250/1.15 + (40-21.5/2) * 8 * 420/1.25

= 380537 N = 380.5 kN

or

$$P_v = \left(0.62\, A_{vn}\, f_u / \gamma_{m1} + A_{tg}\, f_y / \gamma_{m0}\right)$$ (Shear fracture + tensile yield)

= (0.62 (5 * 50 + 30 –5.5 * 21.5) * 8 * 420 / 1.25 + 40 * 8 * 250/ 1.15)

= 339131 N = 339.1

The design tensile strength of the member = 290.4 kN

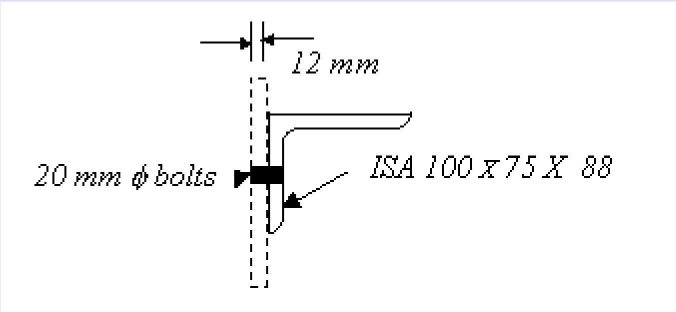
The efficiency of the tension member, is given by

$$\eta = \frac{P_t}{A_g f_y} = \frac{290.4 * 1000}{(100 + 75 - 8) * 8 * 250/1.15} = 1.0$$

b) The 75 mm leg is bolted to the gusset:

A_{nc} = (75 - 8/2 - 21.5) * 8 = 396 mm^2

A_o = (100 - 8/2) * 8 = 768 mm^2



Strength as governed by tearing of net section:

Since the number of bolts = 6, $\beta = 1.0$

$$P_t = A_{nc} f_u / \gamma_{m1} + \beta A_0 f_y / \gamma_{m0}$$

= 396 * 420/1.25 + 1.0 * 768 *250 / 1.15

= 300123 N (or) 300.1 kN

Strength as governed by yielding of gross section:

$$P_t = A_g f_y / \gamma_{m0}$$

= 1336 * 250 / 1.15 = **290435 N (or) 290.4 kN**

Block shear strength:

$$P_v \le \left(0.62\, A_{vg} f_y / \gamma_{m0} + A_{tn} f_u / \gamma_{m1}\right)$$

= 0.62 * (5 *50 +30)* 8 * 250/1.15 + (35-21.5/2) * 8 * 420/1.25

= 367097 N = **367.1 kN**

$$P_v \le \left(0.62\, A_{vn} f_u / \gamma_{m1} + A_{tg} f_y / \gamma_{m0}\right)$$

= (0.62 (5 * 50 + 30 –5.5 *x 21.5) * 8 * 420 / 1.25 + 35 *8 * 250/ 1.15

= 330435 N = **330.4 kN**

The design tensile strength of the member = **290.4 kN**

Even though the tearing strength of the net section is reduced, the yielding of the gross section still governs the design strength.

The efficiency of the tension member is as before 1.0

Note: The design tension strength is more some times if the longer leg of an unequal angle is connected to the gusset (when the tearing strength of the net section governs the design strength).

An understanding about the range of values for the section efficiency, η, is useful to arrive at the trial size of angle members in design problems.

(c & d)The double angle strength would be twice single angle strength as obtained above in case (a)

$P_t = 2 * 290.4 = 580.8$ kN

Design Example 1:

Design a Lap joint between plates 100? 8 so as to transmit a factored load of 100 kN using black bolts of 12mm diameter and grade 4.6. The plates are made of steel of grade ST-42-S.

Solution:

1) Strength Calculations:

Nominal diameter of bolt d= 12 mm

For grade 4.6 bolt, f_u =40 kgf / mm^2=392.4 MPa, $_{mb}$ =1.25

Assuming threads in the shear plane, n_n=1, n_s=0

Shear Area of one bolt A_{nb} = 0.8 A_{sb} = 0.8 x 113.1 = 90.5 mm^2

Design shear strength per bolt $V_{nsb} = f_u A_{nb} / \gamma_{mb} \sqrt{3}$ = 16.4 kN (Cl. 10.3.2)

Design bearing strength per bolt V_{npb} = 2.5 d t f_u

=2.5 x 12 x 8 x 392.4 x 10^{-3} =75.2 kN (Cl. 10.3.3)

Therefore, bolt value = 16.4 kN

No. of bolts required = 100 / 16.4 = 6.1 say 7 bolts

2) Detailing:

Minimum pitch = 2.5 d = 30 mm (Cl. 10.2.1)

Minimum edge distance = 1.4 D = 16.8 mm say 20 mm (Cl. 10.2.3)

Provide 8 bolts as shown in Fig. E1.

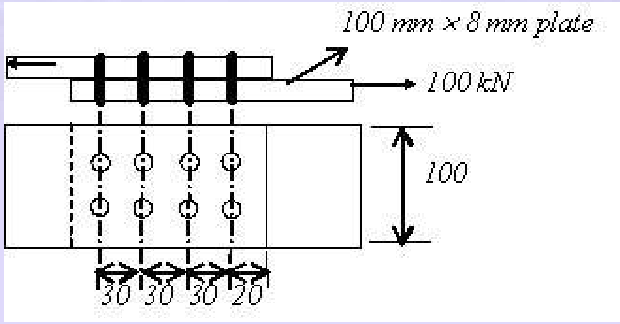


Fig. E1

Design Example 2:

Design a hanger joint along with an end plate to carry a downward load of 2T = 330 kN. Use end plate size 240 mm x 160 mm and appropriate thickness and 2 nos of M25 Gr.8.8 HSFG bolts (fo = 565 MPa).

Solution

Assume 10mm fillet weld between the hanger plate and the end plate

Distance from center line of bolt to toe of fillet weld l_v = 60 mm

1) For minimum thickness design, M = T l_v / 2 = 165 x 60 / 2 = 4950 N-m

$$\therefore t_{\min} = \sqrt{\frac{1.15 \times 4 \times 4950 \times 10^3}{236 \times 160}} = 24.56 \text{ say } 25\, mm$$

$$\mathrm{Mp} = \mathrm{Zp.fy} = \frac{\mathrm{Wt2}}{4} . \frac{\mathrm{fy}}{\gamma \mathrm{m0}}$$

$$\mathrm{t} = \sqrt{4\mathrm{Mp}\, {}^{\gamma \mathrm{m0}}\!/_{\mathrm{fy}} \;\mathrm{x\ w}}$$

2) Check for prying forces distance' l_e' from center line of bolt to prying force is the minimum of edge distance or 1.1t

$\sqrt{(\beta p_o / f_y)}$ = 1.1 x 25 $\sqrt{(2 \times 565 / 236)}$ = 60mm (Cl. 10.4.7)

l_e = 40 mm

prying force Q= M / l_e = 4950 / 40= 123.75 kN

bolt load = 165 + 123.75=288.75 kN (Cl. 10.4.5)

tension capacity of 25 mm dia HSFG bolt = $0.9F_uA_{nb}/\gamma mb$ = 222 kN << 288.75

Load carrying Capacity << Required load Capacity

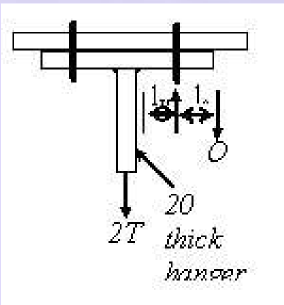


Fig E2

In order to reduce the load on bolt to a value less than the bolt capacity, a thicker end plate will have to be used.

Allowable prying force Q = 222 - 165 = 57 kN

Trying a 36 mm thick end plate gives l_e = 40 mm as before

Moment at toe of weld = T l_v - Q l_e = 165 x 60 – 57 x 40 = 7620 N-m

Moment capacity = (236 / 1.10) (160 x 36^2/4) x 10^{-3} = 11122 N-m > 7620 OK

Minimum prying force

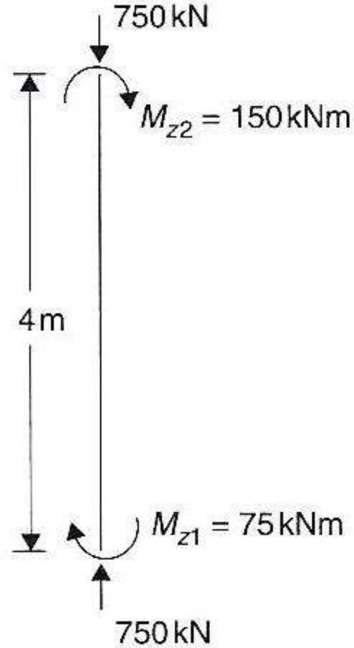
$$Q = \frac{l_v}{2l_e}\left[T - \frac{\beta\gamma p_o b_e t^4}{27 l_e l_v^2}\right] = \frac{60}{2\times 40}\left[165 - \frac{2\times 1.5\times 0.565\times 160\times 36^4}{27\times 40\times 60^2}\right] \qquad \text{(Cl. 10.4.7)}$$

= 36 kN < 57 kN safe!

Therefore, 36 mm end plate needs to be used to avoid significant prying action.

Beam-Column Design Example 2

Design a beam-column of unsupported length 4 m to carry an axial compression of 500 kN and end moments of 50 kNm and 100 kNm which bend the member into a reverse curvature. Assume that the ends of the member are rigidly connected to beams and are prevented from having a side sway. The grade of the steel is E250.



a reverse curvature. Assume that the ends of the member are rigidly connected to beams and are prevented from having a side sway. The grade of the steel is E250.

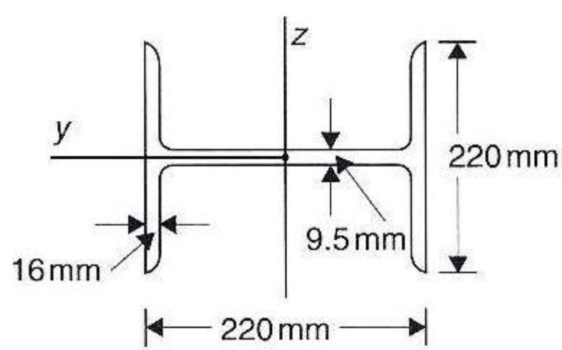
The factored axial compression, $P = 1.5 \times 500 = 750\,\text{kN}$
The factored end moments, $M_{z1} = 1.5 \times 50 = 75\,\text{kNm}$ and $M_{z2} = 1.5 \times 100 = 150\,\text{kNm}$
A trail section for the beam-column is obtained by considering only an axial load of the magnitude of twice the actual axial load. Let $f_{cd} = 180\,\text{MPa}$.

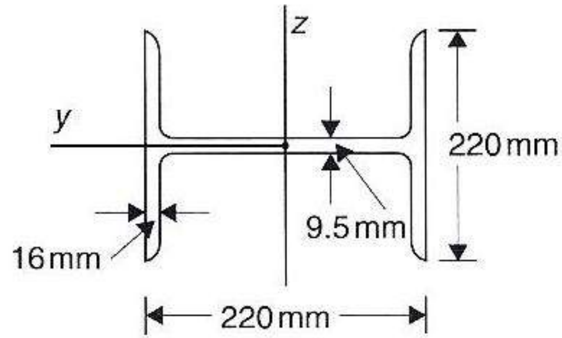
$$\text{The required area of the section} = \frac{2 \times 750 \times 10^3}{180} = 8{,}333\,\text{mm}^2$$

From Appendix A, SC 220 section may be tried. Its properties are

$A = 8{,}980\,\text{mm}^2$, $r_z = 93.5\,\text{mm}$, $r_y = 49\,\text{mm}$, $b_f = 220\,\text{mm}$, $t_f = 16\,\text{mm}$, $t_w = 9.5\,\text{mm}$

$$h/b_f = 220/220 = 1.0 < 1.2$$





∴ From Table 4.3, the buckling class is '*b*' for the *z*-axis and '*c*' for the *y*-axis.

$$\frac{b}{t_f} = \frac{220/2}{16} = 6.9 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{(220 - 2 \times 16)}{9.5} = 19.8 < 84\varepsilon \qquad \text{where } \varepsilon = 1.0$$

∴ From Table 1.7, the section is plastic.

The correct value of the effective length factor K is to be obtained from the Annexure D of IS800. However, in this problem, it is assumed as 0.7 in both the *xz* and the *xy* planes.

$$\frac{KL}{r_z} = \frac{0.7 \times 4{,}000}{93.5} = 30$$

From Table 4.4, for the buckling class 'b', $f_{cdz} = 216\,\text{MPa}$

$$\frac{KL}{r_y} = \frac{0.7 \times 4{,}000}{49} = 57$$

From Table 4.4, for the buckling class 'c', $f_{cdy} = 172.5\,\text{MPa}$

The design strengths in compression about the z and the y axes are

$$P_{dz} = A\, f_{cdz} = 8{,}980 \times 216 = 1{,}940\,\text{kN}$$

$$P_{dy} = A\, f_{cdy} = 8{,}980 \times 172.5 = 1{,}549\,\text{kN}$$

From Table 5.1, assuming that the ends are fully restrained against torsion and warping and the loading condition is normal, the length for the lateral torsional buckling,

$$L_{LT} = 0.7 \times 4{,}000 = 2{,}800\,\text{mm}.$$

$$h_f = 220 - 16 = 204\,\text{mm}$$

The extreme fibre stress corresponding to the lateral torsional buckling is

$$f_{cr,b} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(2{,}800/49)^2}\left[1 + \frac{1}{20}\left(\frac{(2{,}800/49)}{(204/16)}\right)^2\right]^{0.5} = 941.5\,\text{MPa}$$

From Table 13(a) of IS 800, for $f_y = 250\,\text{MPa}$, $\alpha = 0.21$, $f_{cr,b} = 941.5\,\text{MPa}$, $f_{bd} = 206\ \text{MPa}$

$$Z_{pz} = 2\left[220 \times 16 \times (110 - 8) + 9.5 \times \frac{(110-16)^2}{2}\right] = 802\,\text{cm}^3$$

The design bending strength, $M_{dz} = \beta_b Z_{pz} f_{bd} = 1.0 \times 802 \times 10^3 \times 206 = 165.5\,\text{kNm}$

$$\psi = \frac{M_{z1}}{M_{z2}} = \frac{75}{150} = 0.5$$

$$C_{mz} = 0.6 - 0.4 \times 0.5 = 0.2 < 0.4$$

$$\therefore \quad C_{mz} = 0.4$$

Since there are no lateral supports other than at the ends, $C_{mLT} = C_{mz} = 0.4$

$$f_{ccz} = \frac{\pi^2 E}{(KL/r_z)^2} = \frac{\pi^2 \times 2 \times 10^5}{30^2} = 2{,}193\,\text{MPa}$$

$$\therefore \quad \lambda_z = \sqrt{\frac{f_y}{f_{ccz}}} = \sqrt{\frac{250}{2{,}193}} = 0.34$$

$$n_z = P/P_{dz} = 750/1{,}940 = 0.39 \quad \text{and} \quad n_y = P/P_{dy} = 750/1{,}549 = 0.48$$

$$K_z = 1 + (0.34 - 0.2) \times 0.39 = 1.05 < (1 + 0.8 \times 0.39) = 1.31$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}} = \sqrt{\frac{250}{941.5}} = 0.515$$

$$K_{LT} = \left[1 - \frac{(0.1 \times 0.515 \times 0.48)}{(0.4 - 0.25)}\right] = 0.83 > \left[1 - \frac{0.1 \times 0.48}{(0.4 - 0.25)}\right] = 0.68$$

Check for yielding

$$N_d = Af_y/\gamma_{m0} = 8{,}980 \times 250/1.1 = 2{,}041 \text{ kN}$$

$$\frac{N}{N_d} + \frac{M_z}{M_{dz}} = \frac{750}{2{,}041} + \frac{150}{165.5} = 1.27 > 1.0$$

Check for buckling

$$\frac{P}{P_{dy}} + K_{LT}\frac{M_z}{M_{dz}} = \frac{750}{1{,}549} + 0.83 \times \frac{150}{165.5} = 1.23 > 1.0$$

$$\frac{P}{P_{dz}} + K_z\frac{C_{mz}M_z}{M_{dz}} = \frac{750}{1{,}940} + 1.05 \times \frac{0.4 \times 150}{165.5} = 0.77 < 1.0$$

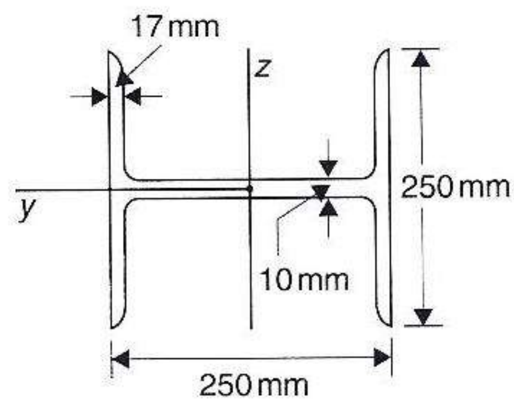
Since two of the interaction equations are not satisfied, a higher section SC250 may be tried.

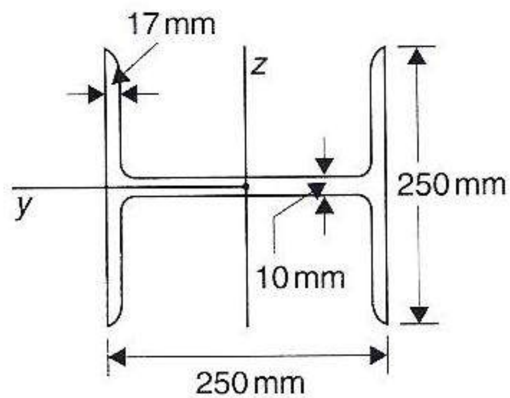
From Appendix A, $A = 10{,}900\,\text{mm}^2$, $r_z = 107\,\text{mm}$, $r_y = 54.6\,\text{mm}$, $b_f = 250\,\text{mm}$,
$t_f = 17\,\text{mm}$, $t_w = 10\,\text{mm}$

$$h/b_f = 250/220 = 1.0 < 1.2$$

$\therefore$ From Table 4.3, the buckling class is '*b*' for the *z*-axis and '*c*' for the *y*-axis.

$$\frac{b}{t_f} = \frac{250}{17} = 7.3 < 9.4\varepsilon$$





$$\frac{d}{t_w} = \frac{(250 - 2 \times 17)}{10} = 21.6 < 84\varepsilon \qquad \text{where } \varepsilon = 1.0$$

∴ From Table 1.7, the section is plastic.

$$\frac{KL}{r_z} = \frac{0.7 \times 4{,}000}{107} = 26$$

From Table 4.4, for the buckling class '*b*', $f_{cdz} = 219\,\text{MPa}$

$$\frac{KL}{r_y} = \frac{0.7 \times 4{,}000}{54.6} = 51$$

From Table 4.4, for the buckling class '*c*', $f_{cdy} = 181\,\text{MPa}$

The design strengths in compression about the z and y axes are

$$P_{dz} = A\, f_{cdz} = 10{,}900 \times 219 = 2{,}387\,\text{kN}$$

$$P_{dy} = A\, f_{cdy} = 10{,}900 \times 181 = 1{,}973\,\text{kN}$$

$$h_f = 250 - 17 = 233\,\text{mm}$$

The extreme fibre stress corresponding to the lateral torsional buckling is

$$f_{cr,b} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(2{,}800/54.6)^2}\left[1 + \frac{1}{20}\left(\frac{(2{,}800/54.6)}{(233/17)}\right)^2\right]^{0.5} = 1{,}073\,\text{MPa}$$

From Table 13(a) of IS 800, for $f_y = 250\,\text{MPa}$, $\alpha = 0.21$, $f_{cr,b} = 1{,}073\,\text{MPa}$, $f_{bd} = 210\,\text{MPa}$

$$Z_{pz} = 2\left[250 \times 17 \times (125 - 8.5) + 10 \times \frac{(125 - 17)^2}{2}\right] = 1{,}107\,\text{cm}^3$$

The design bending strength, $M_{dz} = \beta_b\, Z_{pz}\, f_{bd} = 1.0 \times 1{,}107 \times 10^3 \times 210 = 233\,\text{kNm}$

$$f_{ccz} = \frac{\pi^2 E}{(KL / r_z)^2} = \frac{\pi^2 \times 2 \times 10^5}{26^2} = 2{,}876\,\text{MPa}$$

$$\therefore \quad \lambda_z = \sqrt{\frac{f_y}{f_{ccz}}} = \sqrt{\frac{250}{2{,}876}} = 0.3$$

$$n_z = P/P_{dz} = 750/2{,}387 = 0.31 \quad \text{and} \quad n_y = P/P_{dy} = 750/1{,}973 = 0.38$$

$$K_z = 1 + (0.3 - 0.2) \times 0.31 = 1.03 < (1 + 0.8 \times 0.31) = 1.25$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}} = \sqrt{\frac{250}{1{,}073}} = 0.48$$

$$K_{LT} = \left[1 - \frac{(0.1 \times 0.48 \times 0.38)}{(0.4 - 0.25)}\right] = 0.88 > \left[1 - \frac{0.1 \times 0.38}{(0.4 - 0.25)}\right] = 0.75$$

Check for yielding

$$N_d = A\, f_y/\gamma_{m0} = 10{,}900 \times 250/1.1 = 2{,}477\ \text{kN}$$

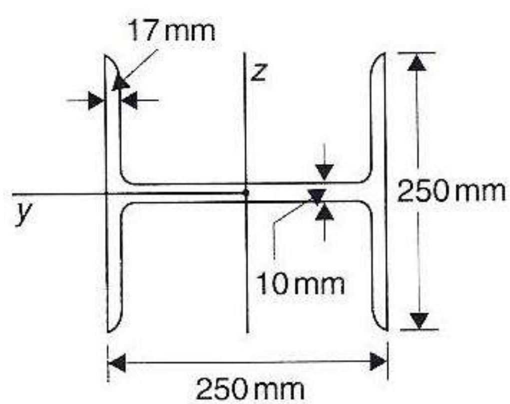
$$\frac{N}{N_d} + \frac{M_z}{M_{dz}} = \frac{750}{2{,}477} + \frac{150}{233} = 0.94 < 1.0$$

Check for buckling

$$\frac{P}{P_{dy}} + K_{LT}\frac{M_z}{M_{dz}} = \frac{750}{1{,}973} + 0.88 \times \frac{150}{233} = 0.95 < 1.0$$

$$\frac{P}{P_{dz}} + K_z\frac{C_{mz}M_z}{M_{dz}} = \frac{750}{2{,}387} + 1.03 \times \frac{0.4 \times 150}{233} = 0.58 < 1.0$$

$\therefore$ SC 250 section may be provided.



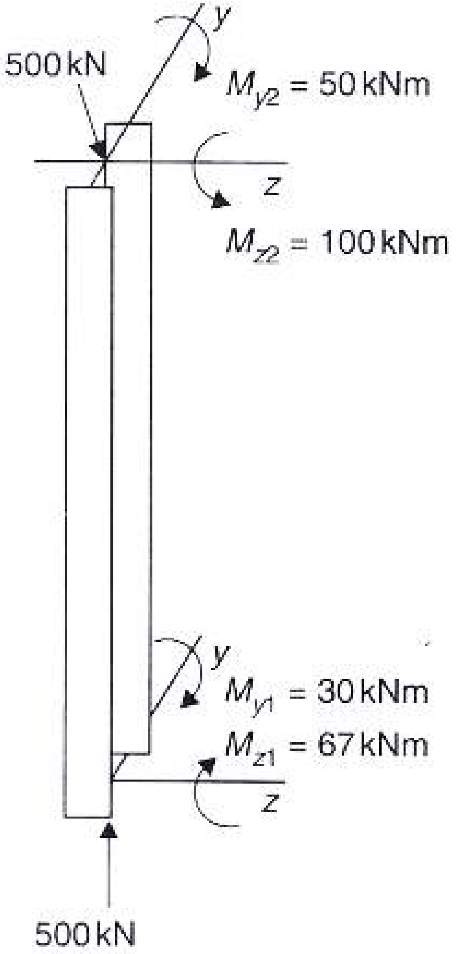
Beam-Column Design Example 3

Design a beam-column for the following data:

$P_u = 500\,\text{kN}$, $M_{uz1} = -67\,\text{kNm}$, $M_{uz2} = 100\,\text{kNm}$, $M_{uy1} = 30\,\text{kNm}$, $M_{uy2} = 50\,\text{kNm}$

Unsupported length = 5 m. The grade of the steel is E250.

Side sway is prevented and the effective length factor, K = 0.7.



As the beam-column is subjected to a biaxial bending and axial compression, the trail section may be obtained by considering only an axial load of magnitude of three times the actual axial load. Let $f_{cd} = 120$ MPa.

$$\text{The required area of the section} = \frac{3 \times 500 \times 10^3}{120} = 12,500 \text{ mm}^2$$

From Appendix A, HB 450 @ 92.5 kg/m may be tried.

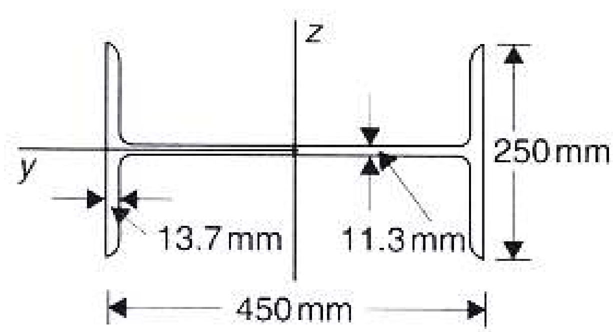
$A = 11,789 \text{ mm}^2$, $b_f = 250$ mm, $t_f = 13.7$ mm, $t_w = 11.3$ mm, $r_z = 185$ mm, $r_y = 50.8$ mm and $Z_{pz} = 2,030.95 \text{ cm}^3$.

$$h/b_f = 450/250 = 1.8 > 1.2 \quad \text{and} \quad t_f = 13.7 \text{ mm} < 40 \text{ mm}.$$

From Table 4.3, the buckling class is '*a*' for the *z*-axis and '*b*' for the *y*-axis.

$$\frac{b}{t_f} = \frac{250/2}{13.7} = 9.1 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{(450 - 2 \times 13.7)}{11.3} = 37.4 < 84\varepsilon$$



From Table 1.7, the section is plastic.

$$\frac{KL}{r_z} = \frac{0.7 \times 5{,}000}{185} = 18.9$$

From Table 4.4, for the buckling class 'a', $f_{cdz} = 226\,\text{MPa}$

$$\frac{KL}{r_y} = \frac{0.7 \times 5{,}000}{50.8} = 69$$

From Table 4.4, for the buckling class 'b', $f_{cdz} = 167.5\,\text{MPa}$

$$P_{dz} = 11{,}789 \times 226 = 2{,}664\,\text{kN}$$

$$P_{dy} = 11{,}789 \times 167.5 = 1{,}975\,\text{kN}$$

$$L_{LT} = 0.7 \times 5{,}000 = 3{,}500\,\text{mm}$$

$$h_f = 450 - 13.7 = 436.3\,\text{mm}$$

$$f_{cr,b} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(3{,}500 / 50.8)^2}\left[1 + \frac{1}{20}\left(\frac{3{,}500 / 50.6}{436.3 / 13.7}\right)^2\right]^{0.5} = 508.5\,\text{MPa}$$

From Table 13(a) of IS800, $\alpha = 0.21$, $f_{cr,b} = 508.5\,\text{MPa}$, $f_{bdz} = 189.4\,\text{MPa}$

$$f_{bdy} = 250/1.1 = 227.3\,\text{MPa}$$

$$M_{dz} = \beta_b \, Z_{pz} \, f_{bd} = 1.0 \times 2{,}030.95 \times 10^3 \times 189.4 = 385 \, \text{kNm}$$

$$Z_{py} = 4 \times 125 \times 13.7 \times \frac{125}{2} + 2 \times (450 - 13.7) \times \frac{11.3}{2} \times \frac{11.3}{4} = 442 \, \text{cm}^3$$

$$M_{dy} = 1.0 \times 442 \times 10^3 \times 227.3 = 100.5 \, \text{kNm}$$

$$f_{ccz} = \frac{\pi^2 \times 2 \times 10^5}{18.9^2} = 5{,}526 \quad \text{and} \quad f_{ccy} = \frac{\pi^2 \times 2 \times 10^5}{69^2} = 414.6$$

$$\lambda_z = \sqrt{\frac{250}{5{,}526}} = 0.213 \quad \text{and} \quad \lambda_y = \sqrt{\frac{250}{414.6}} = 0.776$$

$$n_z = P/P_{dz} = 500/2{,}664 = 0.19 \quad \text{and} \quad n_y = P/P_{dy} = 500/1{,}975 = 0.25$$

$$K_y = 1 + (0.776 - 0.2) \times 0.25 = 1.144 < (1 + 0.8 \times 0.25) = 1.2$$

$$K_z = 1 + (0.213 - 0.2) \times 0.19 = 1.0 < (1 + 0.8 \times 0.19) = 1.15$$

$$\lambda_{LT} = \sqrt{\frac{250}{508.5}} = 0.7$$

$$\psi_z = \frac{M_{z1}}{M_{z2}} = \frac{-67}{100} = -0.67 \quad \text{and} \quad \psi_y = \frac{M_{y1}}{M_{y2}} = \frac{30}{50} = 0.6$$

$$C_{mz} = 0.6 - 0.4\ \psi_z = 0.6 - 0.4 \times (-0.67) = 0.87$$

$$C_{my} = 0.6 - 0.4\ \psi_y = 0.6 - 0.4 \times 0.6 = 0.36 < 0.4$$

$$\therefore \quad C_{my} = 0.4$$

Since there are no lateral supports other than at the ends, $C_{mLT} = C_{mz} = 0.87$

$$K_{LT} = 1 - \frac{0.1 \times 0.7 \times 0.25}{(0.87 - 0.25)} = 0.97 > \left(1 - \frac{0.1 \times 0.25}{(0.87 - 0.25)}\right) = 0.96$$

$$M_z = M_{uz2} = 100\,\text{kNm} \quad \text{and} \quad M_y = M_{uy2} = 50\,\text{kNm}$$

Check for yielding

$$N_d = 11{,}789 \times 250/1.1 = 2{,}679\,\text{kN}$$

$$\frac{N}{N_d} + \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{500}{2{,}679} + \frac{100}{385} + \frac{50}{100.5} = 0.94 < 1.0$$

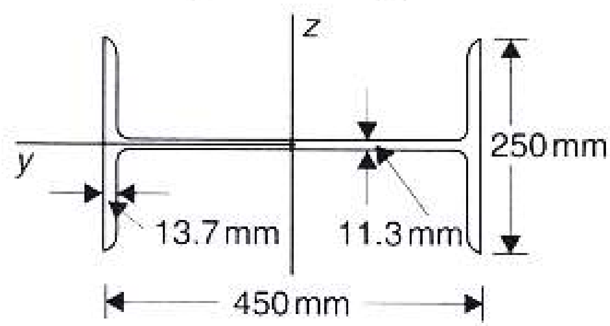
Check for buckling

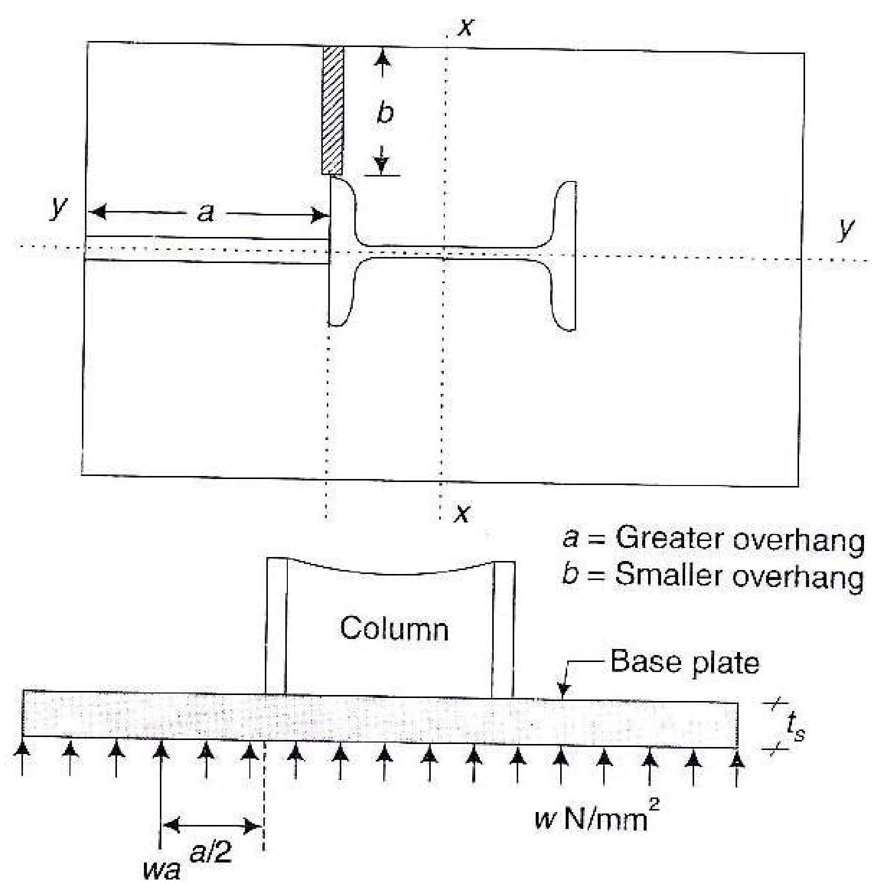
$$\frac{P}{P_{dy}} + K_y \frac{C_{my} M_y}{M_{dy}} + K_{LT} \frac{M_z}{M_{dz}} = \frac{500}{1{,}975} + 1.144 \times \frac{0.4 \times 50}{100.5} + 0.97 \times \frac{100}{385} = 0.73 < 1.0$$

$$\frac{P}{P_{dz}} + 0.6 K_y \frac{C_{my} M_y}{M_{dy}} + K_z \frac{C_{mz} M_z}{M_{dz}} = \frac{500}{2{,}664} + 0.6 \times 1.144 \times \frac{0.4 \times 50}{100.5}$$

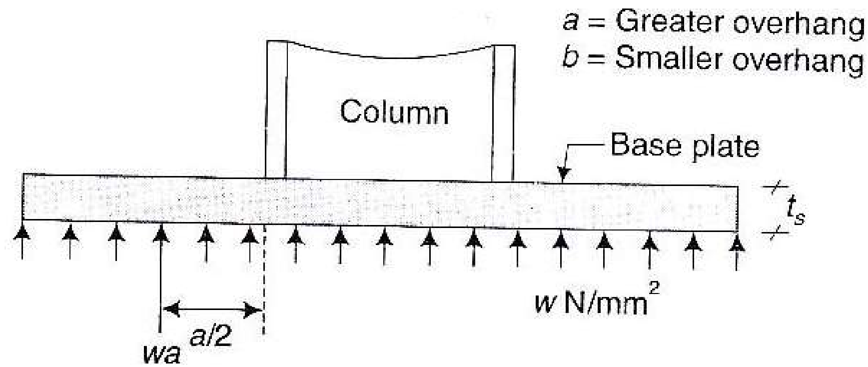
$$+ 1.0 \times \frac{0.87 \times 100}{385} = 0.55 < 1.0 \quad \text{OK}$$

Hence, HB 450 @ 92.5 kg/m is all right.





x
b
y
a
y
x
a = Greater overhang
b = Smaller overhang
Column
Base plate
t_s
w N/mm^2
wa
a/2



Taking 1 mm strip of slab projection along xx-axis

$$\text{Maximum bending moment} = w \times 1 \times a \times \frac{a}{2} = \frac{wa^2}{2}$$

where w = Intensity of bearing pressure from concrete below the base plate.

Taking 1 mm strip of slab projection along yy-axis

$$\text{Maximum bending moment} = w \times 1 \times b \times \frac{b}{2} = \frac{wb^2}{2}$$

The net moment $$M_{x,\,net} = \frac{wa^2}{2} - 0.3\,\frac{wb^2}{2}$$

or $$M_{x,\,net} = \frac{w}{2}\,(a^2 - 0.3\,b^2)$$

The moment capacity of the plate is,

$$M_p = 1.2 f_y Z_e$$

where Z_e = elastic section modulus of the base plate.

The moment capacity of 1 mm strip of plate,

$$M_{ps} = 1.2 f_y \times 1 \times \frac{t_s^2}{6} = 1.2 f_y \frac{t_s^2}{6}$$

where t_s = thickness of the base plate.

From Eqs. (3) and (4)

$$1.2 f_y \frac{t_s^2}{6} = \frac{w}{2} (a^2 - 0.3\, b^2)$$

Applying partial safety factor for material,

$$1.2 \frac{f_y}{\gamma_{m0}} \frac{t_s^2}{6} = \frac{w}{2} (a^2 - 0.3\, b^2)$$

$$t_s^2 = \frac{6 \times w}{2 \times 1.2} (a^2 - 0.3\, b^2) \frac{\gamma_{m0}}{f_y}$$

or

$$t_s^2 = 2.5w\, (a^2 - 0.3\, b^2) \frac{\gamma_{m0}}{f_y}$$

$$t_s = \sqrt{2.5\, w\, (a^2 - 0.3\, b^2) \frac{\gamma_{m0}}{f_y}}$$

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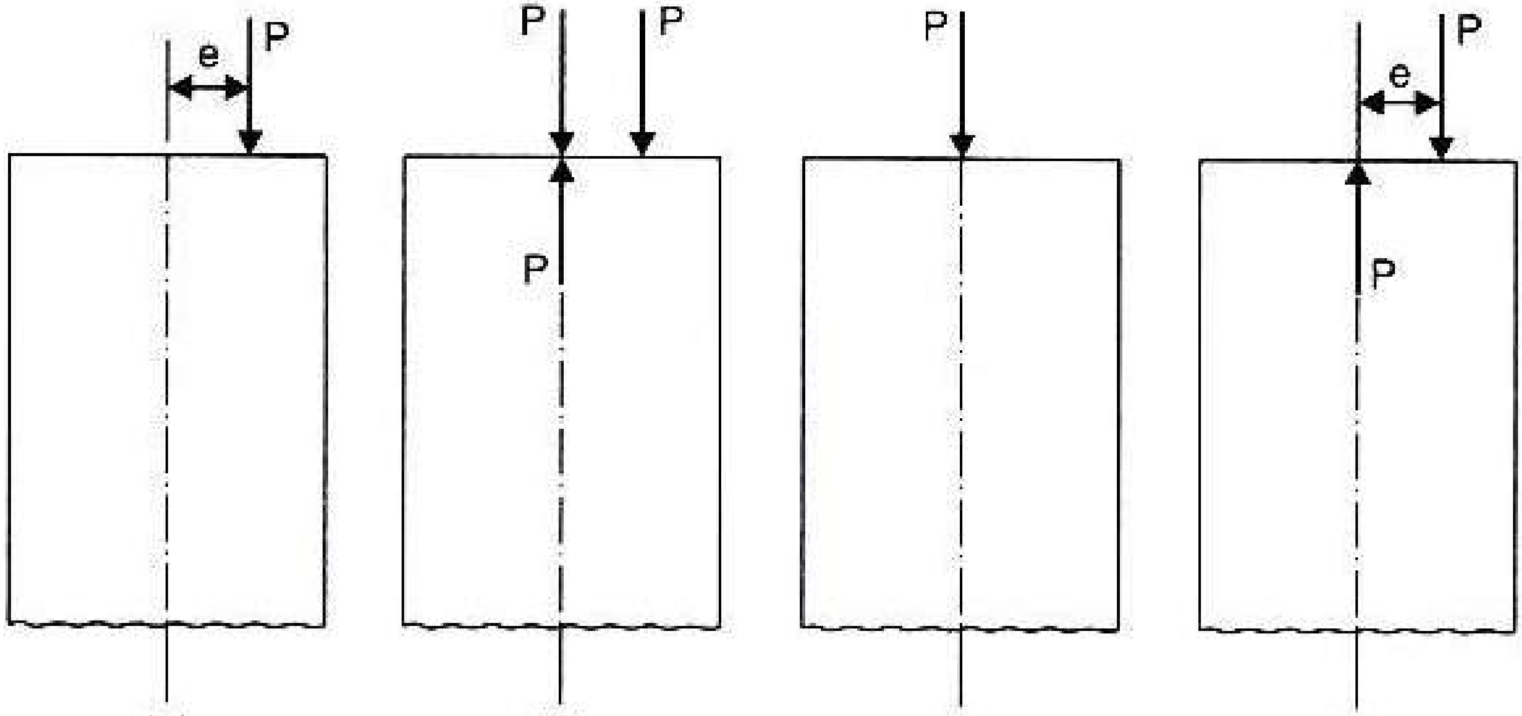
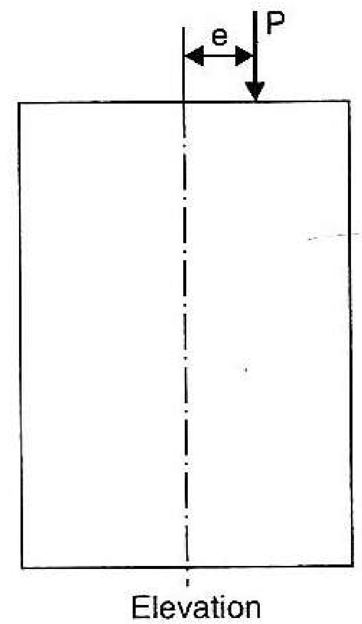
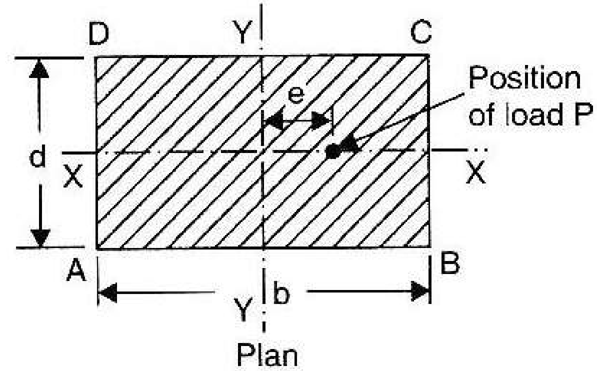


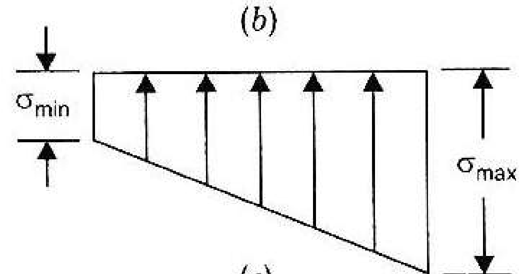
(a) (b) (c) (d)


e
P
Elevation

(a)


D
Y
C
Position of load P
e
d
X
X
A
B
Y
b
Plan

(b)


σ_{min}
σ_{max}

(c)

Let P = Eccentric load on column

e = Eccentricity of the load

σ_0 = Direct stress

σ_b = Bending stress

b = Width of column

d = Depth of column

$\therefore$ Area of column section, $A = b \times d$

Now moment due to eccentric load P is given by,

$$M = \text{Load} \times \text{eccentricity}$$

$$= P \times e$$

The direct stress (σ_0) is given by,

$$\sigma_0 = \frac{\text{Load }(P)}{\text{Area}} = \frac{P}{A} \quad \text{...}(i)$$

This stress is uniform along the cross-section of the column.

The bending stress σ_b due to moment at any point of the column section at a distance y from the neutral axis Y-Y is given by

$$\frac{M}{I} = \frac{\sigma_b}{\pm y}$$

$$\therefore \qquad \sigma_b = \pm \frac{M}{I} \times y \qquad \text{...}(ii)$$

where I = Moment of inertia of the column section about the neutral axis $Y\text{-}Y = \dfrac{d\,.\,b^3}{12}$

Substituting the value of I in equation (ii), we get

$$\sigma_b = \pm \frac{M}{\dfrac{d\,.\,b^3}{12}} \times y = \pm \frac{12\,M}{d\,.\,b^3} \times y$$

The bending stress depends upon the value of y from the axis $Y\text{-}Y$.

The bending stress at the extreme is obtained by substituting $y = \dfrac{b}{2}$ in the above equation.

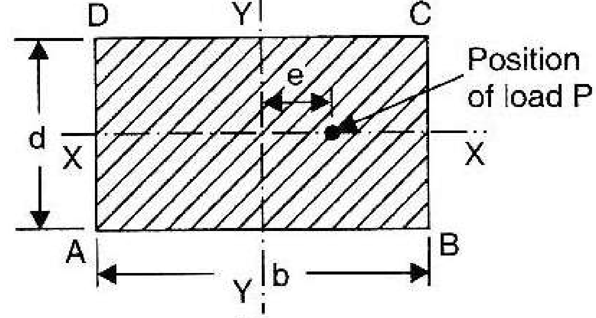
$$\therefore \qquad \sigma_b = \pm \frac{12\,M}{d\,.\,b^3} \times \frac{b}{2} = \pm \frac{6\,M}{d\,.\,b^2}$$

$$= \pm \frac{6\,P \times e}{d\,.\,b^2} \qquad (\because \quad M = P \times e)$$

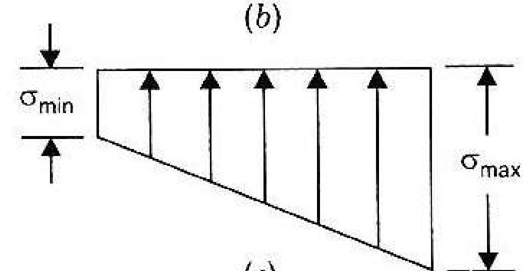
$$= \pm \frac{6\,P \times e}{d\,.\,b\,.\,b} = \pm \frac{6\,P \times e}{A \times b}$$

$$(\because \quad \text{Area} = b \times d = A)$$

The resultant stress at any point will be the algebraic sum of direct stress and bending stress.



(b)



(c)

Let σ_{max} = Maximum stress (*i.e.*, stress along BC)

σ_{min} = Minimum stress (*i.e.*, stress along AD)

Then σ_{max} = Direct stress + Bending stress

$$= \sigma_0 + \sigma_b$$

$$= \frac{P}{A} + \frac{6P.e}{A.b}$$

(Here bending stress is +ve)

$$= \frac{P}{A}\left(1 + \frac{6 \times e}{b}\right) \qquad \ldots(9.1)$$

and σ_{min} = Direct stress – Bending stress

$$= \sigma_0 - \sigma_b$$

$$= \frac{P}{A} - \frac{6P.e}{A.b} = \frac{P}{A}\left(1 - \frac{6 \times e}{b}\right)$$

Column Base Design Example 1

Design a slab base for a column section SC 200 which carries a factored axial compression of 1000 kN. The grade of the steel is E250 and the grade of the concrete pedestal is M20.

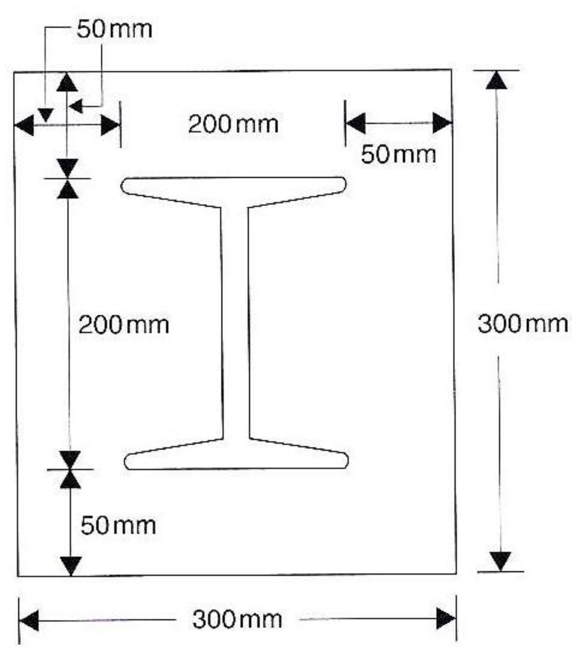
The bearing strength of the concrete $= 0.6 f_{ck} = 0.6 \times 20 = 12$ MPa

$$\text{The required area of the slab base} = \frac{1{,}000 \times 10^3}{12} = 83{,}333\,\text{mm}^3$$

A 300 mm × 300 mm slab base may be provided as shown in Figure 9.8.

$a = b = 50$ mm

$$w = \frac{1{,}000 \times 10^3}{300 \times 300} = 11\,\text{MPa}$$



$$t_s = \sqrt{\frac{2.5 \times 11 \times (50^2 - 0.3 \times 50^2) \times 1.1}{250}} = 14.6\,\text{mm}$$

A 15 mm thick base plate may be provided.
The slab base is directly connected to the column section using a full penetration butt weld.

Column Base Design Example 2

Design a slab base for a beam-column SC 250 to transfer a factored axial compression of 750 kN and a factored bending moment of 75 kNm. The grade of the steel is E250 and the grade of the concrete pedestal is M_{30}.

Eccentricity, $e = 75 / 750 = 0.1$ m

The length of the base plate is kept equal to or more than $6e$ so that the entire plate is subjected to downward pressure and no tension develops in the anchor bolts.

$\therefore$ The length of the base plate, $L = 6 \times 0.1 = 0.6$ m

The bearing strength of the concrete $= 0.6 f_{ck} = 0.6 \times 30 = 18$ MPa

It is assumed that the bearing pressure varies linearly below the base plate.

The maximum bearing pressure, $p_{max} = \dfrac{P}{BL}\left(1 + \dfrac{6e}{L}\right)$

$$\therefore \quad 18 = \frac{750 \times 10^3}{B \times 600}\left(1 + \frac{600}{600}\right)$$

or

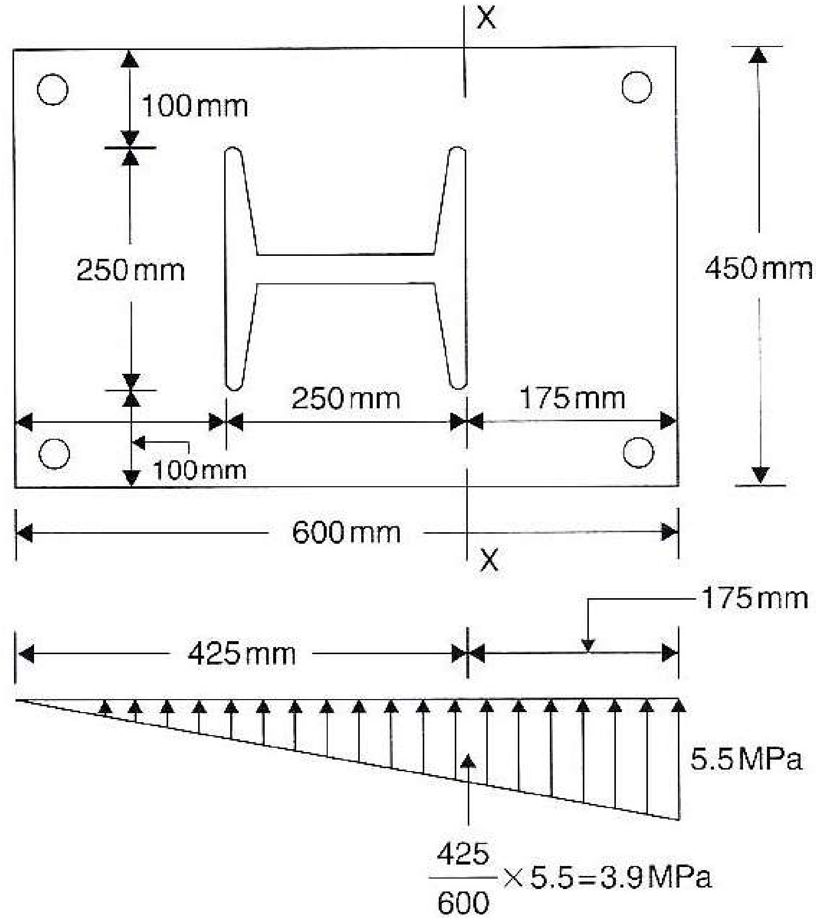
$$B = 139 \text{ mm}$$

The width of the base plate to be provided

= the width of the flange of SC 250 + the projections on either side

= 250 + 2 × 100 = 450 mm

Therefore, a rectangular base plate of 600 mm × 450 mm as shown in Figure 9.9 may be provided.



X
100 mm
250 mm
250 mm
175 mm
450 mm
100 mm
600 mm
X
175 mm
425 mm
5.5 MPa
$\frac{425}{600} \times 5.5 = 3.9$ MPa

For this base plate, $p_{max} = \dfrac{750 \times 10^3}{450 \times 600}\left(1 + \dfrac{600}{600}\right) = 5.5\,\text{MPa}$

The variation bearing pressure is shown in Figure 9.9.
The maximum bending moment in the base plate at Sec. X–X

$$= \frac{3.9 \times 175^2}{2} + \frac{1}{2} \times 175 \times (5.5 - 3.9) \times \frac{2}{3} \times 175 = 76{,}052\,\text{Nmm/mm width}$$

$$1.2 Z_e f_y / \gamma_{m0} = 1.2 \times \left(\frac{1 \times t^2}{6}\right) \times \frac{250}{1.1} = 76{,}052$$

or $\quad t = 41\,\text{mm}$

A base plate of 42 mm thick may be provided. The column section may be directly welded to the base plate by a full penetration butt weld. Four anchor bolts may be provided to keep the beam-column in position as shown in Figure 9.9.

Column Base Design Example 3

Design a gusseted base for the data in Example 9.4. The grade of the steel is E250 and the grade of the concrete pedestal is M_{30}.

The same size of base plate is used as in Example 9.4, i.e. 600 mm × 450 mm. Gusset plates are provided as shown in Figure 9.10. The column section and gusset plates are connected to the base plate by full penetration butt welds.

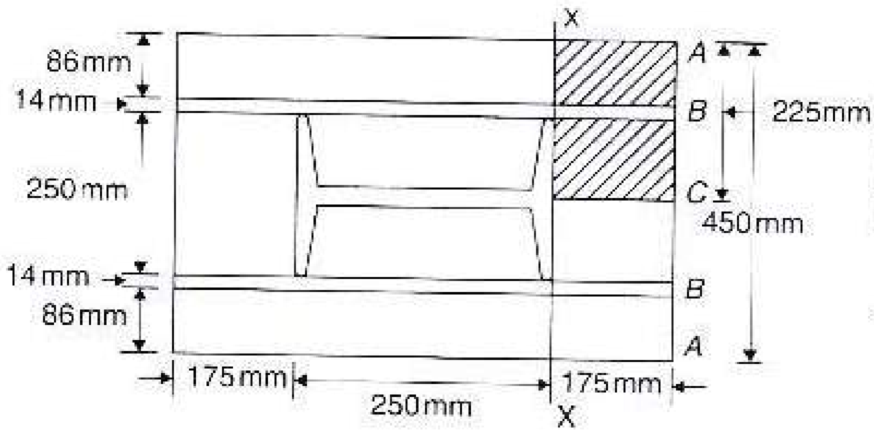
The vertical shear force in the gusset plate at Sec. X–X

= upward force acting on the hatched area as shown in Figure 9.10(a)

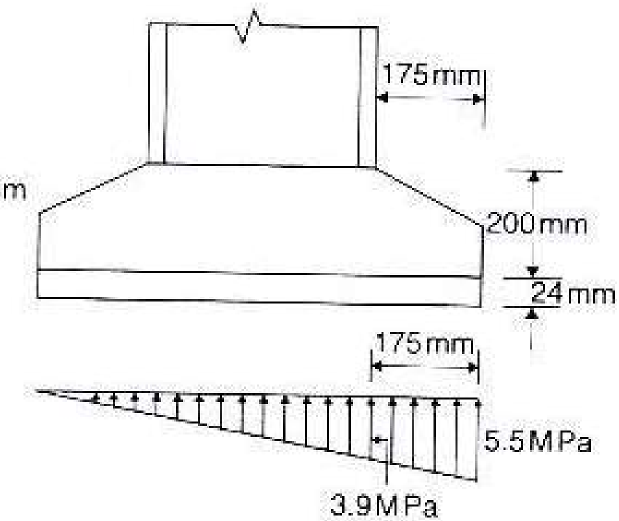
$$= \frac{5.5 + 3.9}{2} \times 175 \times 225 = 185\,\text{kN}$$

The bending moment in the gusset in the vertical plane at Sec. X-X

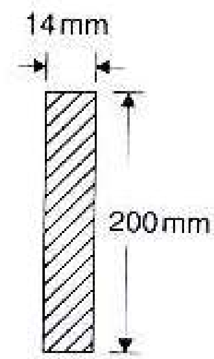
$$= (175 \times 225) \times 3.9 \times \frac{175}{2} + (175 \times 225) \times \frac{1}{2} \times (5.5 - 3.9) \times \frac{2}{3} \times 175 = 17\,\text{kNm}$$



(a)



(b)



(c) Vertical cross section of gusset plate

The shear capacity of a gusset plate $= \dfrac{A_v f_y}{\sqrt{3}\times 1.1} = \dfrac{200\times 14\times 250}{\sqrt{3}\times 1.1} = 367\,\text{kN}$

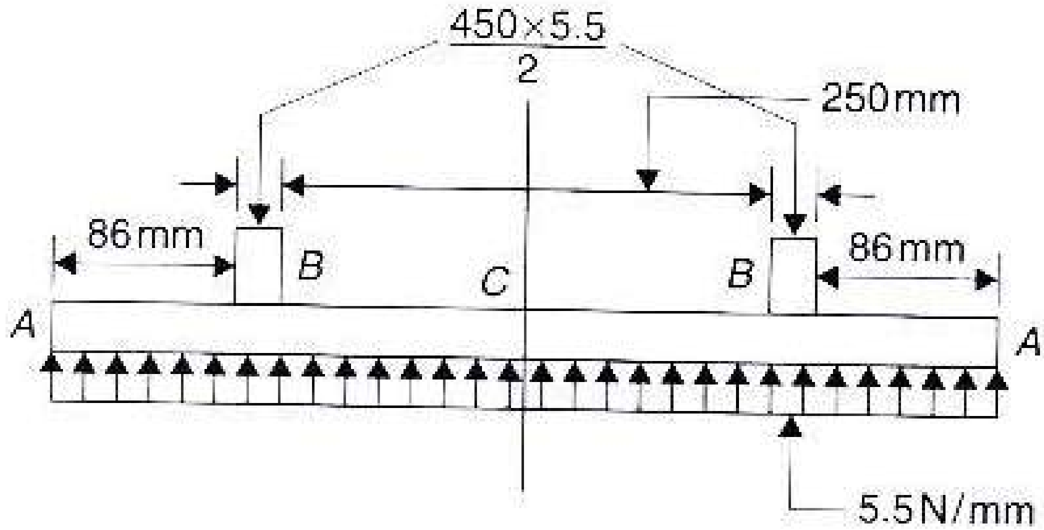
As the shear force in the gusset plate 185 kN is less than $0.6 \times 367 = 220\,\text{kN}$, the moment capacity of the gusset plate is not reduced due to the combined action of the shear force and the bending moment.

The vertical cross section of gusset plate is considered as semi-compact.

The moment capacity of the gusset plate $= \beta_b Z_p f_y / \gamma_{m0}$

$$= \frac{\left(\dfrac{14\times 200^2}{6}\right)}{\left(\dfrac{14\times 200^2}{4}\right)} \times \left(\frac{14\times 200^2}{4}\right) \times \frac{250}{1.1} = \left(\frac{14\times 200^2}{6}\right) \times \frac{250}{1.1} = 21\,\text{kNm} > 17\,\text{kNm}$$

Consider a 1 mm width of the base plate along A–A as shown in Figure 9.11.



The bending moment at B in the base plate $= \dfrac{5.5 \times 86^2}{2} = 20{,}339\,\text{Nmm}$

The bending moment at C in the base plate

$$= \frac{450 \times 5.5}{2}(125 + 7) - \frac{5.5 \times 225^2}{2} = 24{,}131\,\text{Nmm}$$

The moment capacity of the base plate per unit width $= 1.2 Z_e f_y / \gamma_{m0}$

$$= 1.2 \times \frac{1.0 \times t^2}{6} \times \frac{250}{1.1} = 45.5\,t^2$$

$$\therefore \quad 45.5\,t^2 = 24{,}131$$

or $\qquad t = 23\,\text{mm}$

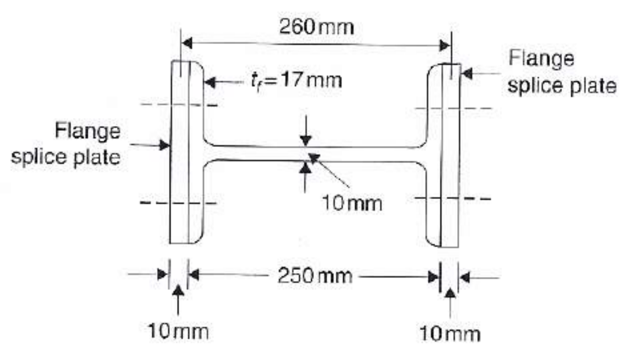
Hence, the thickness of the base plate may be 24 mm.

Column Splices Design Example 1

Design a splice for a beam-column using high strength bolts of the property class 8.8.
The factored axial force = 750 kN

Factored bending moment = 150 kNm
Factored shear force = 75 kN
The section of the column is SC 250. Assume that the ends of the column are milled and the connections are of bearing type. $f_y = 250$ MPa.

As the ends of the column are milled, the splice is designed for 50% of the axial load. Let the thickness of the splice plates on the flanges be 10 mm.



The compressive force in each flange splice plate due to axial load

$$= \frac{0.5 \times 750}{2} = 187.5\,\text{kN}$$

The compressive force in each splice plate due to bending moment

$$\frac{BM}{\text{Lever} - \text{arm}} = \frac{150}{0.26} = 577\,\text{kN}$$

The total compressive force in each flange splice plate = 187.5 + 577 = 764.5 kN

$$f_{cd} = f_y/\gamma_{m0} = 250/1.1 = 227\,\text{MPa}$$

The required area of cross-section of each splice plate $= \dfrac{764.5 \times 10^3}{227} = 3{,}368\,\text{mm}^2$

The width of the splice plate = Width of flange of SC 250 = 250 mm

The thickness of flange splice plate $= \dfrac{3{,}368}{250} = 13.5\,\text{mm}$

The thickness of the flange splice plate may be kept equal to or more than the thickness of the flange of the column section. Hence, 250 mm × 18 mm splice plates may be provided.

Using M16 bolts, $$V_{nsb} = \frac{800}{\sqrt{3}}\left(1 \times \frac{\pi \times 16^2}{4}\right) = 92,867\ \text{N}$$

$$V_{dsb} = 92,867/1.25 = 74,293\ \text{N}$$

k_b is minimum of $\dfrac{e}{3d_0}, \left(\dfrac{p}{3d_0} - 0.25\right), \dfrac{f_{ub}}{f_u}, 1.0$

Assuming $e = 30\ \text{mm}$, $p = 60\ \text{mm}$, $k_b = 0.55$

$$V_{nsb} = 2.5\ k_b\ d\ t\ f_u = 2.5 \times 0.55 \times 16 \times 17 \times 410 = 1,53,340\ \text{N}$$
$$V_{dpb} = 15,53,340/1.25 = 1,22,672\ \text{N}$$

The least design strength of the bolt = 74,293 N = 74 kN

The number of bolts needed $= \dfrac{764.5}{74} = 10.3$

Twelve bolts may be provided to connect each flange as shown in Figure 9.4(a).

To transfer the shear force by the splice, another set of splice plates are provided on the web. The required shearing area of these splice plates is given by

$$V_d = \frac{V_n}{\gamma_{m0}} \quad \text{where} \quad V_n = \frac{A_v f_y}{\sqrt{3}}$$

$$\therefore \quad A_v = \frac{75 \times 10^3 \times \sqrt{3} \times 1.15}{250} = 598\ \text{mm}^2$$

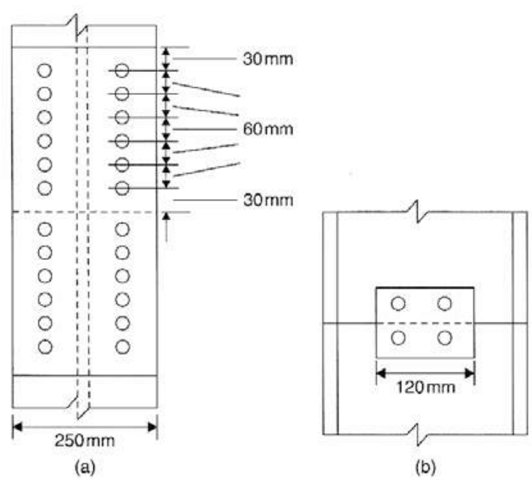
A 120 mm wide and 6mm thick plate may be provided on either side of web. Bolts connecting these plates and the web are, therefore, subjected to double shear.

$$V_{dsb} = 2 \times 74{,}293 = 1{,}48{,}586\,\text{N}$$
$$V_{nsb} = 2.5\; k_b\; d\; t\, f_u = 2.5 \times 0.55 \times 15 \times 10 \times 410 = 90{,}200\,\text{N}$$

$$V_{dsb} = \frac{90{,}200}{1.25} = 72\,\text{kN}$$

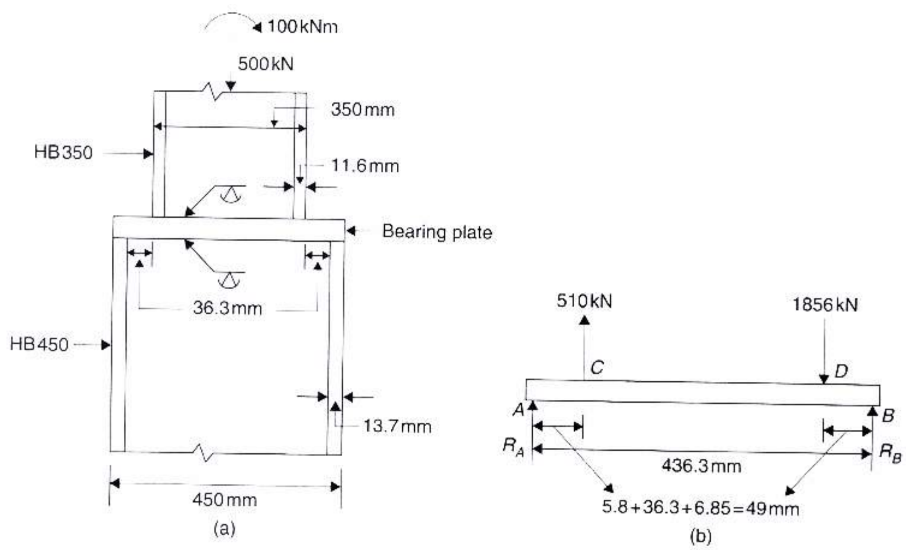
The number of bolts required $= \dfrac{75}{72} = 1.04$

Two bolts may be provided as shown in Figure 9.4(b).



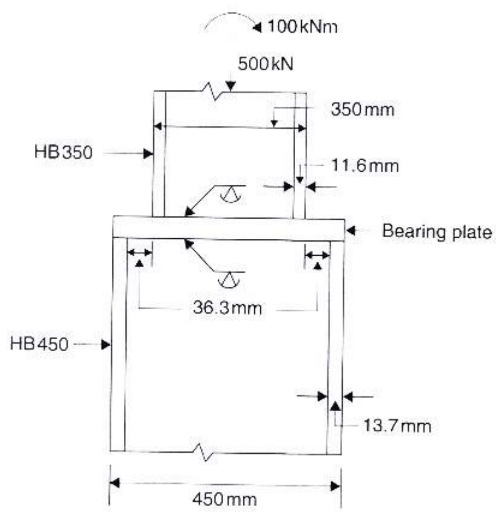
Column Splices Design Example 2

HB 350 @ 67.4 kg/m is to be spliced to HB 450 @ 87.2 kg/m using a butt weld. The factored axial force is 500 kN and the factored bending moment where the column is spliced is 100 kNm.

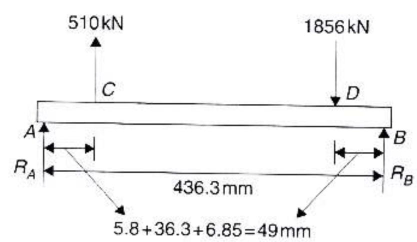


Since two different cross-sections of the column are to be spliced, a bearing plate of size of 250 mm × 450 mm is used shown in Figure 9.5(a). It is assumed that the axial load is resisted by the entire area of the cross-section of the column HB 350 @ 67.4 kg/m whereas only its flanges resist the bending moment.

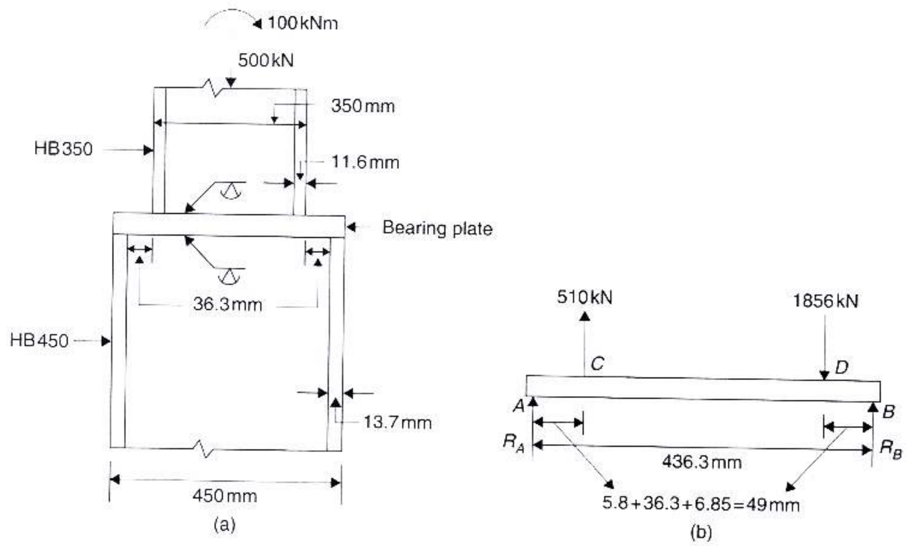
The stress in the column HB 350 due to the axial load $= \dfrac{P}{A} = \dfrac{500 \times 10^3}{85.91 \times 100} = 58\,\text{MPa}$



(a)



(b)



(P = axial load, A = sectional area of HB 350)

The stress in the flanges of HB 350 due to the bending moment

$$= \frac{M}{db_f t_f}$$

$$= \frac{100 \times 10^6}{338.4 \times 250 \times 11.6} = 102\,\text{MPa}$$

(M = bending moment, b_f = width of the flange, t_f = thickness of the flange, d = centre to centre distance between the flanges = 350 − 11.6 = 338.4 mm)

The resultant stress in the left flange = −58 + 102 = 44 MPa

The resultant stress in the right flange = 58 + 102 = 160 MPa

Considering a longitudinal strip of a 1 mm wide plate, the forces acting through the flanges of HB 350 are as follows (Figure 9.5(b))

Force at $C = 11.6 \times 1.0 \times 44 = 510\,\mathrm{N}$

Force at $D = 11.6 \times 1.0 \times 160 = 1{,}856\,\mathrm{N}$

The reactions from the flanges of HB 450, viz. R_A and R_B are calculated from the equilibrium equations.

$$\sum F_y = 0 \Rightarrow R_A + R_B + 510 - 1{,}856 = 0$$

$$\sum M_A = 0 \Rightarrow R_B \times 436.3 + 510 \times 49 - 1{,}856 \times 387.3 = 0$$

$$R_A = -244\,\mathrm{N} \text{ and } R_B = 1{,}590\,\mathrm{N}$$

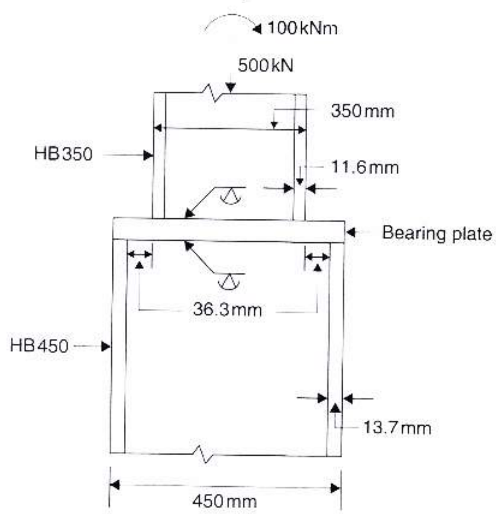
The maximum bending moment is at D, i.e., $M_D = 1{,}590 \times 49 = 77{,}910$ Nmm

$$\therefore \quad 1.2\, Z_e f_y / r_{m0} = M_D$$

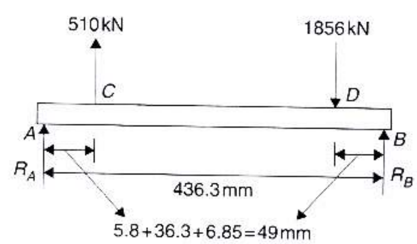
$$1.2 \times \frac{1 \times t^2}{6} \times \frac{250}{1.1} = 77{,}910$$

or $$t = 37.5 \text{ mm}$$

Therefore, a bearing plate of the thickness of 40 mm may be provided and the two parts of the column may be butt welded as shown in Figure 9.5(a).



(a)



(b)

DARBHANGA COLLEGE OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

Subject Code: 011620 Subject Name: Design of Steel Structures

Subject teacher: Ahsan Rabbani

1. **Unit mass of Steel is**
   a) 785 kg/m^3
   b) 450 kg/m^3
   c) 450 kg/cm^3
   d) 7850 kg/m^3

Answer: d

2. **Which of the following is a correct criterion to be considered while designing?**
   a) Structure should be aesthetically pleasing but structurally unsafe
   b) Structure should be cheap in cost even though it may be structurally unsafe
   c) Structure should be structurally safe but less durable
   d) Structure should be adequately safe, should have adequate serviceability

Answer: d

3. **The structure is statically indeterminate when**
   a) static equilibrium equations are insufficient for determining internal forces and reactions on that structure
   b) static equilibrium equations are sufficient for determining internal forces and reactions on that structure
   c) structure is economically viable
   d) structure is environment friendly

Answer: a

4. **Which of the following relation is correct?**
   a) Permissible Stress = Yield Stress x Factor of Safety
   b) Permissible Stress = Yield Stress / Factor of Safety
   c) Yield Stress = Permissible Stress / Factor of Safety
   d) Permissible Stress = Yield Stress – Factor of Safety

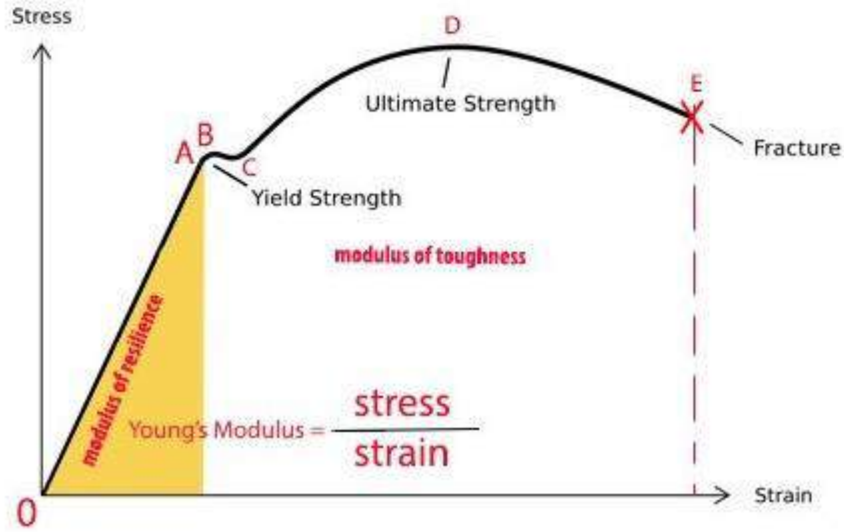
Answer: b

5. **Describe stress strain curve for the Mild Steel with neat sketch.**

Answer:

When steel is curved, it is important to keep the stress-strain curve ratio for mild steel in mind. Below is a stress-strain graph that reviews the properties of steel in detail.

If tensile force is applied to a steel bar, it will have some elongation. If the force is small enough, the ratio of the stress and strain will remain proportional. This can be seen in the graph as a straight line between zero and point A – also called the **limit of proportionality**. If the force is greater, the material will experience elastic deformation, but the ratio of stress and strain will not be proportional. This is between points A and B, known as the elastic limit.



Beyond the elastic limit, the mild steel will experience plastic deformation. This starts the yield point – or the rolling point – which is point B, or the upper yield point. As seen in the graph, from this point on the correlation between the stress and strain is no longer on a straight trajectory. It curves from point C (lower yield point), to D (maximum ultimate stress), ending at E (fracture stress).

Now, we'll look at each individual measure on the graph above and explain how each is derived.

- **Stress**: If an applied force causes a change in the dimension of the material, then the material is in the state of stress. If we divide the applied force (F) by the cross-sectional area (A), we get the stress.

The symbol of stress is σ (Greek letter sigma). For tensile (+) and compressive (-) forces. The standard international unit of stress is the pascal (Pa), where 1 Pa = 1 N/m^2. The formula to derive the stress number is $\sigma = F/A$.

For tensile and compressive forces, the area taken is perpendicular to the applied force. For sheer force, the area is taken parallel to the applied force. The symbol for shear stress is tau (τ).

- **Strain:** Strain is the change in the dimension ($L-L_0$) with respect to the original. It is denoted by the symbol epsilon (ε). The formula is $\varepsilon = (L-L_0) / L_0$. For a shear force, strain is expressed by γ (gamma)

- **Elasticity:** Elasticity is the property of the material which enables the material to return to its original form after the external force is removed.

- **Plasticity:** This is a property that allows the material to remain deformed without fracture even after the force is removed.

The definitions below are important for understanding the Stress-Strain interactions as seen in the graph.

- **Hooke's Law:** Within the proportional limit (straight line between zero and A), strain is proportionate to stress.

- **Young's modulus of elasticity:** Within the proportional limit, stress = E × strain. E is a proportionality constant known as the modulus of elasticity or Young's modulus of elasticity. Young's modulusis a measure of the ability of a material to withstand changes in length when under lengthwise tension or compression. E has the same unit as the unit of stress because the strain is dimensionless. The formula is $E = \sigma / \varepsilon$ Pa.

- **Modulus of Resilience**: The area under the curve which is marked by the yellow area. It is the energy absorbed per volume unit up to the elastic limit. The formula for the modulus of resilience is $1/2 \times \sigma \times \varepsilon = 0.5 \times (FL/AE)$.

- **Modulus of toughness**: This is the area of the whole curve (point zero to E). Energy absorbed at unit volume up to breaking point.

Chicago Metal Rolled Products sets the industry standard for adherence to the stress strain curve for mild steel and other materials.

6. **What are the Advantages and disadvantages of steel as structural materials?**

Answer:
Steel is one of the most generally utilized materials of construction time. Without the use of steel, the structure doesn't make a solid while seismic tremors like earthquakes etc. happen. Steel structures are susceptible to various ecological conditions. There are a few properties wherein solid structures are preferred over steel and the utilization of steel is consistently expanding everywhere throughout the world in development projects and also in civil engineering-related fields. According to 'Lorraine Farrelly', before the utilization of steel in development building, became a common practice, the weight of the structure material and the forces of gravity and pressure defined the endurance, chance of stability in structure, and its architectural possibilities. Each steel structure has some advantages as well as disadvantages. And now we are going to elaborate on the complete description regarding steel here.

ADVANTAGES OF STEEL BUILDINGS

a. Steel is moderately cheap when compared with other structure materials
b. Steel structures are highly fire-resistant when contrasted to a wooden structure as wood is a combustible material and less fire-resistant when contrasted with RCC structure.
c. One of the advantages of using a steel structure in development is the ability of steel to span greater distances with steel ceiling joists. This enables architects to grow their choices, enabling them to make new/huge space utilizing steel items that simply weren't accessible with different materials.
d. Steel can be easily & effectively manufactured and delivered greatly. Steel structures can be delivered off-site at shop floors and after that gathered nearby. This spares time and increases the efficiency of the general development process.
e. Steel structures can withstand outside weights, for example, earthquakes, thunder storms, and cyclones. A well-fabricated steel structure can last more than30 years whenever looked after well.

f. Flexibility is one of the great advantages of steel structure, which means that it tends to be planned according to the design requirements. This plans a steel structure so that it can withstand heavy winds or earthquakes, especially in the case of the bridges or tall towers.
g. Because of simple-to-make portions of a steel structure, it is hassle-free to install and assemble them on-site, and furthermore, there is no need of estimating and cutting of parts nearby.
h. Some of the common advantages of using steel buildings are Design, Strength and Durability, Light in Weight, Easy Installation and Speed in Construction, Versatile, Flexibility, Ductility, Easy Fabrication in Different Sizes, Fire Resistance, Pest and Insect Resistant, Moisture and Weather Resistance, Adaptability, Cost-effective, Environment Friendly, Energy Efficiency, Improved Construction Quality, Temporary Structures, Safe and Resistant and Risk Index.

DISADVANTAGES OF STEEL BUILDINGS

a. Buckling is an issue with steel structures. As the length of the steel segment builds the chances of buckling also increases.
b. Steel is available only at the steel plants where it is produced and should be transported for long distances to the site of construction, not at all like concrete or different materials that might be accessible right at the site of development.
c. Due to the activity of rust in steel, costly paints are required to re-establish from time to time. So that resistance against serious conditions increments.
d. Despite the fact that steel is a flexible material, it is difficult to make field corrections if one or more components do not fit appropriately. Large portions of the metal structure makes perform adhere to strict quality assurance procedure guarantee all pieces of a structure fit accurately. But in actual it is not possible. One can't form it or cut it in the ideal shape on-site once it is fabricated.
e. Steel can't mold in any path you required. It must be utilized in structures in which areas initially exist.
f. Steel is agood conductor of heat, touches off materials in contact and often causes fires, which quickly spread to different segments of a structure. Hence, steel structures may require extra fireproofing treatment.
g. If steel loses its great property of ductility and then there are more chances to increase the fractures.
h. Some of the common disadvantages of using steel buildings are High Maintenance & Capital Cost, Susceptibility to Buckling, Fatigue and Fracture, Fireproof Treatment, Fire Damage and Fabrication Error.

7. **Describe the various types of loads and Load Combinations as per IS code.**

Answer:

Clause 3.2 of IS 800:2007 specifies the various loads and forces that has to be considered while performing the design of steel structures. As per Cl. 3.2.1 of IS 800:2007, for the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable, with partial safety factors and combinations (Cl. 5.3.3 of IS 800:2007). (a) Dead loads; (b) Imposed loads (live load, crane load, snow load, dust load, wave load, earth pressures, etc); (c) Wind loads; (d) Earthquake loads; (e) Erection loads; (f) Accidental loads such as those due to blast, impact of vehicles, etc; and (g) Secondary ffects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components,eccentric connections,rigidity of joints differing from design assumptions.

1. Dead loads (Cl. 3.2.1.1 of IS 800:2007)

Dead loads should be assumed in design as specified in IS 875 (Part 1).

2. Imposed Loads (Cl. 3.2.1.2 of IS 800:2007)

IS 800:2007 specifies in Cl.3.2.1.2 that imposed loads for different types of occupancy and function of structures shall be taken as recommended in IS 875 (Part 2). Imposed loads arising from equipment, such as cranes and machines should be assumed in design as per manufacturers/suppliers data (Cl. 3.5.4 of IS 800:2007). Snow load shall be taken as per IS 875 (Part 4).

3. Wind loads (Cl. 3.2.1.3 of IS 800:2007)

Wind loads on structures shall be taken as per the recommendations of IS 875 (Part 3).

4. Earthquake loads (Cl. 3.2.1.4 of IS 800:2007)

Earthquake loads shall be assumed as per the recommendations of IS 1893 (Part 1).

5. Erection Loads (Cl. 3.3 of IS 800:2007)

All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment, including all loads due to operation of such equipment shall be considered as erection loads. The structure as a whole and all parts of the structure in conjunction with the temporary bracings shall be capable of sustaining these loads during erection.

6. Temperature Effects (Cl. 3.4 of IS 800:2007)

Expansion and contraction due to changes in temperature of the members and elements of a structure shall be considered and adequate provision made for such effect. The co-efficient of thermal expansion for steel is as given in Cl. 2.2.4.l of IS 800:2007.

7. Load Combinations

All structures must be designed to support their own weight along with any superimposed forces, such as the dead loads from other materials, live loads, wind pressures, seismic forces, snow and ice loads, and earth pressures (if buried underground). Because various loads may act on a structure simultaneously, load combinations should be evaluated to determine the most severe conditions for design (worst case scenario). These load combinations vary from one document to another, depending upon the jurisdiction. There are a set of combinations for the allowable stress design and another set that incorporates load factors for strength design.

Load combinations for design purposes shall be those that produce maximum forces and effects and consequently maximum stresses and deformations. The following combination of loads with appropriate partial safety factors as given in Table 4 of IS 800:2007 may be considered. The table is reproduced here as Table 2 for ready reference. a) Dead load + imposed load, b) Dead load + imposed load + wind or earthquake load, c) Dead load + wind or earthquake load, and d) Dead load+ erection load. The effect of wind load and earthquake loads shall not be considered to act simultaneously. The load combinations are outlined in detail in Cl. 3.5 of IS 800:2007.

Table 4 Partial Safety Factors for Loads, γ_f, for Limit States
(*Clauses* 3.5.1 *and* 5.3.3)

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1)]		WL/EL	AL	DL	LL[1)]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+ WL/EL	1.2 1.2	1.2 1.2	1.05 0.53	0.6 1.2	—	1.0	0.8	0.8	0.8
DL+WL/EL	1.5 (0.9)[2)]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2)]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1)] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.
[2)] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.
Abbreviations:
DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.
NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

8. **Explain limit state of serviceability and limit state of collapse briefly.**

Answer:

The most important limit states which are considered in design as are follows:

(i) Limit state of collapse.

(ii) Limit state of serviceability

Limit State of Collapse
This limit state is also called as strength limit state as it corresponds to the maximum load carrying capacity i.e., the safety requirements of the structure. The limit state of collapse is assessed from collapse of the whole or part of the structure. As per this limit state, the resistance to bending, shear, torsion and axial loads at every section shall not be less than that produced by the most unfavorable combination of loads on that structure. The following limit states of collapse are considered in design:

(i) Limit state of collapse in flexure (bending)

(ii) Limit state of collapse in compression

(iii) Limit state of collapse in shear

(iv) Limit state of collapse in torsion.

Limit State of Serviceability

A structure is of no use if it is not serviceable. Thus, this limit state is introduced to prevent excessive deflection and cracking. It ensure the satisfactory performance of the structure at working loads. It is estimated on the basis of elastic theory or working stress method because deformation is of significance under working load and not at collapse. Limit state of serviceability of following limit states:

(i) Limit state of deflection

(ii) Limit state of cracking

(iii) Limit state of vibration

The structure should be designed which considering all the appropriate limit state of safety and serviceability and on the basis of most critical limit state and then checked for all other limit states.

9. **What are the factors to be considered in mechanical properties of structural steel?**

Answer:

Steel derives its mechanical properties from a combination of chemical composition, heat treatment and manufacturing processes. While the major constituent of steel is iron, the addition of very small quantities of other elements can have a marked effect upon the properties of the steel. The strength of steel can be increased by the addition of alloys such as manganese, niobium and vanadium. However, these alloy additions can also adversely affect other properties, such as ductility, toughness and weldability .

Minimizing the sulphur level can enhance ductility , and toughness can be improved by the addition of nickel. The chemical composition for each steel specification is therefore carefully balanced and tested during its production to ensure that the appropriate properties are achieved.

The alloying elements also produce a different response when the material is subjected to heat treatments involving cooling at a prescribed rate from a particular peak temperature. The manufacturing process may involve combinations of heat treatment and mechanical working that are of critical importance to the performance of the steel.

Mechanical working takes place as the steel is being rolled or formed. The more steel is rolled, the stronger it becomes. This effect is apparent in the material standards, which tend to specify reducing levels of yield strength with increasing material thickness.

The effect of heat treatment is best explained by reference to the various production process routes that can be used in steel manufacturing, the principal ones being:

- As-rolled steel
- Normalized steel
- Normalized-rolled steel
- Thermomechanically rolled (TMR) steel
- Quenched and tempered (Q&T) steel.

Steel cools as it is rolled, with a typical rolling finish temperature of around 750°C. Steel that is then allowed to cool naturally is termed 'as-rolled' material. Normalizing takes place when as-rolled material is heated back up to approximately 900°C, and held at that temperature for a specific time, before being allowed to cool naturally. This process refines the grain size and improves the mechanical properties, specifically toughness. Normalized-rolled is a process where the temperature is above 900°C after rolling is completed. This has a similar effect on the properties as normalizing, but it eliminates the extra process of reheating the material. Normalized and normalized-rolled steels have an 'N' designation.

The use of high tensile steel can reduce the volume of steel needed but the steel needs to be tough at operating temperatures, and it should also exhibit sufficient ductility to withstand any ductile crack propagation. Therefore, higher strength steels require improved toughness and ductility, which can be achieved only with low carbon clean steels and by maximizing grain refinement. The implementation of the thermomechanical rolling process (TMR) is an efficient way to achieve this.

Thermomechanically rolled steel utilises a particular chemistry of the steel to permit a lower rolling finish temperature of around 700°C. Greater force is required to roll the steel at these lower temperatures, and the properties are retained unless reheated above 650°C. Thermomechanically rolled steel has an 'M' designation.

The process for Quenched and Tempered steel starts with a normalized material at 900°C. It is rapidly cooled or 'quenched' to produce steel with high strength and hardness, but low toughness. The toughness is restored by reheating it to 600°C, maintaining the temperature for a specific time, and then allowing it to cool naturally (Tempering). Quenched and tempered steels have a 'Q' designation.

Quenching involves cooling a product rapidly by immersion directly into water or oil. It is frequently used in conjunction with tempering which is a second stage heat treatment to temperatures below the austenitizing range. The effect of tempering is to soften previously hardened structures and make them tougher and more ductile.

10. **What are special features of limit state design method compare to other methods of design of steel structures?**

Answer:

Limit state design has advancement over the traditional design philosophies. It considers the safety at the ultimate load and serviceability at the working load, sort of extension of the WSM and ULM.

“Limit state is the state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety or serviceability.”

Unlike WSM which based calculations on service load conditions alone, and unlike ULM, which based calculations on ultimate load conditions alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads.

The LSM philosophy uses a multiple safety factor format which attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads, by considering all possible ‘Limit State’.

A limit state is a state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety of serviceability i.e. it either collapses or becomes unserviceable.There are two types of limit states:

Ultimate limit states (limit states of collapse):- which deal with strength, overturning, sliding, buckling, fatigue fracture etc.

Serviceability limit states: – which deals with discomfort to occupancy and/ or malfunction, caused by excessive deflection, crack width, vibration leakage etc., and also loss of durability etc.

11. **Which of the following is advantage of HSFG bolts over bearing type bolts?**
    a) joints are not rigid
    b) bolts are subjected to shearing and bearing stresses
    c) high strength fatigue
    d) low static strength

Answer: c

12. **Tacking fasteners are used when**
    a) minimum distance between centre of two adjacent fasteners is exceeded
    b) maximum distance between centre of two adjacent fasteners is exceeded
    c) maximum distance between centre of two adjacent fasteners is not exceeded
    d) for aesthetic appearance

Answer: b

13. **Strength of bolt is**
    a) minimum of shear strength and bearing capacity of bolt
    b) maximum of shear strength and bearing capacity of bolt
    c) shear strength of bolt
    d) bearing capacity of bolt

Answer: a

14. **The types of welded joints does not depend on**
    a) size of members connected at joint
    b) type of loading
    c) area available for welding
    d) size of weld

Answer: d

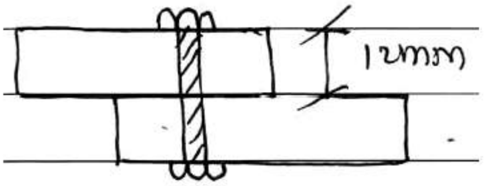
15. **The design nominal strength of fillet weld is given by**
    a) f_u
    b) $\sqrt{3}\, f_u$
    c) $f_u/\sqrt{3}$
    d) $f_u/(1.25 \times \sqrt{3})$

Answer: c

16. **Which of the following is not true regarding effective throat thickness of weld?**
   a) Effective throat thickness should not be less than 3mm
   b) It should not exceed 0.7t or 1t, where t is thickness of thinner plate of elements being welded
   c) Effective throat thickness = K x size of weld, where K is a constant
   d) Effective throat thickness = K x (size of weld)^2 , where K is a constant

Answer: d

17. **A lap joint consists of two plates 200 x 12 mm connected by means of 20 mm diameter bolts of grade 4.6. All bolts are in one line. Calculate strength of single bolt and no. of bolts to be provided in the joint.**



Solution:
Given

Nominal diameter of bolt = 20 mm

$\therefore$ Net area of bolt at thread (A_{nb}) $= 0.78 \times \frac{\pi}{4} \times d^2$

$= 0.78 \times \frac{\pi}{4} \times 20^2$

$A_{nb} = 245.04 \text{ mm}^2$

For fe 410 grade steel plate (assumed)
Ultimate stress for plate $f_y = 410 \text{ N/mm}^2$
For 4.6 grade of bolt
Ultimate stress for bolt (f_{ub}) $= 4 \times 100 = 400 \text{ N/mm}^2$
Yield stress for bolt (f_{yb}) $= 400 \times 0.6 = 240 \text{ N/mm}^2$

Now find design shearing strength of bolt (Vdsb)
$\therefore$ we know that

$$\therefore \quad Vdsb = \frac{fub}{\sqrt{3} \times Y_{mb}} [n_n \times A_{nb} + n_s + A_{ns}]$$

Here number of shear plane with threat intercepting the shear plane $n_n = 1$
Number of shear plane without thread intercepting the shear plane $n_s = 0$

$$\therefore \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 243.04 + 0]$$

γ_{mb} = partial factor of safety for bolt material = 1.25

$V_{dsb} = 45.27 \times 10^3 \text{N}$

Now find design bearing strength of bolt (V_{dsb})

$$V_{dph} = 25 \times kb \times (d \times t) \times \frac{fy}{\gamma_{mb}}$$

Here coefficient k_b is minimum of

(1) $\left[\frac{e}{3dh}, \frac{p}{3dh} - 0.25, \frac{f_{ub}}{f_u}, 1\right]$

(a) Diameter of hole (dh) = Nominal diameter + 2
= 20 + 2 = 22 mm

(b) End distance (e) = 2d = 2×20 = 40 mm

(c) Pitch (p) = 2.5 d
= 2.5×20 = 50 mm

(i) $\frac{e}{3dh} = \frac{40}{3 \times 22} = 0.606$

(ii) $\frac{p}{3dh} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{fub}{fy} = \frac{400}{410} = 0.975$ &

(iv) 1

Hence Kb = 0.507 mm ... take minimum value

Now find design bearing strength of bolt (V_{dpb})

$$= 2.5 \times Kb \times (d \times t) \times \frac{f_y}{r_{mb}}$$

$$= 2.5 \times 0.507 \times (20 \times 12) \times \frac{410}{1.25}$$

$$V_{dph} = 99.77 \times 10^3 \text{ N}$$

Now find bolt value i.e. strength of bolt

$\therefore$ Bolt value = minimum strength between shearing & bearing strength of bolt i.e. minimum between V_{dsb} & V_{dpb}

$= 45.27 \times 10^3$ N

Full strength of member $= 0.9 \times \dfrac{fu}{r_m} \times$ Area of plan

$= \dfrac{0.9 \times 410}{1.25}(250 - 1 \times 22) \times 12$

$= 630.54 \times 10^3$ N

Full strength of plan

$\therefore$ No of bolts $= \dfrac{\text{full strength of plate}}{\text{Bolt value}}$

$= \dfrac{630.54 \times 10^3}{45.27 \times 10^3}$

$= 13.92$ Say 14 Nos

18. **Design the Lap joint for the plates of sizes 100 × 16 mm and 100 × 10 mm thick connected so as to transmit a factored load of 100 kN using single row of 16 mm diameter bolts of grade 4.6 and plate of 410 grade.**

Solution: Given

$f_u = 410$ N/mm^2 $f_{ub} = 400$ N/mm^2

d = 16 mm do = 18mm

$\gamma_{mb} = 1.25$ $P_u = 100$ kN

Strength of bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$A_{nb} = 0.78 \times \dfrac{\pi}{4} \times d^2$

$= 0.78 \times \dfrac{\pi}{4} \times 16^2 = 156.82$

Design

strength of bolt in shear

i.e. $Vdsb = \dfrac{Fub}{\sqrt{3}} \dfrac{(n_n\, Anb + n_s\, A_{sb})}{rmb}$

$= \dfrac{400}{\sqrt{3}} \dfrac{1 \times 157}{1.25} = 29.006 \times 10^3$ N

$\therefore$ Vdsh = 29 N

$\therefore$ No. of bolts required $= \dfrac{Pu}{Vdsb} = \dfrac{100}{29}$

$= 3.4 \cong 4$ No.

No. of bolts required = 4 no.

Arranging bolts in single rows

Equating tensile capacity per pitch length

$Tdn = 0.9 \dfrac{F_u}{rm_1}(P - do) \cdot t$

$$29 \times 10^3 = 0.9 \times \frac{410}{125}(P-18) \times 10$$

$$P = \left(\frac{29 \times 10^3 \times 125}{0.9 \times 410 \times 10}\right) + 18$$

$$= 27.82 < 2.5 \times d = 2.5 \times 16$$

$$= 40$$

$\because$ Provide pitch P = 40 mm

and edge distance = $17 \times$ do [for rough edge]

$= 17 \times 18$

$= 30.6 \cong 30$

kb is smallest of

i) $\frac{e}{3do} = \frac{30}{3 \times 18} = 0.56$

ii) $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25$

$= 0.49$

iii) $\frac{Fub}{Fu} = \frac{400}{410} = 0975$

(iv) 1

Min. Value = 0.49

Hence Kb = 0.49

$\therefore$ Design bearing strength

$$Vdsh = \frac{Vnpb}{rmb} = \frac{2.5 \times kb.d.t\ F_u}{rmb}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25}$$

$$= 64288\ N = 64.29\ kN$$

Vdsb = 64.29 kN > 29 kN

$\because$ Ok no revision is required

Check for the strength of plate

$$Tdn = \frac{0.9\, An \cdot Fu}{rm} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

$$= 188.93\ kN > 110kN$$

safe

Provide 4–16 mm ϕ bolts of 40mm

Pitch with edge distance of 30 mm as shown in fig.

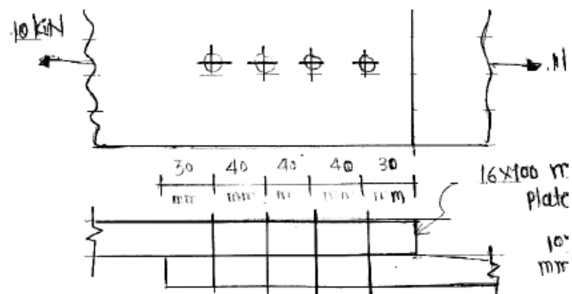


Fig: showing connection of Lap joint

19. **State types of bolted joints and types of failure in case of bolted joints.**

Answer:

i) Types of bolted joint

(a) Lap Joint
- Single line bolting
- Double line bolting

(b) Butt Joint
- Single cover Butt joint
- Double cover Butt joint

ii) Failure of Bolted joint

(a) Failure of plate
- By tearing of plate (shear failure)
- By tensile failure of plate
- By bearing of plate

(b) Failure of bolt
- By shear failure of bolt
- By tensile failure of bolt
- By bearing failure of bolt

20. **State various advantages of welded joints and disadvantages of bolted joints.**

Advantages of Welded Joints

1) The welded structures are usually lighter than riveted structures. This is due to the reason, that in welding, gussets or other connecting components are not used.
2) The welded joints provide maximum efficiency (may be 100%) which is not possible in case of riveted joint.
3) Alterations and additions can be easily made in the existing structures.
4) As the welded structure is smooth in appearance, therefore it looks pleasing.
5) In welded connections, the tension members are not weakened as in the case of riveted joints.
6) A welded joint has a great strength. Often a welded joint has the strength of the parent metal itself.
7) Sometimes, the members are of such a shape (i.e. circular steel pipes) that they afford difficulty for riveting. But they can be easily welded.
8) The welding provides very rigid joints. This is in line with the modern trend of providing rigid frames.
9) It is possible to weld any part of a structure at any point. But riveting requires enough clearance.
10) The process of welding takes less time than the riveting.

Disadvantages of bolted joints :

1) Due to holes made in members to be connected, tensile strength of the members is reduced.
2) Rigidity of joint is affected due to loose fit.
3) Deflection may increase due to affected Rigidity of joint
4) Nuts are likely to loose due to moving load vibration.
5) Bolted structures are heavier than welded structure due to use of connecting angles.
6) Circular section can not be bolted.
7) It is not possible to get 100% efficiency in case of bolted connection
8) Problem may arise in case of mismatching of holes.

21. **List the values of partial safety factor for material strength in case of resistance by yielding, buckling and ultimate stress in bolted connection.**

Answer:

	Descriptions	Partial safety Factor
1	Resistance governed by yielding r_{mo}	1.10
2	Resistance of member to buckling r_{mo}	1.10
3	Resistance governed by ultimate stress r_{m1}	1.25
4	Bolted connection in friction and Bearing r_{mf} and r_{mb}	1.25 [shop and field fabrication]

22. **Explain what do you mean by shear lag?**

Answer:

While transferring the tensile force from gussel plate to tension member through one leg by bolts or welds, the connected leg of section (such as angle, channel) may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus one port behind the other is called as shear lag.

The tearing strength of an angel section connected through one leg is affected by shear lag also. Thus, the design strength, Td_n governed by tearing at net section is given by

$$Td_n = 0.9\frac{Anc\,fu}{vm_j} + \beta\frac{Ago\,fy}{vm_o}$$

Where $\beta = 1.4 = 0.076\frac{w}{t}\times\frac{fy}{fu}\times\frac{bs}{Lc}$

b_3 = Shear log width as shown in fig

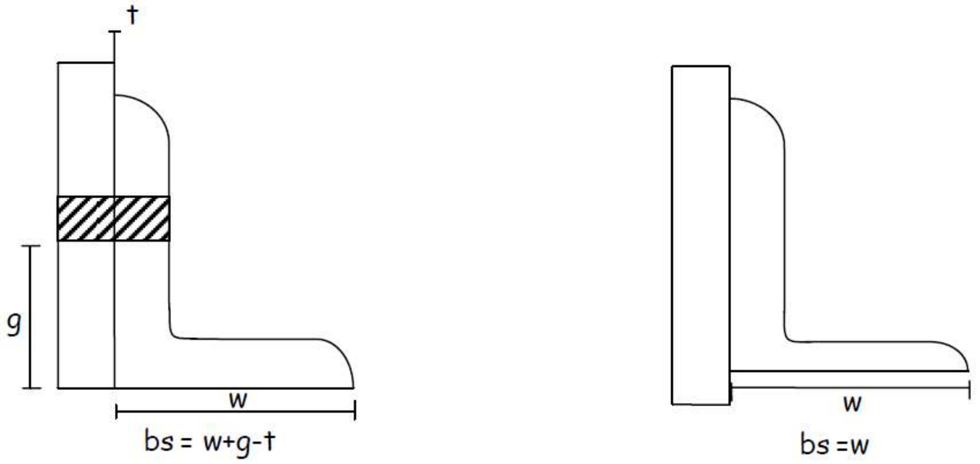


Fig: Shear leg width

23. **A discontinuous compression member consists of 2 ISA 90 × 90 × 10 mm connected back to back on opposite sides of 12 mm thick gusset plate and connected by welding. The length of strut is 3 m. It is welded on either side. Calculate design compressive strength of strut.**
For ISA 90 × 90 × 10, Cxx = Cyy = 25.9 mm Ixx = Iyy = 126.7 × 104 mm4, rzz = 27.3 mm values of f_{cd} are

KL/r	**90**	**100**	**110**	**120**
f_{cd} (N/mm^2)	**121**	**107**	**94.6**	**83.7**

Solution:

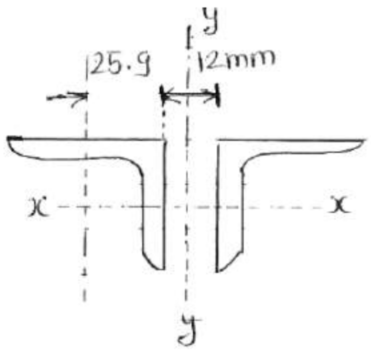
(i) r_{zz} = 27.3 mm (Due to symmetry @ zz axis)

(ii) $I_{yy} = 2[Iy + A \cdot h^2]$
$= 2[126.7 \times 10^4 + 1703\ (25.9 + 12/2)^2]$

(A is calculated by calculating Area of both leg separately and then adding them)
$\therefore I_{yy} = 5999979\ mm^4$

(iii) $\therefore r_{yy} = \sqrt{\dfrac{I_{yy}}{A_g}} = \sqrt{\dfrac{5999979}{2 \times 1703}} = 41.97$ mm

r_{min} = minimum of r_{zz} and r_{yy}
r_{min} = 27.3 mm



(iv) For discontinuous double angle, effective length
KL = 0.85L = 0.85 × 3 = 2.10 m = 2100mm

S.R. = $\dfrac{KL}{r_{min}} = \dfrac{2100}{27.3}$ =76.92

KL/r (SR)	fed
70	152
80	136

Hence,

$$fcd = fcd_1 - \frac{fcd_1 - fcd_2}{SR_2 - SR_1}$$

$$fcd = fs_2 - \frac{152-136}{80-70}(76.92-70)$$

$$fcd = 140.928 \text{ N/mm}^2$$

(v) Design compressive Strength

$Pd = fcd \times Ag$

$Pd = 140.928 \times (2 \times 1703)$

$Pd = 480 \times 10^3$ N

$Pd = 480$ kN

24. **Check whether ISMB250@37.4 kg/m is suitable or not as a simply supported beam over an effective span of 6 m. The compression flange of beam is laterally supported throughout the span. It carries udl of 15 kN/m (including self wt.). Properties of ISMB 250 are bf = 125 mm, tf = 12.5 mm, tw = 6.9 mm, Ixx = 5131.6×104 mm4, Zxx = 410×103 mm3, r1 = 13.0 mm, Z_{px} = 465.71 × 10^3 mm^3, y_{m0} = 1.1, β_b = 1 and fy = 250 MPa.**

Solution:

(i) Loads and factored BMS

$w = 15$ kN/m

Factored udl, $wd = 15 \times 1.5 = 22.5$ kN/m

Factored BM, $Md = \frac{wd.le^2}{8} = \frac{22.5 \times 6^2}{8}$

$= 101.25$ kN/m

Factored S.F. $Vd = \frac{wd.le}{2} = \frac{22.5 \times 6}{2} = 67.5$kN

(ii) Plastic modulus of section required

$Z_p \text{ reqd.} = \frac{Md.\gamma_{mo}}{fy} = \frac{101.25 \times 10^6 \times 1.1}{250}$

$= 445.5 \times 10^3 \text{ mm}^3$

Z_p reqd. $<$ Z_P avil $(= 465.71 \times 10^3 \text{ mm}^3)$

(iii) Classification of beam section

$d = h - 2(f_t + \gamma_1) = 250 - 2(12.5 + 13)$

$= 199$ mm

$$\frac{bh}{tf} = \frac{\frac{125}{2}}{12.5} = 5.0 < 9.4$$

$$\frac{d}{tw} = \frac{199}{6.9} = 28.84 < 67$$

As $\frac{bh}{tf} < 9.4$ and $\frac{d}{tw} < 67$ $\therefore$ Section classification is plastic

(iv) Check for shear

$$Vdr = \frac{f_y \times tw \times h}{\gamma_{mo}\sqrt{3}} \quad OR \quad 0.525 \quad fy.tw.h$$

$$= \frac{250 \times 6.9 \times 250}{1.1 \times \sqrt{3}} = 226348 \text{ N}$$

$$= 226.35 \text{ KN} > Vd \ (=67.5\text{kN})$$

$$\text{Also, } \frac{Vd}{Vdr} = \frac{67.5}{226.35} = 0.298 < 0.6$$

$\therefore$ Check for shear is satisfied.

(v) Check for deflection

$$\delta_{allowable} = \frac{L}{300}$$

$$= \frac{6000}{300}$$

$$= 20 \text{ mm}$$

$$d_{max} = \frac{5}{384} \frac{wL^4}{FI}$$

$$= \frac{5}{384} \times \frac{15 \times 6000^4}{2 \times 10^5 \times 5131.6 \times 10^4}$$

As $\delta_{max} > \delta_{allowable}$

$\therefore$ Deflection check is not O.K.

Hence, ISMB 250 is not a suitable section for given loading and span

25. **State types of bolted joints and types of failure in case of bolted joints.**

Answer:

i) Types of bolted joints

(a) Lap Joint

- Single line bolting
- Double line bolting

(b) Butt Joint

- Single cover Butt joint
- Double cover Butt joint

ii) Failure of Bolted joint

(a) Failure of plate

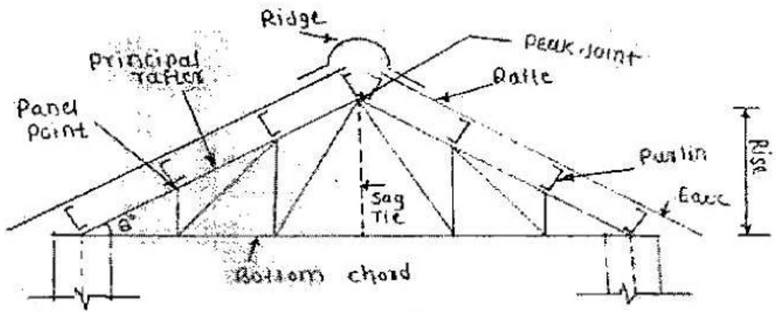
- By tearing of plate (shear failure)
- By tensile failure of plate
- By bearing of plate

(b) Failure of bolt

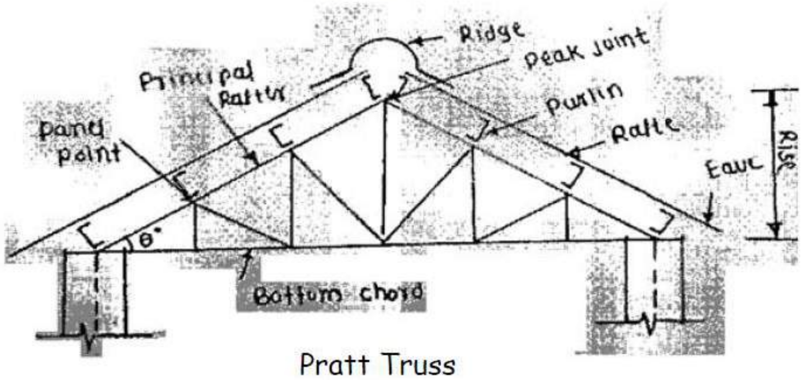
- By shear failure of bolt
- By tensile failure of bolt
- By bearing failure of bolt

26. **Draw sketches of Howe type and Pratt type truss showing pitch, rise, panel point, panel, principal rafters and all members in one of the above types.**

Answer:



Home Truss

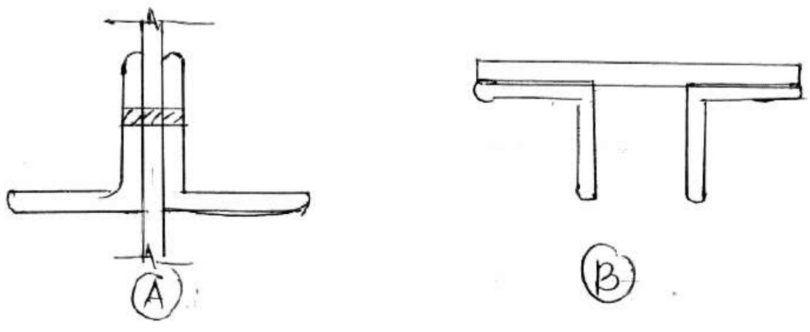


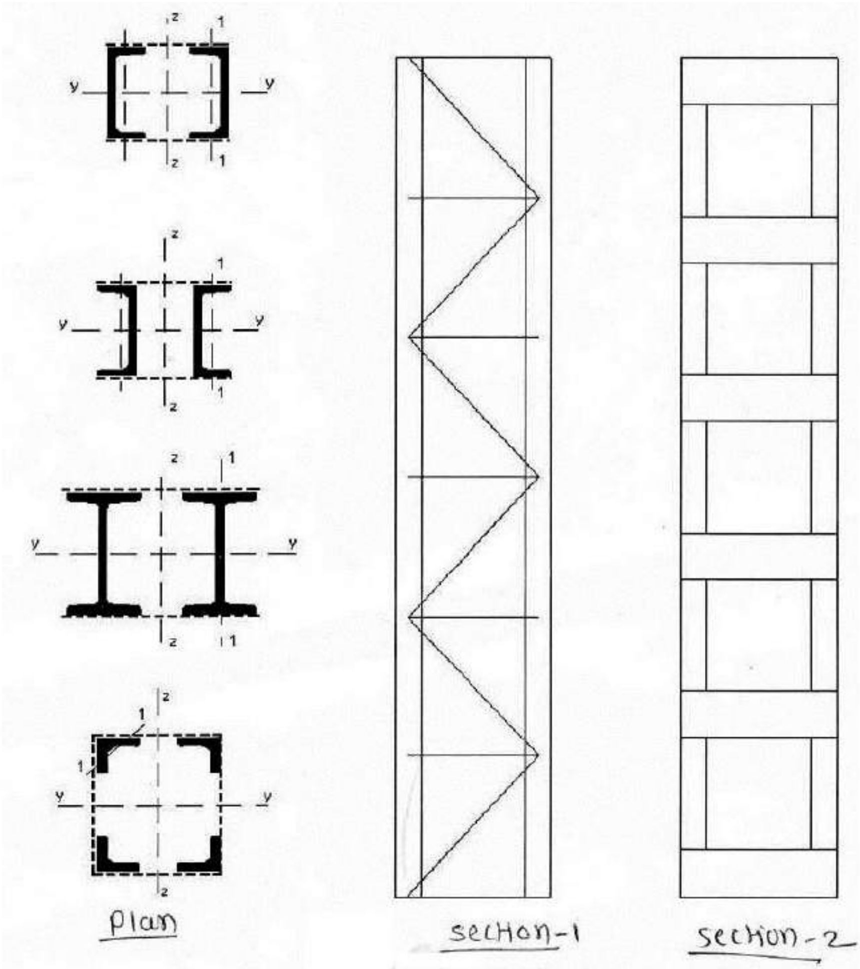
Pratt Truss

27. **Sketch different sections used as built-up strut and built-up column.**

Answer:

<u>Built-up strut</u>



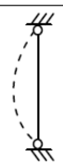


Built-up columns

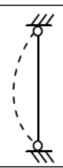
28. **State with a sketch the effective length for a compression member as per IS 800 – 2007 having end conditions as**
   (i) Translation restrained at both ends and rotation free at both ends
   (ii) Translation and rotation restrained at both ends

Answer:

(i) Translation restrained at both ends and rotation free at both ends

Restrained	Free	Restrained	Free		1.0L

(ii) Translation and rotation restrained at both ends

Restrained	Restrained	Restrained	Restrained		0.65 L

29. **State the function of lacing and battening.**

Answer:

1. **Function of lacing**
   - To connect the different components of built up column together so that they will act as one unit
   - To keep the distance between two components of built up column uniform and constant.
   - To keep the distance between two components of built up column uniform and constant.

2. **Function of battening**
   - The batten is placed opposite to each other at each end of the member and at points where the member is proportioned uniform throughout.
   - When battens are used effective length of column should be increased by 10%
   - Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens.

30. **Limiting width to thickness ratio for single beam section of plastic class is 9.4 and d/tw = 84. State whether ISMB 500 @ 852 N/m is of plastic class or not. For ISMB 500; h = 500 mm, bf = 180 mm, tf = 17.2 mm, tw = 10.2 mm, r1 = 17.0 mm, fy = 250 MPa.**

Solution:

$$\frac{h}{bf} < 8.4\,\varepsilon \ldots \text{ For class} -1 \text{ (plastic)}$$

Given section is ISMB500

$\therefore$ h = 500

& bf = 180

$$\frac{h}{bf} = \frac{500}{180} = 2.78 < 8.4\,\varepsilon$$

$$\text{but } \varepsilon = \sqrt{\frac{250}{f_y}}$$

$$\therefore \quad \varepsilon = \sqrt{\frac{250}{250}}$$

$$\varepsilon = 1$$

$$\therefore \frac{h}{bf} = 2.78 < 8.4 \times 1$$

$$= 2.78 < 8.4 \quad \ldots \quad \text{hence the class is plastic}$$

31. **Which of the following statement is correct?**
    a) angles placed on same side of gusset plate produce eccentricity about one plane only
    b) angles placed on same side of gusset plate produce eccentricity about two planes
    c) angles placed on opposite side of gusset plate produce eccentricity about one plane only
    d) angles placed on opposite side of gusset plate produce eccentricity about two planes

Answer: a

32. **Which of the following is true about built up section?**
    a) Built up members are less rigid than single rolled section
    b) Single rolled section are formed to meet required area which cannot be provided by built up members
    c) Built up members can be made sufficiently stiff
    d) Built up sections are not desirable when stress reversal occurs

Answer: c

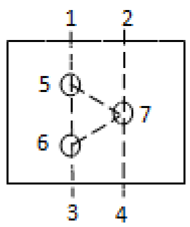
33. **What is the maximum effective slenderness ratio for members always in tension?**
    a) 400
    b) 200
    c) 350
    d) 150

Answer: a

34. **The design tensile strength of tensile member is**
    a) minimum of strength due to gross yielding, net section rupture, block shear
    b) maximum of strength due to gross yielding, net section rupture, block shear
    c) strength due to gross yielding
    d) strength due to block shear

Answer: a

35. **Which section to be considered in the design for the net area of flat?**



    a) 1-5-6-3
    b) 2-7-4
    c) 1-5-7-4
    d) 1-5-7-6-3

    Answer: d

36. **What is the net section area of steel plate 40cm wide and 10mm thick with one bolt if diameter of bolt hole is 18mm?**

    a) 38.2 cm^2
    b) 20 cm^2
    c) 240 mm^2
    d) 480 mm^2

Answer: a

37. **Which of the following is property of compression member?**
    a) member must be sufficiently rigid to prevent general buckling
    b) member must not be sufficiently rigid to prevent local buckling
    c) elements of member should be thin to prevent local buckling
    d) elements of member need not prevent local buckling

Answer: a

38. **Which of the following is true about tubular section?**
    a) tubes have low buckling strength
    b) tubes have same radius of gyration in all direction
    c) tubes do not have torsional resistance
    d) weight of tubular section is more than the weight required for open profile sections

Answer: b

39. **Effective length of compression member is**
    a) distance between ends of members
    b) distance between end point and midpoint of member
    c) distance between points of contraflexure
    d) distance between end point and centroid of member

Answer: c

40. **What is slenderness ratio of compression member?**
    a) ratio of effective length to radius of gyration
    b) ratio of radius of gyration to effective length
    c) difference of radius of gyration and effective length
    d) product of radius of gyration and effective length

Answer: a

41. **Which of the following is true?**
    a) built up column lacings or battens are uneconomical if load carrying members permit greater reduction in weight than what is added by lacing or batten

b) built up column lacings or battens are economical if load carrying members permit greater reduction in weight than what is added by lacing or batten
c) no related shear stress force in plane of cross section
d) built up column designed as axially loaded column can never be eccentrically loaded

Answer: b

42. **Which of the following is true?**
a) in case of rolled section, less thickness of plate is adopted to prevent local buckling
b) for built-up section and cold formed section, longitudinal stiffeners are not provided to reduce width to smaller sizes
c) local buckling cannot be prevented by limiting width-thickness ratio
d) in case of rolled section, high thickness of plate is adopted to prevent local buckling

Answer: d

43. **What are laterally restrained beams?**
a) adequate restraints are provided to beam
b) adequate restraints are not provided to beam
c) economically not viable
d) unstable beams

Answer: a

44. **Critical bending moment capacity of a beam undergoing lateral torsional buckling is a function of**
a) does not depend on anything
b) pure torsional resistance only
c) warping torsional resistance only
d) pure torsional resistance and warping torsional resistance

Answer: d

45. **Which of the following statement is not correct?**
a) Hollow circular tube has more efficiency as flexural member
b) Hollow circular tube has lesser efficiency as flexural member
c) It is the most efficient shape for torsional resistance
d) It is rarely used as a beam element

Answer: a

46. **Which of following statement is correct?**
a) elastic buckling stress may be decreased by using longitudinal stiffeners
b) elastic buckling stress may be decreased by using intermediate stiffeners
c) elastic buckling stress may be increased by using intermediate transverse

stiffeners
d) elastic buckling stress is not affected by intermediate or longitudinal stiffeners

47. **Structural members subjected to bending and large axial compressive loads are known as**
    a) Strut
    b) Purlin
    c) beam-column
    d) lintel

Answer: c

48. **Which of the following assumptions were not made while deriving expression for elastic critical moment?**
a) beam is initially undisturbed and without imperfections
b) behaviour of beam is elastic
c) load acts in plane of web only
d) ends of beam are fixed support

Answer: d

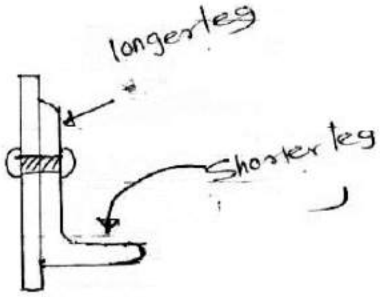
49. **The web is susceptible to shear buckling when d/t_w**
a) <67ε
b) < 2×67ε
c) >67ε
d) < 70ε

Answer: c

50. **Find the value of permissible stress in axial tension (σ_{at}) for fy = 250 MPa. State why unequal angles with long legs connected are more efficient?**

(i) $6at = 0.60 \times fy$
$= 0.60 \times 250$
$6at = 150\ N/mm^2$

(ii) Generally longer legs are connected in case of unequal angle section because of the following reason.
Consider angle is connected in the following manner as shown in fig



In the shorter leg is connected to gusset plate, then the bending stress induced in the section is large due to outstanding longer kg, because of which the stress distribution in the section is no–uniform and hence it may lead to fracture of the member prematurity

51. **Design a tension member consisting of single unequal angle section to carry a tensile load of 340 kN. Assume single row 20 mm bolted connection. The length of member is 2.4 m. Take fu = 410 MPa, α = 0.80**

Section available (mm)	Area (mm^2)
ISA 100 × 75 × 8	**1336**
ISA 125 × 75 × 8	**1538**
ISA 150 × 75 × 8	**1748**

Solution:

(A) Appropriate gross area required

Reqd $Ag = \dfrac{1.1 \times Tag}{fy}$

$= \dfrac{1.1 \times 340 \times 10^3}{250}$

$= 1496\ mm^2$

Try 15A 125 × 75 × 8 mm giving Ag = 1538 mm^2 r_{min} = 16.1 mm. Assuming longer leg connected, check the strength of the section

i) Design strength due to yielding of gross section

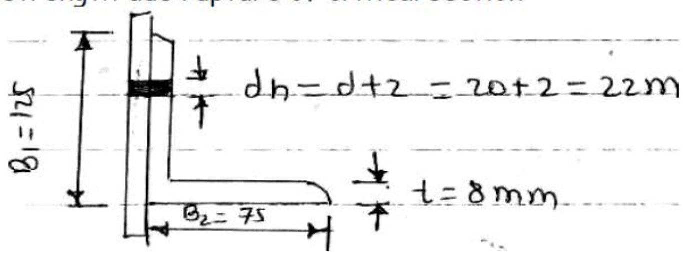
$Tag = \dfrac{Ag \times fy}{r_{mo}}$

$= \dfrac{1538 \times 250}{1.10}$

= 349545.4N

Tag = 349.54 kN

ii) Design strength due rupture of critical section



$Tdn = \alpha An \dfrac{lu}{r_{m1}}$

$An = Anc + Ago$

$Anc = (B_1 - d_h - t/2) \times t$

$= (125 - 22 - 8/2) \times 8$

$Anc = 792\ mm^2$

$Ago = (B_2 - t/2)t$

$= (75 - 8/2)8$

$= 568 mm^2$

An = Anc + Ago

An = 792 + 568

An = 13602 mm^2

Considering more than four bolt's in a raw α = 0.8

$$Tdn = \frac{0.8 \times 1360 \times 410}{1.25}$$

Tdn = 356.864 kN

Design of bolts

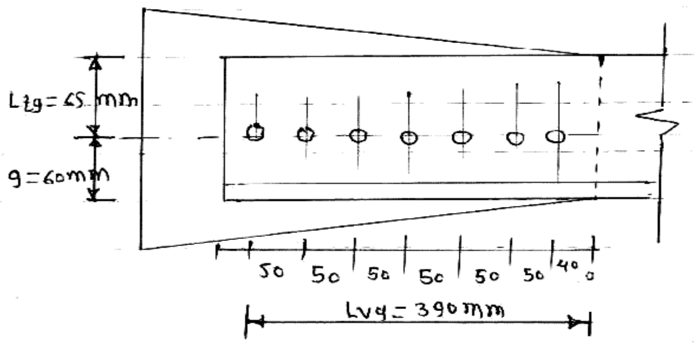
Capacity of bolts in single shear = 45. 3 KN

Capacity of bolt in bearing = $20 \times 8 \times 410 \times 10^{-3}$

= 65.6 kN

least bolt value = 45.3 kN (min of two above)

Number of bolts required = $\frac{340}{45.3}$ = 75 say 8



Assuming edge dist^{n} = 40mm

9 = 60mm

Spacing of bolts = 50mm

Avg = Lvg $\times$ t = 390 $\times$ 8 = 3120mm^2

Avg = 3120 mm^2

$$Avn = \{Lug - [\text{No. of bolts} - 0.5\,dh]\} xt$$

$$Avn = \{390 - [(8 - 0.5)22]\} \times 8 = 1800\text{mm}^2$$

Avn = 1800mm^2

Atg = Ltg $\times$ t = 65 $\times$ 8 = 500 mm^2

Atg = 520 mm^2

$$Atn = (65 - (0.5 \times 22)) \times 8$$

Atn = 432 mm^2

$$Tdb_1 = Avg\, fy / \left(\sqrt{3} \times \gamma_{mc}\right) + 0.9 Atn\, fu / \gamma_{m1}$$

$$= 3120 \times 250 / \left(\sqrt{3} \times 1.10\right) + 0.9 \times 432 \times \frac{410}{1.25}$$

= 409393.8 + 127526.4

= 5369202 N

Tdb_1 = 536.92 kN

Tdb_2 = 424.962 kN

Tdb = lesser than Tdb_1 and Tdb_2 = 424.96 kN

$\therefore$ The tensile strength of angle = lesser of Tag, Tdn and Tdb
(349.54, 356.86 and 426.96)
= 349.54 kN

This is greater than required 340 kN

Check for slenderness ratio $\lambda = \dfrac{L}{r_{min}} = \dfrac{2400}{16.1}$

149.06 $<$ 250

52. **A hall of size 12m x 18m is provided with Fink type trusses at 3 m c/c. Calculate panel point load in case of Dead load and live load from following data.**
   - **a. Unit weight of roofing = 150 N/m²**
   - **b. Self-weight of purlin = 220 N/m²**
   - **c. Weight of bracing = 80 N/m²**
   - **d. Rise to span ratio = 1/5**
   - **e. No. of panels = 6**

Solution:

(A) Span of truss = 12 m

Spacing = 3m /c/c

Types of truss = sink

No. of panel point = 6

$$\text{Rise} = \frac{\text{span}}{5}$$

$$= \frac{12}{5} = 24\text{m}$$

$$\theta = \tan^{-1}\left(\frac{\text{Risc}}{L/2}\right) = \tan^{-1}\left(\frac{2.4}{6}\right) = 21.80^\circ$$

Calculation of dead load

(i) Weight of roofing = 150 N/m^2

(ii) Weight of Purlin = 220 N/m^2

(iii) Weight of truss = $\left(\dfrac{L}{3} + 5\right) \times 10$

= 90 N/m^2

(iv) Weight of bracing = 80 N/m^2

Total dead load = 540 N/m^2

Total dead load on one truss = 540 N/m^2

= $540 \times 12 \times 3$

= 19.44 kN

Dead load on each panel point = $\dfrac{19.44}{2}$ =3.2H kN c

D on end panel point = $\frac{324}{2}$

= 1.62 kN

Live load calculation

L.L. on purlin = 750 – (0 – 10) $\times$ 20)

= 750 – [2180 – 10) $\times$ 20]

= 514 N/m^2 > 400 N/m^2

L.L of truss = 2/3 $\times$ 514 = 342.67 N/m^2

$\therefore$ Total L. L. = L.L. of truss $\times$ span $\times$ spacing

= 342.67 $\times$ 12 $\times$ 3

= 12336 N

L.L. m each panel = $\frac{12336}{6}$ = 2056 N

L.L. m end panel = $\frac{2056}{2}$ = 1028 N

53. **An industrial building has trusses for 14 m span. Trusses are spaced at 4m c/c and rise of truss in 3.6m. Calculate panel point load in case of live load and wind load using following data :**

a. Coefficient of external wind pressure = - 0.7
b. Coefficient of internal wind pressure = $\pm$ 0.2
c. Design wind pressure = 1.5 kPa
d. Number of panels = 08

(A) Span of trus = 14m

Spacing of truss = 3.6 m

No. of panels = 8

Design wind pressure = 1.5 kpa

= 1.5 $\times$ 10^3 N/m^2

$$\theta = \tan^{-1}\left(\frac{\text{Rise}}{\text{Span}/2}\right) = \frac{3.6}{14/2} = 27.22^\circ$$

$\therefore$ θ = 27.22°

Wind load calculation

Coefficient of external wind pressure

Cpe = -0.7

Coefficient of internal wind pressure

Cpi = $\pm$02

Total wind press = [Cpe – Cpi] $\times$ P_2

Wind load combination

i) w.c = [–0.7 – (0.2)] $\times$ 1500 = 750 N/ m^2

ii) w.c = [–0.7 – (+0.2)] $\times$ 1500 = 1350 N/m^2

Max. intensity = –1350 N/m^2

Length of principle dafter = $\frac{L/2}{\cos\theta}$

= [1412]

cos 27.22

Length of principle dafter = 7.87 m

$\because$ Sloping area = $2 \times 7.87 \times 4$

= 62.96 m^2

$\because$ Total wind load = Max. intensity $\times$ sloping area

= 1350×62.96

= 84996 N

Wind

$\because$ load an each panel = $\frac{84996}{8}$

$\therefore$ wind load on end panel = -10624.5 N

$\therefore$ wind load on end panel = $\frac{-10624.5}{2}$

= 5312.25 N

Live load calculation

Live load on purlin = $750 - [(\theta - 10) \times 20]$

= $750 - [(27.22 - 10) \times 20]$

= $405.6 \times 4v$ N/m^2

Hence ok

L.L. on truss

= $2/3 \times 405.6$

= 270.4 N/m^2

$\because$ Total L.L = L.L. intensity $\times$ Span $\times$ spacing

= $270.4 \times 14 \times 4$

= 15142.4 N

$\because$ load on each panel = $\frac{\text{T.L}}{\text{No. of Panel}}$

= $\frac{15142.4}{8}$

= 1892.8 N = 1.892 kN

and load on end panel = $\frac{1892.8}{2}$

= 946.4 N = 0.926 kN

54. **Design a slab base for column ISHB 400 @ 82.2 kg/m to carry factored axial compressive load of 2000 kN. The base rests on concrete pedestal of grade M20. For ISHB 400, bf = 250 mm, fy = 250 MPa, fu = 410 MPa, γ_{mo} = 1.1, tf = 12.7 mm.**

Solution:

(A) Given

Factored load pu = 2000 kN

= 2000×10^3 N

Fck = 20

D = 400

B = 250 i.e bf

v_{mo} = 1.1

tf = 12.7

fy = 250 N/mm^2

Bearing Strength of conc

$= 0.6 \text{ fek}$

$= 0.6 \times 20 = 12 \text{ N/m}^2\text{m}$

Bearing area of base plate

$$A = \frac{Pu}{\text{Bearing strength of conc}}$$

$$A = \frac{2000 \times 10^3}{12} = 166.67 \text{ x } 16^3$$

Size of base plate

length of plate

$$Lp = \frac{D-B}{2} + \sqrt{\left(\frac{D-B}{2}\right)^2 + A}$$

$$= \frac{400-250}{2} + \sqrt{\left(\frac{400-250}{2}\right)^2 + 1666.67 \times 10^3}$$

$$= 490.08 \cong 500$$

$$Bp = \frac{A}{Lp} = \frac{166.67 \times 10^3}{500} = 333.34 \cong 350$$

Larger Projection

$$a = \left(\frac{Lp - p}{2}\right) = \frac{500 - 400}{2} = 50 \text{ mm}$$

Smaller Projection

$$b = \left(\frac{Bp - B}{2}\right) = \frac{350 - 250}{2} = 50 \text{ mm}$$

Area of base plate

$Ap = 500 \times 350 = 175 \times 10^3$

Ultimate Pressure from below m the Slab base

$$w = \frac{Pu}{A} = \frac{2000 \times 10^3}{175 \times 10^3} = 11.42 \text{ N/mm}^2$$

Thickness of slab base

$$f_s = \sqrt{\frac{2.5\,w\left(a^2 - 0.3b^2\right)vm_0}{fy}}$$

$$= \sqrt{\frac{2.5 \times 11.42\left(50^2 - 0.3 \times 50^2\right) \times 1.10}{250}}$$

$= 14.82 \text{ mm} \quad > \quad \text{tf} \quad \text{I.e. } k - 7$

$\cong 15 \text{ mm}$

Hence provide slob base plate having dimension
$500 \times 350 \times 15$

55. **Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base?**

Answer:

Design steps to find thickness

1) To calculate area (A) of base plate

   A = Column load/Bearing strength

   Bearing strength of concrete = $0.6\ f_{ck}$

2) Select the size of base plate.

   L_p & B_p be the sizes of plate

   D = length or longer length

   B = width or shorter side of the column

   Consider square plate

$$L_P = \frac{(D-B)}{2} + \sqrt{\left[\left\{\frac{(D-B)}{2}\right\}^2 + A\right]}$$

$$B_P = \frac{A}{L_p}$$

   Large projection $a = \frac{(L_p - D)}{2}$

   Shorter projection $b = \frac{(B_p - B)}{2}$

   Area of base plate provided = $L_P \times B_P = (D + 2a) \times (B + 2b)$

3) Calculate ultimate bearing pressure

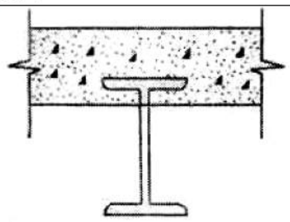
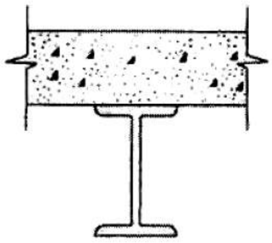
$$w = \frac{P}{(L_p \times B_p)}$$ [1 mark]

4) Calculate thickness of base plate

$$t_s = \left[\left(\frac{2.5 \times w\left(a^2 - 0.3 \times b^2\right) r_{mo}}{f_y}\right)\right]^{0.5}$$ [1 mark]

Function of anchor bolt : To connect concrete pedestal and base plate anchor bolts are used.

56. **Differentiate between Laterally supported and unsupported beams with a neat sketch.**

	Laterally supported beam	Laterally unsupported beam
1)	In laterally supported beam, compression flanges are embedded in concrete.	In laterally unsupported beam, compression flanges are not embedded in concrete.
2)	Compression flange of Beam is restrained against rotation	Compression flange of Beam is free for rotation.
3)	Lateral deflection of compression flange is not occur.	Lateral deflection of compression flange is occur.
4)	 Laterally supported.(it means compression flange is restrained)	 Laterally unsupported.

57. **Define Gusseted base. Also draw its labelled sketch showing all details.**

Answer:

Definition

When the load on column is large or column subjected to moment along with axial load, base is provided called gusseted base.

It consists of base plate, gusset angle, connecting angle on either side of column.

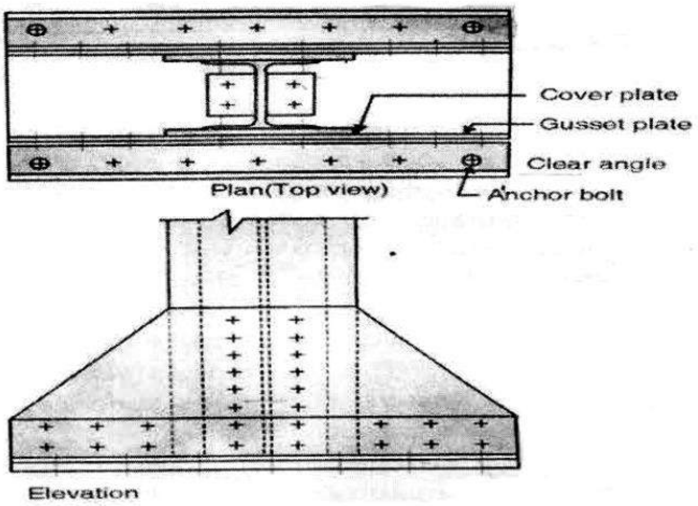


Fig. : Gusseted Base

58. **How beam sections are classified for bending as per IS : 800- 2007. Describe any two of them.**

Answer:

Classification beam:

1) Plaster or class – I
2) Compact or class – II
3) Semi compact or class – III
4) Slender or class – IV

Explain in detail

1) Plastic or class – I

Cross section which can develop plastic hinge, sustain large rotation capacity required to develop plastic mechanism are called as plastic section. These sections are unaffected by local buckling and are able to develop their full plastic moment capacities until a collapse mechanism is formed.

2) Compact or class – II

In compact section, the full cross section forms first plastic hinge but local buckling prevents subsequent moment redistribution. These sections develop full plastic moment capacities M_P but fails by local buckling due to inadequate plastic hinge rotation capacity.

3) Semi compact or class – III

In semi plastic section the extreme fibres reach the yield stress but local buckling prevents the development of plastic moment resistance.

4) Slender or class – IV

The slender section cannot attain even the first yield moment because of premature local buckling of web or flange.

59. **A simply supported beam of 6 m span supports on R. C. C. slab where in compression flange is embedded. The beam is subjected to a dead load of 25 kN/m and super imposed load of 20 kN/m, over entire span. Calculate plastic and elastic modulus required.**

Assume $r_f = 1.5$, $y_m = 1.1$ $f_y = 250$ N/mm^2.

Solution:

1) Calculation of factored load

Dead load = $1.5 \times 25 = 37.5$ KN/m

Live load = $1.5 \times 20 = 30$ KN/m

2) Calculate Maximum bending moment and shear force.

$$\text{B.M.} = \frac{WL^2}{8} + \frac{WL^2}{8} = \frac{37.5 \times 6^2}{8} + \frac{30 \times 6^2}{8} = 303.75 \text{ KN.m}$$

$$\text{S.F.} = \frac{WL}{2} + \frac{WL}{2} = \frac{37.5 \times 6}{2} + \frac{30 \times 6}{2} = 202.5 \text{ KN.m}$$

3) Plastic modulus

$$Z_P = \frac{M \times r_{mo}}{f_y} = \frac{303.75 \times 10^6 \times 1.1}{250} = 1.3365 \times 10^6 \text{ mm}^3$$

4) Elastic modulus

$$Z_e = \frac{Z_P}{1.14} = \frac{1.3365 \times 10^6}{1.14} = 1.17236 \times 10^6 \text{ mm}^3$$

60. **Which of the following are true about roof trusses?**

a) principal rafter are compression members used in buildings
b) principal rafter is bottom chord member of roof truss
c) struts are compression members used in roof trusses
d) struts are tension members used in roof trusses

Answer: c

61. **Which of the following is not a load on columns in buildings?**
a) load from floors
b) load from foundation
c) load from roofs
d) load from walls

Answer: b

62. **What are loads on columns in industrial buildings?**
a) wind load only
b) crane load only
c) wind and crane load
d) load from foundation

Answer: c

63. **Which of the following assumptions is correct for plastic design?**
a) material obeys Hooke's law before the stress reaches f_y
b) yield stress and modulus of elasticity does not have same value in compression and tension
c) material is homogenous and isotropic in both elastic and plastic states.
d) material is not sufficiently ductile to permit large rotations

Answer: c

64. **What is plastic hinge?**
a) zone of bending due to flexure in a structural member
b) zone of yielding due to flexure in a structural member
c) zone of non-yielding due to flexure in a structural member
d) zone of yielding due to twisting in a structural member

Answer: b

65. **What is plastic-collapse load?**
a) load at which sufficient number of elastic hinges are formed
b) load at which sufficient number of plastic hinges are not formed
c) load at which sufficient number of plastic hinges are formed
d) load at which structure fails

Answer: c

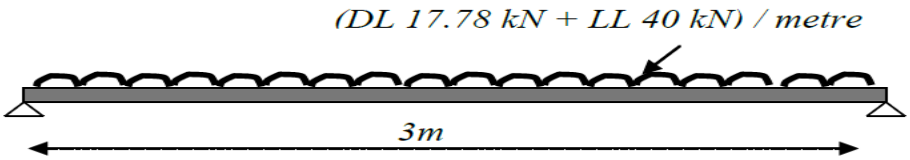
66. **Which of the following is true?**
   a) ultimate load is reached when a mechanism is formed
   b) ultimate load is not reached when a mechanism is formed
   c) plastic hinges are not required for beam to form a mechanism
   d) frictionless hinges are not required for beam to form a mechanism

Answer: a

67. **Which of the following condition is true for kinematic theorem?**
   a) load must be greater than collapse load
   b) load must be less than collapse load
   c) load must be not equal to collapse load
   d) load cannot be related to collapse load

Answer: a

68. **Design a suitable 'I' beam for a simply supported span of 3 m and carrying a dead or permanent load of 17.78 kN/m and an imposed load of 40 kN/m. Assume full lateral restraint and stiff support bearing of 100 mm.**



Solution:

<u>Design load calculation:</u>

$$factored\ load = \gamma_{LD} \times 17.78 + \gamma_{LL} \times 40\ kN$$

in this example the following load factors are chosen.

γ_{LD} *and* γ_{LL} *are taken as 1.35 and 1.50 respectively.*

γ_{LD} – *partial safety factor for dead or permanent loads*
γ_{LL} – *partial safety factor for live or imposed loads*

$$Total\ factored\ load = 1.35 \times 17.78 + 1.5 \times 40.0 = 84\ kN/m$$

$$Factored\ bending\ moment = 84 \times 3^2/8 = 94.50\ kN-m$$

Z—value required for f_y*=250 MPa ;* γ_m *=1.15*

$$Z_{reqd} = \frac{94.5 \times 1000 \times 1000 \times \gamma_m}{250}$$

$$Z_{reqd} = 434.7\ cm^3$$

Try ISMB 250

$\varepsilon = \sqrt{\frac{250}{250}} = 1.0$ $\quad D = 250\ mm$

$B = 125\ mm$

$t = 6.9\ mm$

$T = 12.5\ mm$

$I_{xx} = 5131.6\ cm^4$

$I_{yy} = 334.5\ cm^4$

Section classification:

Flange criterion $= B/2T = 5.$

Web criterion $= (D - 2T)/t = 32.61$

Since $B/2T < 8.92\ \varepsilon$ & $(D-2T)/t < 82.95\ \varepsilon$

The section is classified as '**PLASTIC**'

Moment of resistance of the cross section:

Since the section considered is 'PLASTIC'

$$M_C = \frac{S \times f_y}{\gamma_m}$$

Where S is the plastic modulus

'S' for ISMB 250 $= 459.76\ cm^3$

$M_c = 459.76 \times 1000 \times 250 / 1.15$

$= 99.95\ kN\text{-}m > 94.5\ kN\text{-}m$

Hence ***ISMB-250*** *is adequate.in flexure.*

Shear resistance of the cross section:

This check needs to be considered more importantly in beams where the maximum bending moment and maximum shear force may occur at the same section simultaneously, such as the supports of continuous beams. For the present example this checking is not required. However for completeness this check is presented.

Shear capacity $P_v = \frac{0.6 f_y A_v}{\gamma_m}$

$A_v = 250 \times 6.9 = 1725\ mm^2$
$P_v = 0.6 \times 250 \times 1725 / 1.15 = 225\ kN$
F_v = *factored max shear* $= 84 \times 3 / 2 = 126.\ kN$
$F_v / P_v = 126/225.0 = 0.56 < 0.6$

Hence the effect of shear need not be considered in the moment capacity calculation.

Check for Web Buckling:

The slenderness ratio of the web $= L_E/r_y = 2.5\ d/t = 2.5 \times 194.1/6.9$

$= 70.33$

The corresponding design compressive stress f_c *is found to be*

$f_c = 203\ MPa$ *(Design stress for web as fixed ended column)*

Stiff bearing length $= 100\ mm$

45^0 *dispersion length* $n_1 = 125.0\ mm$

$P_w\ (100 + 125.0) \times 6.9 \times 203.0$

$= 315.16\ kN$

$315.16 > 126$ *Hence web is safe against shear buckling*

Check for web crippling at support

Root radius of ISMB 250 = 13 mm

Thickness of flange + root radius = 25.5 mm

Dispersion length (1:2.5) n_2 = *2.5 x 25.5 = 63.75 mm*

$P_{crip} = (100+63.75) \times 6.9 \times 250 / 1.15$

$= 245.63 \ kN > 126kN$

Hence ISMB 250 has adequate web crippling resistance

Check for serviceability – Deflection:

Load factors for working loads γ_{LD} *and* $\gamma_{LL} = 1.0$

design load = 57.78 kN/m.

$$\delta = \frac{5 \times 57.78 \times 3000^4}{384 \times 2.1 \times 10^5 \times 5131.6 \times 10^4}$$

Max deflection $= 5.65 \ mm$

$$= \frac{L}{531}$$

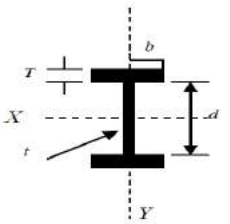
$$\frac{L}{531} < \frac{L}{200}$$

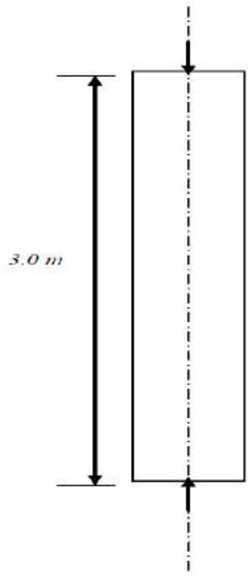
Hence serviceability is satisfied

***Result** :-- Use ISMB – 250.*

69. **Obtain factored axial load on the column section ISHB400. The height of the column is 3.0 m and it is pin-ended. Use fy = 250 N/mm^2, E = 2 x 10^5 N/mm2, ym= 1.15.**

CROSS-SECTION PROPERTIES:





Solution:

$Flange\ thickness = T = 12.7\ mm$

$Clear\ depth\ between\ flanges = d = 400 - (12.7 * 2) = 374.6\ mm$

$Thickness\ of\ web = t = 10.6\ mm$

$Flange\ width = 2b = 250\ mm$

$b = 125\ mm$

$Self\text{-}weight = w = 0.822\ kN/m$

$Area\ of\ cross\text{-}section = A = 10466\ mm^2$

$r_x = 166.1\ mm$

$r_y = 51.6\ mm$

(i) Type of section:

$$\frac{b}{T} = \frac{125}{12.7} = 9.8 < 10 \in$$

$$\frac{d}{t} = \frac{374.6}{10.6} = 35.3 < 41 \in$$

$$where, \in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Hence, cross- section is "COMPACT"

(ii) Effective Length:

As, both ends are pin-jointed effective length $= \ell_x = \ell_y = 3.0\ m$

(iii) Slenderness ratios:

$$\lambda_x = \frac{\ell_x}{r_x} = \frac{3000}{166.1} = 18.1$$

$$\lambda_y = \frac{\ell_y}{r_y} = \frac{3000}{51.6} = 58.1$$

(iv) Values of (α) :

For rolled I-sections,

In x – direction $\alpha_x = 0.0020$

In y – direction $\alpha_y = 0.0035$

$$\lambda_o = 0.2\pi \sqrt{E/f_y} = 0.2 * \frac{22}{7} \sqrt{\frac{200000}{250}}$$

$$= 17.8$$

(v) values of η:

$$\eta = \alpha\ (\lambda - \lambda_o)$$

$$\eta_x = \alpha_x\ (\lambda_x - \lambda_o) = 0.002 * (18.1 - 17.8) = 0.001$$

$$\eta_y = \alpha_y\ (\lambda_y - \lambda_o) = 0.0035 * (58.1 - 17.8) = 0.141$$

(vi) Calculation of maximum compressive stress at failure (σ_c):

We have,

$$\sigma_e = \frac{\pi^2 E}{\lambda^2}$$

$$\sigma_c = \phi \pm \sqrt{\phi^2 - f_y \sigma_e} \ \leq f_y$$

$$where,\ \phi = \frac{f_y + (\eta+1)\ \sigma_e}{2}$$

In x-direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(18.1)^2} = 6025\ N/mm^2$$

$$\varphi_x = \frac{250 + (0.001 + 1)*6025}{2} = 3140\ N/mm^2$$

$$\sigma_{cx} = 3140 \pm \sqrt{(3140)^2 - 250*6025} \leq 250$$

$$= 250\ N/mm^2$$

In y-direction,

$$\sigma_{ey} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\varphi_y = \frac{250 + (0.141 + 1)\,585.0}{2} = 459 N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250*585} \leq 250$$

$$= 205\ N/mm^2$$

Hence, Allowable axial compressive stress, $\sigma_c = 205\ N/mm^2$

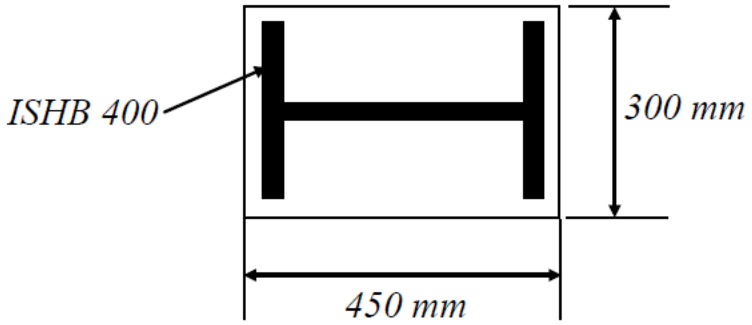
Safe axial compressive stress = $\sigma_c/\gamma_m = 205/1.15 = 178\ N/mm^2$

(vii) Factored Load:

*Factored Load = $\sigma_c\ A/\gamma_m$ = 178 * 10466/1000*

= 1863 kN

70. **Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN. Use fcu = 40 N/mm2 ; fy = 250 N/mm2 ; γm = 1.15**



Solution:

Bearing strength of concrete $= 0.4f_{cu} = 0.4 * 40 = 16\ N/mm^2$

Area required $= 1800*10^3/16 = 112500\ mm^2$

Use plate of 450 X 300 mm (135000 mm²)

Assuming projection of 25 mm on each side

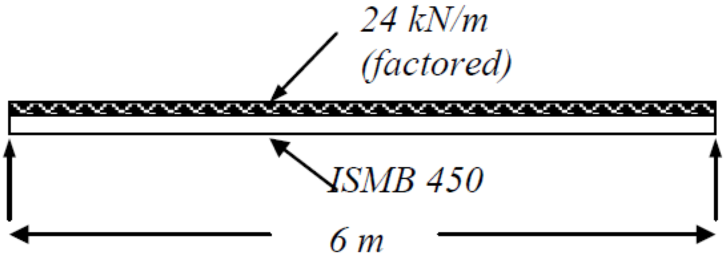
$w = (1800 * 10^3) / (450 * 300) = 13.33\ N/mm^2$

$f_{yp} = 250/1.15 = 217.4\ N/mm^2$

$$t_p = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)}{f_{yp}}} = \sqrt{\frac{2.5\times 13.33\left(25^2 - 0.3\times 25^2\right)}{217.4}} = 8.2\ mm$$

Hence, use 450 X 300 X 10 mm plate.

71. **Check the adequacy of ISMB 450 to carry a uniformly distributed load of 24 kN / m over a span of 6 m. Both ends of the beam are attached to the flanges of columns by double web cleat.**

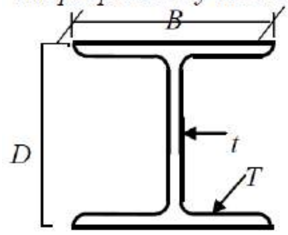


Design check:

For the end conditions given, it is assumed that the beam is simply supported in a vertical plane, and at the ends the beam is fully restrained against lateral deflection and twist with, no rotational restraint in plan at its ends.

Section classification of ISMB 450

The properties of the section are:



Depth, D = 450 mm

Width, B = 150 mm

Web thickness, t = 9.4 mm

Flange thickness, T = 17.4 mm

Depth between fillets, d = 379.2 mm

Radius of gyration about minor axis, $r_y = 30.1$ *mm*

Plastic modulus about major axis, $S_x = 1512.8 * 10^{-3}\ mm^3$

Assume $f_y = 250\ N/mm^2$, $E = 200000\ N/mm^2$, $\gamma_m = 1.15$,

$p_y = f_y / \gamma_m = 250 / 1.15 = 217.4\ N / mm^2$

(I) Type of section

Flange criterion:

$$b = \frac{B}{2} = \frac{150}{2} = 75\ mm$$

$$\frac{b}{T} = \frac{75.0}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon \quad \text{where}\ \varepsilon = \sqrt{\frac{250}{f_y}}$$

Hence O.K.

Web criterion:

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\,\varepsilon$$

Hence O.K.

Since $\frac{b}{T} < 8.92\,\varepsilon$ and $\frac{d}{t} < 82.95\,\varepsilon$, the section is classified as 'plastic'

(II)Check for lateral torsional buckling:

Equivalent slenderness of the beam, $\lambda_{LT} = n\,u\,v\,\lambda$

where, n = *slenderness correction factor (assumed value of 1.0)*

u = *buckling parameter (assumed as 0.9)*

λ = *slenderness of the beam along minor axis*

$$= \frac{6000}{30.1} = 199.33$$

v = *slenderness factor (which is dependent on the proportion of the flanges and the torsional index [D / T])*

= 0.71 *(for equal flanges and* $\lambda = 199.33$*)*

Now, $\lambda_{LT} = 1.0 * 0.9 * 0.71 * 199.33$

$= 127.37$

Bending strength, $p_b = 84$ *Mpa (for* $\lambda_{LT} = 127.37$*) (from Table 11 of BS 5950 Part I)*

Buckling resistance moment $M_b = S_x * p_b$

$$= (1512.78 * 84)/1000$$

$$= 127.07\ kN\,m$$

For the simply supported beam of 6.0 m span with a factored load of 24.0 KN/m

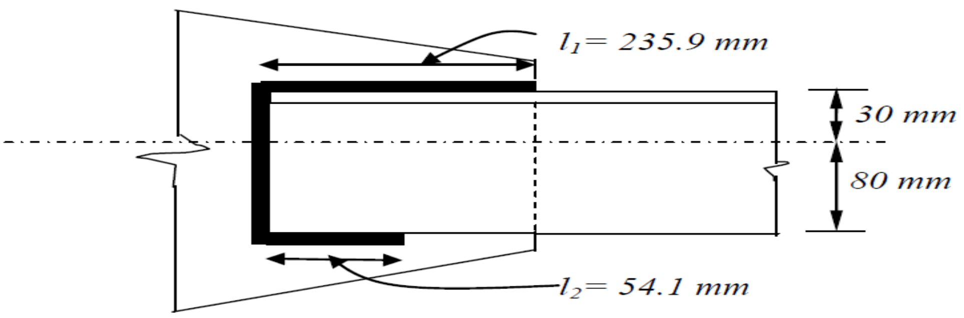
$$M_{max} = \frac{w\ell^2}{8} = \frac{24*6^2}{8}$$

$$= 108.0\ KN\,m < 127.07\ kN\,m$$

$$Hence\ M_b > M_{max}$$

ISMB 450 is adequate against lateral torsional buckling.

72. **In a roof truss, a tie member ISA 110 mm X 110mm X 8 mm carries a factored tension of value 210 kN. The tie is connected to a gusset plate 8 mm thick. Design the welded joint. Factored yield strength of steel is 217.4 N/mm2 and shear strength of weld is 125 N/mm2.**



Fillet weld for tie member of a roof truss

Solution:

For this problem we would provide a weld group consisting of transverse and longitudinal welds and ensure that the CG of the weld group coincides with the line of action of the externally applied load.

First we would decide about the weld size. This is decided by the thickness of the rolled section and the plating. Weld which are applied to rounded toe of rolled section should not be more than ¾ of its thickness or plating and hence we get a weld size of 6mm (3/4 8). The maximum size of the end weld is also limited by the thickness of the plating, which is 8-1.5=6.5 mm. Hence 6 mm fillet welds are O.K.*

Transverse weld is provided equal to the size of the leg = 110 mm.

Force transmitted by transverse weld = (125 0.7* 6* 110) /1000*
=57.75 kN

Remaining force to be transmitted by the longitudinal welds = 210 – 57.75
=152.25 kN

We must ensure that the CG of the welds coincides with line of action of the external force. This could be ensured by providing longitudinal welds along the near and far side of the angle and also by ensuring that the moment of the all the forces about any of the line of the weld vanishes.

Let us assume that the lengths of the welds in the heel and toe sides are l_1 and l_2 respectively.

Total weld length required for 152.25 kN
*=152.25 * 1000 / ((125* 0.7*6))=290 mm*

Taking moment of all forces about the heel side longitudinal weld, we get

$57.75 * 1000 * 55 + l_1 * 0 + l_2 * (125. * 0.7 * 6) * 110$
$= 210 * 1000 * 30.$

Therefore l_2 $= 54.09$ mm

Hence we get the weld length l_2 as say 54.09 mm. The bracketed term in the above expression represents the strength of the weld for 1 mm.

Now we get the length l_1 as 290 – 54.09 =235.91 mm

Alternatively the longitudinal weld length l_1 is obtained by taking moment of all the forces about the toe side weld line. Hence we have demonstrated as to how a weld group could be designed to have a CG coinciding with the externally applied load.

It is also to be noted that in case it is desired to reduce the length of the joint then the heel side weld size can be increased.

73. **Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.**

Solution:

1) Bolt force:

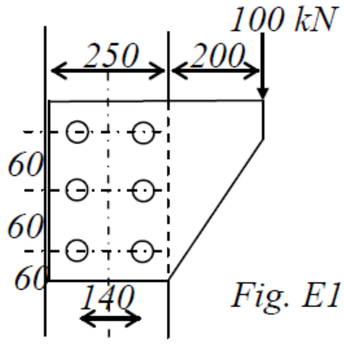


Fig. E1

$P_x = 0;\ P_y = 100\ kN;$

Total eccentricity x'=200+250/2=325 mm

$M = P_y x' = 100x325 = 32500\ kN\text{-}mm$

Try the arrangement shown in Fig. E1
Note: minimum pitch = 60 mm and
minimum edge dist. = 60 mm

$n = 6$

$\Sigma r_i^2 = \Sigma x_i^2 + \Sigma y_i^2 = 6(70)^2+4(60)^2 = 43800\ mm^2$

Shear force on the farthest bolts (corner bolts)

$$R_i = \sqrt{\left\{\left[\frac{32500\times 60}{43800}\right]^2 + \left[\frac{100}{6} + \frac{32500\times 70}{43800}\right]^2\right\}} = 81.79\ kN$$

2) Bolt capacity
Try M20 HSFG bolts

Bolt capacity in single shear $= 1.1\ K\ \mu\ P_o = 1.1 \times 0.45 \times 177 = 87.6\ kN$

ISHB 400 flange is thicker than the bracket plate and so bearing on the bracket plate will govern.
Bolt capacity in bearing $= d\ t\ p_{bg} = 20 \times 8 \times 650 \times 10^{-3} = 104\ kN$

$\therefore$ *Bolt value* $= 87.6\ kN > 81.79$ *safe.*

74. **Design a bolted web cleat beam-to-column connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column. The connection has to transfer a factored shear of 150 kN. Use bolts of diameter 20 mm and grade 4.6.**

Solution:

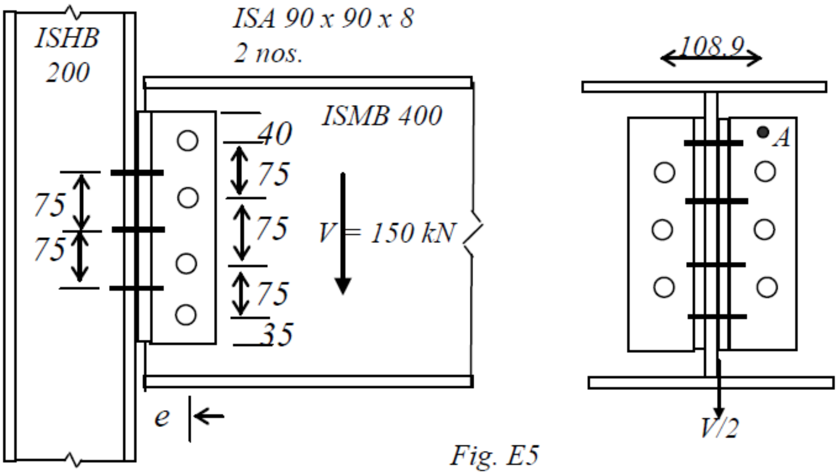


Fig. E5

1) The recommended gauge distance for column flange is 100 mm.
Therefore required angle back mark is 50 mm.
Use web cleats of ISA 90x90x8 giving gauge g = 50+50+8.9=108.9 mm

2) Connection to web of beam- Bolt capacity
shear capacity of bolt in double shear = $2\times160\times245\times10^{-3}=78.4$ kN
bearing capacity of bolt on the beam web = $418\times20\times9.0\times10^{-3}= 75.24$ kN
bolt value = 75.24 kN

Try 4 bolts as shown in the Figure with vertical pitch of 75 mm

Assuming the shear to be acting on the face of the column, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= 150\times50\times112.5/2(37.5^2+112.5^2) = 30.0$ kN

vertical shear force per bolt = 150/4 = 37.5 kN

resultant shear = $\sqrt{(30.0^2+37.5^2)} = 48.0$ kN < bolt value Safe !

3) *Connection to column flange: Bolt capacity*

shear capacity of bolt in single shear = $160 \times 245 \times 10^{-3} = 39.2$ kN
bearing capacity of bolt on column flange = $418 \times 20 \times 9.0 \times 10^{-3} = 75.24$ kN
bolt value = 39.2 kN

Try 6 bolts as shown in the Fig.E5 with vertical pitch of 75 mm

4) *Check bolt force*
Similar to the previous case, the shear transfer between the beam web and the angle cleats can be assumed to take place on the face of the beam web. However, unlike the previous case, no relative rotation is possible between the angle and the beam web.

Assuming centre of pressure 25 mm below top of cleat (point A), horizontal shear force on bolt due to moment due to eccentricity e
$= (150 \times 50/2) \times 200/(50^2+125^2+200^2) = 12.9$ kN

vertical shear force per bolt = 150/6 = 25.0 kN

resultant shear = $\sqrt{(12.9^2+25.0^2)} = 28.13$ kN < *bolt value OK*

Use 2 Nos ISA 90x90x8 of length 375 mm as angle cleats

75. **Design a double web cleat connection for an ISMB 400 coped beam to an ISMB 600 main beam so as to transfer a factored load of 300 kN using HSFG bolts of 20mm diameter and grade 8.8.**

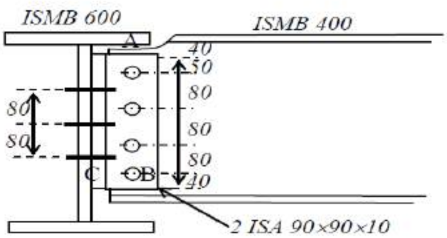


Fig. E7

Solution:
1) *Connection to web of ISMB 400*

For M20 Gr.8.8 HSFG bolts in double shear
Slip resistance per bolt = $2 \times 1.1 \times 0.45 \times 144 = 142.6$ kN
Bearing capacity of web per bolt = $20 \times 8.9 \times 650 \times 10^{-3} = 115.7$ kN
Bolt value = 115.7 kN

Try 4 bolts as shown in the Figure with vertical pitch of 80 mm

Assuming the shear to be acting on the face of the ISMB 600 web, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= (300/2) \times 50 \times 112.5/(37.5^2+112.5^2) = 60.0$ kN

vertical shear force per bolt = 300/4 = 75.0 kN

resultant shear = $\sqrt{(60^2+75^2)} = 96.0$ kN < *bolt value Safe !*

2) *Connection to web of ISMB 600*

Try 6 bolts as shown in the Figure with vertical pitch of 80 mm

For M20 Gr.8.8 HSFG bolts in single shear
Slip resistance per bolt $= 1.1 \times 0.45 \times 144 = 71.28$ *kN*
Bearing capacity of web per bolt $= 20 \times 12 \times 650 \times 10^{-3} = 156$ *kN*
Bolt value $= 71.28$ *kN*

Assuming center of pressure 27.5 mm below the top of the angle

horizontal shear force on bottom bolt due to moment due to eccentricity e
$= (300/2)\times 50\times 200/(50^2+125^2+200^2) = 25.82$ *kN*

vertical shear force per bolt $= 300/6 = 50.0$ *kN*

resultant shear $= \sqrt{(25.82^2+50^2)} = 56.27$ *kN < bolt value Safe !*

3) *Check web of ISMB 400 for block shear*

Block shear capacity = shear capacity of AB + 0.5×tensile capacity of BC
$= 0.6\times 250\times 0.9\times 1.1(3\times 80+50-3.5\times 22)\times 8.9\times 10^{-3}$
$+ 0.5\times 250\times 1.1(45-0.5\times 22)\times 8.9\times 10^{-3} = 323.12 > 300$ *kN* *Safe!*

76. **The shear lag width for ISA 75X75X10 is (Assume gauge distance = 40 mm).**

Solution:

The length of outstanding leg will be w = 75 mm and w1 = 40 mm.

So the shear lag width, bs = w + w1 – t = 75 + 40 – 10 = 105 mm.

77. Explain various types of standard rolled steel sections.

Answer:

Various types of standard rolled steel sections
i) Rolled steel I-sections (Beam sections)
ii) Rolled steel channel sections
iii) Rolled steel Tee sections
iv) Rolled steel angle sections

v) Rolled steel bars
vi) Rolled steel flats
vii) Rolled steel plates
viii) Rolled steel sheets
ix) Rolled steel strips
x) Rolled steel tubular sections

(a) Rolled steel I – sections (Beam sections)

• Indian Standard Junior Beam (ISJB)
• Indian Standard Light Beam (ISLB)
• Indian Standard Medium weight Beam (ISMB)
• Indian Standard Wide flange Beam (ISWB)
• Indian Standard Heavy Beam (ISHB)
• An I – Section is designated by its depth and weight

Eg: An ISLB 500 @ 735.8 N/m means, An I – section is 500 mm deep and self weight is 735.8 N per meter length.
• Special beam section available from Indian rolling mill is Indian Column Section (ISC)

(b) Rolled Steel Channel Sections
• Indian Standard Junior Channel (ISJC)
• Indian Standard Light Channel (ISLC)
• Indian Standard Medium Weight Channel with Sloping Flange (ISMC)
• Indian Standard Medium Weight Channel with parallel flange (ISMCP)
• Indian Standard Gate Channel (ISGC)
• Designated by its depth and weight

Ex: ISLC 350 @ 380.63 N/m

(c) Rolled Steel T – Sections
• Indian Standard rolled Normal T – section (ISNT)
• Indian Standard rolled Deep legged T – (ISDT)
• Indian Standard rolled silt Light weight T – bars (ISLT)
• Indian Standard rolled silt Medium weight T – bars (ISMT)
• Indian Standard rolled silt T – bars from H – section (ISHT)
• Designated by its depth and weight

Ex : ISNT 125 @ 274 N/m

(d) Rolled Steel Angle sections
• Indian standard equal angles, Indian standard unequal angles and Indian standard bulb angles
• Designated by abbreviation ISA along with widths of both legs and thickness.
• Indian equal angles are designated as ISA or ISEA (**Ex.** ISEA 100 x 100 x 10 mm), Indian standard unequal angles are designated as ISA (**Ex.** ISA 125 x 75 x 10 mm) and Indian standard bulb angles are designated as ISBA.

78. **Which of the following is correct in case of angle members?**
  a) connection of lug angle to angle member should be capable of developing a strength of 10% of excess of force of outstanding leg of angle
  b) connection of lug angle to angle member should be capable of developing a strength of 20% of excess of force of outstanding leg of angle
  c) lug angles and their connection to gusset should be capable of developing a strength of less than 20% of excess of force of outstanding leg of angle
  d) lug angles and their connection to gusset should be capable of developing a strength of not less than 20% of excess of force of outstanding leg of angle

Answer: d

79. **The effective length of compression flange of simply supported beam not restrained against torsion at ends is**
   a) 1.2 L
   b) 1.0 L
   c) 0.8 L
   d) 0.5 L

Answer: a

80. **Which of the following assumptions were not made while deriving expression for elastic critical moment?**
   a) beam is initially undisturbed and without imperfections
   b) behaviour of beam is elastic
   c) load acts in plane of web only
   d) ends of beam are fixed support

Answer: d

81. **A single angle section 90X60X10 is connected with gusset plate with 7 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the design tensile strength of the section for rupture of net section? (Assume the section is connected with longer leg and gauge distance = 50 mm)**

Solution:

Anc = (90 - 10/2 - 22) × 10 = 630 mm2
Ago = (60 - 10/2) × 10 = 550 mm2
An = 630 + 550 = 1180 mm2

The length of outstanding leg will be w = 60 mm and w1 = 50 mm. So the shear lag width, bs = w + w1 – t = 60 + 50 – 10 = 100 mm.

Distance between end bolts , Lc = 6 × 50 = 300 mm.

$$\beta = 1.4 - 0.076\frac{b_s}{L_c} \times \frac{w}{t} \times \frac{f_y}{f_u} = 1\,.4 - 0.076 \times \frac{100}{300} \times \frac{60}{10} \times \frac{250}{410} = 1.307$$

$$\text{Thus, } T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta f_y A_{go}}{\gamma_{mo}} = \frac{0.9 \times 410 \times 630}{1.25} + \frac{1.307 \times 250 \times 550}{1.1}$$

$$= 349.35 \times 10^3 \text{ N} = 349.35 \text{ kN.}$$

82. **A single ISA 75 × 50 × 8 is connected (longer leg) with gusset plate using use 4 bolts of 20 mm diameter in one line at pitch of 50 mm and edge distance of 30 mm. What is the Design tensile strength due to block shear failure? (Assume gauge distance = 35 mm)**

Solution:
Avg = 8 × (3 × 50 + 30) = 1440 mm2
Avn = 8 × (3 × 50 + 30 – 3.5 × 22) = 824 mm2
Atg = 8 × 40 = 320 mm2 [assuming gauge *g* = 35 for 75 mm leg]
Atn = 8 × (40 – 0.5 × 22) = 232 mm2

$$T_{db1} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{f_y A_{tg}}{\gamma_{mo}} = \frac{0.9\times410\times824}{\sqrt{3}\times1.25} + \frac{250\times320}{1.1} = 213.16 \times 10^3 \text{ N} = 213.16 \text{ kN}$$

$$T_{db2} = \frac{A_{vg}f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9\, f_u A_{tn}}{\gamma_{m1}} = \frac{1440\times250}{\sqrt{3}\times1.1} + \frac{0.9\times410\times232}{1.25} = 257.44 \times 10^3 \text{ N} = 257.44 \text{ kN}$$

So, T_{db} = 213.16 kN.

83. **An ISA 90 x 90 x 8 used as tension member is connected to a 10 mm gusset plate by fillet weld of size 5 mm. The design strength of the member is 300 kN. Calculate the length of the weld.**

Solution: Force resisted by weld at lower side of angle $P_1 = 300 \times \frac{90-25.1}{90} = 216.33$ kN

Force resisted by weld at upper side of angle $P_2 = 300 \times \frac{25.1}{90} = 83.67$ kN

Assuming size of weld as 5mm, the throat thickness t_e will be $0.707 \times 5 = 3.535$ mm

Length required at lower side $L_{w1} = \frac{P_1}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{216.33 \times 10^3}{\frac{3.535\times410}{\sqrt{3}\times1.25}} = 323.15 \text{ mm} \approx 324 \text{ mm}$

Length required at upper side $L_{w2} = \frac{P_2}{\frac{t_e f_u}{\sqrt{3}\gamma_{mw}}} = \frac{83.67 \times 10^3}{\frac{3.535\times410}{\sqrt{3}\times1.25}} = 124.9 \text{ mm} \approx 125 \text{ mm}$

84. What are the various assumptions used in plastic analysis theory?

Answer:

The following are the assumptions are made in plastic design to simplify computations:

1) The material obeys Hooke, Law till the stress reaches fy.

2) The yield stress and modulus of elasticity have the same value in compression and tension.

3) The material is homogeneous and isotropic in both the elastic and plastic states.

4) The material is assumed to be sufficiently ductile to permit large rotation of the section to take place.

5) Plastic hinge rotation is large compare with the elastic deformations so that all the rotations are concentrated at the plastic hinges. The segments between the plastic hinges are rigid.

6) The magnitude of bending moment caused by the external loads will at the most be equal to the plastic moment reached the capacity of the section.

7) The influence of normal and shear forces on plastic moments is not considered.

8) Plane sections remain plane even after bending and the effect of shear is neglected.

9) The equilibrium of forces at the time of collapse is considered for the undeformed state of the structure.

10) No instability occurs in any member of the structure upto collapse.

85. As per IS: 800 what are the various conditions satisfied in order to use plastic method of analysis?

Answer:

IS: 800 stipulates that the following conditions should be satisfied in order to use the plastic method of analysis:

1) The yield stress of steel used should not be greater than 450MPa.

2) The stress-strain characteristic of the steel used should obey the following conditions, in order to ensure plastic moment redistribution. a) The yield plateau (horizontal portion of the stress-strain curve) should be greater than 6 times the yield strain. b) The ratio of the ultimate tensile stress to the yield stress should be more than 1.2. c) The elongation on the standard gauge length should be more than 15%. d) The steel should exhibit strain-hardening capacity.

3) The members shall be hot-rolled or fabricated using hot-rolled plates.

4) The cross section of the members not containing plastic hinges should be ‘compact’ and those of member containing plastic hinges should be ‘plastic’. 5) The cross-section should be symmetrical about its axis perpendicular to the axis of the plastic hinge rotation.

These limitations are intended to ensure that there is a sufficiently long plastic plateau to enabling a hinge to form and that the steel will not experience premature strain hardening.

86. Write down various advantages and disadvantages of plastic design.

Answer:

Advantages of Plastic Design

Plastic design methods offer the following advantages:
1) Realization of uniform and realistic F.O.S for all parts of the structures (in contrast to elastic methods, where the safely factor varies)
2) Simplified analytical procedure and readily of obtaining design moments, since there is no need to satisfy elastic strain compatibility conditions.
3) Saving of material over elastic methods resulting in lighter structures.
4) No effect due to temperature changes, settlement of supports, imperfection, erection method, etc. (because their only effect is to change the amount of rotation required). This is in contrast to the elastic method, where extra calculation are required. However, calculation for instability and elastic deflection required careful considerations in plastic method. The

plastic design method is very popular for design of some structure, e.g, beams and portal frames.
5) Gives some idea of collapse mode and strength of structure.
6) In the elastic method of design, the design process is repeated several times to obtain an optimum solution, where the plastic method of design produces a balanced section in a single attempt.

Dis-advantages of plastic design:

The disadvantages of plastic design method are the following:
1) Obtaining collapse load is difficult if the structure is reasonably complicated.
2) There is little saving in column design.
3) Difficult to design for fatigue.
4) Lateral bracing requirements are more than stringent than elastic design.
5) Calculations for elastic deformations require careful considerations.
6) When more than one loading condition occurs, it is necessary to perform separate calculations, one for each loading condition; the section requiring the largest plastic moment is selected. Unlike the elastic method of design, wherein the moment produced by different loading condition can be added together, the plastic moment obtained by different loading conditions cannot be combined(i.e, the plastic moment calculated for a given set of loads is valid only for that loading condition). This is because the 'principle of super position' becomes invalid when certain parts of the structure have yielded.

87. Write a short note on plastic hinge and hinge length.

Answer:

Plastic Hinge:
A plastic hinge is a zone of yielding due to flexure in a structural member. Although hinges do not actually form, it cal be seen that large changes of slope occurs over small length of the member at position of maximum moments. A strain hardening action usually occurs at these hinges so that large deflections are accompanied by a slight increase in load.

A structure can support the computed ultimate load due to the formation of plastic hinges at certain critical sections. The member remain elastic until the moment reaches a value Mp, the maximum moment of resistance of a fully yielded cross section or fully plastic moment of a section(Mp = fy Zp). Any additional moment will cause the beam to rotate with little increase in stress. The rotation occurs at a constant moment (Mp). The zone acts as if it was hinges except with a constant restraining moment (Mp). The plastic hinge, therefore, can be defined as a yielded zone due to flexure in a structure in which infinite rotation can take place at a constant restraining moment (Mp) of the section. It is represented normally by a black dot. The value of the moment at the adjacent sections of the yield zone for a certain length is more than the yield moment. This length is known as hinge length, depends upon the loading and geometry of the section. To simplify the analysis, this small length is neglected and the plastic hinge is assumed to be formed at discrete points of zero length. But, it cannot be neglected for the calculation of deflections and the design of bracings as the length over which yielding extends is quite important.

The plastic hinges are formed first at the sections subjected to the greatest deformation (curvature). The possible places for plastic hinges in a structure with prismatic members are points of concentrated loads, at the ends of member meeting at a connection involving a change in geometry and at the point of zero shear in a span under distributed load.

Hinge Length:

Consider a simply supported rectangular beam subjected to a gradually increasing concentrated load P, at the centre. A plastic hinge will be formed at the centre.
Mp = PL/4; My = fy x Ze = fy x bd^2/6 = fy (1/6) x {4x(1/4)} bd^2 = (2/3) x fy xbd^2/4 = (2/3) x fy x Zp = (2/3)Mp, i.e, Mp is 1.5 times more than My.

From the BM diagram, Mp / (L/2) = My / (L/2-x/2) => x=L/3. Therefore, the hinge length of the plasticity zone is equal to 1/3rd of the span.

Similarly, the hinge length of the plasticity zone for a simple beam subjected to uniformly distributed load is L/sqrt(3).

88. Explain in detail shape factor and load factor.

Answer:

Shape Factor (v):

The ratio Mp / My is a property of a cross sectional shape and is independent of the material properties. This ration is known as the shape factor v and is given by v = Mp / My = = fy Zp / fy Ze.= Zp / Ze For wide-flange I-section in flexure about the strong axis, the shape factor ranges from 1.09 to about 1.18 with the average value being 1.14. One may conservatively take the plastic moment strength Mp of I-section bent about their strong axis to be at least 15% greater than the strength My. On the other hand, the shape factor for I-section bent about their minor axis is about the same as for a rectangular section, i.e, about 1.5.

Load Factor:

Load factor is defined as the ratio of the collapse load to the working load(service load) and is represented by F, i.e, F = Pc / Pw
If a collection of beams having different end conditions (free or fixed) and the working load W were first design elastically and then plastically, the ratio Pc / Pw will not be identical. Only the beams that are simply supported will produce a constant ratio of Pc / Pw and for these cases the values of Pc / Pw will be the lowest.

From a practical point of view, a minimum acceptable and constant load factor is required, and that found for a simply supported beam may be regarded satisfactory. For a simple beam the variation of the bending moment with the load is linear.

In actual practice a load factor varying from 1.7 to 2.0 is assumed depending upon the designer’s judgment. When the structures are subjected to wind the corresponding load factor for plastic design is reduced by 25%. The prime function of the load factor is to ensure that the structure will be safe under the collapse load. Therefore, it may be regarded as a factor of safety based upon the collapse load. It depends upon the nature of loading, the support conditions, and the geometrical shape of the structural members. Uncertainty of the loads, imperfection in workmanship and error in fabrication are some of the other factors which influence the choice of the load factor.

89. Explain various mechanism of plastic analysis.

Answer:
When a structure is subjected to a system of loads, it is stable and hence functional until a sufficient number of plastic hinges have been formed to render the structure unstable. As soon as the structure reaches an unstable condition, it is considered to have been failed. The segments of the beams between the plastic hinges are able to move without an increase of load. This condition in a member is called mechanism. The concept of mechanism formation in a structure due to loading beyond the elastic limit and of virtual work is used in the plastic analysis and design of steel structures. If an indeterminate structure has the redundancy r, the insertion of r plastic hinges makes it statically determinate. Any further hinge converts this statically determinate structure into mechanism. Hence, for collapse, the numbers of plastic hinges required are (r+1).

Types of Mechanism: Various possible mechanism are listed below:
a) Beam mechanism
b) Panel / sway mechanism
c) Joint mechanism
d) Gable mechanism
e) Composite (combined) mechanism

Number of Independent Mechanism: Let,

N = number of possible plastic hinges
r = number of redundancies
n = possible independent mechanism.

Then, $n = N - r$

After finding out the number of independent mechanism all the possible combinations are made in such a way so as to make the external works maximum or the internal work a minimum. This is done to obtain the lowest load.

90. What are the different theorems of plastic analysis? Explain

Answer:
The plastic analysis of a structure is govern by three theorem, which are as follows:
1) The static or lower bound theorem: states that a load ($P<P_c$)computed on the basis of an assumed equilibrium moment diagram, in which the moments(M) are nowhere greater than the plastic moment($M<M_p$), is less than, at the best equal to, the correct collapse load. Hence the static method represents the lower limit to the true ultimate load and has a maximum factor of safety. The static theorem satisfies the equilibrium and yield conditions.

2) Kinematic or Upper bound theorem: states that a load computed on the basis of an assumed mechanism will always be greater than or at the best equal to , the true collapse load ($P>P_c$). Hence the kinematic method represents an upper limit to the true ultimate load and has a smaller factor of safety. The kinematic theorem satisfies the equilibrium and continuity conditions.

3) Uniqueness theorem: The lower and upper bound theorems can be combined to produce the uniqueness theorem, which states that the load that satisfies both the theorems at the same time is the correct collapse load. When both the theorems are satisfied in a given problem then the solution is said to be the correct (unique) one.

Using the principle of virtual work and the upper and lower bound theorems, a structure can be analysed for its ultimate load by any of the following methods:

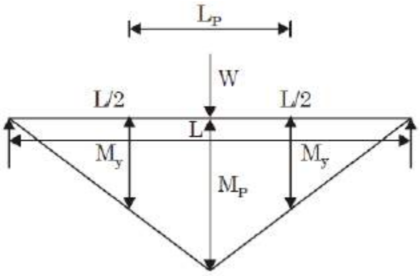
1) Static method: This consists of selecting the redundant forces, The free and redundant bending moment diagram is drawn for the structure. A combined bending moment diagram is drawn in such a way that a mechanism is formed. The collapse load is found by working out the equilibrium equation. It is checked that the bending moment is not more than the fully plastic moment at any section.

2) Kinematic method: This consists of locating the possible places of plastic hinges. The possible independent and combined mechanism are ascertained. The collapse load is found by applying the principle of virtual work. A bending moment diagram corresponding to the collapse mechanism is drawn and it is checked that the bending moment is not more than the fully plastic moment at any section.

For complicated frames, the static method of analysis is more difficult, and finding the correct equilibrium equation becomes illusive. In these cases, the kinematic method is more practical.

91. **A simply supported beam of span L supports a concentrated load W at its midspan. If the cross-section of the beam is circular, then the length of elastic-plastic zone of the plastic hinge will be**

Solution:



$$\frac{M_P}{M_y} = \frac{\frac{L}{2}}{\frac{L-L_P}{2}}$$

$$\frac{Mp}{My} = \frac{1}{1-\frac{L_P}{L}}$$

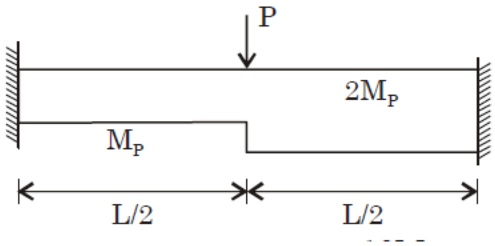
$$1-\frac{Lp}{L} = \frac{1}{\frac{M_P}{M_y}}$$

$$L_P = L\left[1-\frac{1}{\frac{M_P}{M_y}}\right]$$

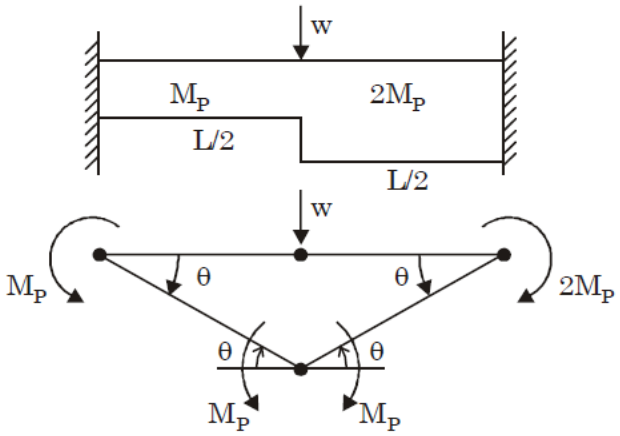
$$= L\left[1-\frac{1}{\text{Shape factor}}\right]$$

$$= L\left[1-\frac{1}{1.7}\right] = \frac{7L}{17}$$

92. **A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to**



Solution:



$$5M_P\theta = \frac{WL}{2}\theta$$

$$\Rightarrow \qquad W = \frac{10M_P}{l}$$

93. **Explain various types of bolt briefly.**

Answer:

<u>Types of bolts</u>
There are several types of bolts used to connect structural members. Some of them are listed below

(a) Black bolts or unfinished bolts

• Black bolts are referred to as ordinary, rough or common bolts. They are least expensive bolts and are made of low carbon steels (mild steel) with square or hexagonal head. The diameter of the hole is about 1.0 to 2.0 mm larger than the bolt diameter for ease in fitting. They are designated as Md× l, ‘d’ – shank diameter of bolt and , l – length of the bolt.

They are primarily used in light structures under static loads such as small trusses, purlins, bracings. They are also used as temporary fasteners during erection where HSFG bolts or welding are used as permanent fasteners.

• These bolts are not recommended for connections, which are subjected to impact load, vibration and fatigue.

• For bolt of a grade or property class 4.6 represents the ultimate tensile strength is 400 N/mm2 and yield strength is 0.6 times 400 which is 240N/mm2.

• Ordinary bolted joints, the force transfer through interlocking and bearing of bolts and joint is called bearing type joint.

(b) High Strength Friction Grip (HSFG) bolts

• High strength friction grip bolts are made from bars of medium carbon heat treated steel (high tensile steel). The bolt property class 10.9S and 12.9S are commonly used in steel connections.

• The HSFG bolts are available in sizes from 16mm to 36mm and are designated as M16, M20, M24 and M30.

• These bolts tightened (by torque wrenches) until they have very high tensile stresses, so that connected parts are clamped tightly together between the bolt head and nut, this permits load to be transferred primarily by friction not by shear.

• These bolts are most suitable for bridges where the stress reversal may occur or slippage is undesirable also for seismic loading and for fatigue load.

• High strength bolts have replaced rivets and black bolts are being used in structures, high raised building, bridges etc.

94. **Explain various steps involved in the design of laterally unsupported beam.**

Answer:

Steps involved are:

STEP 1: FIND OUT ULTIMATE LOAD ON BEAM.
Factored Ultimate Load (Factored Load) w = **1.5** × Working Load

STEP 2: FIND OUT MAXIMUM BENDING MOMENT (M) AND SHEAR FORCE (V) ON BEAM.

STEP 3: CALCULATE PLASTIC SECTION MODULUS REQURIED FOR TRIAL SECTION.

$$Z_{P(required)} = 1.3\frac{M\gamma_{mO}}{f_y}$$

STEP 4: SELECT SUITABLE SECTION BASED ON Z_p FROM IS: 800: 2007, PAGE NO. 138, 139. WRITE DOWN SE TIONAL PROPERTIES.

STEP 5: SECTION CLASSIFICATION.
a. Find out value of b/t_f and d/t_w . (refer Figure. 2, Page no. 19, IS 800: 2007 to find b and d)
t_f = thickness of flange t_w = thickness of web.
b. Refer Table 2, Page no. 18, IS 800: 2007 and classify the section semi-compact, compact, plastic or slender.

STEP 6: CHECK FOR SHEAR. (Clause no. 8.4.1., Page no. 59, IS 800: 2007)
a. Find out design shear strength V_d.

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{mo}} h t_w$$

b. Beam is checked for high / low shear case
$V \le 0.6\ V_d$ low shear case
$V > 0.6\ V_d$ high shear case

STEP 7: CHECK FOR BENDING.

a. For low shear Case (Clause no. 8.2.2, Page no. 54, IS 800: 2007)

$M_d > M$

M_d = Design Bending Strength and M = Bending Moment

$$M_d = \beta_b Z_p f_{bd}$$

$$f_{bd} = \frac{\chi_{LT} f_y}{\gamma_{mo}}$$

$$\chi_{LT} = \frac{1}{\varphi_{LT} + (\varphi^2{}_{LT} - \lambda^2{}_{LT})^{0.5}}$$

β_b = 1 for plastic and compact sections.

= Z_e/Z_p for semi compact sections.

Z_e = Elastic section Modulus

Z_p = Plastic section Modulus

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}}$$

$$M_{cr} = \sqrt{\frac{\pi^2 EI}{L_{LT}{}^2}\left(GI_t + \frac{\pi^2 EI_w}{L_{LT}{}^2}\right)}$$

Where,

$$I_w = (1 - B_f) B_f I_y h_y{}^2$$

$$I_t = \Sigma \frac{b_i t_i^3}{3}$$

$$G = \frac{E}{2(1+\mu)}$$

I_w = warping constant (page no. 129 , IS 800)

$B_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5$ (for symmetrical section $I_{fc} = I_{ft}$).

h_y = distance between shear center of flanges.

L_{LT} =eff. len. for lateral torsional buckling (table15,pg58)

$\phi = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda^2{}_{LT}]$

α_{LT} = 0.21 for rolled section.

b. For High shear Case (Clause no. 8.2.1.3, Page no. 53, IS 800: 2007)

Refer Clause no. 8.2.1.3, Page no. 53, IS 800: 2007. Generally low shear case is preferred.

STEP 8: CHECK FOR WEB BUCKLING AT SUPPORT (Clause no. 8.7.3.1, Page no. 67, IS 800: 2007)

a. Capacity of section = $A_b f_{cd} > V$

b. $A_b = (b_1 + n_1)\, t_w$ when load is at support

$A_b = (b_1 + 2n_1)\, t_w$ when load is not at support

Where, b_1= stiff bearing length of load = assume between 0 to 100mm

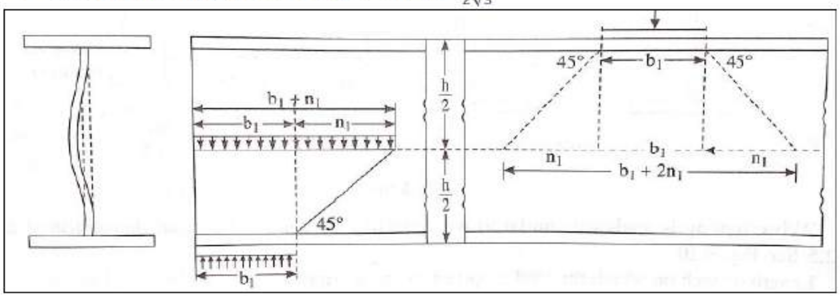
n_1 = for 45° dispersion consider h/2

d. Find out F_{cd} = Design Compressive Stress considering class c and f_y = 250 MPa.

Slenderness ratio $= \frac{kl}{r} = \frac{0.7d}{r}$

D = depth of the web between the flanges

r = least radius of gyration of the section = $\frac{t_w}{2\sqrt{3}}$



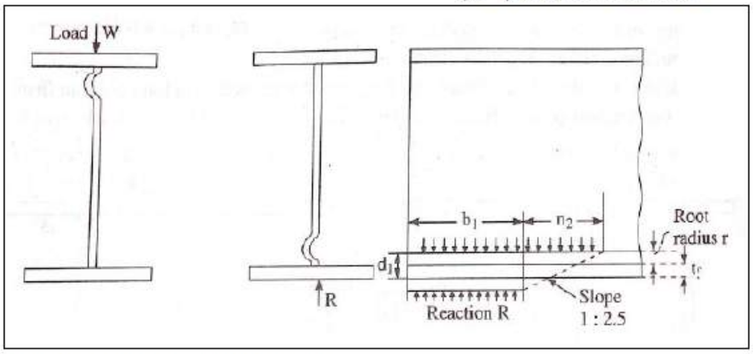
STEP 9: CHECK FOR WEB CRIPPLING (Clause no. 8.7.4, Page no. 67, IS 800: 2007)

Design crippling strength $F_w = \frac{(b_1+n_2)t_w f_{yw}}{\gamma_{mo}} > V$

Where, b_1 = stiff bearing length = 0 to 100 mm

$n_2 = 2.5\ (t_f + r_1)$

f_{yw} = yield stress of web



STEP 10: CHECK FOR DEFLECTION

a. Actual deflection for simply supported

$$\delta_{max} = \frac{5}{384}\frac{wl^4}{EI}$$

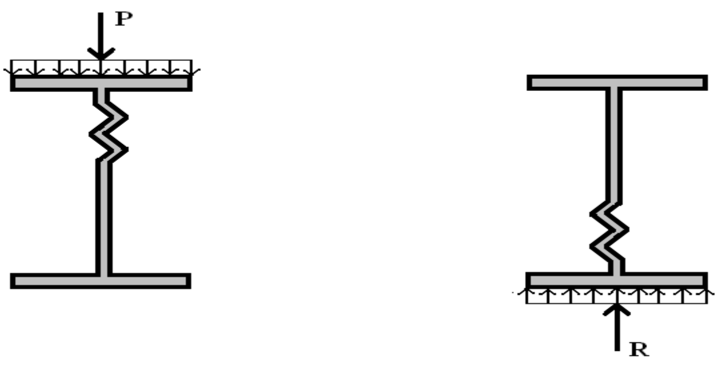
b. Permissible deflection = Span/300 (table 6, Page no. 31, IS 800: 2007)

95. **Explain web crippling and web buckling with the help of sketch.**

Answer:

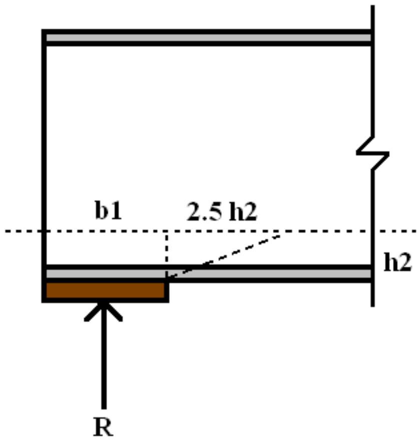
<u>Web crippling:</u>

Web crippling causes local crushing failure of web due to large bearing stresses under reactions at supports or concentrated loads. This occurs due to stress concentration because of the bottle neck condition at the junction between flanges and web. It is due to the large localized bearing stress caused by the transfer of compression from relatively wide flange to narrow and thin web. Web crippling is the crushing failure of the metal at the junction of flange and web. Web crippling causes local buckling of web at the junction of web and flange.



Under Concentrated Load   Under Support

WEB CRIPPLING OF BEAMS



Web crippling calculations

For safety against web crippling, the resisting force shall be greater than the reaction or the concentrated load. It will be assumed that the reaction or concentrated load is dispersed into the web with a slope of 1 in 2.5 as shown in the figure

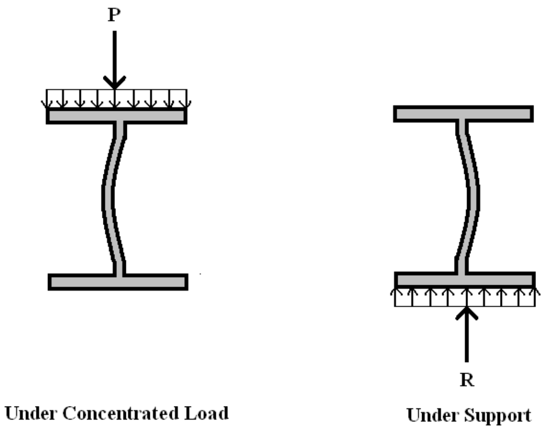
Let Resisting force = Fwc
Thickness of web = tw
Yield stress in web = fyw
Width of bearing plate = b1
Width of dispersion = n2 = 2.5 h2
Depth of fillet = h2 (from SP [6])
Fwc = [(b1 + n2) tw fyw] / y_{mo} ≥ Reaction, R_u

For concentrated loads, the dispersion is on both sides and the resisting force can be expressed as

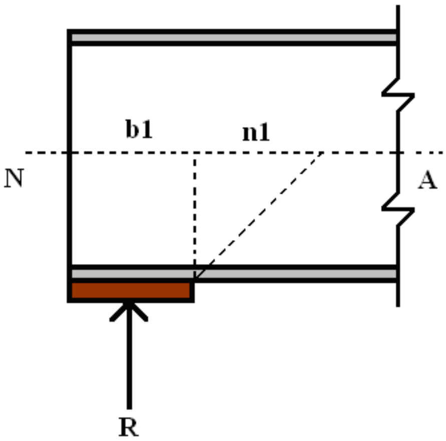
Fwc = [(b1 + 2 n2) tw fyw] / y_{mo} ≥Concentrated load, W_u

Web Buckling:

The web of the beam is thin and can buckle under reactions and concentrated loads with the web behaving like a short column fixed at the flanges. The unsupported length between the fillet lines for I sections and the vertical distance between the flanges or flange angles in built up sections can buckle due to reactions or concentrated loads. This is called web buckling.



WEB BUCKLING OF BEAMS



Web Buckling Calculations

For safety against web buckling, the resisting force shall be greater than the reaction or the concentrated load. It will be assumed that the reaction or concentrated load is dispersed into the web at 45° as shown in the figure.

Let Resisting force = Fwb
Thickness of web = tw
Design compressive stress in web = fcd
Width of bearing plate = b1
Width of dispersion = n1
Fwb = (b1 + n1) tw fcd ≥ Reaction, R_u

For concentrated loads, the dispersion is on both sides and the resisting force can be expressed as

Fwc = [(b1 + 2 n1) tw fcd] ≥Concentrated load, W_u

The design compressive stress fcd is calculated based on a effective slenderness ratio of 0.7 d / r_y, where d = clear depth of web between the flanges.
r_y = radius of gyration about y-y axis and is expressed as
= sqrt (Iyy / area) = sqrt [$(t_w)^3$ / 12/ t] = sqrt [$(t_w)^2$ / 12]

kl / r_y = (0.7 d) / sqrt [$(t_w)^2$ / 12] = 2.425 * d / tw

96. **Explain various steps involved in the design of laterally supported beam as per IS 800 (2007).**

Answer:

Various steps involved are:

1. Calculate the factored load and the maximum bending moment and shear force
2. Obtain the plastic section modulus required

$$Z_{req} = \frac{(M \times \gamma_{mo})}{fy}$$

Select a suitable section for the beam-ISLB, ISMB, ISWB or suitable built up sections (doubly symmetric only). (Doubly symmetric, singly symmetric and asymmetric- procedures are different)

3. Check for section classification such as plastic, compact, semi-compact or slender. Most of the sections are either plastic or compact. Flange and web criteria.

$$\frac{d}{t_w}, \frac{b}{t_f}, \varepsilon = \sqrt{\frac{250}{fy}} = 1$$

4. Calculate the design shear for the web and is given by

$$V_{dp} = \frac{(Av \times fy)}{\sqrt{3} \times \gamma_{mo}} > V_d \; and, V < 0.6V_d$$

5. Calculate the design bending moment or moment resisted by the section (for plastic and compact)

$M_d = \beta_p$ x Z_p x f_y / γ_{mo}

6. Check for buckling
7. Check for crippling or bearing
8. Check for deflection

97. **At the location of plastic hinge**
(a) Radius of curvature is infinite
(b) Curvature is infinite
(c) Moment is infinite
(d) Flexural stress is infinite

Answer: b

98. **A ductile structure is defined as one for which the plastic deformation before fracture**
(a) is smaller than the elastic deformation
(b) vanishes
(c) is equal to the elastic deformation
(d) is much larger than the elastic deformation

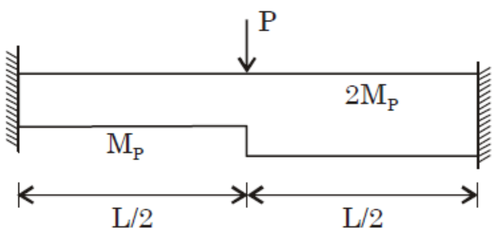
Answer: d

99. **Assertion (A):** The shape factor of a circular section is less than that of a rectangular section.
Reason (R): Compared to rectangular section, a circular section has more area near the neutral axis than at the extreme fibre.
Of these statements:
(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explantion of A

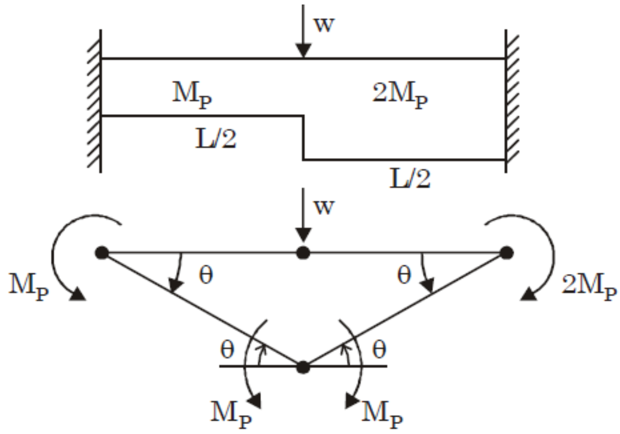
(c) A is true but R is false
(d) A is false but R is true

Answer: d

100. A fixed beam made of steel is shown in the figure below. At collapse, the value of load P will be equal to



Solution:



$$5M_p\theta = \frac{WL}{2}\theta$$

$$\Rightarrow \qquad W = \frac{10M_P}{l}$$

Structural Steel Design Project	Job No:	Sheet *1 of 2*	Rev
	Job Title: *MAXIMUM FACTORED LOADS*		
	Worked Example - 1		
Calculation Sheet		Made by *SSSR*	Date *15-09-99*
		Checked by *RN*	Date *20-09-99*

A frame sketched in Fig. 2 is loaded by a dead load of 6 kN/m, imposed load of 20 kN/m and wind load of 10 kN/m. The example below illustrates the checks in respect of the following.

- *Imposed load + Dead load*
- *Wind load + Dead load*
- *Imposed load + Wind load + Dead load*

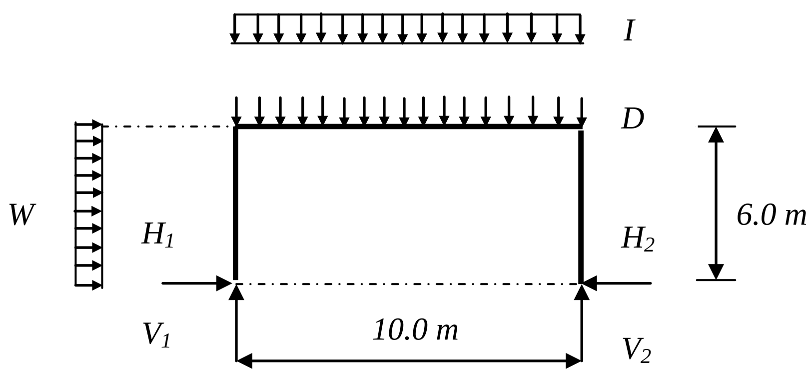


Fig. 2 Portal frame subject to loading

Dead Load (D) 6 kN/m

Imposed Load (I) 20 kN/m

Wind Load (W) 10 kN/m

Case1 - Dead plus imposed loads

$$V_1 = V_2 = (1.5I + 1.35D) * span/2$$
$$= (1.5 * 20 + 1.35 * 6) * 5 = 190.5 \text{ kN}$$

$\gamma_{fDL} = 1.35$
$\gamma_{fIL} = 1.50$

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet *2 of 2*	Rev
	Job Title: *MAXIMUM FACTORED LOADS*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-09-00*
		Checked by *RN*	Date *20-09-99*

Case 2 - Dead plus wind

Taking moments about right support,

$V_1 = [1.35\ D\ span^2/2 - 1.50\ W * height^2/2]\ 1/10$

$= [1.35 * 6 * 100/2 - 1.50 * 10 * 36/2]\ 1/10$

$= 13.5\ kN$

$V_2 = 1.35D * span - V_1$

$= 1.35 * 6 * 10 - 13.5 = 67.5\ kN$

$H_1 + H_2 = 1.35\ W * height = 1.50 * 10 * 6 = 90\ kN$

(Note: The evaluation of H_1 and H_2 will depend on the stiffnesses of the members as the problem is statically indeterminate)

$\gamma_{fDL} = 1.35$
$\gamma_{fWL} = 1.50$

Case 3 - Dead plus imposed plus wind

$\gamma_{fDL} = 1.35$
$\gamma_{fIL} = 1.50$
$\gamma_{fWL} = 1.05$

$V_1 = 1.35 * D * span/2 + 1.5 * I * span/2 - 1.05 * W * height^2 / (2 * span)$

$= 1.35 * 6 * 5 + 1.5 * 20 * 5 - 1.05 * 10 * 36/20$

$= 171.6\ kN$

$V_2 = 1.35 * D * span/2 + 1.5 * I * span/2 + 1.05 * W * height^2/2 * span$

$= 1.35 * 6 * 5 + 1.5 * 20 * 5 + 1.05 * 10 * 36/20$

$= 209.4\ kN$

The worst value for design purposes are;

$V_1 = 190.5\ kN$; $V_2 = 209.4\ kN$

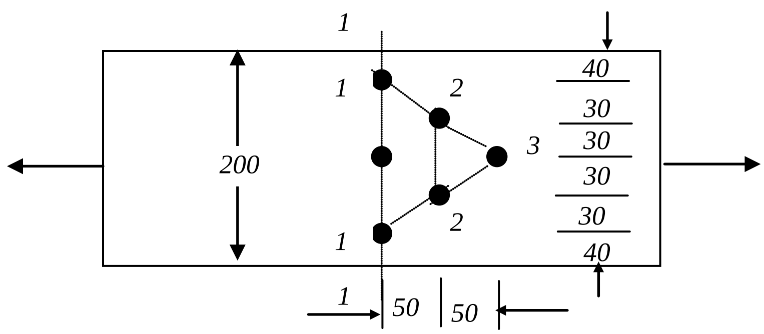
Structural Steel Design Project	Job No:	Sheet: *1 of 1*	Rev:
	Job Title: *Tension Member Example*		
	Worked Example*: 1*		
		Made by *SSSR*	Date: *3-1-2000*
Calculation Sheet		Checked by VK	Date

PROBLEM 1:

Determine the desig tensile strength of the plate (200 X 10 mm) with the holes as shown below, if the yield strength and the ultimate strength of the steel used are 250 MPa and 420 MPa and 20 mm diameter bolts are used.

$f_y = 250\ MPa$

$f_u = 420\ MPa$



Calculation of net area, A_{net}:

$$A_n \text{ (section 11)} = (200-3*21.5)*10 = 1355\ mm^2\ (governs)$$

$$A_n \text{ (section 1221)} = \left(200-4*21.5+\frac{2*50^2}{4*30}\right)*10 = 1557\ mm^2$$

$$A_n \text{ (section 12321)} = \left(200-5*21.5+\frac{4*50^2}{4*30}\right)*10 = 1758\ mm^2$$

P_t is lesser of

$$(i)\ A_g\ f_y/\gamma M_0 = \frac{200*10*250/1.15}{1000} = 434.8\ kN$$

$$(ii)\ 0.9\, A_n\, f_u/\gamma M_1 = \frac{0.9*1342*420/1.25}{1000} = 405.8\ kN$$

$$P_t = 405.8\ kN$$

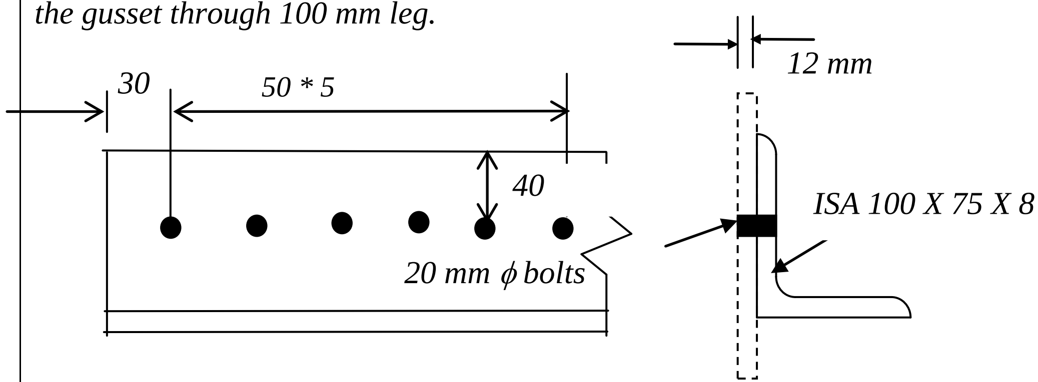
Efficiency of the plate with holes $= \dfrac{P_t}{A_g\ f_y/\gamma_{M_0}} = \dfrac{409.8}{434.8} = 0.93$

Structural Steel Design Project	Job No:	Sheet: *1 of 4*	Rev
	Job Title: *Tension Member Example*		
	Worked Example*: 2*		
		Made by *SSSR*	Date *3-1-2000*
Calculation Sheet		Checked by VK	Date

PROBLEM 2:

Analysis of single angle tension members

A single unequal angle 100 X 75 X 8 mm is connected to a 12 mm thick gusset plate at the ends with 6 nos. 20 mm diameter bolts to transfer tension. Determine the design tensile strength of the angle. (a) if the gusset is connected to the 100 mm leg, (b) if the gusset is connected to the 70 mm leg, (c) if two such angles are connected to the same side of the gusset through the 100 mm leg. (d) if two such angles are connected to the opposite sides of the gusset through 100 mm leg.



a) *The 100mm leg bolted to the gusset* :

A_{nc} = (100 - 8/2 - 21.5) *8 = 596 mm^2.

A_o = (75 - 8/2) * 8 = 568.mm^2

A_g = ((100-8/2) + (75 – 8/2)) * 8 =1336 mm^2

Strength as governed by tearing of net section:

Since the number of bolts = 4; β = 1.0

$$P_t = A_{nc}\, f_u/\gamma_{M1} + \beta A_o\, f_y/\gamma_{M0}$$

$$= 596 * 420/1.25 + 1.0 * 568 * 250 / 1.15$$

$$= 323734 \text{ N (or) } 323.7 \text{ kN}$$

Structural Steel Design Project	Job No:	Sheet: *2* of *4*	Rev
	Job Title: *Tension Member Example*		
	Worked Example *:2*		
		Made by *SSSR*	Date *3-1-2000*
Calculation Sheet		Checked by VK	Date

<u>Strength as governed by yielding of gross section:</u>

$$P_t = A_g f_y/\gamma_{M0}$$

$$= 1336 * 250 / 1.15 = 290435 \text{ N (or) } 290.4 \text{ kN}$$

<u>Block shear strength</u>

$$P_v = (0.62 A_{vg}\ f_y/\gamma_{M0} + A_{tn}\ f_u/\gamma_{M1})$$
$$= 0.62 * (5 * 50 + 30) * 8 * 250/1.15 + (40-21.5/2) * 8 * 420/1.25$$
$$= 380537 \text{ N} = 380.5 \text{ kN}$$

or

$$P_v = (0.62 A_{vn}\ f_u/\gamma_{M1} + A_{tg}\ f_y/\gamma_{M0})$$

$$= (0.62 (5 * 50 + 30 - 5.5 * 21.5) * 8 * 420 / 1.25$$

$$+ 40 * 8 * 250/ 1.15)$$

$$= 339131 \text{ N} = 339.1$$

The design tensile strength of the member = 290.4 kN

The efficiency of the tension member, is given by

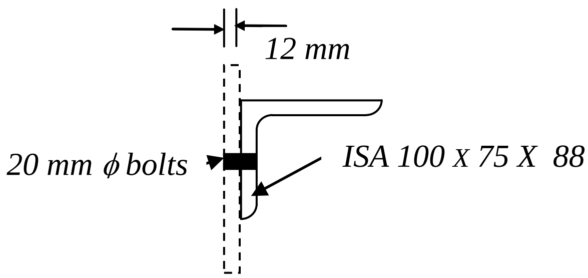
$$\eta = \frac{P_t}{A_g\ f_y} = \frac{290.4 * 1000}{(100 + 75 - 8) * 8 * 250/1.15} = 1.0$$

b) <u>The 75 mm leg is bolted to the gusset:</u>

$$A_{nc} = (75 - 8/2 - 21.5) * 8 = 396 \text{ mm}^2$$

$$A_o = (100 - 8/2) * 8 = 768 \text{ mm}^2$$

Structural Steel Design Project	Job No:	Sheet: *3* of *4*	Rev
	Job Title: *Tension Member Example*		
	Worked Example*: 2*		
		Made by *SSSR*	Date *3-1-2000*
Calculation Sheet		Checked by VK	Date



Strength as governed by tearing of net section:

Since the number of bolts = 6 $\quad\beta = 1.0$

$$P_t = A_{nc}\, f_u/\gamma_{M1} + \beta A_o\, f_y/\gamma_{M0}$$

$$= 396 * 420/1.25 + 1.0 * 768 * 250 / 1.15$$

$$= 300123 \text{ N (or) } 300.1 \text{ kN}$$

Strength as governed by yielding of gross section:

$$P_t = A_g\, f_y / \gamma_{M0}$$

$$= 1336 * 250 / 1.15 \quad = 290435 \text{ N (or) } 290.4 \text{ kN}$$

Block shear strength:

$$P_v \le (0.62\, A_{vg}\, f_y/\gamma_{M0} + A_{tn}\, f_u/\gamma_{M1})$$

$$= 0.62 * (5 * 50 + 30) * 8 * 250/1.15$$

$$+ (35 - 21.5/2) * 8 * 420/1.25$$

$$= 367097 \text{ N} = 367.1 \text{ kN}$$

$$P_v \le (0.62\, A_{vn}\, f_u/\gamma_{M1} + A_{tg}\, f_y/\gamma_{M0})$$

$$= (0.62\, (5 * 50 + 30 - 5.5 * x\ 21.5) * 8 * 420 / 1.25$$

$$+ 35 * 8 * 250/ 1.15$$

$$= 330435 \text{ N} = 330.4 \text{ kN}$$

The design tensile strength of the member = 290.4 kN

Structural Steel Design Project	Job No:	Sheet: *4* of *4*	Rev
	Job Title: *Tension Member Example*		
	Worked Example *: 2*		
		Made by *SSSR*	Date *3-1-2000*
Calculation Sheet		Checked by VK	Date

Even though the tearing strength of the net section is reduced, the yielding of the gross section still governs the design strength.

The efficiency of the tension member is as before 1.0

Note: *The design tension strength is more some times if the longer leg of an unequal angle is connected to the gusset (when the tearing strength of the net section governs the design strength).*

An understanding about the range of values for the section efficiency, η, is useful to arrive at the trial size of angle members in design problems.

(c & d)The double angle strength would be twice single angle strength as obtained above in case (a)

$P_t = 2 * 290.4 = 580.8$ kN

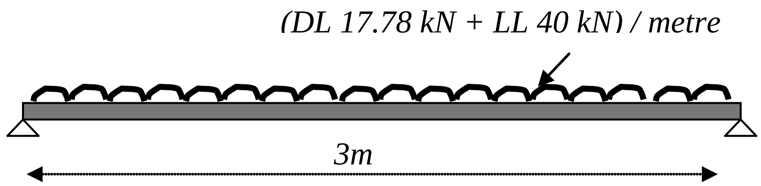
Structural Steel Design Project	Job No. Ex .1	Sheet *1 of 4*	Rev.
	Job. Title: *LATERALLY RESTRAINED BEAMS*		
	Worked Example -1		
	Made by *SAJ*	Date *21.03.2000*	
Calculation Sheet	Checked by *SS*	Date *26.03.2000*	

$\gamma_{LD} = 1.35$

$\gamma_{LL} = 1.50$

EXAMPLE: 1

Design a suitable 'I' beam for a simply supported span of 3 m and carrying a dead or permanent load of 17.78 kN/m and an imposed load of 40 kN/m. Assume full lateral restraint and stiff support bearing of 100 mm.



Design load calculation:

factored load = $\gamma_{LD} \times 17.78 + \gamma_{LL} \times 40$ *kN*

in this example the following load factors are chosen.

γ_{LD} *and* γ_{LL} *are taken as 1.35 and 1.50 respectively.*

γ_{LD} *– partial safety factor for dead or permanent loads*
γ_{LL} *– partial safety factor for live or imposed loads*

Total factored load = $1.35 \times 17.78 + 1.5 \times 40.0 = 84$ *kN / m*

Factored bending moment $= 84 \times 3^2 / 8 = 94.50$ *kN – m*

Z—value required for $f_y = 250$ *MPa ;* $\gamma_m = 1.15$

$$Z_{reqd} = \frac{94.5 \times 1000 \times 1000 \times \gamma_m}{250}$$

$$Z_{reqd} = 434.7 \text{ cm}^3$$

Structural Steel Design Project	Job No. Ex .1	Sheet 2 of 4	Rev.
	Job. Title: *LATERALLY RESTRAINED BEAMS*		
	Worked Example - 1		
Calculation sheet	Made by *SAJ*	Date *21.03.2000*	
	Checked by *SS*	Date *26.03.2000*	

Try ISMB 250

$$\varepsilon = \sqrt{\frac{250}{250}} = 1.0 \qquad D = 250\ mm$$

$B = 125\ mm$

$t = 6.9\ mm$

$T = 12.5\ mm$

$I_{xx} = 5131.6\ cm^4$

$I_{yy} = 334.5\ cm^4$

Section classification:

Flange criterion $= B/2T = 5.$

Web criterion $= (D - 2T)/t = 32.61$

Since $B/2T < 8.92\ \varepsilon$ & $(D-2T)/t < 82.95\ \varepsilon$

The section is classified as ' ***PLASTIC*** *'*

Moment of resistance of the cross section:

Since the section considered is 'PLASTIC'

$$M_C = \frac{S \times f_y}{\gamma_m}$$

Where S is the plastic modulus

'S' for ISMB 250 $= 459.76\ cm^3$

$$M_c = 459.76 \times 1000 \times 250 / 1.15$$

$$= 99.95\ kN\text{-}m > 94.5\ kN\text{-}m$$

Hence ***ISMB-250*** *is adequate.in flexure.*

Structural Steel Design Project **Calculation sheet**	Job No. Ex .1	Sheet *3 of 4*	Rev.
	Job. Title: *LATERALLY RESTRAINED BEAMS*		
	Worked Example- 1		
	Made by *SAJ*	Date *21.03.2000*	
	Checked by *SS*	Date *26.03.2000*	

Shear resistance of the cross section:

This check needs to be considered more importantly in beams where the maximum bending moment and maximum shear force may occur at the same section simultaneously, such as the supports of continuous beams. For the present example this checking is not required. However for completeness this check is presented.

Shear capacity $P_v = \dfrac{0.6\, f_y A_v}{\gamma_m}$

$A_v = 250 \times 6.9 = 1725\ mm^2$

$P_v = 0.6 \times 250 \times 1725 / 1.15 = 225\ kN$

F_v = *factored max shear* $= 84 \times 3 / 2 = 126.\ kN$

$F_v / P_v = 126/225.0 = 0.56 < 0.6$

Hence the effect of shear need not be considered in the moment capacity calculation.

Check for Web Buckling:

The slenderness ratio of the web $= L_E/r_y = 2.5\ d/t = 2.5 \times 194.1/6.9$

$= 70.33$

The corresponding design compressive stress f_c *is found to be*

$f_c = 203\ MPa$ *(Design stress for web as fixed ended column)*

Stiff bearing length = 100 mm

45^0 *dispersion length* $n_1 = 125.0\ mm$

$P_w \quad (100 + 125.0) \times 6.9 \times 203.0$

$= 315.16\ kN$

$315.16 > 126$ *Hence web is safe against shear buckling*

Structural Steel Design Project	Job No. Ex .1	Sheet *4 of 4*	Rev.
	Job. Title: *LATERALLY RESTRAINED BEAMS*		
	Worked Example - 1		
	Made by *SAJ*	Date *21.03.2000*	
Calculation sheet	Checked by *SS*	Date *26.03.2000*	

Check for web crippling at support

Root radius of ISMB 250 = 13 mm

Thickness of flange + root radius = 25.5 mm

Dispersion length (1:2.5) n_2 *= 2.5 x 25.5 = 63.75 mm*

P_{crip} = $(100+63.75) \times 6.9 \times 250 / 1.15$

= *245.63 kN > 126kN*

Hence ISMB 250 has adequate web crippling resistance

Check for serviceability – Deflection:

Load factors for working loads γ_{LD} *and* γ_{LL} *= 1.0*

design load = 57.78 kN/m.

$$\delta = \frac{5 \times 57.78 \times 3000^4}{384 \times 2.1 \times 10^5 \times 5131.6 \times 10^4}$$

Max deflection $= 5.65\,mm$

$$= \frac{L}{531}$$

$$\frac{L}{531} < \frac{L}{200}$$

Hence serviceability is satisfied

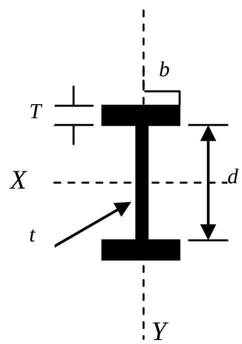
Result :-- Use ISMB – 250.

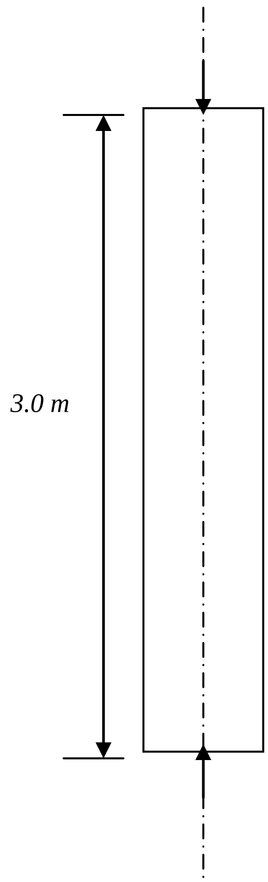
Structural Steel Design Project Calculation Sheet	Job No:	Sheet ***1 of 5***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date 2*3-09-99*
		Checked by *RN*	Date 2*8-09-99*

Obtain factored axial load on the column section ISHB400. The height of the column is 3.0m and it is pin-ended.

[$f_y = 250\ N/mm^2$; $E = 2*10^5\ N/mm^2$; $\gamma_m = 1.15$]

CROSS-SECTION PROPERTIES:





Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***2 of 5***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date *23-09-99*
		Checked by *RN*	Date *28-09-99*

Flange thickness = T = *12.7 mm*

Clear depth between flanges = d = *400 – (12.7 * 2) = 374.6 mm*

Thickness of web = t = *10.6 mm*

Flange width = $2b$ = *250 mm*

b = *125 mm*

Self –weight = w = *0.822 kN/m*

Area of cross-section = A = *10466 mm²*

r_x = *166.1 mm*

r_y = *51.6 mm*

(i) Type of section:

$$\frac{b}{T} = \frac{125}{12.7} = 9.8 < 10\in$$

$$\frac{d}{t} = \frac{374.6}{10.6} = 35.3 < 41\in$$

BS: 5950
Part - 1
Table - 7

$$where, \in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Hence, cross- section is "COMPACT"

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***3 of 5***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date 2*3-09-99*
		Checked by *RN*	Date 2*8-09-99*

(ii) Effective Length:

As, both ends are pin-jointed effective length = $\ell_x = \ell_y = 3.0$ *m*

(iii) Slenderness ratios:

$$\lambda_x = \frac{\ell_x}{r_x} = \frac{3000}{166.1} = 18.1$$

$$\lambda_y = \frac{\ell_y}{r_y} = \frac{3000}{51.6} = 58.1$$

(iv) Values of (α) :

For rolled I-sections,

In x – direction $\alpha_x = 0.0020$
In y – direction $\alpha_y = 0.0035$

$$\lambda_o = 0.2\pi \sqrt{E/f_y} = 0.2 * \frac{22}{7}\sqrt{\frac{200000}{250}}$$

$$= 17.8$$

(v) values of η:

$$\eta = \alpha\ (\lambda - \lambda_0)$$

$$\eta_x = \alpha_x\ (\lambda_x - \lambda_0) = 0.002 * (18.1 - 17.8) = 0.001$$

$$\eta_y = \alpha_y\ (\lambda_y - \lambda_0) = 0.0035 * (58.1 - 17.8) = 0.141$$

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***4 of 5***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date *23-09-99*
		Checked by *RN*	Date *28-09-99*

(vi) Calculation of maximum compressive stress at failure (σ_c):

We have,

$$\sigma_e = \frac{\pi^2 E}{\lambda^2}$$

$$\sigma_c = \phi \pm \sqrt{\phi^2 - f_y \sigma_e} \le f_y$$

$$\text{where, } \phi = \frac{f_y + (\eta + 1)\,\sigma_e}{2}$$

In x-direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(18.1)^2} = 6025 \ N/mm^2$$

$$\varphi_x = \frac{250 + (0.001 + 1)*6025}{2} = 3140 \ N/mm^2$$

$$\sigma_{cx} = 3140 \pm \sqrt{(3140)^2 - 250*6025} \le 250$$

$$= 250 \ N/mm^2$$

Structural Steel Design Project	Job No:	Sheet *5 of 5*	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date 23-09-99
Calculation Sheet		Checked by *RN*	Date 28-09-99

In y-direction,

$$\sigma_{ey} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\varphi_y = \frac{250 + (0.141 + 1)585.0}{2} = 459 N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250 * 585} \leq 250$$

$$= 205\ N/mm^2$$

Hence, Allowable axial compressive stress, $\sigma_c = 205\ N/mm^2$

Safe axial compressive stress $= \sigma_c/\gamma_m = 205/1.15 = 178\ N/mm^2$

(vii) Factored Load:

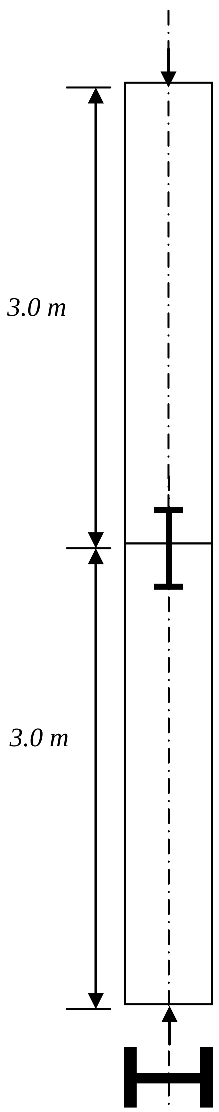
$$\text{Factored Load} = \sigma_c A/\gamma_m = 178 * 10466/1000$$

$$= 1863\ kN$$

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *AXIALLY COMPRESSED COLUMN*		
	Worked Example - 2		
Calculation Sheet		Made by *SSSR*	Date 2*3-09-99*
		Checked by *RN*	Date 2*8-09-99*

Obtain maximum axial load carried by the column shown when ISHB 400 is employed. The column is effectively restrained at mid-height in the y-direction, but is free in x-axis. The data is the same as in problem1.

[$f_y = 250\ N/mm^2$; $E = 2.0*10^5\ N/mm^2$; $\gamma_m = 1.15$]



(i) Type of section:

Section is "COMPACT" from previous example.

(ii) Effective lengths:

$$\ell_x = 6000\ mm$$
$$\ell_y = 3000\ mm$$

(iii) Slenderness ratios:

$$\lambda_x = \frac{6000}{166.1} = 36.1$$

$$\lambda_y = \frac{3000}{51.6} = 58.1$$

(iv) Values of (α) and (η):

In x – direction $\alpha_x = 0.002$
In y – direction $\alpha_y = 0.003$

and $\lambda_0 = 17.8$

$$\eta_x = 0.002\,(36.1 - 17.8) = 0.037$$
$$\eta_y = 0.0035\,(58.1 - 17.8) = 0.141$$

<table>
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</table>

(v) Calculation of σ_c :

In x- direction,

$$\sigma_{ex} = \frac{\pi^2 E}{\lambda_x^2} = \frac{\pi^2 * 200000}{(36.1)^2} = 1515\, N/mm^2$$

$$\phi_x = \frac{250 + (0.037+1)*1515}{2} = 911\, N/mm^2$$

$$\sigma_{cx} = 911 \pm \sqrt{(911)^2 - 250*1515} \le 250$$

$$= 239\ N/mm$$

In y-direction

$$\sigma_{cy} = \frac{\pi^2 E}{\lambda_y^2} = \frac{\pi^2 * 200000}{(58.1)^2} = 585\ N/mm^2$$

$$\phi_y = \frac{250 + (0.141+1)*585}{2} = 459\, N/mm^2$$

$$\sigma_{cy} = 459 \pm \sqrt{(459)^2 - 250*585} \le 250$$

$$= 205\, N/mm^2$$

(vi) Factored Load:

Factored Load = $\sigma_c A/\gamma_m$ = 205/1.15 10466/1000 = 1863 kN*

<table>
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<tr><td colspan="3">Job Title: BASE PLATE</td></tr>
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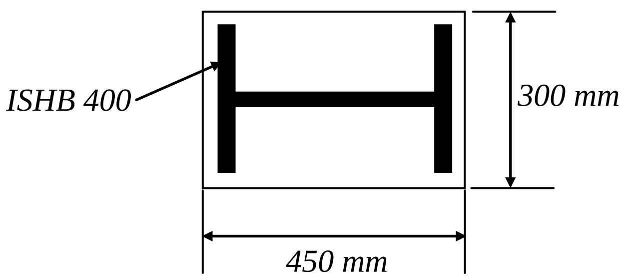
<tr><td>Checked by RN</td><td>Date 28-2-00</td></tr>
</table>

Design a simple base plate for a ISHB400 @ 0.822 kN/m column to carry a factored load of 1800 kN.

[f_{cu} = 40 N/mm^2 ; f_y = 250 N/mm^2 ; γ_m = 1.15]



Bearing strength of concrete = $0.4f_{cu}$ = 0.4 * 40 = 16 N/mm^2

Area required = 1800*10^3/16 = 112500 mm^2

Use plate of 450 X 300 mm (135000 mm^2)

Assuming projection of 25 mm on each side

w = (1800 * 10^3) / (450 * 300) = 13.33 N/mm^2

f_{yp} = 250/1.15 = 217.4 N/mm^2

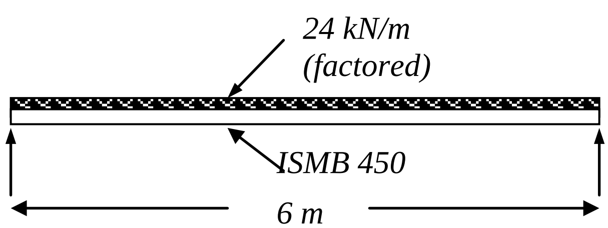
$$t_p = \sqrt{\frac{2.5w\left(a^2 - 0.3b^2\right)}{f_{yp}}} = \sqrt{\frac{2.5 \times 13.33\left(25^2 - 0.3 \times 25^2\right)}{217.4}} = 8.2\ mm$$

Hence, use 450 X 300 X 10 mm plate.
(Design of connections are discussed in later chapters)

Structural Steel Design Project Calculation sheet	Job No.	Sheet *1* of *4*	Rev.
	Job title: *UNRESTRAINED BEAM DESIGN*		
	Worked example: *1*		
		Made by. *SSR*	Date.*1/3/2000*
		Checked by. *SAJ*	Date.*5/3/ 2000*

Problem - 1

Check the adequacy of ISMB 450 to carry a uniformly distributed load of 24 kN / m over a span of 6 m. Both ends of the beam are attached to the flanges of columns by double web cleat.

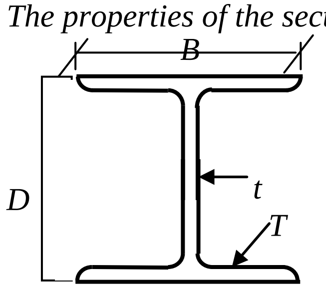


Design check:

For the end conditions given, it is assumed that the beam is simply supported in a vertical plane, and at the ends the beam is fully restrained against lateral deflection and twist with, no rotational restraint in plan at its ends.

Section classification of ISMB 450

The properties of the section are:



Depth, D = 450 mm

Width, B = 150 mm

Web thickness, t = 9.4 mm

Flange thickness, T = 17.4 mm

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	Job title: *UNRESTRAINED BEAM DESIGN*		
	Worked example*: 1*		
		Made by. *SSR*	Date.*1/3/2000*
Calculation sheet		Checked by. *SAJ*	Date. *5/3/ 2000*

Depth between fillets, d = 379.2 mm

Radius of gyration about minor axis, $r_y = 30.1$ *mm*

Plastic modulus about major axis, $S_x = 1512.8 * 10^{-3}$ mm^3

Assume $f_y = 250$ N/mm^2, $E = 200000$ N/mm^2, $\gamma_m = 1.15$,

$p_y = f_y / \gamma_m = 250 / 1.15 = 217.4$ N / mm^2

(I) Type of section

Flange criterion:

$$b = \frac{B}{2} = \frac{150}{2} = 75 \text{ mm}$$

$$\frac{b}{T} = \frac{75.0}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon \quad \text{where } \varepsilon = \sqrt{\frac{250}{f_y}}$$

Hence O.K.

Web criterion:

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\,\varepsilon$$

Hence O.K.

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</table>

Since $\frac{b}{T} < 8.92\,\varepsilon$ *and* $\frac{d}{t} < 82.95\,\varepsilon$, *the section is classified as 'plastic'*

(II)Check for lateral torsional buckling:

Equivalent slenderness of the beam, $\lambda_{LT} = n\,u\,v\,\lambda$

where, n *= slenderness correction factor (assumed value of 1.0)*

u *= buckling parameter (assumed as 0.9)*

λ *= slenderness of the beam along minor axis*

$$= \frac{6000}{30.1} = 199.33$$

v *= slenderness factor (which is dependent on the proportion of the flanges and the torsional index [D / T])*

= 0.71 (for equal flanges and λ *= 199.33)* — *Table 14 of BS5950 Part I*

Now, $\lambda_{LT} = 1.0 * 0.9 * 0.71 * 199.33$

$= 127.37$

Bending strength, p_b *= 84 Mpa (for* λ_{LT} *= 127.37) (from Table 11 of BS 5950 Part I)*

Buckling resistance moment $M_b = S_x * p_b$

$= (1512.78 * 84)/1000$

<table>
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<tr><td colspan="3">Job title: UNRESTRAINED BEAM DESIGN</td></tr>
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Date. 5/3/ 2000</td></tr>
</table>

$$= 127.07\ kN\ m$$

For the simply supported beam of 6.0 m span with a factored load of 24.0 KN/m

$$M_{max} = \frac{w\ell^2}{8} = \frac{24*6^2}{8}$$

$$= 108.0\ KN\ m\ <\ 127.07\ kN\ m$$

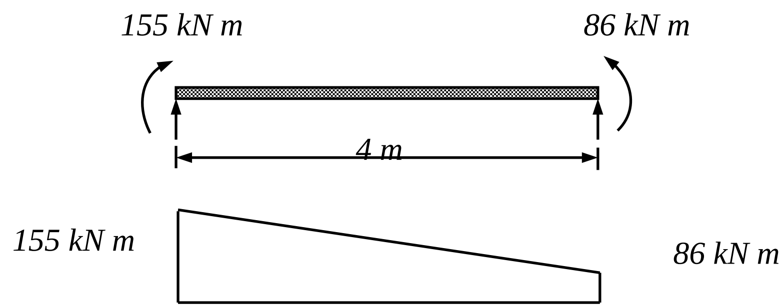
$$Hence\ \ M_b\ >\ M_{max}$$

ISMB 450 is adequate against lateral torsional buckling.

Structural Steel Design Project	Job No.	Sheet *1* of *5*		Rev.
	Job title: *UNRESTRAINED BEAM DESIGN*			
	Worked example*: 2*			
Calculation sheet		Made by. *SSR*		Date.*23/3/2000*
		Checked by. *SAJ*		Date.*26/3/2000*

Problem-2

(i) A simply supported beam of span 4 m is subjected to end moments of 155 kN m (clockwise) and 86 k N m (anticlockwise) under factored - applied loading. Check whether ISMB 450 is safe with regard to lateral buckling.



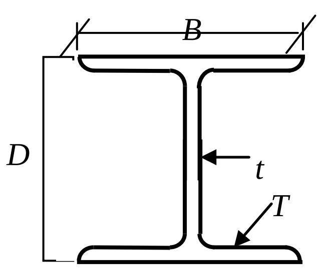
B.M Diagram

Design check:

For the end conditions given, it is assumed that the beam is simply supported in a vertical plane, and at the ends the beam is fully restrained against lateral deflection and twist with, no rotational restraint in plan at its ends.

Section classification of ISMB 450

The properties of the section are:



Depth, D = 450 mm.

Width, B = 150 mm.

Web thickness, t = 9.4 mm
Flange thickness, T = 17.4 mm

Structural Steel Design Project	Job No.	Sheet *2* of *5*		Rev.
	Job title: *UNRESTRAINED BEAM DESIGN*			
	Worked example*: 2*			
Calculation sheet		Made by. *SSR*	Date*23/34/2000*	
		Checked by. *SAJ*	Date.*26/3/2000*	

Depth between fillets, d = 379.2 mm

Radius of gyration about minor axis, $r_y = 30.1$ *mm*

Plastic modulus about major axis, $S_x = 1512.8 * 10^{-3}\ mm^3$

Assume $f_y = 250\ N/mm^2$, $E=200000\ N/mm^2$, $\gamma_m = 1.15$,

$p_y = f_y / \gamma_m = 250 / 1.15 = 217.4\ N / mm^2$

(II) Type of section

Flange criterion:

$$b = \frac{B}{2} = \frac{150}{2} = 75\ mm$$

$$\frac{b}{T} = \frac{75.0}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon \quad where\ \varepsilon = \sqrt{\frac{250}{f_y}}$$

Hence O.K.

Web criterion:

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\,\varepsilon$$

Hence O.K

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Since $\frac{b}{T} < 8.92\,\varepsilon$ *and* $\frac{d}{t} < 82.95\,\varepsilon$, *the section is classified as 'plastic.' Section should be plastic or compact to attain plastic moments. Most of the hot - rolled sections are classified as plastic or compact.*

(II)Check for lateral torsional buckling:

Equivalent slenderness of the beam, $\lambda_{LT} = n\,u\,v\,\lambda$

Where, n = slenderness correction factor (assumed value of 1.0)

u = buckling parameter (assumed as 0.9)

λ *= slenderness of the beam along minor axis,* ℓ_e/r_y

$$= \frac{6000}{30.1} = 199.33$$

v = slenderness factor (which is dependent on the proportion of the flanges and the torsional index [D / T])

= 0.71 (for equal flanges and λ *= 199.33)* — *Table 14 of BS5950 Part I*

Now, $\lambda_{LT} = 1.0 * 0.9 * 0.71 * 199.33$

$= 127.37$

Bending strength, p_b *= 84 Mpa (for* λ_{LT} *= 127.37)* — *Table 11 of BS5950 Part I*

Buckling resistance moment $M_b = S_x * p_b$

$= (1512.78 * 84)/1000$

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$= 127.07$ *kN m*

For the given beam of 4 m span,

$\beta = 86 / 155 = 0.555$

Using the equation to find the value of m

$m = 0.57 + 0.33 * 0.555 + 0.1 * 0.555^2$

$= 0.784$

Equivalent uniform moment $\overline{M} = 0.784 * 155$

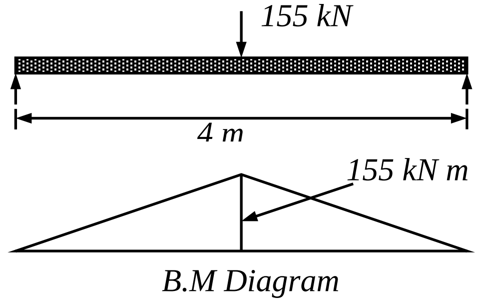
$= 122$ *kN m*

$127.07 > 122.$

Therefore the capacity of the beam exceeds the design moment.

"ISMB 450 is adequate against lateral torsional buckling"

(ii) If the beam of problem (i) is subjected to a central load producing a maximum factored moment of 155 kN m check whether the beam is still safe.



B.M Diagram

<table>
<tr><td rowspan="4">Structural Steel Design Project

Calculation sheet</td><td>Job No.</td><td>Sheet 5 of 5</td><td colspan="2">Rev.</td></tr>
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Date. 26/3/2000</td></tr>
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For this problem,

m =0.74 (see Fig. 9 of the text)

Therefore $n = \sqrt{m} = \sqrt{0.74} = 0.86$ *(see section 5.4.2 of the text)*

Therefore $\lambda'_{LT} = n\lambda_{LT} = 0.86 * 127.37 = 109.54$

$p_b = 105\ N/mm^2$

Therefore $M_b = 105 * 1512.78 / 1000 = 158.84$ *kN m.*

Therefore the $M_b > M_{max}$ *(158.84 > 155)*

Therefore the section ISMB 450 is adequate against lateral torsional buckling.

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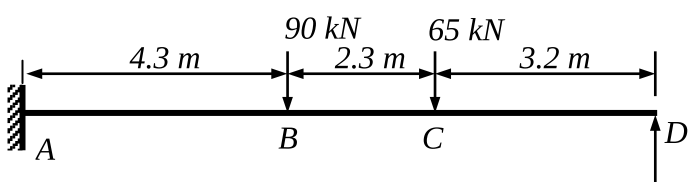
Problem I :

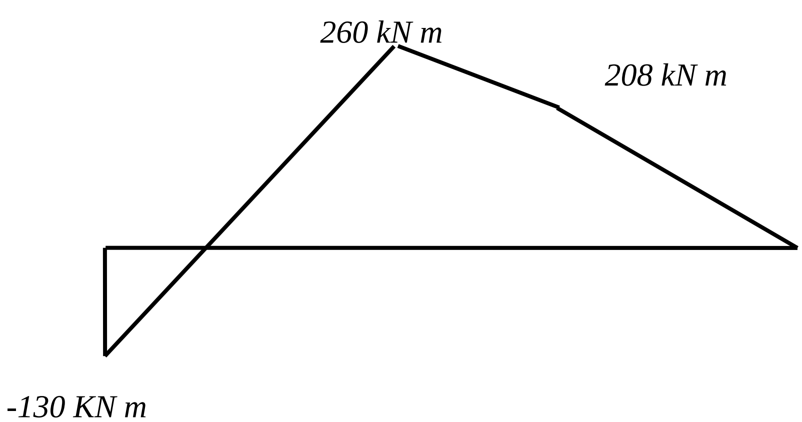
A propped cantilever has a span of 9.8 m. it is loaded by cross beams at 4.3 m and 6.6 m from its left hand end. The ends of the beam and the loaded points are assumed to be fully braced laterally and torsionally.



The loads as given are factored.

Design a suitable section for the beam:

The bending moment diagram of the beam is as shown below.



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Section properties of ISMB 450 are :

Depth D = 450 mm.

Width B = 150 mm.

Web thickness t = 9.4 mm.

Flange thickness T = 17.4 mm.

Depth between fillets, d = 379.2 mm.

Radius of gyration about

Minor axis, $r_y = 30.1$ *mm.*

Plastic modulus about
major axis, $S_x = 1512.78\ cm^3$.

Assume $f_y = 250\ N/mm$, $E = 200000\ N/mm^2$, $\gamma_m = 1.15$

$P_y = f_y / \gamma_m = 250/1.15 = 217.4\ N/mm^2$

(1) ***Type of section***

(i) *flange criterion:*

$$b = \frac{B}{2} = \frac{150}{2} = 75\ mm$$

$$\frac{b}{T} = \frac{75}{17.4} = 4.31$$

$$\frac{b}{T} < 8.92\varepsilon\,,\ \text{where}\ \ \varepsilon = \sqrt{\frac{250}{f_y}}$$

Hence o.k.

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(ii) *Web criterion:*

$$\frac{d}{t} = \frac{379.2}{9.4} = 40.3$$

$$\frac{d}{t} < 82.95\varepsilon$$

Hence o.k.

Since

$\frac{b}{t} < 8.92\varepsilon$ *and* $\frac{d}{t} < 82.95\ \varepsilon$ *, the section is classified as " plastic"*

Check for moment capacity:

$$M_C = S_x * p_y$$

$$= \frac{1512.78 * 217.4}{1000} = 328.87\ kN\ m$$

Maximum applied moment $= 260\ KN - m < 328.87\ KN\ m$

Hence o.k.

(iii) *Lateral torsional buckling:*

The beam length AB, BC and CD will be treated separately using the equivalent uniform method.

Effective lengths:

$$\ell_{AB} = 4.3\ m.$$

$$\ell_{BC} = 2.3\ m.$$

$$\ell_{CD} = 3.2\ m.$$

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Length L_{AB}:

The equivalent uniform moment should be less than the lateral torsional buckling resistance moment

$$\overline{M} \leq M_b$$

where,

$\overline{M}$ *is equivalent uniform moment*

M_b *is lateral torsional buckling resistance moment*

$$\overline{M} = m\,M_A$$

where,

M_A *is the maximum moment in the member*

m *is the equivalent uniform moment factor*

To determine 'm' :

$$m = 0.57 + 0.33\,\beta + 0.1\,\beta^2 \nless 0.43\text{ , where } \beta = \frac{M_{min}}{M_{max}}$$

$$\beta = -\frac{130}{260} = -0.5\text{, then } m = 0.43$$

$$\overline{M} = 0.43 * 260 = 112\text{ kN m}$$

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Length L_{BC}:

$$\beta = \frac{208}{260} = 0.8; \text{ Then } m = 0.9$$

$$\overline{M} = 0.9 * 260 = 234 \text{ kN m}$$

length L_{CD}:

$$\beta = 0; m = 0.57. \ \overline{M} = 0.57 * 208 = 119 \text{ kN m}$$

For the purpose of determining the governing values, all the three segments are checked separately.

$$\lambda = \frac{\ell_{AB}}{r_y} = \frac{4.3 * 1000}{30.1} = 142.86$$

$$x = \frac{D}{T} = 25.86$$

$$\frac{\lambda}{x} = \frac{142.86}{25.86} = 5.52$$

$$v = 0.79$$

$$\lambda_{LT} = n\,u\,v\,\lambda \;; \quad u = 0.9, n = 1.0$$

$$= 1.0 * 0.9 * 0.79 * 142.86$$

$$= 101.57$$

Bending strength, $P_b = 117$ *Mpa* (*for* $\lambda_{LT} = 101.57$)

Buckling resistance moment

$$M_b = \frac{1512.78 * 117}{1000} = 177 > 112 \text{ kN m}$$

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	Worked example:*1*		
Calculation sheet		Made by. *SSR*	Date.*1/3/2000*
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Length ℓ_{AB} is safe against lateral torsional buckling

Length ℓ_{BC} :

$$\lambda = \frac{\ell_{BC}}{r_y} = \frac{2300}{30.1} = 76.41$$

$$x = \frac{D}{T} = 25.86$$

$$\frac{\lambda}{x} = \frac{76.41}{25.86} = 2.95$$

$$v = 0.91$$

$$\lambda_{LT} = 1.0 * 0.9 * 0.91 * 76.41 = 62.58$$

Bending strength , p_b = 190 Mpa

Buckling resistance moment M_b

$$= \frac{1512.78 * 190}{1000} = 287.43 > 234 \text{ kN m}$$

Length ℓ_{BC} is safe against lateral torsional buckling

Length ℓ_{CD} :

$$\lambda = \frac{\ell_{CD}}{r_y} = \frac{3.2 * 1000}{30.1} = 106.31$$

$$x = \frac{D}{T} = 25.86$$

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	Job title: *UNRESTRAINED BEAM DESIGN*		
	Worked example:*1*		
Calculation sheet		Made by. *SSR*	Date.*1/3/2000*
		Checked by. *SAJ*	Date. *7/3/2000*

$$\frac{\lambda}{x} = \frac{106.31}{25.86} = 4.11$$

$$v = 0.70$$

$$\lambda_{LT} = 1 * 0.9 * 0.7 * 106.31 = 66.97$$

bending strength, $p_b = 174\ Mpa$

$$\text{bending resistance moment} = \frac{174 * 1512.78}{1000}$$

$$= 263.22 > 119\ kN\ m$$

Length L_{CD} *is safe against lateral torsional buckling.*

There fore the section chosen 'ISMB 450' is o.k.

The shortest segment BC, which has the most severe pattern of moments controls the design.

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	Job Title: *BEAM COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date *3-1-00*
		Checked by *VK*	Date *10-1-00*

PROBLEM: 1

A non – sway intermediate column in a building frame with flexible joints is 4.0 m high and it is ISHB 300 @ 588 N/m steel section. Check the adequacy of the section when the column is subjected to following load:

Factored axial load = 500 kN

Factored moments:

	M_x	M_y
Bottom	*+ 7.0 kN –m*	*- 1.0 kN - m*
Top	*+ 15.0 kN – m*	*+ 0.75 kN - m*

[$f_y = 250$ N/mm^2 ; $E = 2 * 10^5$ N/mm^2]

Assume effective length of the column as 3.4 m along both the axes.

CROSS-SECTION PROPERTIES:

Flange thickness = T = 10.6 mm

Clear depth between flanges = d = 300 – (10.6 * 2) = 278.8 mm

Thickness of web = t = 7.6 mm

Flange width = $2b$ = 250 mm

b = 125 mm

Area of cross-section = A_g = 7485 mm^2

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	Job Title: *BEAM COLUMN*		
	Worked Example - 1		
		Made by *SSSR*	Date *3-1-00*
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r_x = 129.5 mm

r_y = 54.1 mm

I_x = $12545.2 * 10^4$ mm^4

I_y = $2193.6 * 10^4$ mm^4

Z_x = $836.3 * 10^3$ mm^3

Z_y = $175.5 * 10^3$ mm^3

Z_{px} = $953.4 * 10^3$ mm^3

Z_{py} = $200.1 * 10^3$ mm^3

(i) Type of section:

$$\frac{b}{T} = \frac{125}{10.6} = 11.8 < 13.65 \in$$

$$\frac{d}{t} = \frac{278.8}{7.6} = 36.7 < 40.95 \in$$

$$where, \in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Hence, cross- section is " SEMI-COMPACT" (Class 3)

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	Job Title: *BEAM COLUMN*		
	Worked Example - 1		
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(ii) Check for resistance of cross-section to the combined effects *for yielding:*

$f_{yd} = f_y/\gamma_a = 250/1.15$

$= 217.4\ N/mm^2$

$A_g = 7485\ mm^2$

$Z_x = 836.3*10^3\ mm^3$

$Z_y = 175.5*10^3\ mm^3$

$F_c = 500\ kN$

$M_x = 15\ kN\text{-}m$

$M_y = 1.0\ kN\text{-}m$

The interaction equation is:

$$\frac{F_c}{A_g f_{yd}} + \frac{M_x}{Z_x f_{yd}} + \frac{M_y}{Z_y f_{yd}} \le 1$$

$$= \frac{500\times10^3}{7485\times217.4} + \frac{15\times10^6}{836.3\times10^3\times217.4} + \frac{1\times10^6}{175.5\times10^3\times217.4}$$

$= 0.307 + 0.083 + 0.026 = 0.416 < 1.0$

Hence, section is O.K. against combined effects

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	Job Title: *BEAM COLUMN*		
	Worked Example - 1		
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Calculation Sheet		Checked by *VK*	Date *10-1-00*

(iii) Check for resistance of cross-section to the combined effects for buckling:

Slenderness ratios:

Effective length of the column = 3.4 m

$\lambda_x = 3400/129.5 = 26.3$

$\lambda_y = 3400/54.1 = 62.8$

$\lambda_1 = \pi(E/f_y)^{1/2} = \pi(200000/250)^{1/2}$

$= 88.9$

Non-dimensional slenderness ratios:

$$\bar{\lambda} = \frac{\lambda}{\lambda_1}$$

$$\bar{\lambda}_x = \frac{26.3}{88.9} = 0.296$$

$$\bar{\lambda}_y = \frac{62.8}{88.9} = 0.706$$

Calculation of χ *:*

Imperfection factors:

$\alpha_x = 0.21$; $\alpha_y = 0.34$

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	Worked Example - 1		
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ϕ - values:

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$$\phi_x = 0.5[1 + 0.21(0.296 - 0.2) + (0.296)^2] = 0.554$$

$$\phi_y = 0.5[1 + 0.34(0.706 - 0.2) + (0.706)^2] = 1.006$$

χ - values:

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{\frac{1}{2}}} \le 1.0$$

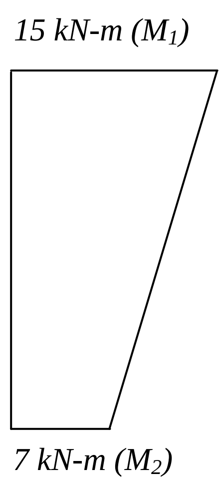
$$\chi_x = 1/[0.554 + (0.554^2 - 0.296^2)^{1/2}] = 0.978$$

$$\chi_y = 1/[1.006 + (1.006^2 - 0.706^2)^{1/2}] = 0.580$$

The interaction equation is

Eurocode 3 Clause 5.5.4

$$\frac{F_c}{F_{cl}} + \frac{k_x\, M_x}{M_{ux}} + \frac{k_y\, M_y}{M_{uy}} \le 1$$



$$\psi_x = M_2/M_1 = 7/15 = 0.467$$

$$\beta_{Mx} = 1.8 - 0.7\psi = 1.8 - 0.7 \times 0.467 = 1.473$$

$$\mu_x = \bar{\lambda}_x\left(2\beta_{Mx} - 4\right) = 0.296\,(2 \times 1.473 - 4) = -0.312$$

$$k_x = 1 - \frac{\mu_x F_c}{P_{cx}} = 1 - \frac{\mu_x F_c}{\chi_x A f_y} = 1 - \frac{(-0.312) \times 500 \times 10^3}{0.978 \times 7485 \times 250} = 1.085$$

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	Job Title: *BEAM COLUMN*		
	Worked Example - 1		
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$\psi_y = 0.75/(-1.0) = -0.75$

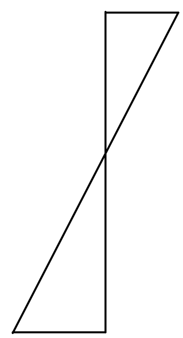
$\beta_{My} = 1.8 - 0.7\psi$

$= 1.8 + 0.7 \times 0.75 = 2.325$

$\mu_y = \bar{\lambda}_y \left(2\beta_{My} - 4\right)$

$= 0.706\ (2\times 2.325 - 4) = 0.459$

0.75 kN-m (M_2)



- 1.0 kN-m (M_1)

$$k_y = 1 - \frac{\mu_y F_c}{P_{cy}} = 1 - \frac{\mu_y F_c}{\chi_y A f_y} = 1 - \frac{0.459 \times 500 \times 10^3}{0.58 \times 7485 \times 250} = 0.788$$

Note: $F_{cl} = \chi_{min} A_g f_{yd}$

$M_{ux} = Z_x f_{yd}$

$M_{uy} = Z_y f_{yd}$

Substituting the interaction equation,

$$\frac{500\times 10^3}{7485\times 217.4 \times 0.58} + \frac{15\times 10^6 \times 1.085}{836.3\times 10^3 \times 217.4} + \frac{1\times 10^6 \times 0.788}{175.5\times 10^3 \times 217.4}$$

$= 0.530 + 0.089 + 0.021 = 0.640 < 1.0$

Hence, section is O.K. against combined effects for buckling.

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	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
		Checked by *PU*	Date *25-04-00*

PROBLEM:

The girder showed in Fig. E1 is fully restrained against lateral buckling throughout its span. The span is 36 m and carries two concentrated loads as shown in Fig. E1. Design a plate girder.

Yield stress of steel, f_y = *250 N/mm*^{2}
Material factor for steel, γ_m = *1.15*
Dead Load factor, γ_{fd} = *1.35*
Imposed load factor, $\gamma_{f\ell}$ = *1.50*

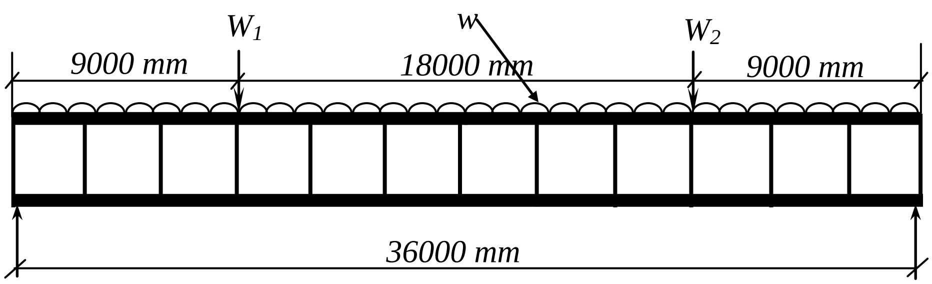

Fig. E1 Example plate girder

1.0 LOADING

Dead load:

Uniformly distributed load, w_d = *20 kN/ m (Including self-weight)*
Concentrated load, W_{1d} = *200 kN*
Concentrated load, W_{2d} = *200 kN*

Live load:

Uniformly distributed load, w_ℓ = *35 kN/m*
Concentrated load, $W_{1\ell}$ = *400 kN*
Concentrated load, $W_{2\ell}$ = *400 kN*

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Factored Loads:

$w' = w_d * \gamma_{fd} + w_\ell * \gamma_{f\ell} = 20 * 1.35 + 35 * 1.5 = 79.5 \text{ kN/m}$

$W'_1 = W_{1d} * \gamma_{fd} + W_{1\ell} * \gamma_{f\ell} = 200 * 1.35 + 400 * 1.5 = 870 \text{ kN}$

$W'_2 = W_{2d} * \gamma_{fd} + W_{2\ell} * \gamma_{f\ell} = 200 * 1.35 + 400 * 1.5 = 870 \text{ kN}$

2.0 BENDING MOMENT AND SHEAR FORCE

	Bending moment (kN-m)	***Shear force (kN)***
UDL effect	$\frac{w^1\ell^2}{8} = \frac{79.5*36*36}{8} = 12879$	$\frac{w^1\ell}{2} = 1431$
Concentrated load effect	$\frac{W\ell}{4} = 870*9 = 7830$	*W = 870*
TOTAL	*20709*	*2301*

The design shear forces and bending moments are shown in Fig. E2.

3.0 INITIAL SIZING OF PLATE GIRDER

Depth of the plate girder:

The recommended span/depth ratio for simply supported girder varies between 12 for short span and 20 for long span girder. Let us consider depth of the girder as 2400 mm.

$$\frac{\ell}{d} = \frac{36000}{2400} = 15.0$$

Depth of 2400 mm is acceptable.

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	Job Title: *PLATE GIRDER*		
	Worked Example - 1		
		Made by *SSSR*	Date *15-04-00*
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(For drawing the bending moment and shear force diagrams, factored loads are considered)

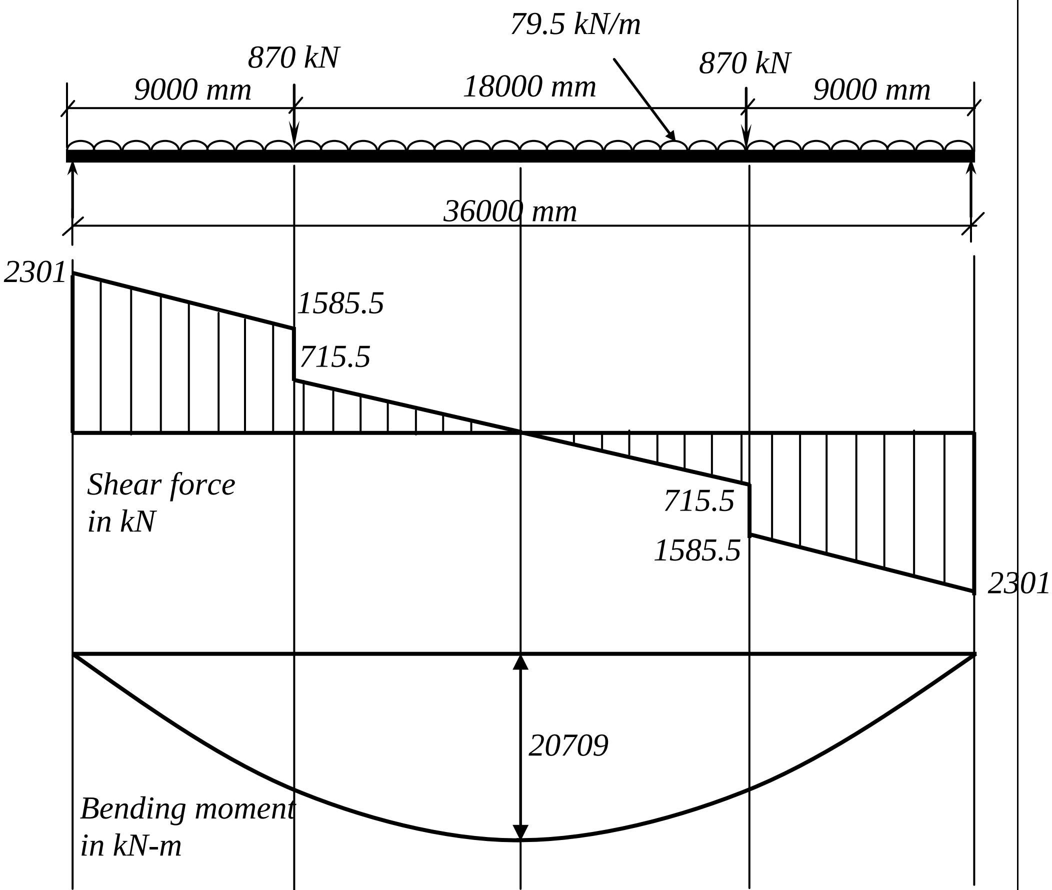

Fig. E2 Bending moment and shear force diagrams

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	Worked Example - 1		
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		Checked by *PU*	Date *25-04-00*

Flange:

$p_y = 250/1.15 = 217.4\ N/mm^2$

Single flange area,

$$A_f = \frac{M_{\max}}{d\ p_y} = \frac{20709*10^6}{2400*217.4} = 39690.7\ mm^2$$

By thumb rule, the flange width is assumed as 0.3 times the depth of the section. Try 720 X 60 mm, giving an area = 43200 mm^2.

Web:

Minimum web thickness for plate girder in buildings usually varies between 10 mm to 20 mm. Here, thickness is assumed as 14 mm.

Hence, web size is 2400 X 14 mm

4.0 SECTION CLASSIFICATION

Flange:

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{1/2} = \left\{\frac{250}{250}\right\}^{1/2} = 1.0$$

$$b = \frac{B-t}{2} = \frac{720-14}{2} = 353$$

$$\frac{b}{T} = \frac{353}{60} = 5.9 < 7.9\varepsilon$$

Hence, Flange is PLASTIC SECTION.

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Web:

$$\frac{d}{t} = \frac{2400}{14} = 171.4 > 66.2\varepsilon$$

Hence, the web is checked for shear buckling.

5.0 CHECKS

Check for serviceability:

$$\frac{d}{250} = \frac{2400}{250} = 9.6\ mm < t$$

$$Since,\ t > \frac{d}{250}$$

Web is adequate for serviceability.

Check for flange buckling in to web:

Assuming stiffener spacing, a > 1.5 d

$$t \geq \frac{d}{294}\left(\frac{p_{yf}}{250}\right)^{1/2} = \frac{2400}{294} \times \left(\frac{217.4}{250}\right)^{1/2} = 7.6\ \ mm$$

Since, t (= 14 mm) > 7.6 mm, the web is adequate to avoid flange buckling into the web.

Check for moment carrying capacity of the flanges:

The moment is assumed to be resisted by flanges alone and the web resists shear only.

Distance between centroid of flanges, $h_s = d + T = 2400 + 60 = 2460\ mm$

$$A_f = B * T = 720 * 60 = 43200\ mm^2$$

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$M_c = p_{yf} * A_f * h_s = 217.4 * 43200 * 2460 * 10^{-6} = 23103.5$ *kN-m*
> 20709 *kN-m*

Hence, the section in adequate for carrying moment and web is designed for shear.

6.0 WEB DESIGN

The stiffeners are spaced as shown in Fig. E5. The spacing of stiffeners is taken as 3000 mm. The spacing can be increased towards the centre of the span for economy.

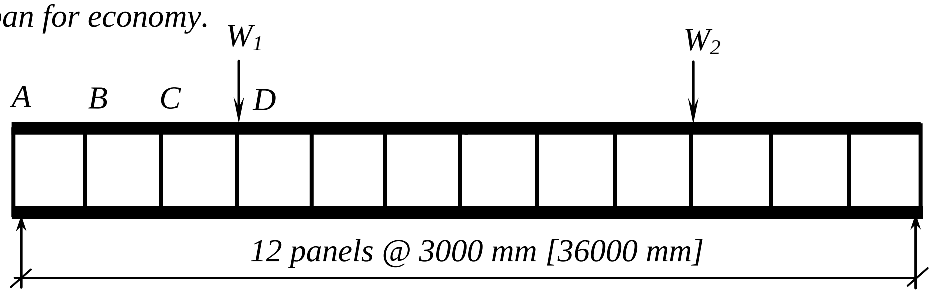

Fig.E3 Trial stiffener arrangement

Panel AB is the most critical panel (Maximum shear zone), so design checks for the web are made for panel AB only.

<u>End panel (AB) design:</u>

$d = 2400$ *mm*
$t = 14$ *mm*

$$\frac{a}{d} = \frac{3000}{2400} = 1.25$$

$$\frac{d}{t} = \frac{2400}{14} = 171.4$$

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Calculation of critical shear strength, q_{cr}:

Elastic critical stress, q_e (when $a/d > 1$) $= [1.0 + 0.75/(a/d)^2][1000/(d/t)]^2$

$= [1 + 0.75/(1.25)^2][1000/(171.4)]^2$

$= 50.4\ N/mm^2$

Slenderness parameter, λ_w $= [0.6(f_{yw}/\gamma_m)/q_e]^{1/2}$

$= [0.6(250/1.15)/50.4]^{1/2}$

$= 1.61 > 1.25$

Hence, Critical shear strength ($q_{cr} = q_e$) = 50.4 N/mm²

$$f_v = \frac{F_{VA}}{dt} = \frac{2301*10^3}{2400*14} = 68.5\ N/mm^2$$

Since, $f_v > q_{cr}$ (68.5 > 50.4)

Panel AB is designed using tension field action.

Calculation of basic shear strength, q_b:

$$\phi_t = \frac{1.5q_{cr}}{\sqrt{1+\left(\frac{a}{d}\right)^2}} = \frac{1.5*50.4}{\sqrt{1+(1.25)^2}} = 47.2$$

$y_b = (p_{yw}^2 - 3q_{cr}^2 + \phi_t^2)^{1/2} - \phi_t = (217.4^2 - 3*50.4^2 + 47.2^2)^{1/2} - 47.2 = 157.4$

$$q_b = q_{cr} + \frac{y_b}{2\left[\frac{a}{d} + \sqrt{1+\left(\frac{a}{d}\right)^2}\right]} = 50.4 + \frac{157.4}{2\left[1.25 + \sqrt{1+(1.25)^2}\right]} = 78.0\ N/mm^2$$

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Since, $q_b > f_v$ *(78.0 > 68.5)*

Panel AB is safe against shear buckling.

Checks for the web panel:

End panel AB should also be checked as a beam (Spanning between the flanges of the girder) capable of resisting a shear force R_{tf} *and a moment* M_{tf} *due to anchor forces.*
(In the following calculations boundary stiffeners are omitted for simplicity)

Check for shear capacity of the end panel:

$$H_q = 0.75\,dt\;p_y\left[1-\frac{q_{cr}}{0.6\,p_y}\right]^{1/2}\left[\frac{f_v-q_{cr}}{q_b-q_{cr}}\right]$$

$$q_{cr} = 50.4\,N/mm^2$$

$$H_q = 0.75*2400*14*217.4\left[1-\frac{50.4}{0.6*(250/1.15)}\right]^{1/2}\left[\frac{68.5-50.4}{78-50.4}\right] = 2814\;kN.$$

$$R_{tf} = \frac{H_q}{2} = \frac{2814}{2} = 1407\,kN$$

$A_v = t\,.a = 14 * 3000 = 42000\ mm^2$

$P_v = 0.6\,p_{yw}\,A_v = 0.6 * (250/1.15) * 42000/1000 = 5478\ kN$

Since, $R_{tf} < P_v$*, the end panel can carry the shear force.*

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Check for moment capacity of end panel AB:

$$M_{tf} = \frac{H_q d}{10} = \frac{2814 * 2400}{10} * 10^{-3} = 675.4 \; kN - m$$

$$y = \frac{a}{2} = \frac{3000}{2} = 1500$$

$$I = \frac{1}{12} t a^3 = \frac{1}{12} * 14 * 3000^3 = 3150 * 10^7 \; mm^4$$

$$M_q = \frac{I}{y} p_y = \frac{3150 * 10^7}{1500} * (250/1.15) * 10^{-6} = 4565 \; kN - m$$

Since, $M_{tf} < M_q$ *(675.4 < 4565)*

$\therefore$ *The end panel can carry the bending moment.*

7.0 DESIGN OF STIFFENERS

Load bearing stiffener at A:

Design should be made for compression force due to bearing and moment.

Design force due to bearing, F_b *= 2301 kN*

Force(F_m*) due to moment* M_{tf}*, is*

$$F_m = \frac{M_{tf}}{a} = \frac{675.4}{3000} * 10^3 = 225 \; kN$$

Total compression = $F_c = F_b + F_m = 2301 + 225 = 2526$ *kN*

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Area of stiffener in contact with the flange, A:

Area (A) *should be greater than* $\dfrac{0.8\,F_c}{p_{ys}}$

$$\frac{0.8 F_c}{p_{ys}} = \frac{0.8*2526}{217.4}*10^3 = 9295\ mm^2$$

Try stiffener of 2 flats of size 240 X 25 mm thick

Allow 15 mm to cope for web/flange weld

*A = 225 * 25 * 2 = 11250 mm^2 > 9295 mm^2*

$\therefore$ *Bearing check is ok.*

Check for outstand:

Outstand from face of web should not be greater than 20 $t_s\varepsilon$.

$$\varepsilon = \left\{\frac{250}{f_y}\right\}^{1/2} = \left\{\frac{250}{250}\right\}^{1/2} = 1.0$$

Outstand b_s = *240 mm <* $20\,t_s\varepsilon$ *(= 20 * 25 * 1.0 = 500)*

b_s = *240 mm <* $13.7\,t_s\varepsilon$ *(= 13.7 * 25 * 1.0 = 342.5)*

Hence, outstand criteria is satisfied.

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Check stiffener for buckling: (*The effective stiffener section is shown in Fig. E4*)

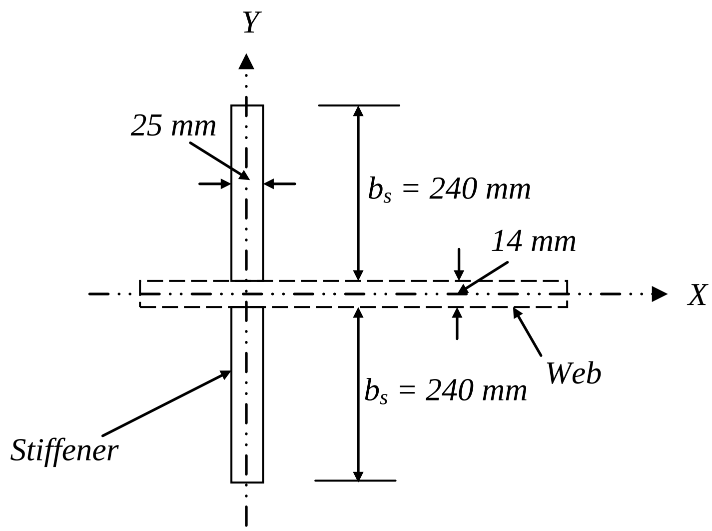

Fig. E4 End bearing stiffener

The buckling resistance due to web is neglected here for the sake of simplicity.

$$I_x = \frac{25*494^3}{12} - \frac{1}{12}*25*14^3 = 25115*10^4\ mm^4$$

A_e = *Effective area = 240 * 25 * 2 = 12000* mm^2

$$r_x = \left[\frac{I_x}{A_e}\right]^{1/2} = \left[\frac{25115*10^4}{12000}\right]^{1/2} = 144.7\ mm$$

Flange is restrained against rotation in the plane of stiffener, then

l_e = *0.7 l = 0.7 * 2400 = 1680 mm*

$$\lambda = \frac{l_e}{r_x} = \frac{1680}{144.7} = 11.6$$

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For $f_y = 250\ N/mm^2$ *and* $\lambda = 11.6$

$\sigma_c = 250\ N/mm^2$ *from table (3) of chapter on axially compressed columns*

Buckling resistance of stiffener is

$P_c = \sigma_c A_e / \gamma_m = (250/1.15) * 12000 * 10^{-3} = 2609\ kN$

Since $F_c < P_c$ *(2526 < 2609), stiffener provided is safe against buckling.*

<u>Check stiffener A as a bearing stiffener:</u>

Local capacity of the web:

Assume, stiff bearing length $b_1 = 0$

$n_2 = 2.5 * 60 * 2 = 300$ *BS 5950: Part – 1, Clause 4.5.3*

$P_{crip} = (b_1 + n_2)\ t\ p_{yw}$

$= (0 + 300) * 14 * (250/1.15) * 10^{-3} = 913\ kN$

Bearing stiffener is designed for F_A

$F_A = F_c - P_{crip} = 2526 - 913 = 1613\ kN$

Bearing capacity of stiffener alone

$P_A = p_{ys} * A = (250/1.15) * 12000/1000 = 2609\ kN$

Since, $F_A < P_A$ *(1613 < 2609)*

The designed stiffener is OK in bearing.

<u>Stiffener A</u> *– Adopt 2 flats 240 mm X 25 mm thick*

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Design of intermediate stiffener at B:

Stiffener at B is the most critical intermediate stiffener, hence it will be chosen for the design.

Minimum Stiffness:

$$I_s \geq 0.75 dt^3 \quad for \;\; a \geq d\sqrt{2}$$

$$I_s \geq \frac{0.75 dt^3}{a^3} \quad for \;\; a < d\sqrt{2}$$

$$d\sqrt{2} = \sqrt{2} * 2400 = 3394 \; mm$$

$$\therefore a < d\sqrt{2} \qquad\qquad (3000 < 3394)$$

Conservatively' t' is taken as actual web thickness and minimum' a' is used.

$$\frac{1.5 d^3 t^3}{a^2} = \frac{1.5 * 2400^3 * 14^3}{3000^2} = 632 * 10^4 \; mm^4$$

Try intermediate stiffener of 2 flats 90 mm X 12 mm

$$(I_s)_{Provided} = \frac{12*194^3}{12} - \frac{12*14^3}{12} = 730 * 10^4 \; mm^4$$

The section provided satisfies the minimum required stiffness.

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Check for outstand:

Outstand of the stiffener $\leq 13.7\ t_s\varepsilon$

$13.7\ t_s\varepsilon = 13.7 * 14 * 1.0 = 192$ *mm*

Outstand $= 90$ *mm* $(90 < 192)$

Hence, outstand criteria is satisfied.

Buckling check:

Stiffener force, $F_q = V - V_s$

where, V = *Total shear force*
$V_s = V_{cr}$ *of the web.*

Elastic critical stress, $q_e = 50.4\ N/mm^2$

$V_{cr} = q_{cr}\ d\ t = 50.4 * 2400 * 14 * 10^{-3} = 1693\ kN$

Shear force at B, V_B **= 2301 – [(2301 – 1585.5)*(3000/9000)] = 2062.5 kN**

Stiffener force, $F_q = [2062.5 - 1693] = 369.5\ kN$

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Buckling resistance of intermediate stiffener at B:

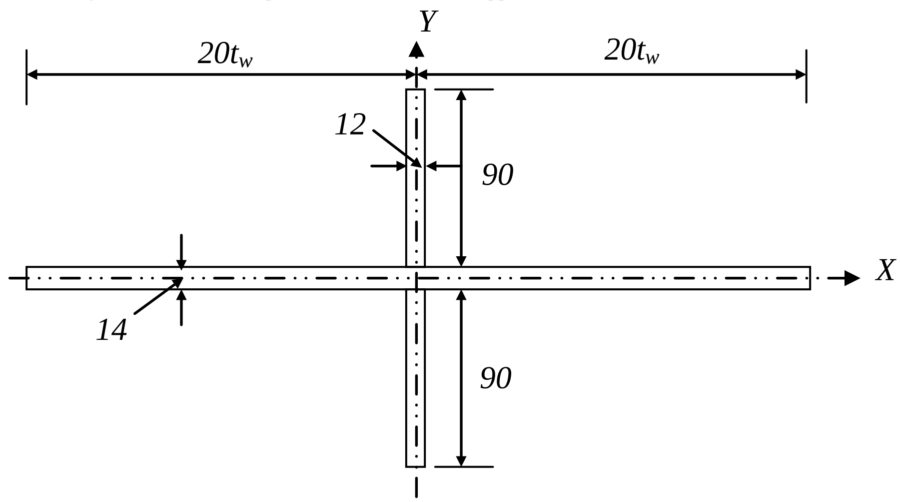

Fig.E5 Effective section

$20\ t_w\ = 20 * 14\ = 280\ mm$

$$I_x = \frac{1}{12} * 12 * 194^3 + \frac{560*14^3}{12} - \frac{12*14^3}{12} = 743*10^4\ mm^4$$

$A\ = 180 * 12 + 560 * 14 = 10000\ mm^2$

$$r_x = \left[\frac{743*10^4}{10000}\right]^{1/2} = 27.3$$

$l_e\ = 0.7 * 2400 = 1680$

$$\lambda = \frac{l_e}{r_x} = \frac{1680}{27.3} = 61.5$$

For $f_y\ = 250\ N/mm^2$ *and* $\lambda = 61.5$

From table3 of chapter on axially compressed columns,

$\sigma_c = 182.3\ N/mm^2$

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*Buckling resistance = (182.3/1.15) * 10000 * 10^{-3} = 1585 kN*

F_q < Buckling resistance. (369.5 < 1585)

Hence, intermediate stiffener is adequate

Intermediate stiffener at B *- Adopt 2 flats 90 mm X 12 mm*

Intermediate Stiffener at D (Stiffener subjected to external load):

Try intermediate stiffener 2 flats 90 mm X 12 mm thick

It satisfies the minimum stiffness requirement as in case of stiffener at B.

Buckling check:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \leq 1$$

$F_q = V - V_s$ $\quad\quad$ $V = 1585.5$ kN

$V_s = V_{cr} = q_{cr}\, d\, t = 50.4 * 2400 * 14 * 10^{-3} = 1693$ kN

F_q is negative and so we can take $F_q - F_x = 0$

$M_s = 0$

F_x = 870 kN

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<u>Buckling resistance of load carrying stiffener at D:</u>

(Calculation is similar to stiffener at B)

Buckling resistance, $P_x = (182.3/1.15) * 10000 * 10^{-3} = 1585\ kN$

$F_x / P_x = 870/1585 = 0.55 < 1.0$

Hence, stiffener at D is OK against buckling

<u>Stiffener at D</u> *- Adopt flats 90 mm X 12 mm thick*

<u>Web check between stiffeners:</u>

$f_{ed} \le p_{ed}$

$f_{ed} = w^1 / t = 79.5/14 = 5.7\ N/mm^2$

when compression flange is restrained against rotation relative to the web

$$p_{ed} = \left[2.75 + \frac{2}{\left(a/d\right)^2}\right]\frac{E}{\left(d/t\right)^2} = \left[2.75 + \frac{2}{\left(\frac{3000}{2400}\right)^2}\right]\frac{200000}{\left(\frac{2400}{14}\right)^2}$$

$$= \frac{3.79 * 20000}{26406} = 27.4\ N/mm^2$$

Since, $f_{ed} < p_{ed}$ *[5.7 < 27.4], the web is OK for all panels.*

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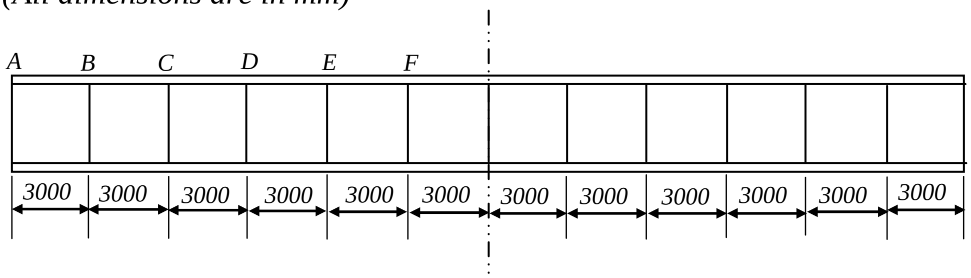
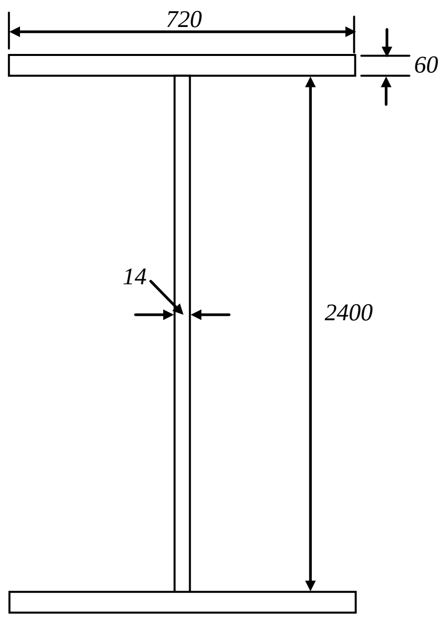
8.0 FINAL GIRDER

(All dimensions are in mm)



(a) Longitudinal section of plate girder



(b) Flange and web proportions

Fig. E6 Final girder

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PROBLEM 1:

Design a roof truss for an industrial building with 25 m span and 120 m long. The roofing is galvanized iron sheeting. The basic wind speed is 50 m/s and terrain is open industrial area and building is class A building. The building clear height at the eaves is 9 m.

Structural form:

For the purpose of this design example a trapezoidal truss is adopted with a roof slope of 1 to 5 and end depth of 1 m. For this span range the trapezoidal trusses would be normally efficient and economical.

Economical span to depth ratio is around 10.

Then, Span/depth = 25/3.5 = 7.1

Hence, depth is acceptable.

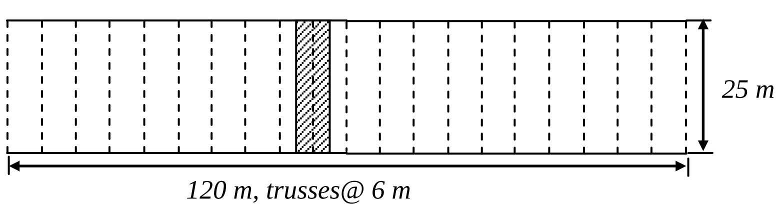
Truss spacing:

Truss spacing should be in the region of $1/4^{th}$ to $1/5^{th}$ of the span length.

For 6 m spacing,

Spacing/span = 6/25 = 1/4.17 (acceptable)

Then, number of bays = 120/6 = 20



Plan

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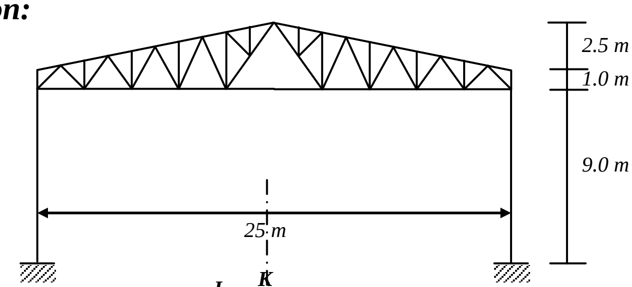
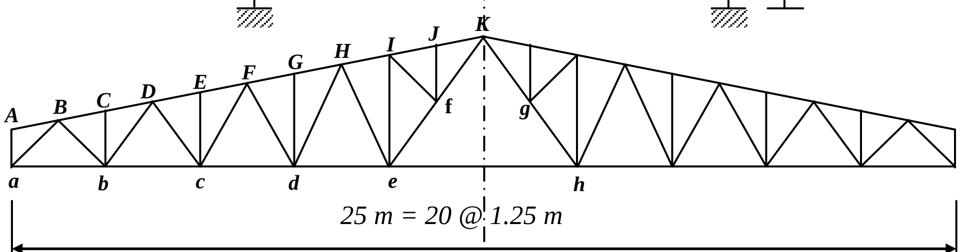
Truss configuration:

Elevation





Truss

Loading:

kN/m^2

Dead load:	*GI sheeting*	*= 0.085*
	Fixings	*= 0.025*
	Services	*= 0.100*
	Total load	*= 0.210*

For 6 m bays,

Roof dead load	*= 0.21 * 25 * 6*	*= 31.5 kN*
Weight of purlin	*= 0.07 * 6 * 25*	*= 10.5 kN*
(Assuming 70 N/m^2)		
** Self-weight of truss*	*= 0.133 * 6 * 25*	*= 20.0 kN*
Total dead load		*= 62.0 kN*

**[For welded sheeted roof trusses, the self-weight is given approximately by*

$$w = (1/100)\,(5.37 + 0.053A)\ kN/m^2$$
$$= (5.37 + 0.053 * 6 * 25) = 0.133\ kN/m^2$$

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Dead Loads

Intermediate nodal dead load (W_1) = 62.0/20 = 3.1 kN

Dead load at end nodes (W_1 / 2) = 3.1/2 = 1.55 kN

(Acts vertically downwards at all nodes)

Wind load (IS: 875-1987):

Basic wind speed = 50 m/s

Wind load F on a roof truss by static wind method is given by

$F = (C_{pe} - C_{pi}) * A * p_d$

where, C_{pe}, C_{pi} *are force co-efficient for exterior and interior of the building.*

Value of C_{pi}:

Assume wall openings between 5-20% of wall area.

Then, $C_{pi} = \pm 0.5$

Value of C_{pe}:

Roof angle = $\alpha = tan^{-1}\frac{1}{5} = 11.3^0$

Height of the building to eaves, h = 9 m

Lesser dimension of the building in plan, w = 25 m

Building height to width ratio is given by,

$$\frac{h}{w} = \frac{9}{25} = 0.36 < 0.5$$

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h/w	*Roof angle*	*Wind angle*		*Wind angle*	
	α	*Windward side*	*Leeward side*	*Windward side*	*Leeward side*
0.36	10^0	*- 1.2*	*- 0.4*	*- 0.8*	*- 0.8*
	20^0	*- 0.4*	*- 0.4*	*- 0.7*	*- 0.7*
	Here, $\alpha = 11.3^0$*, then by interpolation we get*				
	11.3^0	*- 1.1*	*- 0.4*	*- 0.79*	*- 0.79*

Risk Co-efficient, k_1 = *1.0*

(Assuming the industrial building as general building and its probable life about 50 years)

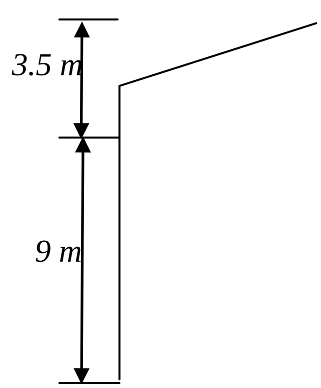
Terrain, height, structure size factor, k_2*:*

Roof elevation - 9 m to 12.5 m.

Height (m)	*Terrain category and class of building*
10	*0.91*
15	*0.97*

For 12.5 m, $k_2 = 0.94$

Assume, topography factor = $k_3 = 1.0$



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Design Project Calculation Sheet	*Worked Example - 1*		
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Wind pressure:

Total height of the building = 12.5 m

Basic wind speed, v_b = 50 m/s

Design wind speed v_Z is given by,

$v_Z = k_1 * k_2 * k_3 * v_b.$

$k_1 = 1.0$
$k_2 = 0.94$
$k_3 = 1.0$

$v_Z = 0.94 * 1 * 1 * 50 = 47$ m/s

Design wind pressure $(p_d) = 0.6\, v_Z^2 = 0.6 * (47)^2$
$= 1325\ N/m^2$
$= 1.325\ kN/m^2$

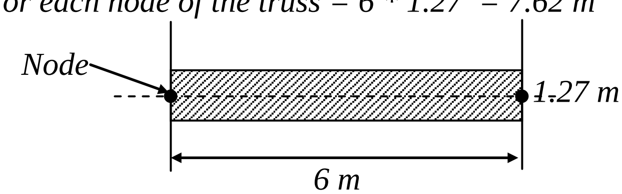
Tributary area for each node of the truss:

Length of each panel along sloping roof

$$= \frac{1.25}{\cos\ 11.3^0} = 1.27\ m \leq 1.4\ m$$

Spacing of trusses = 6m

*Tributary area for each node of the truss = 6 * 1.27 = 7.62 m^2*



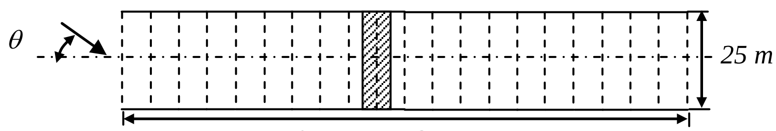
Area

Structural Steel	Job No:	Sheet *6 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

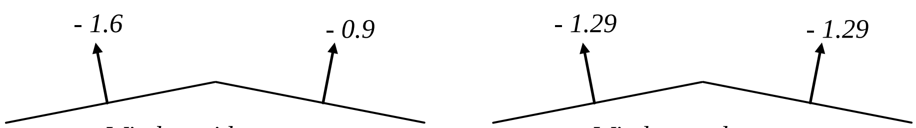
Design Project **Calculation Sheet**		Made by *SSSR*	Date *9-2-2000*
		Checked by *PU*	Date *16-08-00*

Wind load on roof truss:

Wind angle	**Pressure co-efficient**			**(C_{pe}-C_{pi})**		**A p_d (kN)**	**Wind load F (kN)**	
	C_{pe}		**C_{pi}**	**Wind ward**	**Lee ward**		**Wind ward**	*Lee ward*
	Wind ward	**Lee ward**						
0^0	- 1.10	- 0.4	0.5 - 0.5	-1.6 - 0.6	- 0.9 0.1	10.1 10.1	- 16.2 - 6.1	- 9.1 1.0
90^0	- 0.79	- 0.79	0.5 - 0.5	- 1.29 - 0.29	- 1.29 - 0.29	10.1 10.1	- 13.0 - 2.9	- 13.0 - 2.9



Maximum $C_{pe} - C_{pi}$:



Critical wind loads to be considered for analysis:

Wind angle	**Wind ward side (W_3)**		**Lee ward side (W_4)**	
	Intermediate nodes W_3	**End and apex nodes W_3 /2**	**Intermediate nodes W_4**	**End and apex nodes W_4 / 2**
0^0	- 16.2	- 8.1	- 9.1	- 4.55
90^0	- 13.0	- 6.5	- 13.0	- 6.5
***Loads in kN**				

Structural Steel	Job No:	Sheet *7 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project		Made by SSSR	Date *9-2-2000*
Calculation Sheet		Checked by PU	Date *16-08-00*

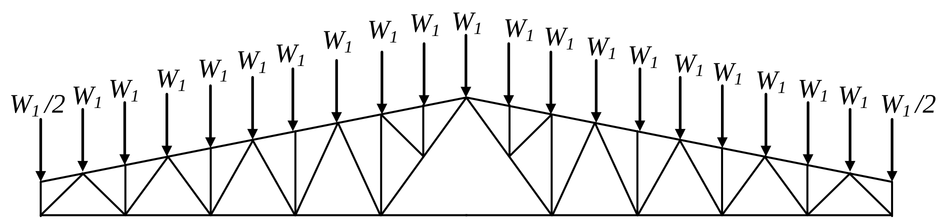
Imposed load:

Live load = 0.35 kN/m^2 *[From IS: 875 – 1964]*

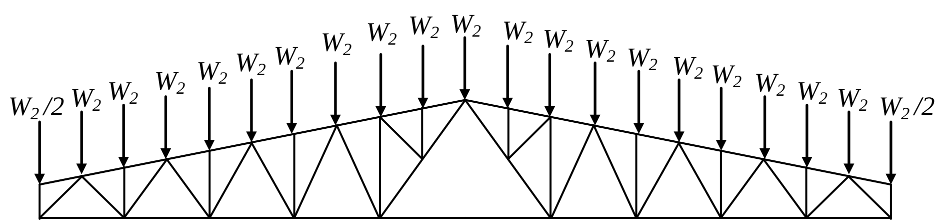
*Load at intermediate nodes, W_2 = 0.35 * 6 * 1.25*
= 2.63 kN
Load at intermediate nodes, W_2 / 2 = 1.32 kN

(Acts vertically downwards)

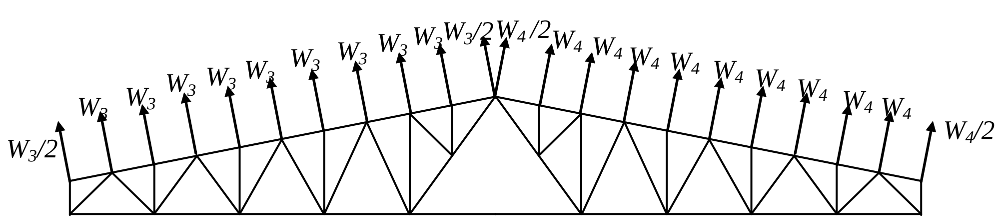
Loading pattern:



(a) Dead load



(b) Live load



(c) Wind load

Structural Steel	Job No:	Sheet *8 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project Calculation Sheet		Made by *SSSR*	Date *9-2-2000*
		Checked by *PU*	Date *16-08-00*

Forces in the members:

The truss has been modeled as a pin jointed plane truss and analysed using SAP90 software. The analysis results are tabulated below.
[See truss configuration for member ID]

Member	***Member Forces (kN)***			
	Dead load	***Live load***	***Wind on side***	***Wind on end***
A-B	*0*	*0*	*1.6*	*1.3*
B-C	*-47.4*	*-40.2*	*214.9*	*172.5*
C-D	*-47.4*	*-40.2*	*218.1*	*175.0*
D-E	*-63.2*	*-53.6*	*284.3*	*228.1*
E-F	*-63.2*	*-53.6*	*287.5*	*230.7*
F-G	*-66.4*	*-56.3*	*294.8*	*236.6*
G-H	*-66.4*	*-56.3*	*298*	*239.1*
H-I	*-63.2*	*-53.6*	*276*	*221.5*
I-J	*-64.5*	*-54.8*	*286.2*	*229.7*
J-K	*-64.5*	*-54.8*	*289.4*	*232.2*
a-A	*-1.6*	*-1.3*	*8.3*	*6.7*
a-B	*-41.6*	*-35.3*	*186.5*	*149.7*
a-b	*29.5*	*25*	*-131.8*	*-105.8*
b-B	*24.1*	*20.5*	*-104.8*	*-84.1*
b-C	*-3.1*	*-2.6*	*16.5*	*13.2*
b-D	*-17.1*	*-14.5*	*70.8*	*56.8*
b-c	*56.5*	*47.9*	*-247.1*	*-198.3*
c-D	*9.5*	*8.1*	*-35.4*	*-28.4*
c-E	*-3.1*	*-2.6*	*16.5*	*13.2*
c-F	*-5.3*	*-4.5*	*14*	*11.2*
c-d	*64.6*	*54.8*	*-274.5*	*-220.3*
d-F	*1*	*0.9*	*5.8*	*4.7*
d-G	*-3.1*	*-2.63*	*16.5*	*13.2*
d-H	*2.4*	*2*	*-23.7*	*-19.0*
d-e	*64.1*	*54.4*	*-262*	*-210.2*
e-H	*-5.1*	*-4.3*	*36.4*	*29.2*
e-I	*-4.6*	*-3.9*	*24.8*	*19.9*
e-f	*11.4*	*9.7*	*-71.1*	*-57.1*
e-h	*55.4*	*47*	*-205.5*	*-164.9*
f-I	*1.8*	*1.6*	*-9.7*	*-7.8*
f-J	*-3.1*	*-2.6*	*16.5*	*13.2*
f-K	*13.6*	*11.6*	*-83*	*-66.6*

Structural Steel	Job No:	Sheet *9 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project **Calculation Sheet**		Made by *SSSR*	Date *9-2-2000*
		Checked by *PU*	Date *16-08-00*

Load factors and combinations:

For dead + imposed
*1.35*DL + 1.5*LL*

For dead + wind
*1.35*DL + 1.5* WL*

For dead + imposed + wind
Not critical as wind loads act in opposite direction to dead and imposed loads

Member Forces under Factored loads in kN:

Member	***Member Design Forces (kN)***	
	DL + WL	***DL + LL***
A-B	*2.4*	*0*
B-C	*258.4*	*-124.3*
C-D	*263.2*	*-124.3*
D-E	*341.1*	*-165.7*
E-F	*345.9*	*-165.7*
F-G	*352.6*	*-174.1*
G-H	*357.4*	*-174.1*
H-I	*328.7*	*-165.7*
I-J	*342.2*	*-169.3*
J-K	*347.0*	*-169.3*
a-A	*10.3*	*-4.1*
a-B	*223.6*	*-109.1*
a-b	*-157.9*	*77.3*

Structural Steel	Job No:	Sheet *10 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project Calculation Sheet		Made by *SSSR*	Date *9-2-2000*
		Checked by *PU*	Date *16-08-00*

Member	*DL + WL*	*DL + LL*
b-B	*-124.7*	*63.3*
b-C	*20.6*	*-8.1*
b-D	*83.1*	*-44.8*
b-c	*-294.4*	*148.1*
c-D	*-40.3*	*25.0*
c-E	*20.6*	*-8.1*
c-F	*13.8*	*-13.9*
c-d	*-324.5*	*169.4*
d-F	*10.1*	*2.7*
d-G	*20.6*	*-8.1*
d-H	*-32.3*	*6.2*
d-e	*-306.5*	*168.1*
e-H	*47.7*	*-13.3*
e-I	*31.0*	*-12.1*
e-f	*-91.3*	*29.9*
e-h	*-233.5*	*145.3*
f-I	*-12.1*	*4.8*
f-J	*20.6*	*-8.1*
f-K	*-106.1*	*35.8*

Top Chord Design:(G-H)

Maximum compressive force = 174.1 kN
Maximum tensile force = 357.4 kN

Trying ISNT 150 X 150 X 10 mm @ 0. 228 kN/ m

Sectional Properties:

Area of Cross section	$= A_t$	$= 2908\ mm^2$
Width of Section	$= 2B$	$= 150$ mm
Thickness of the flange	$= T$	$= 10$ mm
Thickness of the web	$= t$	$= 10$ mm
Radii of gyration:	r_{xx}	$= 45.6$ mm
	r_{yy}	$= 30.3$ mm

Structural Steel	Job No:	Sheet *11 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project **Calculation Sheet**		Made by *SSSR*	Date *9-2-2000*
		Checked by *PU*	Date *16-08-00*

Section classification:

$$\varepsilon = (250/f_y)^{0.5} = (250/250)^{1/2} = 1.0$$

Flange:

$B/T = 75/10 = 7.5 < 8.9\varepsilon$ (*Flange is plastic*)

Web:

$d/t = 140/10 = 14$ $[> 9.975\varepsilon$ and $< 19.95\varepsilon]$

(*Web is semi-compact*)

As no member in the section is slender, the full section is effective and there is no need to adopt reduction factor.

Maximum unrestrained length $= \ell_y = 3810$ *mm*

(Assuming every two alternative nodes are restrained)

$r_{yy} = 30.3$ *mm*

$\lambda_y = 3810/30.3 = 125.7$

Then, $\sigma_c = 84.3$ N/mm^2

*Axial capacity = (84.3/1.15)*2908/1000 = 213.2 kN > 174.1 kN*

Hence, section is safe against axial compression

*Axial tension capacity of the section = 2908 * 250/1.15 = 632 kN > 357.4 kN*

Hence, section is safe in tension.

Bottom chord design:(c-d)

Maximum compressive force = 324.5 kN
Maximum tensile force = 169.4 kN [Try same section as top chord]
*Axial tension capacity of the selected section = 2908 * 250/1.15 = 632 kN*

Hence, section is safe in tension.

Structural Steel	Job No:	Sheet *12 of 13*	Rev
	Job Title: *ROOF TRUSS*		
	Worked Example - 1		

Design Project Calculation Sheet		Made by SSSR	Date 9-2-2000
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Maximum unrestrained length = ℓ_y = 2500 mm

(Assuming every node is restrained by longitudinal tie runner)

r_{yy} = 30.3m

λ_y = 2500/30.3 = 82.5

Then, σ_c = 145.5 N/mm^2

*Axial capacity = (145.5/1.15)*2908/1000 = 368 kN > 324.5 kN*

Hence, section is safe against axial compression also.

Web member design:(b-B)

Maximum compressive force = 124.7 kN

Maximum tensile force = 63.3 kN

Try – ISA 80 X 80 X 8.0

A = 1221 mm^2

r_{xx} = 24.4 mm

r_{uu} = 30.8 mm

Section classification:

b/t = 80/8 = 10.0 <14.0

Hence, the section is not slender

Length of member $= (1250^2 + 1250^2)^{0.5} = 1767.5$ mm

Slenderness ratio is taken as the greater of

0.85 * 1767.5/24.4 = 61.6

1.0 * 1767/ 30.8 = 57.4

Structural Steel	Job No:	Sheet 13 of 13	Rev
	Job Title: ROOF TRUSS		
	Worked Example - 1		

Design Project Calculation Sheet		Made by *SSSR*	Date *9-2-2000*
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Then, σ_c $= 182.1\ N/mm^2$

Design compressive strength = *1221 * (182.1/1.15)/1000*

=*193.3 kN > 124.7 kN*

Hence, safe in compression.

Tensile capacity of the section = *(250/1.15)*1221/1000*

= *265.4 kN > 63.3 kN*

Hence ISA 80 X 80 X 8.0 is adequate for the web member

(The web members away from the support would have lesser axial force but longer and can be redesigned, if so desired)

Structural Steel	Job No:	Sheet *1 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		

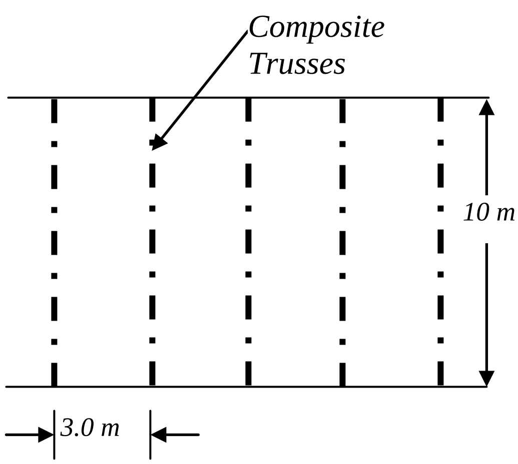
Design Project **Calculation Sheet**		Made by *SSSR*	Date *17-10 -99*
		Checked by *PU*	Date *16-08-00*

PROBLEM 2:

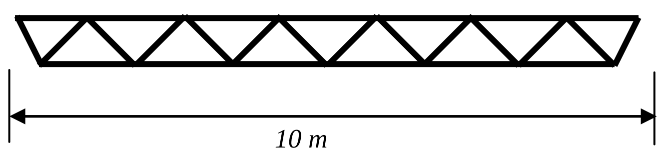
Design a composite truss of span 10.0 m with following data:

DATA:

Span $= \ell = 10.0$ *m*
Truss spacing $= 3.0$ *m*
Slab thickness $= D_s = 150$ *mm*
Profile depth $= D_p = 75.0$ *mm*
Self weight of deck slab $= 2.80$ *kN/m*^{2}
Maximum laterally un-restrained length in top chord is 1.5 m.
Grade of concrete, M20 $=(f_{ck})_{cu}=20$ *MPa*



FLOOR PLAN



TRUSS ELEVATION

Structural Steel Design Project	Job No:	Sheet *2 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
		Made by *SSSR*	Date *17-10 -99*
Calculation Sheet		Checked by *PU*	Date *16-08-00*

Loading:

	kN/m^2	Factored Load (kN/m^2)
Deck slab weight	*2.8*	*2.8*1.35 = 3.78*
Truss weight (assumed)	*0.4*	*0.4*1.35 = 0.54*
Ceiling, floor finish and Services	*1.0*	*1.0*1.35 = 1.35*
Construction Load	*1.0*	*1.0*1.5 = 1.5*
Superimposed live load	*5.0*	*5.0*1.5 = 7.5*

PRE-COMPOSITE STAGE:

Loading	kN/m^2	Factored Load (kN/m^2)
Deck slab weight	*2.8*	*2.8*1.35 = 3.78*
Truss weight	*0.4*	*0.4*1.35 = 0.54*
Construction load	*1.0*	*1.0*1.5 = 1.5*

Total factored load = $5.82\ kN/m^2$

Choose depth of truss = *Span/20* = *10000/20* = *500 mm*

Total factored load = 5.82 * 3 = 17.5 kN/m

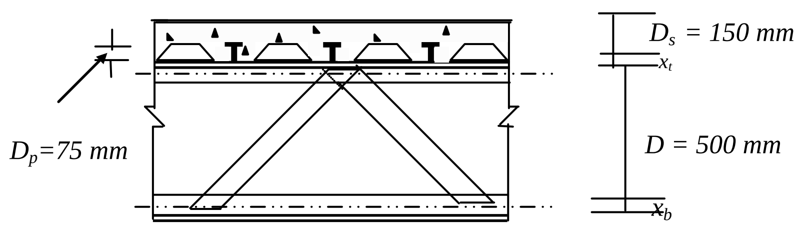
Maximum bending moment = $w\ell^2/8$ = $17.5 \times 10^2/8 = 218.7$ kN-m

Maximum shear = $w\ell/2 = 17.5 \times 10/2 = 87.5$ kN

Depth of truss (centre to centre distance of chords) = 0.5 m

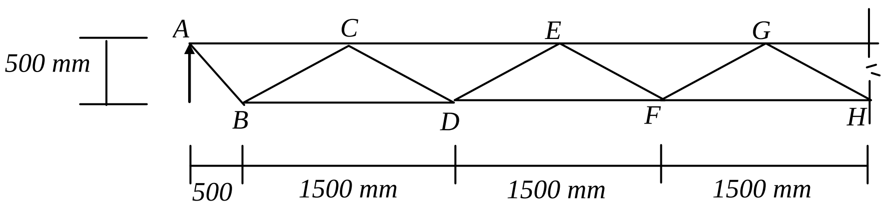
Maximum axial compressive force in top chord = 218.7/0.5 =437.4 kN

Structural Steel Design Project	Job No:	Sheet *3 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
		Made by *SSSR*	Date *17-10 -99*
Calculation Sheet		Checked by *PU*	Date *16-08-00*



$D_t = 500 + x_t + x_b$

Truss configuration: *Choose the following truss configuration*



Top chord design:

Try ISNT 150 X 150 X 10 mm @ 0.228 kN/m

Sectional properties:

Area of cross-section $= A_t = 2908\ mm^2$

Depth of section $= 150\ mm$

Width of section, b $= 2b^1 = 150\ mm$

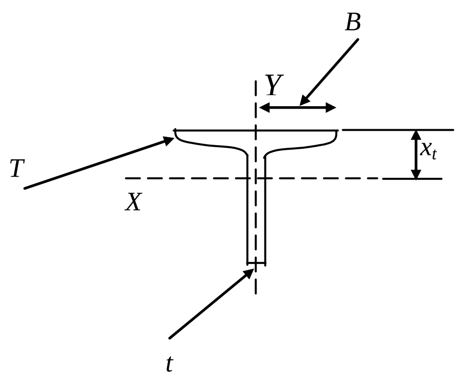
Thickness of flange $= T = 10.0\ mm$

Thickness of web $= t = 10.0\ mm$

Centre of gravity $= x_t = 39.5\ mm$

$r_{xx} = 45.6\ mm$

$r_{yy} = 30.3\ mm$



Structural Steel Design Project	Job No:	Sheet *4 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
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Calculation Sheet		Checked by *PU*	Date *16-08-00*

Section classification:

$$\varepsilon \quad =(250/f_y)^{0.5} \quad = (250/250)^{1/2} = 1.0$$

Flange:

$b^1/T = 75/10 = 7.5 < 8.9\varepsilon$ *Flange is plastic*

Web:

$d/t = 140/10 = 14\ (> 9.98\varepsilon \text{ and } < 19.95\varepsilon)$ *Web is semi-compact*

As no member in the section is slender, there is no need of adopting reduction factor (Yielding govern).

Given, maximum un-restrained length of top chord is 1.5 m during construction stage.

Maximum unrestrained length = $\ell_y = 1500$ mm

$\ell_x = 0.85*1500 = 1275$ mm

$r_{xx} = 45.6$ mm

$r_{yy} = 30.3$ mm

$\lambda_x = 1275/45.6 = 28$

$\lambda_y = 1500/30.3 = 49.5$

Then, $\sigma_c = 202.8$ N/mm^2 *[From Table - 3 of Chapter on axially compressed Columns]*

*Axial capacity = (202.8/1.15)*2908/1000 = 512.8 kN > 437.4 kN*

Hence, section is safe against axial compression at construction stage.
[Other member design is governed by composite loading]

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet *5 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
		Made by *SSSR*	Date *17-10 -99*
		Checked by *PU*	Date *16-08-00*

COMPOSITE STATE:

	kN/m^2	Factored Load (kN/m^2)
Deck slab weight	2.8	2.8*1.35 = 3.78
Truss weight (assumed)	0.4	0.4*1.35 = 0.54
Ceiling, floor finish and Services	1.0	1.0*1.35 = 1.35
Superimposed live load	5.0	5.0*1.5 = 7.5

Total factored load = (3.78+0.54+1.35+7.5)* 3
= 13.2*3 = 39.5 kN/m

Maximum bending moment (M_c) $= w\ell^2/8 = 39.5*10^2/8 = 493.7$ kN-m

Maximum shear $= w\ell/2 = 39.5*10/2 = 197.5$ kN

Bottom chord design:

Force in bottom chord, $R_{b,req}$ *is given by: [See Fig. of the text]*

$R_{b,req}\{D + x_t + D_s-(D_s-D_p)/2\} = M_c$

[Assume NA is in the concrete slab]

$R_{b,req}(500+39.5+(150-37.5))/1000 = 493.7$

$R_{b,req}(652/1000) = 493.7$ kN-m

$R_{b,req} = 493.7/0.652 = 757.2$ kN

Area required $= 757.2*1000/(f_y/1.15)$
$= 757.2*1000/(250/1.15) = 3483\ mm^2$

Trial-1 *Trying ISHT 150 @ 0.294kN/m*

Sectional properties:

$A = 3742\ mm^2$; x_b = Centre of gravity = 26.6 mm

Width of the section, $b = 2b_1 = 250$ mm

Structural Steel Design Project	Job No:	Sheet *6 of 12*	Rev
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	Worked Example - 2		
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Axial tension capacity of the selected section (R_b):

$R_b = (250/1.15)* 3742/1000 = 814 \text{ kN} > 757.2 \text{ kN}$

<u>Hence, O.K.</u>

<u>Capacity of Composite Section in Compression:</u>

Capacity of concrete slab, R_c, is given by

$R_c = 0.45\ (f_{ck})_{cu}*b_{eff}*(D_s - D_p)$

Effective width of the slab, b_{eff}: *[See the chapter Composite beams – II]*

$b_{eff} \leq \ell/4 = 10000/4 = 2500 \text{ mm}$

Therefore, $b_{eff} = 2500 \text{ mm}$

$R_c = 0.45*20*2500*75/1000$ $\{f_{ck} = 20 \text{ N/mm}^2\}$
$= 1687.5 \text{ kN} > R_b$ *(tension governs)*

Neutral axis depth :

$x_c = (D_s - D_p)*814/1687.5 = 75*814/1687.5 = 36.2 \text{ mm}$
$D_t = 0.5+0.0266+0.0395 = 0.566 \text{ mm}$

Then, maximum moment it can carry

$M_{u,\ design} = 814(0.566+0.15-0.5*0.0362-0.0266)$
$= 546 \text{ kN-m} > 493.7 \text{ kN-m}$

<u>Hence, the slab and chord members are designed.</u>

Structural Steel Design Project	Job No:	Sheet *7 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
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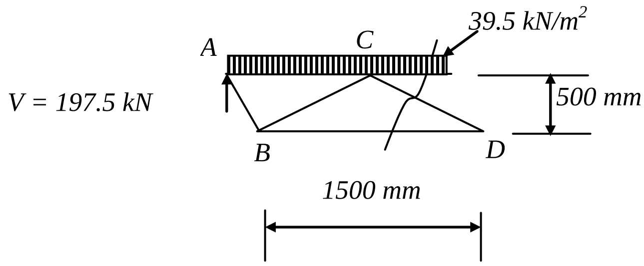
Web members:



F_{AB} $= V(1.414) = 197.5(1.414)$ $= 279$ *kN (tension)*

F_{BC} $= (V-0.5*39.5)(500^2+750^2)^{0.5}/500$ $=320$ *kN (compression)*

F_{CD} $=(V-2.0*39.5)(500^2+750^2)^{0.5}/500$ $= 214$ *kN (tension)*

Hence, maximum tensile force in bracing members =279 kN
Maximum compressive force in bracing members =320 kN

Design of tension members:

Trial gross area required $=279*10^3/(250/1.15)$ $=1283.4\ mm^2$

Trying 2 – ISA 70 X70 X6 .0 @ 0.126 kN/m

A_{gross} *provided* $=2*806$ $=1612\ mm^2$

Effective area:

(Assume, angle is welded to T- section)

$A_{net\ effective}$ $= 1612\ mm^2$

Axial tension capacity $= A_e*(f_y/\gamma_m)$

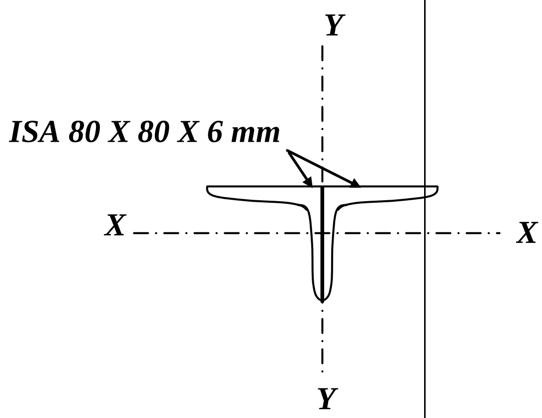
$= 1612*250/1.15$

$= 350.4\ kN > 279\ kN$

<u>*Hence, 2 – ISA 70 X 70 X 6.0 are adequate*</u>

Structural Steel Design Project	Job No:	Sheet *8 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
		Made by *SSSR*	Date *17-10 -99*
Calculation Sheet		Checked by *PU*	Date *16-08-00*

Design of compression member:



Maximum compressive load = 320 kN
Trying 2 – ISA 80 X 80 X 6.0 @ 0.146 kN/m
$A = 1858\ mm^2$
$r_{xx} = 24.6\ mm$
$r_{uu} = 34.9\ mm$

Section classification:

$b/t = 80/6 = 13.3 < 15.75\varepsilon$

Hence, the section is not slender and no need to apply any reduction factor.

Slenderness ratio is taken as the greater of

Length of member $= (750^2 + 500^2)^{0.5} = 901\ mm$
$\lambda_{xx} = 0.85 * 901/24.6 = 31.1$
$\lambda_{xx} = 1.0 * 901/34.9 = 25.8$

Design buckling strength = $\sigma_c = 231.2$ *Mpa*
[Table – 3 of chapter on axially compressed columns]

Design compressive strength $= 1858 * (231.2/1.15)/10^3 = 373.6\ kN > 320\ kN$

<u>*Hence the 2 – ISA 80 X 80 X 6.0 are adequate for the web members*</u>

(The web members away from the support would have lesser axial force and can be redesigned, if so desired. Preferably use the same section for all web members)

Structural Steel Design Project	Job No:	Sheet *9 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
		Made by *SSSR*	Date *17-10 -99*
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Weight Schedule:

Description	*Section mm X mm X mm*	*Weight kN/m*	*Number*	*Length (m)*	*Total Length (m)*	*Weight kN*
Top Chord	*ISNT 150 X150X10*	*0.228*	*1*	*10.0*	*10.0*	*2.28*
Bottom Chord	*ISHT 150*	*0.294*	*1*	*10.0*	*10.0*	*2.94*
Bracing Members	*2-ISA 70 X 70 X 6*	*0.126*	*2*	*0.71*	*1.42*	*0.18*
Tension Members	*2-ISA 70 X 70 X 6*	*0.126*	*6*	*0.9*	*5.4*	*0.68*
Compression Members	*2-ISA 80 X 80 X 6*	*0.146*	*6*	*0.9*	*5.4*	*0.79*
						6.87
Allow 2 ½ % Extras						*0.17*
						7.04

Average weight per unit area of floor

$$= \frac{7.04}{10*3} = 0.23\ kN/m^2 < 0.4\ kN/m^2\ (Assumed)$$

Hence, O.K.

Structural Steel Design Project	Job No:	Sheet *10 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 2		
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Deflection:

Pre-composite stage:

The second moment of area of the steel truss, I_t *can be calculated from the following equation.*

$$I_t = \frac{A_b A_t}{(A_b + A_t)} [D_t - x_b - x_t]^2$$

Where,

A_b - *Cross-sectional area of bottom chord.*

A_t - *Cross-sectional area of top chord.*

In this problem,

$A_b = 3742\ mm^2$

$x_b = 26.6\ mm$

$A_t = 2908\ mm^2$

$x_t = 39.5\ mm$

$D_t = 566\ mm$

$$I_t = \frac{3742 \times 2908}{(3742 + 2908)} [566 - 26.6 - 39.5]^2$$

$$= 409 \times 10^6\ mm^4$$

Loading:

	kN/m^2
Deck slab weight	*2.80*
Truss weight	*0.23*
Construction load	*1.00*

	4.03

Total Load = *4.03*3*10=121 kN*

Structural Steel Design Project	Job No:	Sheet *11 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 11		
		Made by *SSSR*	Date *17-10 -99*
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Deflection at pre composite state is given by

$$\delta_0 = (5*121*10000^3)/(384*200*409*10^6) = 19.3\ mm$$

Deflection at composite state due to dead load = δ_1 *= (3.03/4.03)*19.3*
= 14.5 mm

[For composite stage construction load has to removed for calculating deflections]

Deflection - Composite stage:

The second moment of area, I_c*, of a composite truss can be calculated from the following equation*

$$I_c = \frac{A_b A_c / m}{(A_b + A_c / m)} \left[D_t + (D_s + D_p) / 2 - x_b\right]^2$$

Where,

A_c = *Cross-sectional area of the concrete in the effective breadth of slab*
= $(D_s - D_p) b_{eff}$
m = *modular ratio*

In this problem,

A_b = $3742\ mm^2$; $b_{eff} = 2500\ mm$
A_c = $(150 - 75) * 2500 = 1875*10^2\ mm^2$
m = *15(light weight concrete)*
D_t = $566\ mm$
x_b = $26.6\ mm$

$$I_c = \frac{3742 \times 1875 \times 10^2 / 15}{(3742 + 1875 \times 10^2 / 15)} \left[566 + \frac{225}{2} - 26.6\right]^2$$

$$= 1224 \times 10^6\ mm^4$$

Structural Steel Design Project	Job No:	Sheet *12 of 12*	Rev
	Job Title: *COMPOSITE TRUSS*		
	Worked Example - 12		
		Made by *SSSR*	Date *17-10 -99*
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Loading:

Super Imposed load $= 5.0\ kN/m^2$

Total Load $= 5.0*3*10=150\ kN$

Deflection at composite state due to superimposed load is given by

$\delta_2 = (5*150*10000^3)/(384* 200* 1224*10^6) = 8.0\ mm$

10% allowance is given

Then, $\delta_2 = 8.8\ mm < \ell/360 = 10000/360 = 28\ mm$

Total deflection $= \delta_1 + \delta_2 = 14.5 + 8.8 = 23.3\ mm\ (\ell/429) < (\ell/325)$

Hence, design is O.K.

<table>
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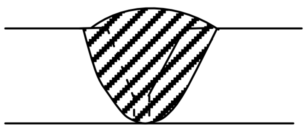
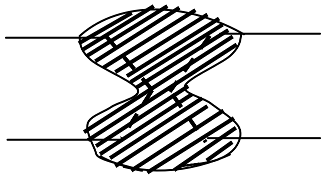
PROBLEM 1:

Two plates 14 mm thick are joined by (I) a double-V butt weld, (II) a single-V butt weld. Determine the strength of the welded joint in tension in each case. Effective length of the weld is 20 cm. Yield strength of steel = 250 N/mm^{2,} Partial safety factor for strength = 1.15.

Solution:



Single V butt weld



Double V butt weld

I. *In the case of double V butt weld, complete penetration of the weld would take place.*
Therefore effective throat thickness = 14 mm (thickness of the parent metal
Effective length of weld = 200 mm
Factored yield stress of the member = 250/1.15 = 217.4 N/mm^2
Design ultimate Strength of the
*Single V butt weld = (217.4 *14 * 200)/1000*
= 608.72 kN

II *For single V butt weld the penetration would be incomplete*
*Therefore effective throat thickness = 5/8 * 14 = 8.75 mm*
(IS: 816-1969)
Effective length of weld = 200 mm
Design ultimate Strength of the
*double V butt weld = (217.4 * 8.75 * 200)/1000*
= 380.45 kN

Note: The design ultimate strength of the welds are presented along with the factored strength and in no case these should be used with working loads.

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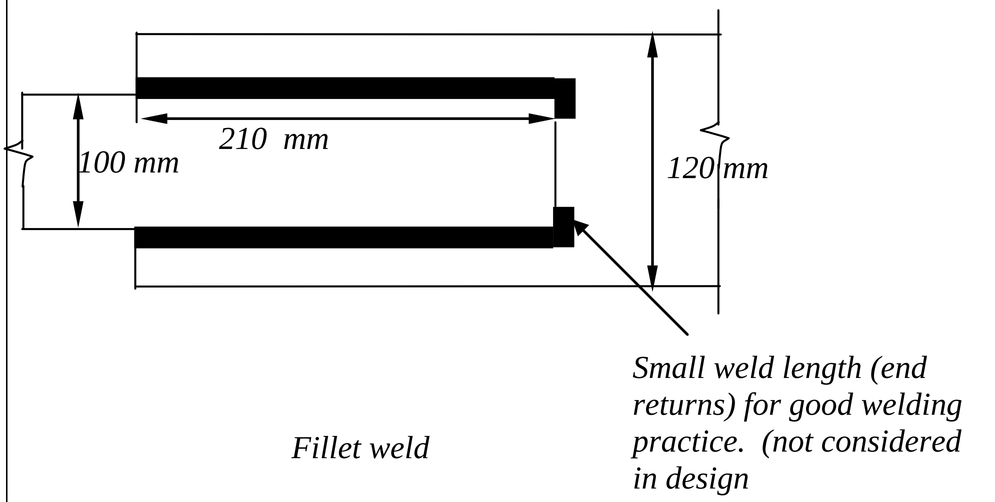
Date.</td></tr>
</table>

PROBLEM 2:

Design a suitable side fillet weld to connect two plates 100 mm × 10 mm and 120 mm × 12 mm, and to transmit tension equal to the full strength of the thin plate. Factored shear stress in the weld is 125.5 N/mm² and factored yield stress of the plate is 217.4 N/mm².



Solution:

Minimum size of the weld specified in IS:816(1969) for 12 mm thick plate is 5 mm.

Maximum size of fillet weld based on the thickness of the plate, i.e.,
= (10.0 – 1.5) = 8.5 mm

A fillet weld of 6 mm may be suitable.

Tension transmitted by 1 mm of weld length = 125.5 1* 0.7*6*
= 527.1 N

Tensile strength based on the thinner plate = 217.4 100* 10*
= 217.4 kN

Structural Steel Design Project	Job No:	Sheet: *2* of *2*	Rev:
	Job title: *Weld Design (Static)*		
	Worked example: *2*		
		Made by *SSR*	Date.
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Length of weld required $= 217400 / 527.1$
$= 413$ *mm*

Therefore, a weld length of 210 mm each on either longitudinal side can be provided.

<u>To ensure good fabrication practice the following checks are to be made</u>

- *The welds are to be checked for, a) whether sufficient weld lengths are provided on either side, b) This is greater than the width of the thinner plate. In this particular case 210 mm > 100 mm.*

- *The spacing of the longitudinal welds should be checked so as to be less than 16 times the thickness of the plate. In the present case 100 mm < 16 * 10 mm.*

- *It is also a good fabrication practice to take the weld round the corner for a small distance, normally twice the weld thickness (end return).*

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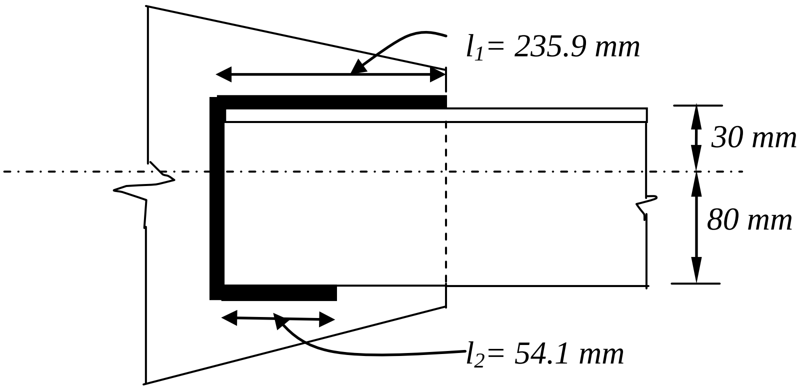
	Job title: *Weld Design (Static)*		
	Worked example: *3*		
		Made by *SSR*	Date
Calculation Sheet		Checked by *SAJ*	Date

PROBLEM 3:

In a roof truss, a tie member ISA 110 mm x *110mm* x *8 mm carries a factored tension of value 210 kN. The tie is connected to a gusset plate 8 mm thick. Design the welded joint. Factored yield strength of steel is 217.4 N/mm^2 and shear strength of weld is 125 N/mm^2.*



Fillet weld for tie member of a roof truss

Solution:

For this problem we would provide a weld group consisting of transverse and longitudinal welds and ensure that the CG of the weld group coincides with the line of action of the externally applied load.

First we would decide about the weld size. This is decided by the thickness of the rolled section and the plating. Weld which are applied to rounded toe of rolled section should not be more than ¾ of its thickness or plating and hence we get a weld size of 6mm (3/4 8). The maximum size of the end weld is also limited by the thickness of the plating, which is 8-1.5=6.5 mm. Hence 6 mm fillet welds are O.K.*

Transverse weld is provided equal to the size of the leg = 110 mm.

Structural Steel Design Project	Job No:	Sheet : *2* of *2*	Rev:
	Job. Title: *Weld Design (Static)*		
	Worked Example: *3*		
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Force transmitted by transverse weld = (125 0.7* 6* 110) /1000*
=57.75 kN

Remaining force to be transmitted by the longitudinal welds = 210 – 57.75
=152.25 kN

We must ensure that the CG of the welds coincides with line of action of the external force. This could be ensured by providing longitudinal welds along the near and far side of the angle and also by ensuring that the moment of the all the forces about any of the line of the weld vanishes.

Let us assume that the lengths of the welds in the heel and toe sides are l_1 *and* l_2 *respectively.*

Total weld length required for 152.25 kN
*=152.25 * 1000 / ((125* 0.7*6))=290 mm*

Taking moment of all forces about the heel side longitudinal weld, we get

$57.75 * 1000 * 55 + l_1 * 0 + l_2 * (125. * 0.7 * 6) * 110$
$= 210 * 1000 * 30.$
Therefore l_2 $= 54.09$ *mm*

Hence we get the weld length l_2 *as say 54.09 mm. The bracketed term in the above expression represents the strength of the weld for 1 mm.*

Now we get the length l_1 *as 290 – 54.09 =235.91 mm*

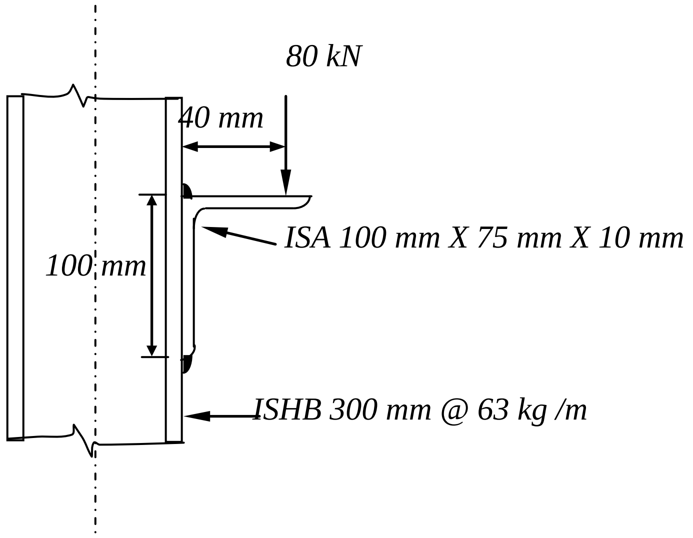
Alternatively the longitudinal weld length l_1 *is obtained by taking moment of all the forces about the toe side weld line. Hence we have demonstrated as to how a weld group could be designed to have a CG coinciding with the externally applied load.*

It is also to be noted that in case it is desired to reduce the length of the joint then the heel side weld size can be increased.

Structural Steel Design Project	Job No:	Sheet: *1* of *2*	Rev:
	Job. Title: *Weld Design (Static)*		
	Worked Example: *4*		
Calculation Sheet		Made by *SSR*	Date
		Checked by *SAJ*	Date

PROBLEM 4:

An ISA 100 mm X 75 mm X 10 mm is welded with the flange of a column ISHB 300, @63 kg/mm. The bracket carries factored load of 80 kN at a distance of 40 mm from the face of the column. Design the bracket connection. Factored yield stress of steel is 217.4 N/mm^2.



Welded bracket connection

Solution:
For the rolled section ISHB 300 @63 kg/ m

Flange width = 250 mm

We see that the external load not only causes shear but also a bending moment in the plane of the web. Hence it is necessary to provide two weld lines such that they produce a resisting couple to oppose the applied bending moment. In this particular case, as shown in the figure, we can provide weld at the top and bottom of the seating angle or bracket. Hence the top weld would be under tension and bottom weld would be under compression producing a couple whose lever arm would be 100 mm (length of the connected leg).

We also see that both welds share the shear force. Hence the welds are subjected to simultaneous action of shear and tension or compression.

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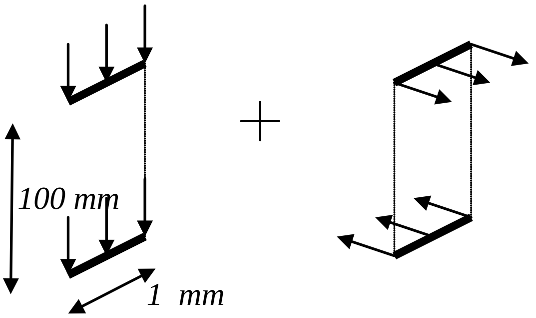
	Job. Title: *Weld Design (Static)*		
	Worked Example: *4*		
Calculation Sheet		Made by *SSR*	Date
		Checked by *SAJ*	Date

Having decided about the weld lengths (250 mm at top and bottom of the bracket) we can calculate the size of the weld 't'.

Shear force in 1mm length size of the weld. = 80000/(2 250)=160 N*



*Bending moment about the face of the column=40 * 80000*
$= 32 * 10^5$ *N-mm*

Force in the weld per 1 mm length due to bending moment $= 32 * 10^5 / (250 * 100) = 128$ *N*

Resultant force due shear and bending = $\sqrt{160^2 + 128^2} = 205$ *N*

This resultant force should not exceed the design strength of the weld.

Therefore, $t * 0.7 * 125 \geq 205$

Solving for t,

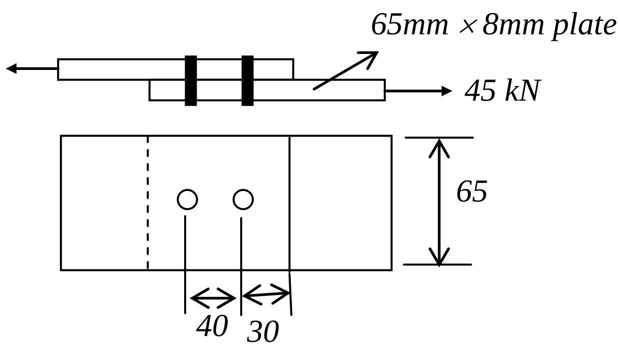
$t \geq 2.34$ *mm*

Hence provide a weld size of 3 mm.

Structural Steel Design Project	Job No:	Sheet *1 of 1*	Rev
	Job Title: ***Bolted Connections***		
	Worked Example – 1 Lap joint		
Calculation Sheet		Made by *SRSK*	Date *15-07-00*
		Checked by *RN*	Date

	Remarks
Design Example 1*: Design a Lap joint between plates 65 × 8 as shown in Fig. E1 below so as to transmit a factored load of 45 kN using black bolts of 16mm diameter and grade 4.6. The plates are made of steel of grade ST-42-S.*	

Fig. E1



Solution:

1) Strength Calculations:

Nominal diameter of bolt d= 16 mm,
hole diameter D = 16 +1.5 = 17.5 mm

Shear Area of one bolt $A_s = 0.8A = 0.8 \times 201.06 = 161 \text{ mm}^2$

Shear strength of each bolt $= p_s A_s = 160 \times 161 \times 10^{-3} = 25.76$ *kN* — Eq. (7)

Since p_{bb} *for bolt is greater than* p_{bs} *of plate, plate will fail in bearing before the bolt.*

Bearing strength of plate $= p_{bs}\, d\, t = 418 \times 16 \times 8 \times 10^{-3} = 53.5$ *kN* — Eq. (9)

Therefore, bolt value = 25.76 kN

No. of bolts required = 45/25.76 = 1.75 say 2 bolts

2)Detailing:

Minimum pitch = 2.5 d = 40 mm

Minimum edge distance = 1.4 D = 24.5 mm say 25 mm

Provide 2 bolts as shown.

Structural Steel Design Project	Job No:	Sheet *1 of 1*	Rev
	Job Title: ***Bolted Connections***		
	Worked Example – 2 Hanger joint		
		Made by *SRSK*	Date *15-7-00*
Calculation Sheet		Checked by *RN*	Date

Remarks

***Design Example 2**: Design a hanger joint along with an end plate to carry a downward load of 330 kN. Use end plate size 240 mm × 160 mm and appropriate thickness and M25 HSFG bolts (2 nos).*

Solution

Assume 10mm fillet weld between the hanger plate and the end plate
Distance from center line of bolt to toe of fillet weld b = 60 mm

1) For minimum thickness design, M = Tb/2 = 165 × 60/2 = 4950 N-m

$$\therefore t_{\min} = \sqrt{\frac{1.15 \times 4 \times 4950 \times 10^3}{236 \times 160}} = 24.56 \text{ say } 25\, mm$$

2) Check for prying forces

distance'n' from center line of bolt to prying force is the minimum of edge distance or 1.1t $\sqrt{(\beta p_o/f_y)}$ *= 1.1 x 25* $\sqrt{(2 \times 510/236)}$ *= 57 mm* — Eq. (6)

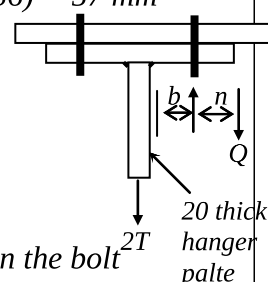
$\therefore$ *n = 40 mm*

prying force = M/n = 4950/40= 123.75 kN

bolt load = 165+123.75=288.75 kN

tension capacity of 25 mm dia HSFG bolt = 0.9Po — Eq. (2), Table 3

=0.9×195.6=176 kN << 288.75 unsafe !



3) In order to reduce the load on bolt to a value less than the bolt capacity, a thicker end plate will have to be used.

Allowable prying force Q = 176-165 = 11 kN

Trying a 40 mm thick end plate gives n = 40 mm as before

Moment at toe of weld = Tb-Qn = 165 × 60 – 11 × 40 = 9460 N-m

Moment capacity = $(236/1.15)(160 \times 40^2/4) \times 10^{-3}$

= 13134 N-m > 9460 OK

Minimum prying force — Eq. (3)

$$= \frac{b}{2n}\left[T - \frac{\beta \gamma p_o w t^4}{27 n b^2}\right] = \frac{60}{2 \times 40}\left[165 - \frac{2 \times 1.5 \times 0.512 \times 160 \times 40^4}{27 \times 40 \times 60^2}\right]$$

= 2.4 kN < 11 kN $\therefore$ *safe!*

Therefore, 40 mm end plate needs to be used to avoid significant prying action. — Eq. (1)

Structural Steel Design Project Calculation Sheet	Job No:	Sheet **1 of 1**	Rev
	Job Title: *Eccentrically Loaded Bolt Group*		
	Worked Example – 1		
		Made by *SRSK*	Date *01-10-00*
		Checked by VK	Date

Design Example 1: *Design a bolted connection between a bracket 8 mm thick and the flange of an ISHB 400 column using HSFG bolts, so as to carry a vertical load of 100 kN at a distance of 200 mm from the face of the column as shown in Fig. E1.*

Remarks Ref: Section 2.1

Solution:

1) Bolt force:

$P_x = 0$; $P_y = 100$ kN;

Total eccentricity $x' = 200 + 250/2 = 325$ mm

$M = P_y x' = 100 \times 325 = 32500$ kN-mm

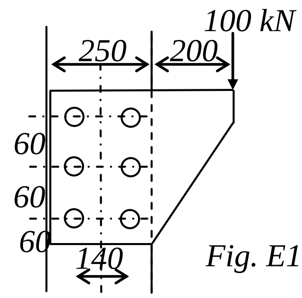


Fig. E1

Try the arrangement shown in Fig. E1
Note: minimum pitch = 60 mm and
minimum edge dist. = 60 mm

$n = 6$

$\sum r_i^2 = \sum x_i^2 + \sum y_i^2 = 6(70)^2 + 4(60)^2 = 43800$ mm^2 — *Equation (8)*

Shear force on the farthest bolts (corner bolts)

$$R_i = \sqrt{\left\{\left[\frac{32500 \times 60}{43800}\right]^2 + \left[\frac{100}{6} + \frac{32500 \times 70}{43800}\right]^2\right\}} = 81.79\ kN$$

2) Bolt capacity

Try M20 HSFG bolts

Bolt capacity in single shear $= 1.1\, K\, \mu\, P_o = 1.1 \times 0.45 \times 177 = 87.6$ kN

ISHB 400 flange is thicker than the bracket plate and so bearing on the bracket plate will govern.

Bolt capacity in bearing $= d\, t\, p_{bg} = 20 \times 8 \times 650 \times 10^{-3} = 104$ kN

$\therefore$ *Bolt value = 87.6 kN > 81.79 safe.*

Use 6 M20 HSFG bolts as shown.

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Beam Splice*		
	Worked Example – 2		
		Made by *SRSK*	Date *01-10-00*
Calculation Sheet		Checked by *VK*	Date

Design Example 2: *Design a bolted splice for an ISMB 450 section to transfer a factored bending moment of 150 kN-m and a factored shear of 100 kN. Assume that the flange splices carry all of the moment and that the web splice carries only the shear.*

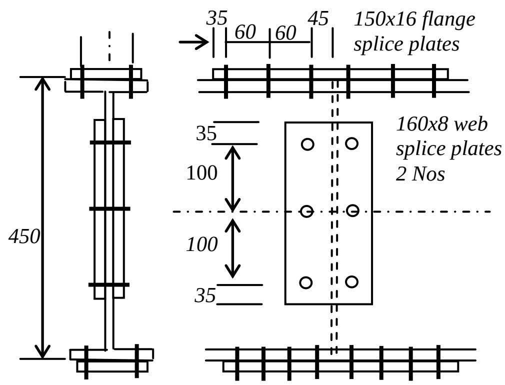


Fig. E2

Solution:

1) Flange Splices :

Flange force $=BM/(D-t_f) = 150 \times 10^3/(450-17.4) = 346.7$ *kN*

For M20 Gr.8.8 HSFG bolts in single shear

Slip resistance per bolt $= 1.1 \times 0.45 \times 144 = 71.3$ *kN* — $1.1K_s\mu P_o$

Bearing resistance on flange per bolt $= 20 \times 17.4 \times 650 \times 10^{-3} = 226.2$ *kN* — dtp_{bg}

Bolt value = 71.3 kN

Use 3 rows of 2 bolts at a pitch of 60 mm

Net area of flange $= (150-2 \times 22)\ 17.4 = 1844.4\ mm^2$

Flange capacity $= (250/1.15) \times 1844 \times 10^{-3} = 400.9$ *kN > flange force OK* — $\gamma_m=1.15$

Try 150 mm wide splice plate

Thickness of splice plate required

$= 346.7 \times 10^3/1.0 \times 250(150-2 \times 22)/1.15 = 15.1$ *mm Use 16 mm*

Use flange splice plate of size $400\times150 \times 16$ — *Flange splice plate of size* $400\times 150 \times 16$

<table>
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<tr><td colspan="3">Worked Example – 2</td></tr>
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</table>

2) Web Splice

For M20 HSFG bolts of Gr.8.8 in double shear
Slip resistance per bolt $= 2 \times 1.1 \times 0.45 \times 144 = 142.6$ *kN*

Try 8 mm thick web splice plates on both sides of the web.

Therefore bearing on web will govern
Bearing Resistance per bolt $= 20 \times 9.4 \times 650 \times 10^{-3} == 122.2$ *kN*
Bolt value = 122.2 kN

Try 3 bolts at 100 mm vertical pitch and 45 mm from the center of joint.

Horizontal shear force on bolt due to moment due to eccentricity
$= 100 \times 45 \times 100/(2 \times 100^2) = 22.5$ *kN*

Vertical Shear force per bolt = 100/3 = 33.3 kN

Resultant shear force $= \sqrt{(22.5^2+33.3^2)} = 40.2$ *kN < 122.2(bolt cap) OK*

Use web splice plate of size $270 \times 160 \times 8$ *- 2 nos.*

Web splice plate of size $270 \times 160 \times 8$ *with 3 M20 bolts on each side of the splice.*

Structural Steel Design Project		Sheet **1 of 2**	Rev
	Job Title: *Column Splice*		
	Worked Example – 3		
		Made by *SRSK*	Date *01-10-00*
Calculation Sheet		Checked by *VK*	Date

Design Example 3:*Design a bolted cover plate splice for an ISHB 200 @ 50.94 kg/m column supported by an ISHB 200 @ 47.54 kg/m column so as to tramsfer a factored axial load of 440 kN. The splice is near a point of lateral restraint. The ends are not prepared for full contact in bearing.*

Remarks

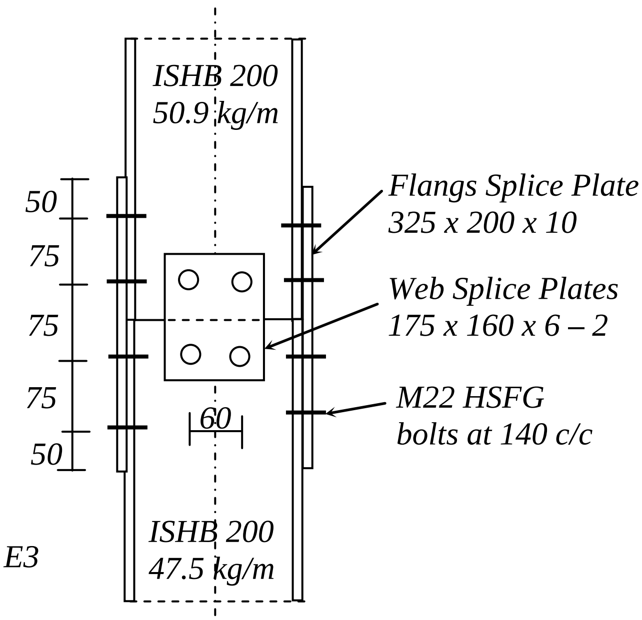

Fig. E3

Solution:

1) Area of ISHB 200 @ 47.54 kg/m section = 4754 mm^2

Area of web = $(200-2\times 9)\times 6.1 = 1110.2\ mm^2$

2) Web Splice

Portion of load carried by web = $440 \times 1110.2/4754 = 102.8$ kN

For M22 HSFG bolts, 2 Nos in double shear

Shear force /bolt = 102.8/2 = 51.4 kN

Slip resistance/bolt = $2 \times 1.1 \times 0.45 \times 177 = 175.2$ kN

Bearing resistance/bolt = $22 \times 6.1 \times 650 \times 10^{-3} = 87.62$ kN

Bolt value = 87.62 kN > bolt force of 51.4 kN $\therefore$ OK

End distance > $51.4 \times 10^3/(1/3 \times 6.1 \times 650) = 39$ mm

Also end distance > 1.4(22+1.5) = 35 mm Use 50 mm

Use $175 \times 160 \times 6$ mm web splice plates – 2 Nos.

Web splice
$175 \times 160 \times 6$

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***2 of 2***	Rev
	Job Title: *Column Splice*		
	Worked Example - 4		
		Made by *SRSK*	Date 0*1-10-00*
		Checked by *VK*	Date

3) Flange Splice *Portion of load carried by each flange = 0.5(440-102.8) = 168.6 kN* *For M22 HSFG bolts, 4 Nos in single shear* *Shear force /bolt = 168.6/4 = 42.15 kN* *Slip resistance/bolt* $= 1.1 \times 0.45 \times 177 = 87.62$ *kN* *Bearing resistance/bolt* $= 22 \times 9 \times 650 \times 10^{-3} = 128.7$ *kN* *Bolt value = 87.62 kN > bolt force of 42.15 kN* $\therefore$ *OK* *End distance* $> 42.15 \times 10^{3} / (1/3 \times 9 \times 650) = 21.62$ *mm* *Also end distance > 1.4(22+1.5) = 35 mm Use 50 mm* *Use 325×200×10 mm flange splice with bolts at 140 mm gauge,* *75 mm pitch*	*flange splice* *325×200×10*

Structural Steel Design Project **Calculation Sheet**	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Bolted Seating Angle Connection*		
	Worked Example – 4		
		Made by *SRSK*	Date *01-10-00*
		Checked by *VK*	Date

	Remarks
Design Example 4: *Design a Seating angle connection for an ISMB 400 beam to an ISHB 200 column so as to transfer a shear of 150 kN.*	*Remarks*

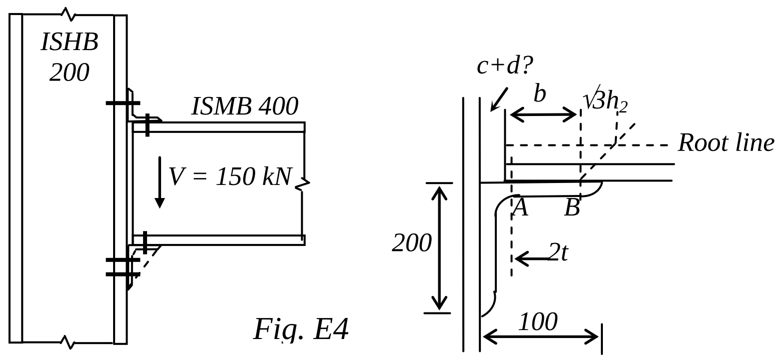

Fig. E4

1) Seating Angle

The support reaction acts as a UDL over length $(b+\sqrt{3}h_2)$ on the web

Length of bearing required at root line of beam $(b+\sqrt{3}\,h_2)$

$= V/(t_w p_{yw}) = 150\times10^3/(8.9\times250/1.15) = 77.53$ *mm*

Length of bearing on cleat $= b = 77.53-\sqrt{3}h_2 = 77.53-(\sqrt{3})32.8 = 20.7$ *mm*

end clearance of beam from the face of the column c= 5 mm

allow tolerance d = 5 mm

minimum length of angle leg required for seating = b+c+d = 30.7 mm

Try ISA $200\times100\times12$ *angle of length* $w = b_f = 140$ *mm*

Distance from end of bearing on cleat to root of angle (A to B)

$= b + c + d - (t+r)$ *of angle; assuming r = t for angle*

$= b + 5 + 5 - (2t) = 20.7 + 5 + 5 - 24 = 6.7$ *mm*

assuming the load to be uniformly distributed over the bearing length b

moment at the root of angle $=(150/20.7)\times6.7^2/2 = 162.6$ *N-m*

Moment capacity $= 1.2p_yZ = 1.2\times(250/1.15)\times(140\times12^2/6)\times10^{-3}$

$= 876.5$ *N-m OK*

Note :[The maximum moment occurs to the left of point A. To account for it the section modulus is taken as $1.2wT^2/6$ *instead of* $wT^2/4$*].*

Shear Capacity of outstanding leg of cleat

$= 0.6p_y\times0.9wt = 0.6\times(250/1.15)\times0.9\times140\times12\times10^{-3}$

$= 197.2$ *kN >150 kN OK*

Use ISA $200\times100\times12$

Structural Steel Design Project	*Job No:*	Sheet ***2 of 2***	Rev
	Job Title: *Bolted Seating Angle Connection*		
	Worked Example – 4		
		Made by *SRSK*	Date *15-04-00*
Calculation Sheet		Checked by *VK*	Date

2) Connection of seating angle to column flange

Bolts required to resist only shear

Try 4 bolts of 20mm dia and grade 4.6 at angle back marks

Total shear capacity $= 4 \times 160 \times 245 \times 10^{-3} = 156.8$ *kN > 150 kN OK*

Column flange critical for bearing of bolts

Total bearing capacity $= 4 \times 418 \times 20 \times 9.0 \times 10^{-3} = 301$ *kN > 150 kN OK* $\quad Np_{bs}dt_f$

3) Provide nominal clip angle of ISA 50 $\times$ *50* $\times$ *8 at the top*

<table>
<tr><td rowspan="5">Structural Steel Design Project

Calculation Sheet</td><td>Job No:</td><td>Sheet 1 of 2</td><td>Rev</td></tr>
<tr><td colspan="3">Job Title: Bolted Web Cleats Connection</td></tr>
<tr><td colspan="3">Worked Example – 5</td></tr>
<tr><td rowspan="2"></td><td>Made by SRSK</td><td>Date 01-10-00</td></tr>
<tr><td>Checked by VK</td><td>Date</td></tr>
</table>

Design Example 5: *Design a bolted web cleat beam-to-column connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column. The connection has to transfer a factored shear of 150 kN. Use bolts of diameter 20 mm and grade 4.6.*

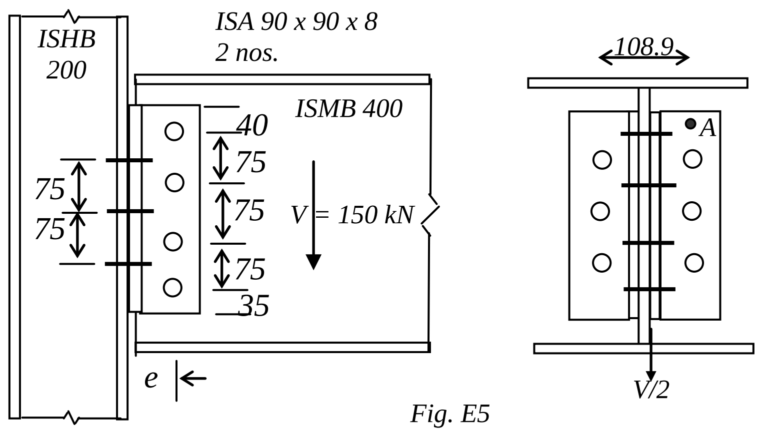

Fig. E5

1) The recommended gauge distance for column flange is 100 mm.
Therefore required angle back mark is 50 mm.
Use web cleats of ISA 90x90x8 giving gauge g = 50+50+8.9=108.9 mm

g for ISHB200 is 100 mm OK

2) Connection to web of beam- Bolt capacity
shear capacity of bolt in double shear $= 2\times160\times245\times10^{-3} = 78.4$ *kN*
bearing capacity of bolt on the beam web $= 418\times20\times9.0\times10^{-3} = 75.24$ *kN*
bolt value = 75.24 kN

Try 4 bolts as shown in the Figure with vertical pitch of 75 mm

Assuming the shear to be acting on the face of the column, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= 150\times50\times112.5/2(37.5^2+112.5^2) = 30.0$ *kN*

$P_x\, e\, r_i / \Sigma r_i^2$

vertical shear force per bolt = 150/4 = 37.5 kN

resultant shear = $\sqrt{(30.0^2+37.5^2)} = 48.0$ *kN < bolt value Safe !*

<table>
<tr><td rowspan="5">Structural Steel Design Project

Calculation Sheet</td><td>Job No:</td><td>Sheet 2 of 2</td><td>Rev</td></tr>
<tr><td colspan="3">Job Title: Bolted Web Cleats Connection</td></tr>
<tr><td colspan="3">Worked Example – 5</td></tr>
<tr><td rowspan="2"></td><td>Made by SRSK</td><td>Date 01-10-00</td></tr>
<tr><td>Checked by VK</td><td>Date</td></tr>
</table>

3) Connection to column flange: Bolt capacity	
shear capacity of bolt in single shear = $160 \times 245 \times 10^{-3} = 39.2$ *kN*	
bearing capacity of bolt on column flange = $418 \times 20 \times 9.0 \times 10^{-3} = 75.24$ *kN*	
bolt value = 39.2 kN	
Try 6 bolts as shown in the Fig.E5 with vertical pitch of 75 mm	
4) Check bolt force	
Similar to the previous case, the shear transfer between the beam web and the angle cleats can be assumed to take place on the face of the beam web. However, unlike the previous case, no relative rotation is possible between the angle and the beam web.	
Assuming centre of pressure 25 mm below top of cleat (point A), horizontal shear force on bolt due to moment due to eccentricity e $= (150 \times 50/2) \times 200/(50^2+125^2+200^2) = 12.9$ *kN*	$(V/2)e_x r_i/\Sigma r_i^2$
vertical shear force per bolt = 150/6 = 25.0 kN	
resultant shear = $\sqrt{(12.9^2+25.0^2)} = 28.13$ *kN < bolt value OK*	
Use 2 Nos ISA 90x90x8 of length 375 mm as angle cleats	*ISA 90x90x8* *Length 375mm*

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Bolted End Plate Connection*		
	Worked Example - 6		
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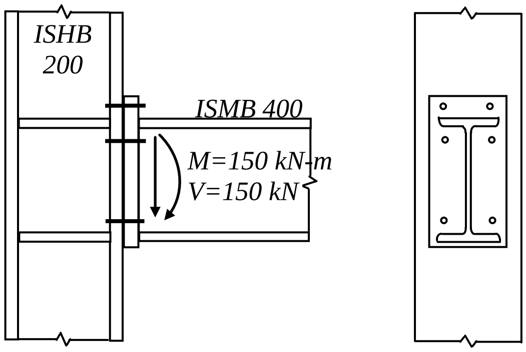
Calculation Sheet		Checked by *VK*	Date

Design Example 6: *Design a bolted end plate connection between an ISMB 400 beam and an ISHB 200 @ 40 kg/m column so as to transfer a hogging factored bending moment of 150 kN-m and a vartical factored shear of 150 kN. Use HSFG bolts of diameter 22 mm.*



1) bolt forces

taking moment about the centre of the bottom flange and neglecting the contribution of bottom bolts and denoting the force in the top bolts by F

$4F \times 384 = 150 \times 10^3$

$F = 97.6\ kN$

tension capacity of M22 bolt $= 0.9P_o = 159.3\ kN$

allowable prying force $Q = 159.3-97.6 = 61.7\ kN$

2) design for prying action

try 30 mm thick end plate of width $b_e = 180\ mm$

distance from the centre line of bolt to prying force n is the minimum of edge distance or $1.1T\sqrt{\beta Po/Py} = 1.1 \times 30\ \sqrt{(2 \times 512/250)} = 55.66\ mm$

$n = 40\ mm$

assuming 10 mm fillet weld,

distance from center line of bolt to toe of fillet weld $b = 60-10 = 50\ mm$;

moment at the toe of the weld $= Fb-Qn = 97.6 \times 50-61.7 \times 40 = 2412\ N\text{-}m$

effective width of end plate per bolt $w = be/2 = 180/2 = 90\ mm$

moment capacity $= (250/1.15)(90 \times 30^2/4) = 4402\ N\text{-}m > 2412\ N\text{-}m$ *Safe !*

$(py/1.15) \times (wT^2/4)$

Structural Steel Design Project	Job No:	Sheet *2 of 2*	Rev
	Job Title: *Bolted End Plate Connection*		
	Worked Example - 6		
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$min\ Q = \frac{50}{2\times 40}\left[97.6 - \frac{2\times 1.5\times 0.587\times 90\times 30^4}{27\times 40\times 50^2}\right]$	$Q = \frac{b}{2n}\left[F - \frac{\beta\gamma P_o w T^4}{27nb^2}\right]$
Q = 31.8 kN < 61.7 kN OK	*β=2 (non-preloaded)* *γ =1.5 (for factored load)*
3) Check for combined shear and tension	
Shear capacity of M20 HSFG Ps l= 87.6 kN	
Shear per bolt Fs = 150/6 = 25 kN	
= (25.0/87.6) + (97.6+31.8)/159.3 = 0.936 < 1.0 Safe !	$F_s/P_{sl} + 0.8f_t/P_t$

Structural Steel Design Project	Job No:	Sheet ***1 of 2***	Rev
	Job Title: *Beam to Beam Connection*		
	Worked Example - 7		
		Made by *SRSK*	Date *1-10-00*
Calculation Sheet		Checked by *VK*	Date

Design Example 7*: Design a double web cleat connection for an ISMB 400 coped beam to an ISMB 600 main beam so as to transfer a factored load of 300 kN using HSFG bolts of 20mm diameter and grade 8.8.*

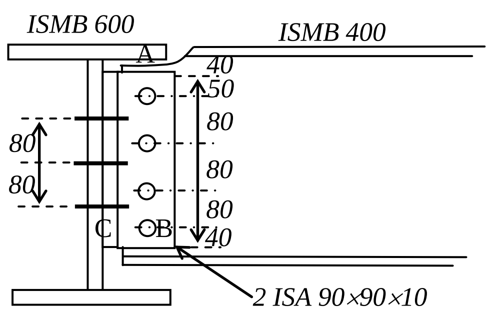

Fig. E7

Solution:

1) Connection to web of ISMB 400

For M20 Gr.8.8 HSFG bolts in double shear
Slip resistance per bolt = $2\times1.1\times0.45\times144 = 142.6$ kN
Bearing capacity of web per bolt = $20\times8.9\times650\times10^{-3} = 115.7$ kN
Bolt value = 115.7 kN

Try 4 bolts as shown in the Figure with vertical pitch of 80 mm

Assuming the shear to be acting on the face of the ISMB 600 web, its eccentricity with the centre of the bolt group will produce horizontal shear forces in the bolts in addition to the vertical shear.

horizontal shear force on top bolt due to moment due to eccentricity e
$= (300/2)\times50\times112.5/(37.5^2+112.5^2) = 60.0$ kN

vertical shear force per bolt = 300/4 = 75.0 kN

resultant shear = $\sqrt{(60^2+75^2)} = 96.0$ kN < bolt value Safe !

<table>
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<tr><td colspan="3">Job Title: Beam to Beam Connection</td></tr>
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<tr><td></td><td>Checked by VK</td><td>Date</td></tr>
</table>

2) *Connection to web of ISMB 600*

Try 6 bolts as shown in the Figure with vertical pitch of 80 mm

For M20 Gr.8.8 HSFG bolts in single shear
Slip resistance per bolt = $1.1 \times 0.45 \times 144 = 71.28$ kN
Bearing capacity of web per bolt = $20 \times 12 \times 650 \times 10^{-3} = 156$ kN
Bolt value = 71.28 kN

Assuming center of pressure 27.5 mm below the top of the angle

horizontal shear force on bottom bolt due to moment due to eccentricity e
$= (300/2)\times 50\times 200/(50^2+125^2+200^2) = 25.82$ *kN*

vertical shear force per bolt = 300/6 = 50.0 kN

resultant shear = $\sqrt{(25.82^2+50^2)} = 56.27$ *kN < bolt value Safe !*

3) *Check web of ISMB 400 for block shear*

Block shear capacity = shear capacity of AB + 0.5×tensile capacity of BC
$= 0.6\times 250\times 0.9\times 1.1(3\times 80+50-3.5\times 22)\times 8.9\times 10^{-3}$
$+ 0.5\times 250\times 1.1(45-0.5\times 22)\times 8.9\times 10^{-3} = 323.12 > 300$ *kN Safe!*

<table>
<tr><td rowspan="4">Structural Steel
Design Project

CALCULATION SHEET</td><td>Job No.</td><td>Sheet 1 of 2</td><td>Rev.</td></tr>
<tr><td colspan="3">Job title: PLASTIC ANALYSIS</td></tr>
<tr><td colspan="3">Worked Example. 1</td></tr>
<tr><td rowspan="2"></td><td>Made by RSP</td><td>Date April 2000</td></tr>
<tr><td></td><td>Checked by RN</td><td>Date April 2000</td></tr>
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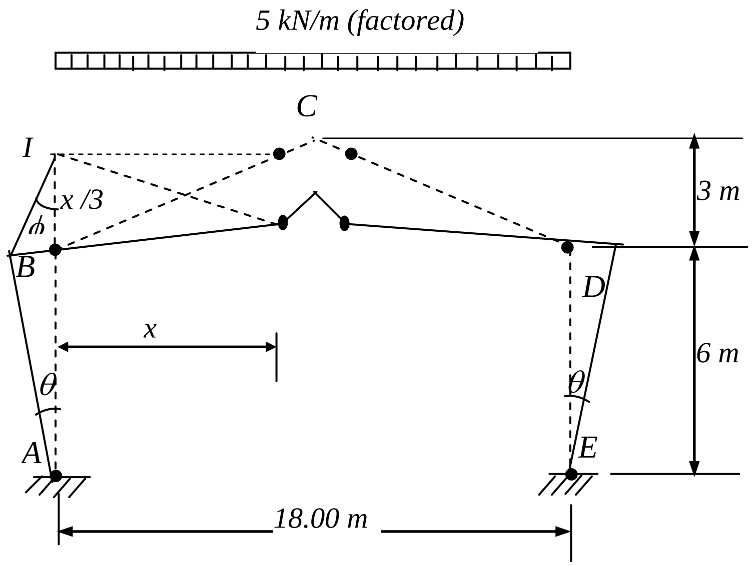

Fig. 22 Portal with fixed base and symmetrical vertical loading

Determine the M_p required for a symmetric single bay pitched portal frame with a factored UDL of 5 kN /m by instantaneous centre method.

In the case of gable frames, computation of the geometrical relationship of the displacement in the direction of the load as the structure moves through the mechanism may become somewhat tedious. In such cases the method of instantaneous centres may be used. This method is discussed below along with its use for solving practical problems.

In the following problem, when the structure moves under loading, the point B will move in a direction perpendicular to line AB . Then its centre of rotation should be along line AB extended. The point C will move vertically downwards and its centre of rotation should lie in a horizontal line. The point I satisfies both the conditions. Thus I is the centre of rotation of the member BC . The rotation at both the column bases is taken as θ. Assume that the hinges will form in the rafter at a distance of x from B and very close to roof apex.

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	Job title: ***PLASTIC ANALYSIS***		
	Worked Example. ***1***		
		Made by ***RSP***	Date ***April 2000***
CALCULATION SHEET		Checked by ***RN***	Date ***April 2000***

Data (as shown in Fig. 20).

Frame centres	*5.0 m*
Span of portal	*18.0 m*
Eves height	*6.0 m*
Eves to ridge height	*3.0 m*
Purlin spacing	*1.5 m*

Solution:

$$(x/3)\,\phi = 6\theta$$

$$\therefore \phi = 18\,\theta/x$$

$$M_p\,(\theta + 18\,\theta/x + \theta + 18\,\theta/x) = 18\,\theta/x\,[\,5x^2/2 + (9 - x)\,5x]$$

$$M_p = \frac{9\left(-2.5x^2 + 45x\right)}{\left(18 + x\right)}$$

For maximum value of M_p **,** $\dfrac{dM_p}{dx} = 0$

$$(18 + x)(-5x + 45) - (-2.5x^2 + 45x) = 0$$

$$-2.5x^2 - 90x - 810 = 0$$

$$x = 7.5\ m$$

Substituting in eqn. for M_p *,* $M_p = \underline{69.5\ kNm}$

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	Job title: ***PLASTIC ANALYSIS***		
	Worked Example. ***2***		
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Find M_p for the portal frame with electrically operated travelling crane as shown in Fig. 21 by 'Reactant moment diagram' method. The roof pitch is 30°. Neglect the effect of wind acting vertically on the roof.

Horizontal wind pressure is = 1 kN/m^2
γ_f = *1.2 for the combined effects of wind, crane, dead load and live load.*

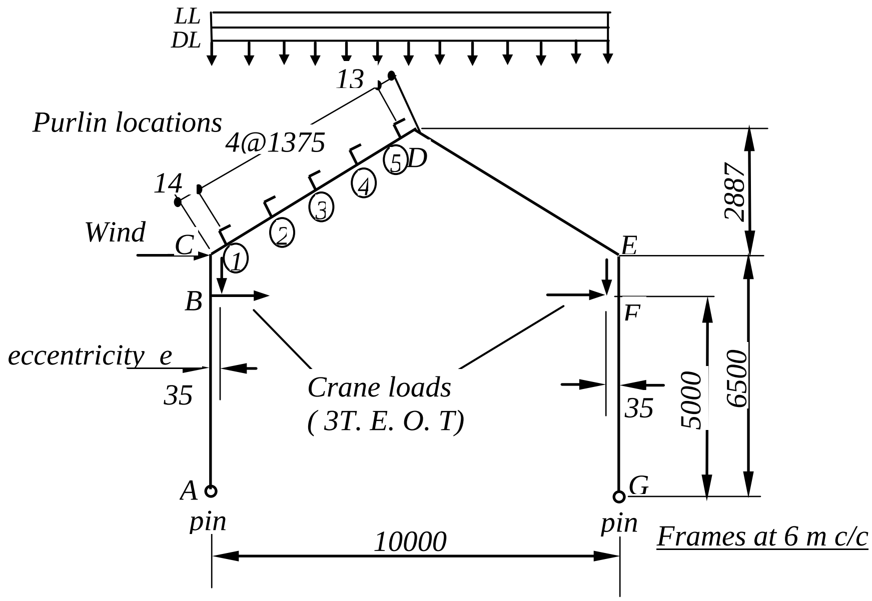

Fig. 23 Summary of typical loads and dimensions for Example 2

PRELIMINARY CALCULATIONS

(1) Forces due to dead load and live load on roof

Superimposed load = *0.6 kN/m^2*
Dead load = *0.5 kN/m^2*
Total load = $(0.6 + 0.5) \times 1.2 \times 6$ = *7.92 kN/m*

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(2) Crane loading

3 ton capacity crane, 9.3 m span.

Horizontal crane loading

This may be shared between each side of the portal, based on the assumption that the crane wheels are flanged, and in effect share the load between the two rails. Check that the crane wheels are flanged when the vendor is selected, or place entire horizontal crane load at point B for a more onerous case.

Vertical crane loading
Maximum wheel load = 26.5 kN (2 wheels)
Minimum wheel load = 7.25 kN (2 wheels)
Maximum reaction at column due to loaded crane $= 2 \times 26.5 = 53$ *kN*
Minimum reaction at column due to loaded crane $= 2 \times 7.25 = 14.5$ *kN*

Moment due to vertical crane loading (unfactored)
Moment at B $= 53 \times 0.35 \times 1.2 = 22.3$ *kNm*
Moment at F $= 14.5 \times 0.35 \times 1.2 = 6.1$ *kNm*
Load on the crane bracket is 350 mm eccentric from column centre line.

Transverse crane loading
Transverse load due to crab and load $= 0.1\,(6.0 + 3.0) = 3.6$ *kN*
Shared between points B and F, i.e. 1.8 kN each.

Moment due to transverse crane loading
Moment at B $= 1.8 \times 5 \times 1.2 = 10.8$ *kNm*
$5 \left(113 + \frac{1191}{2}\right) \frac{7.92}{1000}$
Split the frame at the apex, then it can be treated as two cantilevers.

Total roof load = 7.92 kN/m

Load at purlin point = = 5.61 kN

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Design Project	Job title: ***PLASTIC ANALYSIS***		
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Load at purlin point 4 $= \left(\frac{1191}{2} + \frac{1191}{2}\right)\frac{7.92}{1000} = 9.43\ kN$

Moment at purlin point 3 $= \left[9.43 \times 1191 + 5.61 \times (1191 \times 2)\right]\frac{1}{1000}$

$= 24.59\ kNm$

<u>Moment at A</u>

Moment due to roof load $= 7.92 \times 5 \times 2.5 = 99\ kNm$

Moment due to wind load on ABC $= (1 \times 6 \times 6.5)\ \frac{6.5}{2} \times 1.2 = 152.1\ kNm$

Moment due to vertical crane loading $= 53 \times 0.35 \times 1.2 = 22.3\ kNm$

Moment due to transverse crane load $= 1.8 \times 1.2 \times 5 = 10.8\ kNm$

Total $= 284.2\ kNm$

<u>Moment at G</u>

Moment due to roof load $= 99\ kNm$

Moment due to vertical crane loading $= 6.1\ kNm$

Moment due to transverse crane load $= 10.8\ kNm$

Total $= 115.9\ kNm$

Summary of the reactant moment diagram method for the portal frame is shown in Fig. 22(a) and 22(b). For solution by calculation, let purlins be numbered 1 to 'n' from roof to apex. Put moment for each purlin point into the reactant moment diagram equations and solve for successive purlin points. The largest value of M_p *found by this method is the design case.*

Design Project	Job title: ***PLASTIC ANALYSIS***		
	Worked Example. ***2***		
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For this design the equations for M_p are:

At A : $284.2 - m - 9.387\,R - 5\,S = 0$

At point 3 : $24.59 - m - 1.42\,R - 2.495\,S = -M_p$

At E : $99 - m - 2.887\,R + 5\,S = +M_p$

At G : $115.9 - m - 9.387\,R + 5\,S = 0$

Using the method and equations illustrated in Fig. 22(b) these equations can be solved simultaneously (or by matrix) to give $R = 16.2$ kN, $S = 16.83$ kN, $m = 48$ kN, and
$M_p = 88.4$ kNm.

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	Worked Example. ***2***			
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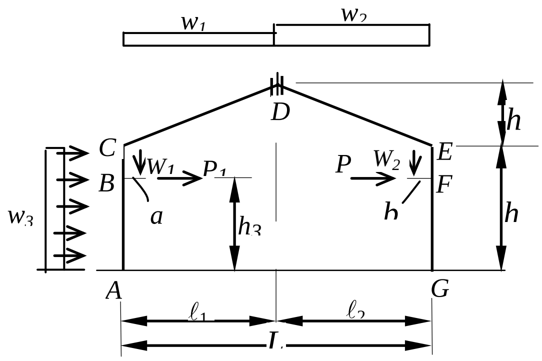
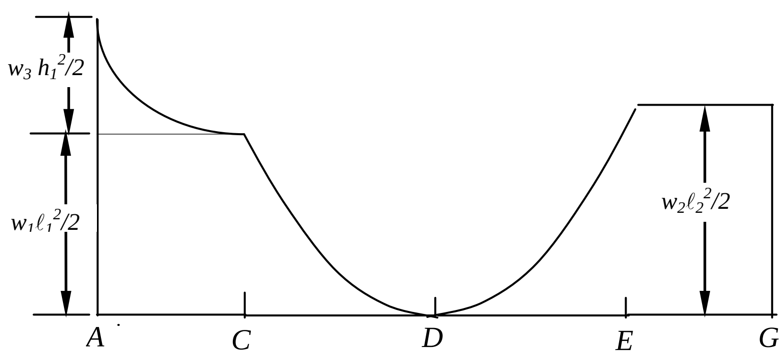
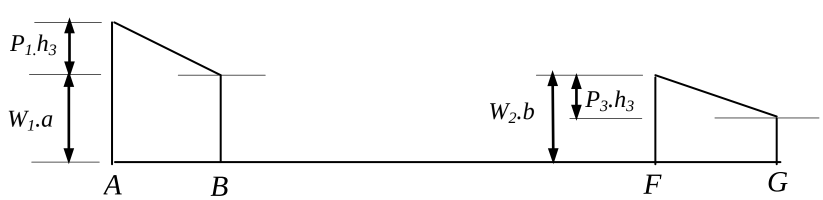


Loading and geometry



Free moment diagram for UDLs



Free moment diagram due to crane loads

Fig. 24(a) Loadings and free moment diagrams for the portal frame

Structural Steel Design Project	Job No.	Sheet 6 of 6	Rev.
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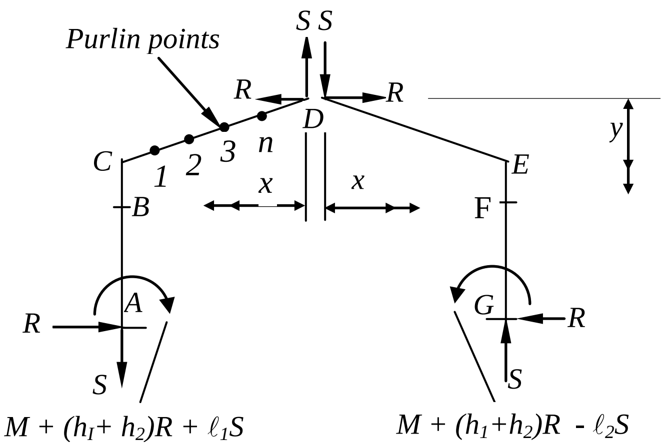
	Worked Example. ***2***		
		Made by ***RSP***	Date ***April 2000***
CALCULATION SHEET		Checked by ***RN***	Date ***April 2000***

For the purpose of analysis, frame is notionally split at apex, and treated as two cantilevers



Redundant reactants

<u>*Equations for fixed bases*</u>

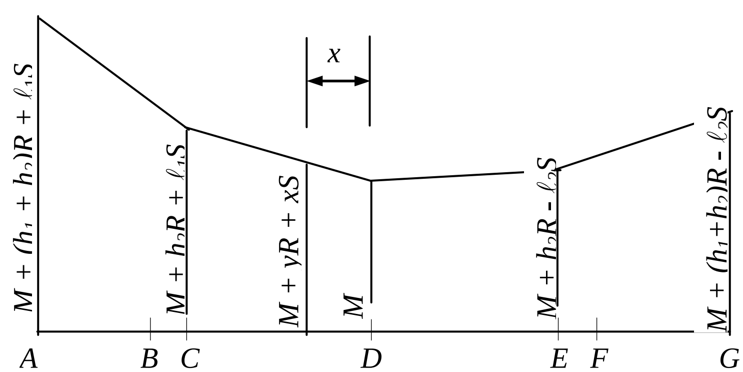
at C ; Free M – M – $h_2R - \ell_1S = +M_p$

at n ; Free M – M – $yR - xS = -M_p$

at E ; Free M – M – $h_2R + \ell_2S = +M_p$

at G ; Free M – M – $(h_1+h_2)R + \ell_2S = -M_p$

Equations for pinned bases are as for fixed bases but with moment at the bases set to zero. Put the factored values of free moment into the equations and solve simultaneously or by matrix. Solve for successive purlin points, thus finding the largest value of M_p



Redundant reactant moment diagram

Fig. 24(b) Summary of the reactant moment diagram method for a portal frame

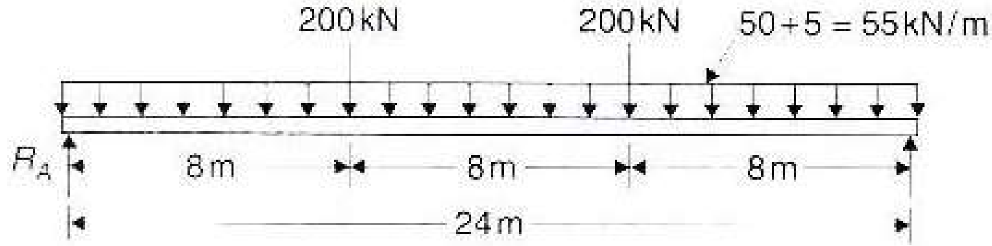
Welded Plate Girder Design Example

Design a welded plate girder to carry a superimposed load of 50 kN/m and two concentrated loads of 200 kN each at one-third points of the span. The effective span of the plate girder is 24 m. Assume that the girder is laterally supported throughout its length. The yield strength of the steel (of both the flanges and the web), $f_y = 250$ MPa.

Maximum shear force and the bending moment

The self weight of the plate girder may be taken as W/300 where W is the total load on the girder.

$$\text{Self weight} = \frac{50 \times 24 + 2 \times 200}{300} \approx 5.0\,\text{kN/m}$$



The reaction at the support A, $R_A = 55 \times 24/2 + 200 = 860$ kN

∴ The maximum factored shear force, $V = 1.5 \times 860 = 1{,}290$ kN

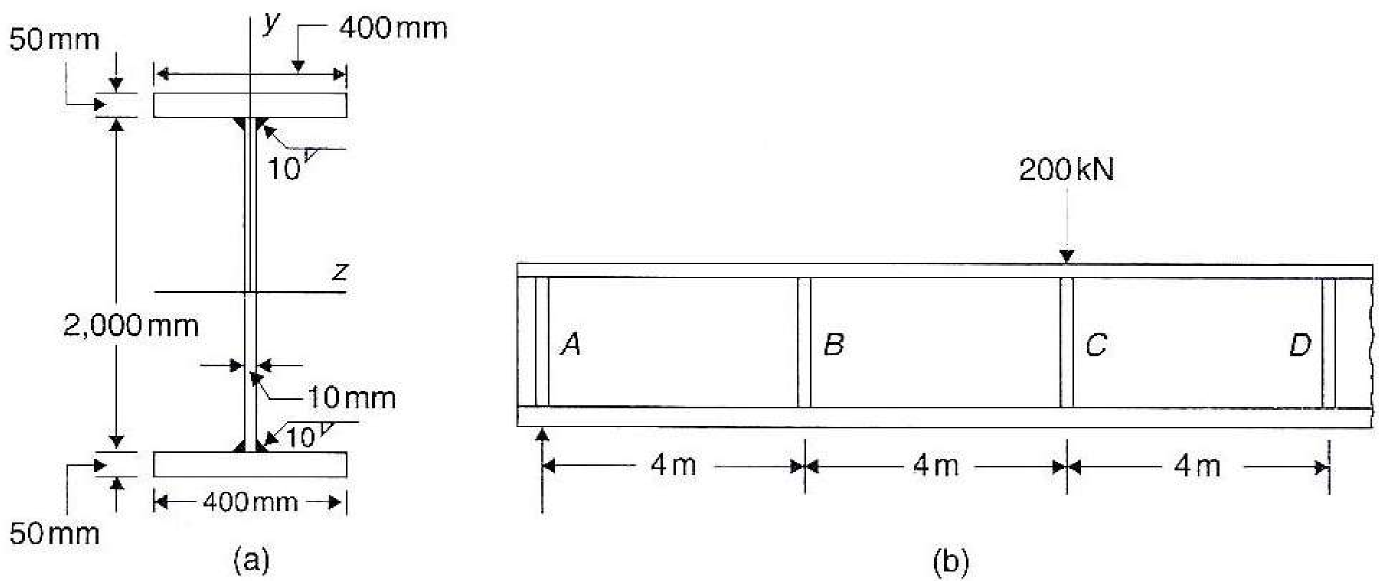
The maximum factored bending moment (at mid-span)

$$M = 1.5\left(860 \times 12 - \frac{55 \times 12^2}{2} - 200 \times 4\right) = 8{,}340 \text{ kNm}$$

Dimensions of the flanges and the web

The span to depth ratio may vary from 8 to 12. Hence, a web of depth, $d = 2{,}000$ mm may be considered.

The area of the flange needed, $A_f = \dfrac{M}{df_y/\gamma_{m0}} = \dfrac{8{,}340 \times 10^6}{2{,}000 \times 250/1.1} = 18{,}348 \text{ mm}^2$



Two flanges of each cross-section 400 mm × 50 mm may be provided as shown in Figure 7.8a. For serviceability limit state, when only intermediate transverse stiffeners with a spacing greater than or equal to the depth of the web are provided,

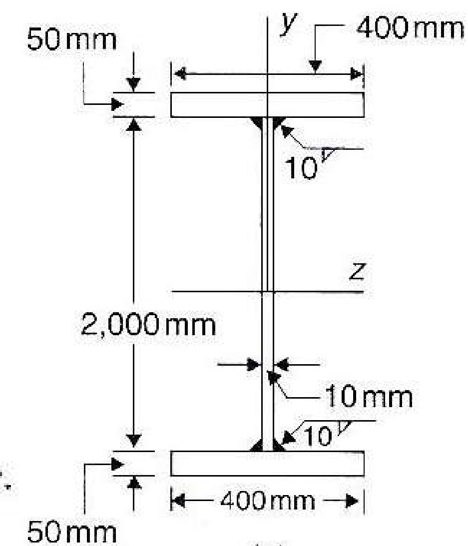
$$t_w \geq \frac{d}{200\varepsilon}$$

$$\geq \frac{2{,}000}{200 \times 1.0}, \quad \text{i.e. } 10 \text{ mm}$$

Hence, a thickness of 10 mm may be provided to the web. This also ensures the condition for the flange does not buckle into the web is satisfied, i.e. $d/t_w < 345\,\varepsilon$, where $\varepsilon = 1.0$.

Bearing/Load carrying stiffeners (end post) are provided at the supports. Since, two concentrated loads are acting at 8,000 mm from each support, intermediate transverse stiffeners are provided at a spacing of 4,000 mm (i.e. 2d) as shown in Figure 7.8b which ensures an intermediate transverse stiffener under a concentrated load. This intermediate stiffener is also designed to function as a bearing/load carrying stiffener.

Section classification



$$b = (400 - 10)/2 = 195$$

$$b/t_f = 195/50 = 3.9 < 8.4\,\varepsilon \qquad \text{where } \varepsilon = 1.0$$

flanges are plastic.

$d/t_w = 2000/10 = 200 > 126\,\varepsilon$

$\therefore$ From Table 1.7, the web is slender and hence shear buckling takes place.

The design moment carrying capacity of the flanges

$$M_d = \beta_b Z_p f_y / \gamma_{m0} = 1.0 \times (2 \times 400 \times 50 \times 1{,}025) \times 250/1.1$$
$$= 9{,}318\,\text{kNm} > 8{,}340\,\text{kNm}$$

End panel design

The end panels are designed using the tension field method as described in Sec. 7.3. Spacing intermediate transverse stiffeners, $c = 4{,}000$ mm

$$\therefore \quad c/d = 4{,}000/2{,}000 = 2.0$$

$$K_v = 5.35 + \frac{4}{2.0^2} = 5.35$$

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1 - 0.3^2) \times 200^2} = 24.2\,\text{MPa}$$

$$\lambda_w = \left(\frac{250}{\sqrt{3} \times 24.2}\right)^{0.5} = 2.44 > 1.2$$

$$\therefore \quad \tau_b = \frac{250}{\sqrt{3} \times 2.44^2} = 24.2\,\text{MPa}$$

$$\phi = \tan^{-1}\frac{2{,}000}{4{,}000} = 26.6^0$$

$$\psi = 1.5 \times 24.2 \times \sin 2 \times 26.6^0 = 29.0\,\text{MPa}$$

$$f_v = [250^2 - 3 \times 24.2^2 + 29.0^2]^{0.5} - 29.0 = 219\,\text{MPa}$$

The bending moment in the plate girder at 4 m from the support

$$= 1.5\left(860 \times 4 - \frac{55 \times 4^2}{2}\right) = 4,500\,\text{kNm}$$

The axial force in the flange due to the bending moment, $N_f = \dfrac{bending - moment}{lever - arm}$

$$= \frac{4,500 \times 10^6}{2,050} = 2,195\,\text{kN}$$

$$M_{fr} = 0.25 \times 400 \times 50^2 \times 250 \times \left[1 - \left(\frac{2,195 \times 10^3}{400 \times 50 \times 250/1.1}\right)^2\right] = 48\,\text{kNm}$$

$$s_c = s_t = \frac{2}{\sin 45^0}\left[\frac{48.3 \times 10^6}{250 \times 10}\right]^{0.5} = 391.5\,\text{mm} < c$$

$$w_{tf} = 2{,}000 \cos 26.6° + (4{,}000 - 391.5 - 391.5) \sin 26.6° = 3{,}229\,\text{mm}$$

$$A_v = 2{,}000 \times 10 = 20{,}000\,\text{mm}^2$$

$$V_p = \frac{20{,}000 \times 250}{\sqrt{3}} = 2{,}887\,\text{kN}$$

$$V_{tf} = 20{,}000 \times 24.2 + 0.9 \times 3{,}229 \times 10 \times 219 \times \sin 26.6^0 = 3{,}333\,\text{kN} > V_p$$

$$\therefore \quad V_{tf} = V_p = 2{,}887\,\text{kN}$$

$$V_d = \frac{V_{tf}}{\gamma_{m0}} = \frac{2{,}887}{1.1} = 2{,}624\,\text{kN} > (V = 1{,}290\,\text{kN}) \qquad \text{OK}$$

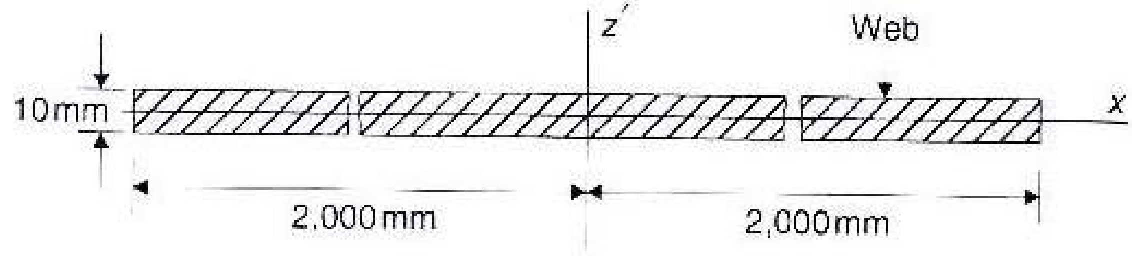


FIGURE 7.9 Longitudinal section of web

Check for the shear capacity of the end panel

Shear capacity of longitudinal section (Figure 7.9) of the end panel $= A_v f_y/\sqrt{3}$

$$= 4{,}000 \times 10 \times 250/\sqrt{3}$$

$$= 5{,}774\,\text{kN}$$

$$V_{cr} = 20{,}000 \times 24.2 = 484\,\text{kN}$$

$$H_q = 1.25 \times 2{,}887 \times 10^3 \times \left(1 - \frac{4{,}84{,}000}{2{,}887 \times 10^3}\right)^{0.5} = 3{,}292\,\text{kN}$$

Since $V < V_{tf}$, H_q may be reduced by the ratio

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}} = \frac{1{,}290 - 484}{2{,}887 - 484} = 0.34$$

$$\therefore \quad H_q = 0.34 \times 3{,}292 = 1{,}119\,\text{kN}$$

$$R_{tf} = 1{,}119/2 = 559.5\,\text{kN} < 5{,}774\,\text{kN} \qquad \text{OK}$$

Check for the moment capacity of the end panel

$$M_{tf} = \frac{H_q d}{10} = \frac{1{,}119 \times 2{,}000}{10} = 224\,\text{kNm}$$

$$x = c/2 = 4{,}000/2 = 2{,}000\,\text{mm}$$

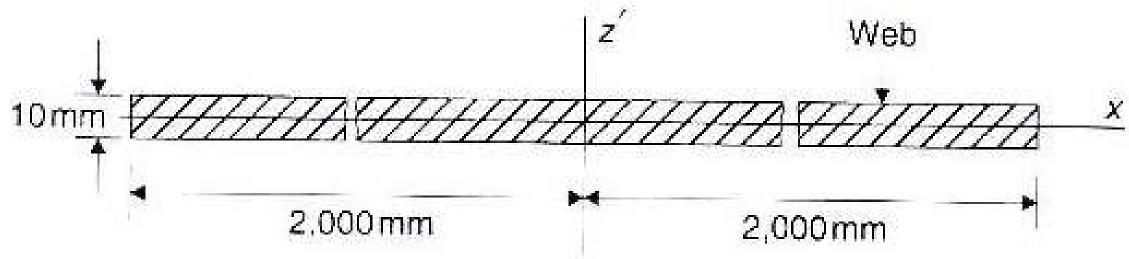


FIGURE 7.9 Longitudinal section of web

$$I_{z'} = \frac{tc^3}{12} = \frac{10 \times 4{,}000^3}{12} = 5.3 \times 10^{10}\,\text{mm}^4$$

Moment capacity of longitudinal section (Figure 7.9) of the end panel is given by

$$M_q = \frac{I_{z'}}{x}\frac{f_y}{\gamma_{m0}} = \frac{5.3 \times 10^{10} \times 250}{2{,}000 \times 1.1} = 6{,}023\,\text{kNm} > 224\,\text{kNm} \qquad \text{OK}$$

Connection between the web and the flange

The moment of inertia of the girder cross-section

$$I_Z = \frac{10 \times 2{,}000^3}{12} + 2 \times 400 \times 50 \times 1{,}025^2 = 4.87 \times 10^{10} \text{ mm}^4$$

The horizontal shear force acting on the welds per unit length

$$q = \frac{VA\overline{y}}{I} = \frac{1{,}290 \times 10^3 \times (400 \times 50) \times 1{,}025}{4.87 \times 10^{10}} = 543 \text{ N/mm}$$

The force acting on the welds per unit length due to the tension field action

$$= f_y\, t_w = 250 \times 10 = 2{,}500 \text{ N/mm} > 543 \text{ N/mm}$$

$$\therefore\ 2 \times 0.7 \times s \times 189.4 = 2{,}500 \qquad \text{(refer Example 2.1)}$$

$$\text{or} \quad s = 9.4 \text{ mm}$$

10 mm size fillet welds may be provided on either side of the web.

Connection between the web and the flange

The moment of inertia of the girder cross-section

$$I_Z = \frac{10 \times 2{,}000^3}{12} + 2 \times 400 \times 50 \times 1{,}025^2 = 4.87 \times 10^{10}\ \text{mm}^4$$

The horizontal shear force acting on the welds per unit length

$$q = \frac{VA\overline{y}}{I} = \frac{1{,}290 \times 10^3 \times (400 \times 50) \times 1{,}025}{4.87 \times 10^{10}} = 543\ \text{N/mm}$$

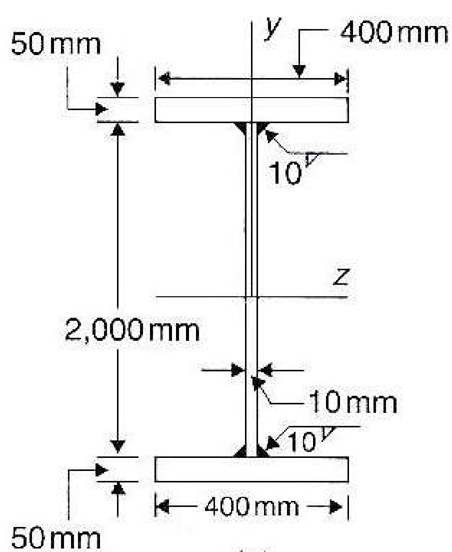
The force acting on the welds per unit length due to the tension field action

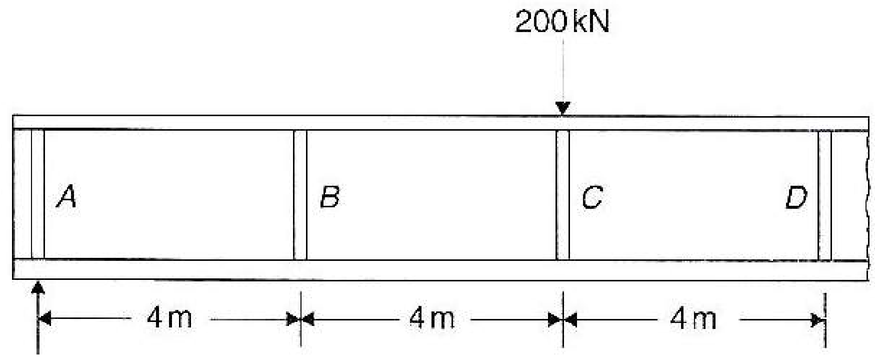
$$= f_y\, t_w = 250 \times 10 = 2{,}500\ \text{N/mm} > 543\ \text{N/mm}$$

$$\therefore\ 2 \times 0.7 \times s \times 189.4 = 2{,}500 \qquad \text{(refer Example 2.1)}$$

or $\quad s = 9.4\ \text{mm}$

10 mm size fillet welds may be provided on either side of the web.





Design of bearing/load carrying stiffener at A (end post)

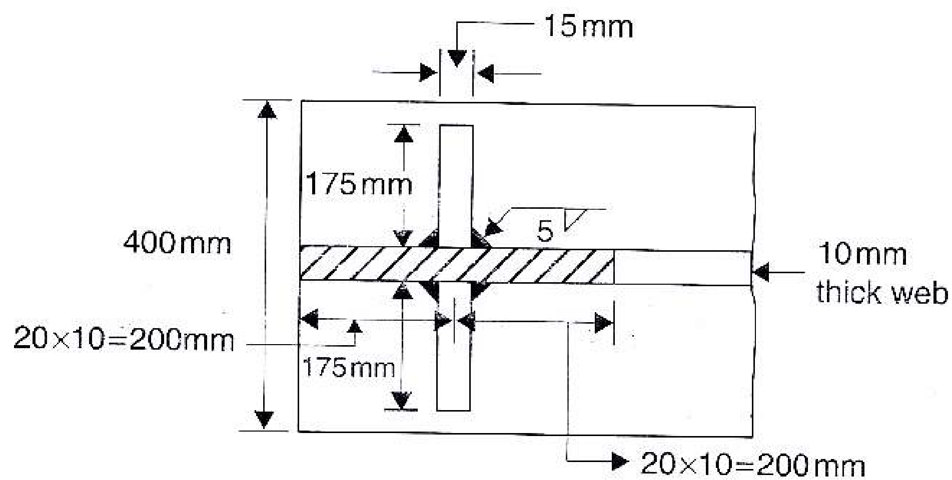
The force on the end post due to the moment $2M_q/3 = \dfrac{bending-moment}{lever-arm}$

$$= \frac{2 \times 224 \times 10^6}{3 \times 4{,}000} = 37\,\text{kN}$$

The design force on the end post, $F_x = 1{,}290 + 37 = 1{,}327$ kN
The required bearing area of the stiffener is obtained from

$$\frac{A_q f_y}{0.8\gamma_{m0}} \geq F_x$$

or $\quad A_q = \dfrac{1{,}327 \times 10^3 \times 0.8 \times 1.1}{250} = 4{,}671\,\text{mm}^2$



Two flats of each cross-section 175 mm × 15 mm may be tried (Figure 7.10).
The effective width of the stiffener is equal to the actual width minus the size of the welds provided between the web and the flange.

The bearing area provided = $2 \times (175 - 10) \times 15 = 4{,}950\ \text{mm}^2 > 4{,}671\ \text{mm}^2$ OK

The outstand of the stiffener = 175mm < ($14\,t_q\varepsilon = 14 \times 15 \times 1.0 = 210$ mm)

$\therefore$ The full outstand is effective.

$b_1 = 0$

$n_2 = 2.5 \times 40 \times 2 = 200\,\text{mm}$

Local bearing capacity of the web, $F_w = (b_1 + n_2)\, t_w f_y / \gamma_{m0}$

$= 200 \times 10 \times 250/1.1 = 455\,\text{kN}$

Bearing force on the stiffener $= 1327 - 455 = 872\,\text{kN}$

Bearing capacity of the stiffener $= (175 - 10) \times 15 \times 2 \times 250/1.1$

$= 1{,}125\,\text{kN} > 872\,\text{kN}$ OK

Moment of inertia of the stiffener, $I = 15 \times (2 \times 175 + 10)^3 / 12 = 5{,}832 \times 10^4\,\text{mm}^4$

Area of cross-section of the stiffener, $A = 2 \times 175 \times 15 + 2 \times 200 \times 10 = 9{,}250\,\text{mm}^2$

Radius of the gyration of the stiffener, $r = \sqrt{\dfrac{5{,}832 \times 10^4}{9{,}250}} = 79.4\,\text{mm}$

$$\text{KL} = 0.7 \times 2{,}000 = 1{,}400\,\text{mm}$$

$$\text{KL/r} = 1{,}400 / 79.4 = 17.6$$

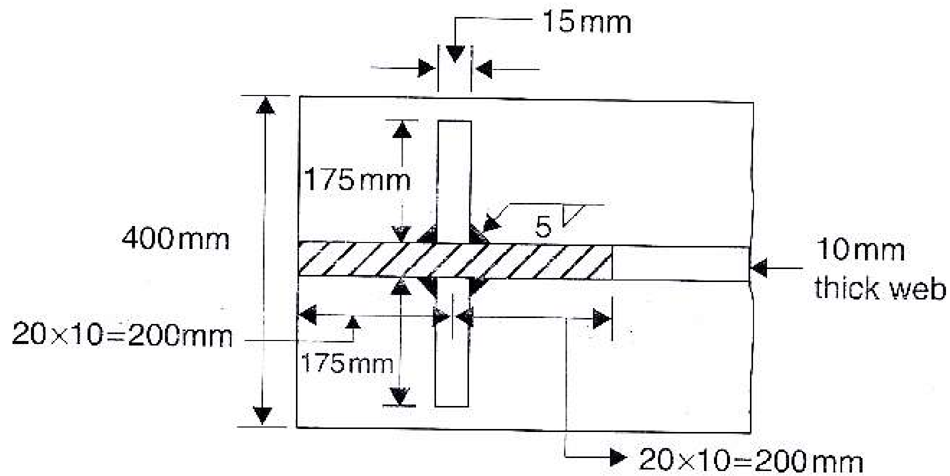
From Table 4.3, for the buckling class c, $f_{cd} = 224.7\,\text{MPa}$

Design compressive strength of the stiffener (F_{xd}) $= 9{,}250 \times 224.7$

$= 2{,}078\,\text{kN} > 1{,}327\,\text{kN}$ OK

Connection between the stiffener and the web

Tensile strength of the stiffener $= 2 \times 175 \times 15 \times 250 / 1.1 = 1{,}193\,\text{kN} < 1{,}327\,\text{kN}$



By connecting each of the flats by two fillet welds shown in Figure 7.10(a), size of the welds is given by

$$4\,(2{,}000 - 2 \times 10) \times 0.7s \times 189.4 = 1{,}193 \times 10^3$$
$$s = 4.5\,\text{mm}$$

Hence, bearing/load carrying stiffener may be connected to the web by 5 mm size fillet welds.

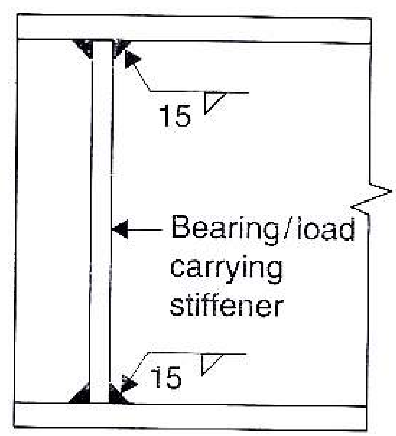
Connection between the stiffener and the flange

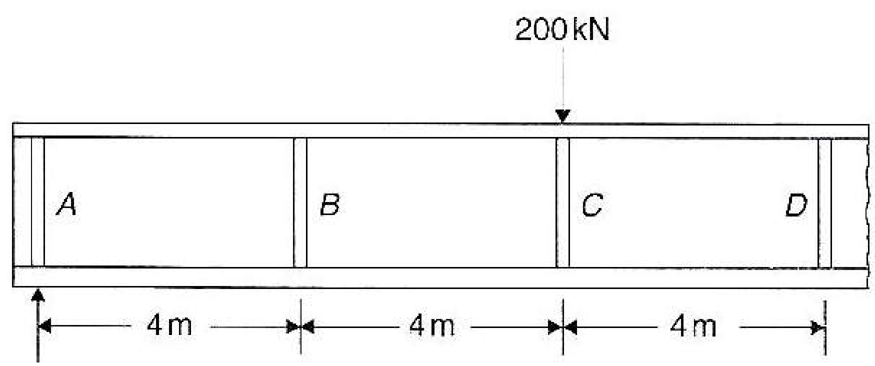
Length of the weld = 4 (175 − 10) = 660 mm

$$660 \times 0.7s \times 189.4 = 1{,}193 \times 10^3$$

or $$s = 13.6\ \text{mm}$$

The stiffener may be connected to the flanges using a 15 mm size fillet weld as shown in Figure 7.10(b).





Design of the intermediate stiffener at B

$$c/d = 4{,}000 / 2{,}000 = 2 > \sqrt{2}$$

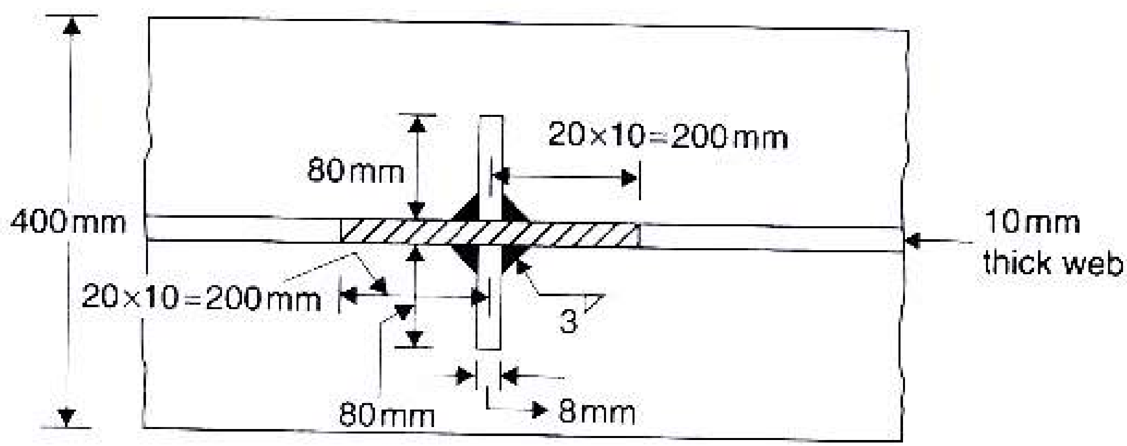
Required moment of inertia of the stiffener, $I_s = 0.75 d t_w^3$

$$= 0.75 \times 2{,}000 \times 10^3 = 1.5 \times 10^6 \text{ mm}^4$$

Two flats of cross-section 80 mm × 8 mm may be tried as shown in Figure 7.11.

The outstand of the stiffener = 80 mm < (14 × 8 × 1.0 = 112 mm)

∴ The full outstand is effective.



Buckling check

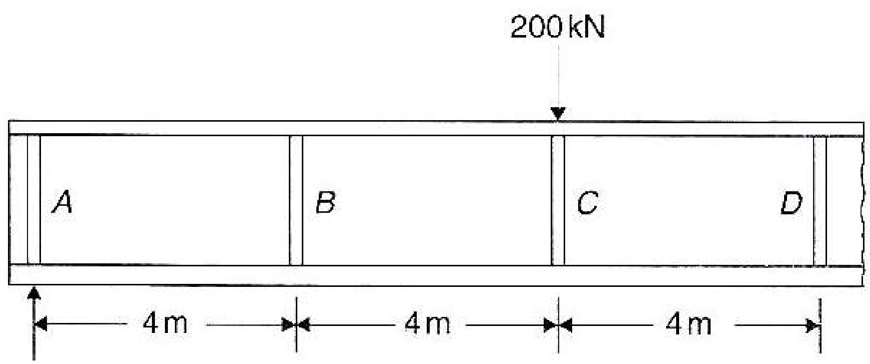
The factored shear force at B, i.e. at 4 m from the support $= 1.5\ (860 - 4 \times 55) = 960\,\text{kN}$

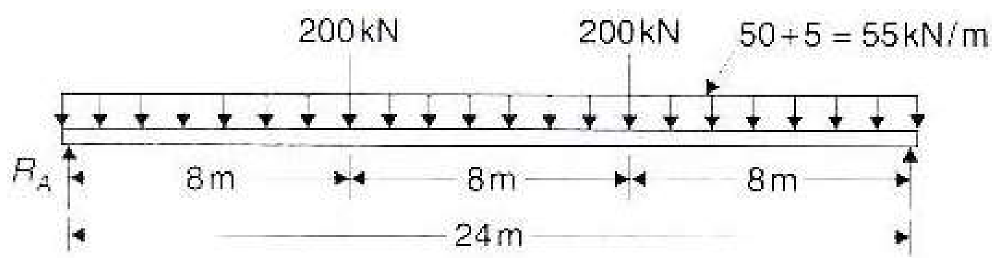
$$V_{cr} = 484\,\text{kN}$$

Force on the stiffener, $F_q = 960 - 484/1.1 = 520\,\text{kN}$

Moment of inertia of the stiffener, $I_s = 8 \times (2 \times 80 + 10)^3/12 = 3.3 \times 10^6\,\text{mm}^4$

Area of cross-section of the stiffener, $A = 2 \times 80 \times 8 + 2 \times 200 \times 10 = 5{,}280\,\text{mm}^2$





$$r = \sqrt{\frac{3.3 \times 10^6}{5,280}} = 24.9\,\text{mm}$$

$KL = 0.7 \times 2,000 = 1,400\,\text{mm}$

$$\frac{KL}{r} = \frac{1,400}{24.9} = 56$$

From Table 4.3, for the buckling class c, $f_{cd} = 174\,\text{MPa}$

The design compressive strength of the stiffener, $F_{qd} = 5,280 \times 174$
$= 919\,\text{kN} > 520\,\text{kN}$ OK

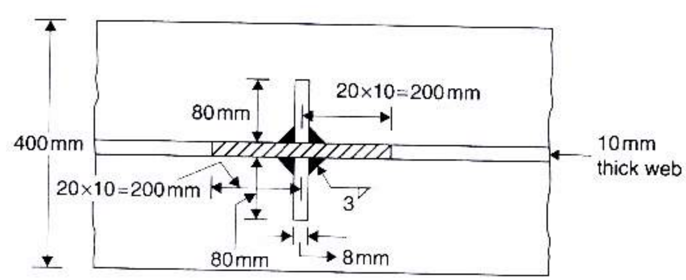
Shear force per unit length between each component of the stiffener and the web

$$= \frac{t_w^2}{5b_s} = \frac{10^2}{5 \times 80} = 0.25\,\text{kN/mm} = 250\,\text{N/mm}$$

Each component of the stiffener is connected by two fillet welds

$$\therefore \quad 2 \times 0.7 \times s \times 189.4 = 250$$
$$s = 0.94\,\text{mm}$$

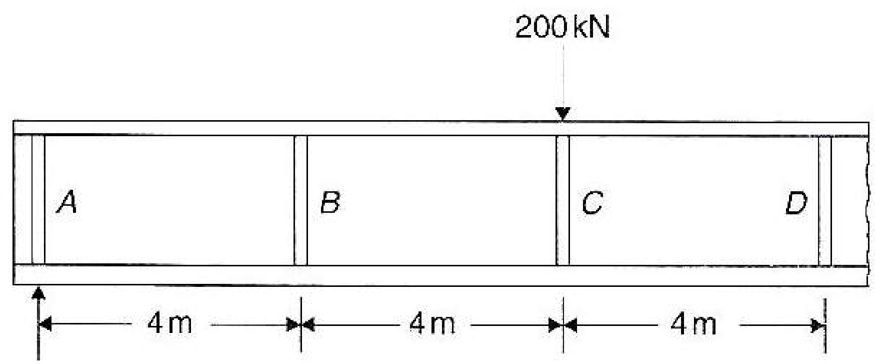
3 mm size fillet welds may be provided as shown in Figure 7.11.

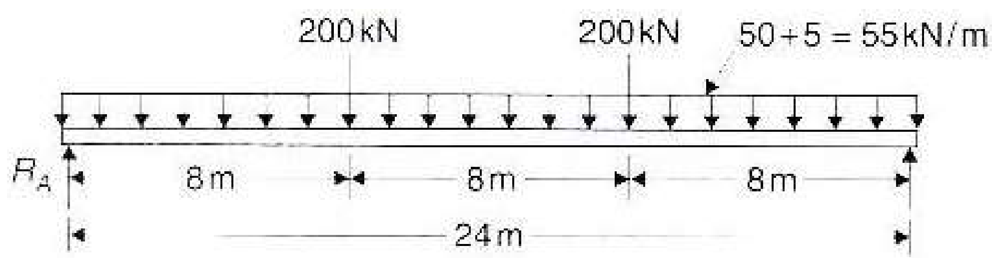


Design of the intermediate stiffener at C

Since a concentrated load of 200 kN acts at C, the stiffener should be designed as intermediate transverse stiffener and bearing/load carrying stiffener. For this, the following interaction condition should be satisfied.

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} \leq 1.0$$



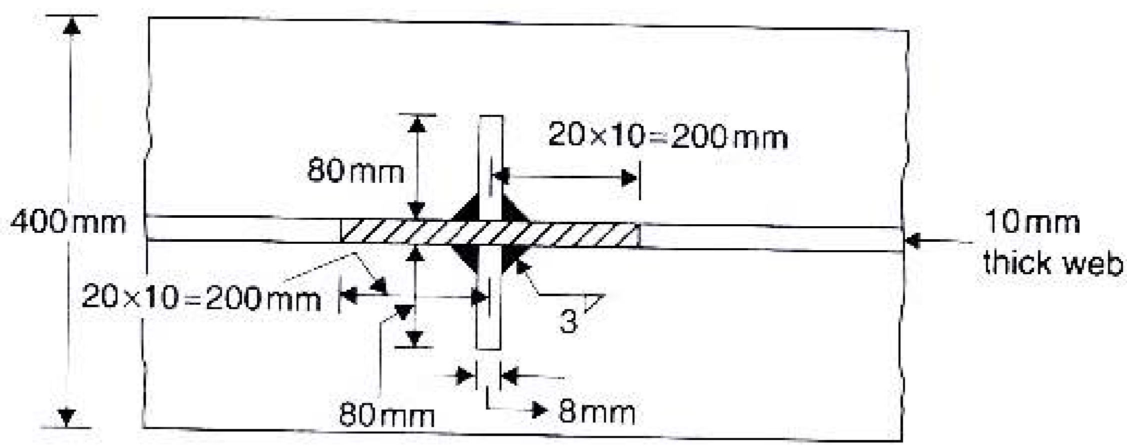


The factored shear force at C (at 8 m from the support), $V = 1.5\,(860 - 8 \times 55) = 630\,\text{kN}$
The same section provided for the stiffener at B may be considered at C.

$V_{cr} = 484\,\text{kN}$
$F_q = 630 - 484/1.1 = 190\,\text{kN}$
$F_x = 1.5 \times 200 = 300\,\text{kN}$
$F_{xd} = F_{qd} = 919\,\text{kN}$

$$\therefore \quad \frac{190 - 300}{919} + \frac{300}{919} = 0 + 0.33 = 0.33 < 1.0 \qquad \text{OK}$$

Since $F_q < F_x$, $(F_q - F_x)$ is considered zero.
So, two 80 mm × 8 mm flats may be provided at C as also shown in Figure 7.11.
The stiffener need not be checked for bearing since the local bearing capacity of web (455 kN) is more than factored load (300 kN).



Connection

The shear force between each component of the stiffener and the web due to the external load

$$= \frac{1.5 \times 200 \times 10^3}{2 \times 2{,}000} = 75\ \text{N/mm}$$

The total shear force $= \dfrac{t_w^2}{5b_s} + 75 = 250 + 75 = 325$ N/mm

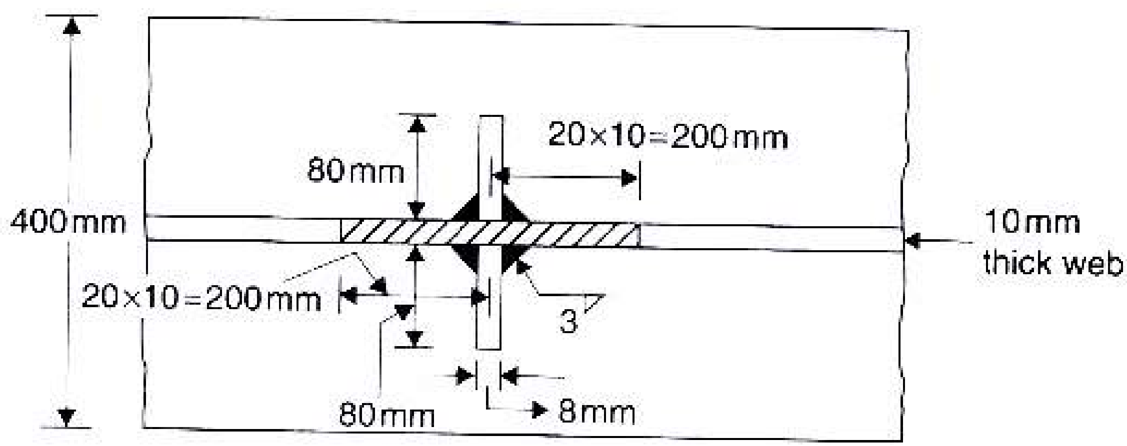
Again, 3 mm size fillet welds will be sufficient.

The stiffener may be connected to the flanges.

$4 \times (175 - 10) \times 0.7s \times 189.4 = 1.5 \times 200 \times 10^3$

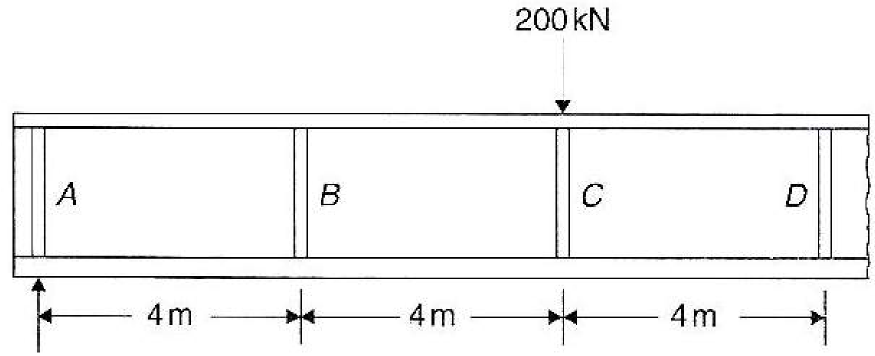
or $\quad s = 3.43$ mm

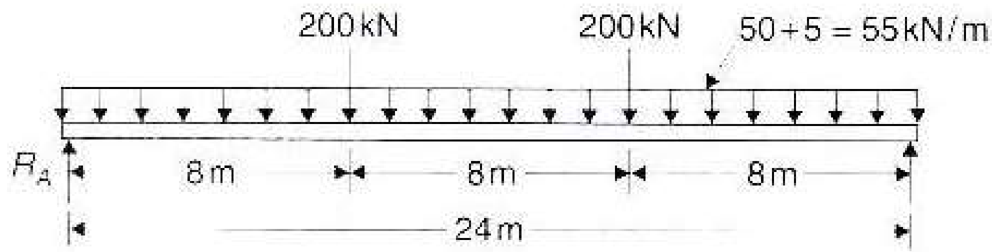
5 mm size fillet weld may be provided.

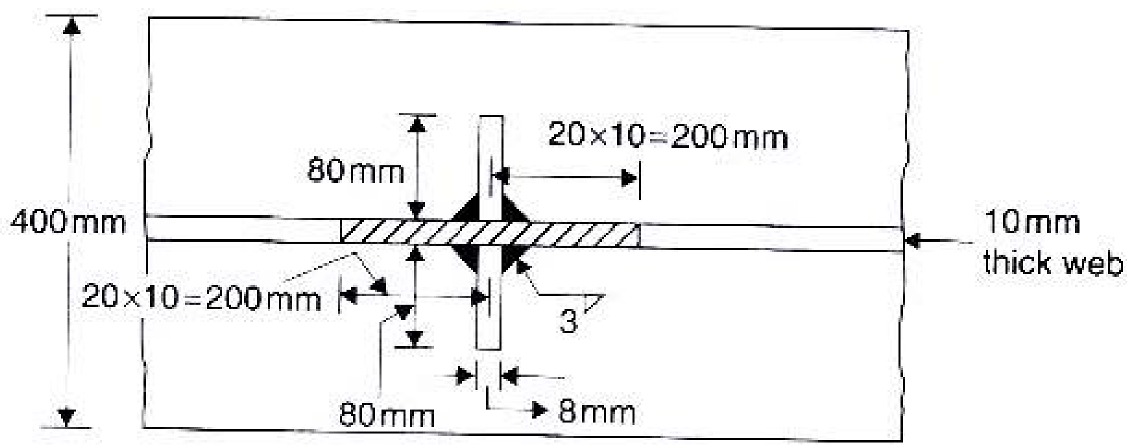


Intermediate stiffener at D

The same stiffener provided at *B* and *C* may also be provided at *D*.







Darbhanga College of Engineering, Darbhanga

B.Tech - 6th Sem. (2017-21), Civil Engineering Dept.

Subject (Theory):- DESIGN OF STEEL STRUCTURES

S.N.	Registration No.	Student Name	5 Attendance	5 Assignment	20 Online_Exam	Total (30)
1	17101111001	ARUN KUMAR	3	5	19	27
2	17101111002	PRADUMN KUMAR	5	5	20	30
3	17101111003	KETAN SINHA	5	4	17	26
4	17101111004	MD MATLUB NEYAZ	5	5	20	30
5	17101111005	DEEPAK KUMAR	3	4	18	25
6	17101111006	MANIKANT KUMAR	5	5	17	27
7	17101111007	SURAJ KUMAR	3	5	20	28
8	17101111008	AKASH RAMAN	5	4	16	25
9	17101111009	SURENDRA KUMAR SAHU	3	5	19	27
10	17101111010	ASHUTOSH KUMAR	5	3	18	26
11	17101111011	NITISH KUMAR	5	5	20	30
12	17101111012	BAUSKI KUMAR	5	5	16	26
13	17101111013	SANJEEV KUMAR SINGH	5	5	20	30
14	17101111014	SANDEEP KUMAR	5	5	20	30
15	17101111015	SHUBHAM KUMAR	3	4	18	25
16	17101111016	DEEPAK KUMAR	3	4	16	23
17	17101111017	REKESH KUMAR MAHTO	3	5	15	23
18	17101111018	UMA SHANKAR	5	5	17	27
19	17101111020	AMIT KUMAR	3	4	19	26
20	17101111021	RADHESH JHA SUMAN	3	5	20	28
21	17101111022	ANISH ANAND	3	3	16	22
22	17101111023	RAM BILASH YADAV	3	5	16	24

23	17101111024	SACHIN KUMAR	3	5	19	27
24	17101111025	ANSHU KUMAR	3	5	17	25
25	17101111026	UDAY LAL DAS	5	5	20	30
26	17101111028	CHANDAN KUMAR	3	4	17	24
27	17101111029	SANJEET KR PRABHAKAR	3	5	20	28
28	17101111030	NAVEEN KUMAR	3	4	19	26
29	17101111031	RAKESH KUMAR	5	5	19	29
30	17101111032	RANJEET KUMAR PATHAK	4	5	16	25
31	17101111033	MANISH KUMAR	5	5	20	30
32	17101111034	PRAVEEN KUMAR	5	5	19	29
33	17101111035	NAVIN KUMAR	5	5	18	28
34	17101111036	VISHAL KUMAR	3	4	19	26
35	17101111037	ISAAC ARYA	4	5	19	28
36	17101111038	VIKASH KUMAR	5	5	20	30
37	17101111039	MEERA KUMARI	5	5	20	30
38	17101111040	NEHAL RAJ	3	5	20	28
39	17101111041	SNEHLATA	5	5	19	29
40	17101111042	ASHISH KUMAR	3	5	13	21
41	17101111043	OM PRAKASH	5	5	20	30
42	17101111044	DARPAN KUMAR	5	5	17	27
43	17101111045	VIKASH KUMAR	5	5	18	28
44	17101111046	PRINCE KUMAR	5	5	19	29
45	17101111047	LALU KUMAR	5	4	18	27
46	17101111048	SANTOSH ANAND	5	5	20	30
47	17101111049	MD ESTEKHAR ALAM	5	5	18	28
48	17101111050	TARUN KUMAR	5	5	19	29
49	17101111052	MASOOM AKHTAR	5	5	20	30

50	17101111054	MUKUL KUMAR	3	4	20	27
51	17101111055	SHUBHAM RAJ	3	4	18	25
52	17101111056	KRISHNA ARYA	5	5	18	28
53	17101111057	PRATIK KUMAR	5	5	20	30
54	17101111058	AJIT KUMAR YADAV	3	5	20	28
55	17101111059	ABHISHEK RAJ	5	5	19	29
56	17101111060	RAHUL KUMAR	3	4	17	24
57	17101111061	VIKRAM KUMAR	5	5	20	30
58	17101111062	RAKESH KUMAR	5	5	16	26
59	17101111063	NIRANJAN KUMAR	3	3	20	26
60	17101111064	NITISH KUMAR	3	5	15	23
61	17101111065	MANISH KUMAR SINGH	3	5	17	25
62	17101111066	VIVEK MANI	3	5	19	27
63	18101111001	AMIT KUMAR	3	4	17	24
64	18101111901	SITA RAM YADAV	5	5	16	26
65	18101111902	RANVEER KUMAR YADAV	3	5	18	26
66	18101111903	NITISH KUMAR	3	5	19	27
67	18101111904	SONIKA NIRANJAN	5	5	19	29
68	18101111905	INDRA KUMAR	3	3	19	25
69	18101111906	SUMIT KUMAR	5	5	19	29
70	18101111908	NAVEEN KUMAR	3	4	20	27
71	18101111909	DEEPAK KUMAR	3	4	15	22

Sign:-

Darbhanga College of Engineering, Darbhanga

B.Tech - 6th Sem. (2017-21), Civil Engineering Dept.

Subject (Lab):- Design of Steel Structures

S.N.	Registration No.	Student Name	5	5	10	Total (20)
1	17101111001	ARUN KUMAR	3	5	8	16
2	17101111002	PRADUMN KUMAR	5	5	9	19
3	17101111003	KETAN SINHA	5	4	8	17
4	17101111004	MD MATLUB NEYAZ	5	5	9	19
5	17101111005	DEEPAK KUMAR	4	4	8	16
6	17101111006	MANIKANT KUMAR	5	5	9	19
7	17101111007	SURAJ KUMAR	3	5	8	16
8	17101111008	AKASH RAMAN	5	4	9	18
9	17101111009	SURENDRA KUMAR SAHU	3	5	8	16
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11	17101111011	NITISH KUMAR	5	5	9	19
12	17101111012	BAUSKI KUMAR	5	5	9	19
13	17101111013	SANJEEV KUMAR SINGH	5	5	9	19
14	17101111014	SANDEEP KUMAR	5	5	9	19
15	17101111015	SHUBHAM KUMAR	4	4	8	16
16	17101111016	DEEPAK KUMAR	4	4	8	16
17	17101111017	REKESH KUMAR MAHTO	3	5	8	16
18	17101111018	UMA SHANKAR	5	5	9	19
19	17101111020	AMIT KUMAR	4	4	8	16
20	17101111021	RADHESH JHA SUMAN	3	5	8	16
21	17101111022	ANISH ANAND	3	3	8	14
22	17101111023	RAM BILASH YADAV	3	5	8	16
23	17101111024	SACHIN KUMAR	3	5	8	16

24	17101111025	ANSHU KUMAR	3	5	8	16
25	17101111026	UDAY LAL DAS	5	5	9	19
26	17101111028	CHANDAN KUMAR	4	4	8	16
27	17101111029	SANJEET KR PRABHAKAR	3	5	9	17
28	17101111030	NAVEEN KUMAR	4	4	8	16
29	17101111031	RAKESH KUMAR	5	5	9	19
30	17101111032	RANJEET KUMAR PATHAK	4	5	9	18
31	17101111033	MANISH KUMAR	5	5	9	19
32	17101111034	PRAVEEN KUMAR	5	5	9	19
33	17101111035	NAVIN KUMAR	5	5	9	19
34	17101111036	VISHAL KUMAR	4	4	8	16
35	17101111037	ISAAC ARYA	4	5	9	18
36	17101111038	VIKASH KUMAR	5	5	9	19
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45	17101111047	LALU KUMAR	5	4	8	17
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47	17101111049	MD ESTEKHAR ALAM	5	5	9	19
48	17101111050	TARUN KUMAR	5	5	9	19
49	17101111052	MASOOM AKHTAR	5	5	9	19
50	17101111054	MUKUL KUMAR	3	4	9	16

51	17101111055	SHUBHAM RAJ	4	4	8	16
52	17101111056	KRISHNA ARYA	5	5	9	19
53	17101111057	PRATIK KUMAR	5	5	9	19
54	17101111058	AJIT KUMAR YADAV	3	5	8	16
55	17101111059	ABHISHEK RAJ	5	5	9	19
56	17101111060	RAHUL KUMAR	4	4	8	16
57	17101111061	VIKRAM KUMAR	5	5	9	19
58	17101111062	RAKESH KUMAR	5	5	8	18
59	17101111063	NIRANJAN KUMAR	4	4	8	16
60	17101111064	NITISH KUMAR	3	5	8	16
61	17101111065	MANISH KUMAR SINGH	3	5	8	16
62	17101111066	VIVEK MANI	3	5	9	17
63	18101111001	AMIT KUMAR	4	4	8	16
64	18101111901	SITA RAM YADAV	5	5	9	19
65	18101111902	RANVEER KUMAR YADAV	3	5	9	17
66	18101111903	NITISH KUMAR	3	5	8	16
67	18101111904	SONIKA NIRANJAN	5	5	9	19
68	18101111905	INDRA KUMAR	3	3	8	14
69	18101111906	SUMIT KUMAR	5	5	9	19
70	18101111908	NAVEEN KUMAR	3	4	8	15
71	18101111909	DEEPAK KUMAR	3	4	8	15

Sign:-

Darbhanga College of Engineering, Darbhanga

B.Tech - 6th Sem. (2017-21), Civil Engineering Dept.

Subject (Theory):- DESIGN OF STEEL STRUCTURES

S.N.	Registration No.	Student Name	5 Attendance	5 Assignment	20 Online_Exam	20 IA_Lockdown	Total (50)
1	17101111001	ARUN KUMAR	3	5	19	16	43
2	17101111002	PRADUMN KUMAR	5	5	20	18	48
3	17101111003	KETAN SINHA	5	4	17	16	42
4	17101111004	MD MATLUB NEYAZ	5	5	20	19	49
5	17101111005	DEEPAK KUMAR	3	4	18	15	40
6	17101111006	MANIKANT KUMAR	5	5	17	18	45
7	17101111007	SURAJ KUMAR	3	5	20	16	44
8	17101111008	AKASH RAMAN	5	4	16	17	42
9	17101111009	SURENDRA KUMAR SAHU	3	5	19	16	43
10	17101111010	ASHUTOSH KUMAR	5	3	18	18	44
11	17101111011	NITISH KUMAR	5	5	20	18	48
12	17101111012	BAUSKI KUMAR	5	5	16	18	44
13	17101111013	SANJEEV KUMAR SINGH	5	5	20	18	48
14	17101111014	SANDEEP KUMAR	5	5	20	18	48
15	17101111015	SHUBHAM KUMAR	3	4	18	16	41
16	17101111016	DEEPAK KUMAR	3	4	16	15	38
17	17101111017	REKESH KUMAR MAHTO	3	5	15	16	39
18	17101111018	UMA SHANKAR	5	5	17	19	46
19	17101111020	AMIT KUMAR	3	4	19	15	41
20	17101111021	RADHESH JHA SUMAN	3	5	20	16	44
21	17101111022	ANISH ANAND	3	3	16	15	37
22	17101111023	RAM BILASH YADAV	3	5	16	16	40

23	17101111024	SACHIN KUMAR	3	5	19	16	43
24	17101111025	ANSHU KUMAR	3	5	17	16	41
25	17101111026	UDAY LAL DAS	5	5	20	19	49
26	17101111028	CHANDAN KUMAR	3	4	17	16	40
27	17101111029	SANJEET KR PRABHAKAR	3	5	20	19	47
28	17101111030	NAVEEN KUMAR	3	4	19	16	42
29	17101111031	RAKESH KUMAR	5	5	19	17	46
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32	17101111034	PRAVEEN KUMAR	5	5	19	18	47
33	17101111035	NAVIN KUMAR	5	5	18	17	45
34	17101111036	VISHAL KUMAR	3	4	19	16	42
35	17101111037	ISAAC ARYA	4	5	19	18	46
36	17101111038	VIKASH KUMAR	5	5	20	18	48
37	17101111039	MEERA KUMARI	5	5	20	19	49
38	17101111040	NEHAL RAJ	3	5	20	16	44
39	17101111041	SNEHLATA	5	5	19	19	48
40	17101111042	ASHISH KUMAR	3	5	13	16	37
41	17101111043	OM PRAKASH	5	5	20	18	48
42	17101111044	DARPAN KUMAR	5	5	17	18	45
43	17101111045	VIKASH KUMAR	5	5	18	16	44
44	17101111046	PRINCE KUMAR	5	5	19	18	47
45	17101111047	LALU KUMAR	5	4	18	17	44
46	17101111048	SANTOSH ANAND	5	5	20	19	49
47	17101111049	MD ESTEKHAR ALAM	5	5	18	18	46
48	17101111050	TARUN KUMAR	5	5	19	19	48
49	17101111052	MASOOM AKHTAR	5	5	20	19	49

50	17101111054	MUKUL KUMAR	3	4	20	18	45
51	17101111055	SHUBHAM RAJ	3	4	18	16	41
52	17101111056	KRISHNA ARYA	5	5	18	18	46
53	17101111057	PRATIK KUMAR	5	5	20	18	48
54	17101111058	AJIT KUMAR YADAV	3	5	20	16	44
55	17101111059	ABHISHEK RAJ	5	5	19	18	47
56	17101111060	RAHUL KUMAR	3	4	17	16	40
57	17101111061	VIKRAM KUMAR	5	5	20	18	48
58	17101111062	RAKESH KUMAR	5	5	16	16	42
59	17101111063	NIRANJAN KUMAR	3	3	20	16	42
60	17101111064	NITISH KUMAR	3	5	15	16	39
61	17101111065	MANISH KUMAR SINGH	3	5	17	15	40
62	17101111066	VIVEK MANI	3	5	19	19	46
63	18101111001	AMIT KUMAR	3	4	17	16	40
64	18101111901	SITA RAM YADAV	5	5	16	17	43
65	18101111902	RANVEER KUMAR YADAV	3	5	18	17	43
66	18101111903	NITISH KUMAR	3	5	19	16	43
67	18101111904	SONIKA NIRANJAN	5	5	19	18	47
68	18101111905	INDRA KUMAR	3	3	19	16	41
69	18101111906	SUMIT KUMAR	5	5	19	19	48
70	18101111908	NAVEEN KUMAR	3	4	20	16	43
71	18101111909	DEEPAK KUMAR	3	4	15	15	37

Sign:-

Institute/college Name	Darbhanga College of Engineering, Darbhanga
Corse/Branch	B.Tech./Civil Engineering
Year/Semester	III/VI
Course Code/Choice	011620/ Core
Course credits	4
Course Name	Design of Steel Structure
Lecture/ Sessional (per week)	4/0
Course Teacher name	Ahsan Rabbani
Deptt./Designation	Civil Engineering/Assistant Professor

1. Scope and Objectives of the Course

Many civil engineering structures are made up of steel. Knowledge of designing and detailing of steel structures is very important for civil engineers in order to make structures safe and serviceable during its life span. Limit State design philosophy is currently used worldwide for design of steel structures and its various components. Also precise and correct detailing of structural drawing is necessary in order to get the correct behavior of structures and leads to smooth construction of structures. This course will provide detailed knowledge of design and detailing of steel structures as per Indian standards.

The concepts of this course are applicable in all civil engineering structures. The Design of Steel Structures curriculum is designed to prepare interested students for a future career in the field of Structural Engineering, Earthquake and Wind Engineering. The course deals with design of steel structures using "Limit State Design Method". The design methodology is based on the latest Indian Standard Code of Practice for general construction (IS 800:2007). The subject covers all the necessary components such as material specifications, connections and elementary design of structural members for designing industrial steel structures. The course provides material specifications and design considerations. It provides relevant material properties of different types of steel. It deals with two types of connections namely welded and bolted connections.

At the end of this course, the students will be able to

CO1: Understand the knowledge of different connections used in steel structures

CO2: Evaluate how to determine the design strength of tensile members

CO3: Evaluate how to determine the design strength of compression members

CO4: Understand about laterally supported, laterally un-supported beam, plate girder and design of column bases.

CO5: Analyze the plastic theory on steel structures.

2. Teaching/Learning Methodology

Interactive lectures will enable students to understand the basic design concepts and learn how to design basic structural members with due consideration to their service conditions; Tutorial will enable students to consolidate the structural design concept through design problem-solving assignments and discussions.

3. Expected outcome

The student will be able to: Understanding of the WSM, LSM and LRFD design philosophies and behavior of structural steel; Ability to analyze and design of tension members, columns, beams, beam-columns; Ability to analyze and design of simple bolted and welded connections; Ability to design steel framing system and connections of a building in a team setting; Familiarity with structural steel fabrication process and construction through field trip and/or speaker presentation; Familiarity with professional and ethical issues and the importance of lifelong learning in structural engineering.

4. Textbooks

TB1: Bhavikatti.S.S, "Design of Steel Structures" By Limit State Method as per IS: 800–2007, IK International Publishing House Pvt. Ltd., 2009.

TB2: Duggal. S.K, "Limit State Design of Steel Structures", Tata McGraw Hill Publishing Company.

TB3: Chandak, N.R., “Design of Steel Structures", Katson Publication.

TB4: Subramanian.N, "Design of Steel Structures", Oxford University Press, New Delhi, 2013.

5. Reference Books

RB1: Shah.V.L. and Veena Gore, "Limit State Design of Steel Structures", IS 800–2007 Structures Publications, 2009.

RB2: Negi, L. S., “Design of Steel Structures”, Tata McGraw Hill.

6. Required Code

1. IS 800: 2007, General Construction in Steel - Code of Practice, (Third Revision), Bureau of Indian Standards, New Delhi, 2007.
2. IS 875 (Part 1): Indian Standard Code of Practice for Dead Loads, Bureau of Indian Standards, New Delhi.
3. IS 875 (Part 2): Indian Standard Code of Practice for Imposed Loads, Bureau of Indian Standards, New Delhi.
4. IS 875 (Part 3): Indian Standard Code of Practice for Wind Loads, Bureau of Indian Standards, New Delhi.

Note: IS: 800(2007), Steel table are permitted in the examination.

Other readings and relevant websites:

S.No.	Link of Journals, Magazines, websites and Research Papers
1.	https://nptel.ac.in/courses/105105162/
2.	elearning.vtu.ac.in
3.	http://www.steel-insdag.org/new/contents.asp

4. Course Plan

Lecture Number	Date of Lecture	Topics	Web Links for lectures	Text Book / Reference Book	Page numbers of Text Book(s)
~~1-4~~		**Introduction**			
		Importance of steel structure, Type of steel structure and its properties, type of load acting on the steel structures, calculation of the loads and load combination Working stress design, plastic design and LRFD method	http://mzsengineeringtechnologies.blogspot.com/2013/11/welded-connections.html https://nptel.ac.in/courses/105105162/		
~~4-8~~		**Design of structural fasteners**			
		Type of connection, advantages and disadvantages of riveted connection.	https://nptel.ac.in/courses/105105162/		

		Introduction to bolted connection, types of bolts, advantages and disadvantages of bolted connection, terminology used in bolted connection, Numerical question related to design of bolted connection. Introduction to welded connection, advantages and disadvantages of welded connection, Types of welded joints, weld size for butt weld, Introduction to fillet weld, specification of fillet weld, Design stresses in weld, Numerical on design of welded connection. Design of eccentric connection	https://www.slideshare.net/ahsanrabbani/design-of-steel-structure-as-per-is-8002007 https://www.youtube.com/watch?v=OTr8G2ITxIl https://www.youtube.com/watch?v=3XffzIsYCx0 https://www.youtube.com/watch?v=cg9VSJTvuZk		
~~9-13~~		**Design of tension members**			
		Different types of tension member, Design steps as per IS code, Numerical questions on design of tension member, Introduction to Lug Angle.	https://nptel.ac.in/courses/105105162/		
~~14-19~~		**Design of Compression members**			
		Introduction to compression member, buckling of column, Buckling class of cross section, Slenderness ratio, actual length and effective length, Design of compression member, Numerical question on design of compression member. Designing of built-up member, Designing of single lacing & double lacing column, batten etc.	https://nptel.ac.in/courses/105105162/		
~~20-23~~		**Design of flexural member**			
		Design of laterally supported and laterally unsupported beam as per IS Code, Numerical questions on beam, design of built-up section, Introduction to plate girder, Design of eccentric connection.	https://nptel.ac.in/courses/105105162/ https://www.slideshare.net/ahsanrabbani/welded-plate-girder https://www.youtube.com/watch?v=rxFP-cYpbLw http://www.engineeringenotes.com/civil-engineering/girders/plate-girder-components-and-design-construction-civil-engineering/38215 https://theconstructor.org/structural-engg/plate-girder-bridges/2066/		
~~24-28~~		**Design of Beam**			
		Design of column and slab bases, moment resistant connection, semi rigid connection, design of supports etc.	https://nptel.ac.in/courses/105105162/ https://www.slideshare.net/ahsanrabbani/beam-column-design		

			https://www.slideshare.net/secret/teb9GFKUnBQ0OG https://www.slideshare.net/ahsanrabbani/column-splices-design https://freevideolectures.com/course/2679/design-of-steel-structures/29 https://www.steelconstruction.info/Moment_resisting_connections		
~~28-32~~		**Designing of industrial shed**			
		Introduction to design of steel industrial shed and design of structural member due to wind load.	https://nptel.ac.in/courses/105105162/ https://www.slideshare.net/SudhirGayake/design-of-industrial-roof-truss https://www.steelconstruction.info/Single_storey_industrial_buildings http://www.iitk.ac.in/nicee/IITK-GSDMA/W06.pdf https://www.youtube.com/watch?v=aqEQFiMc2RE		
~~32-35~~		**Plastic analysis**			
		Introduction to inelastic action and plastic hinges. Determination of plastic section modulus, moment resistance, theory of plastic analysis, numerical on plastic theory. Concept of LRFD	https://nptel.ac.in/courses/105105162/		

Syllabus:

Sl. No.	Topics	No. of Lectures	Weightage (%)
1	**Introduction to Design:** Design Loads and Load combinations, Working Stress Design, Plastic Design, LRFD Method, Introduction to steel and steel structures.	4	5
2	**Design of structural Fasteners:** rivets, bolts and welds.	6	15
	Design of tension members.	4	15
3	**Design of compression member:** laced and battened columns.	6	20
4	**Design of flexure members:** Beams- rolled sections, built up section, plate Girders- riveted/ bolted and welded, Design of eccentric connections: riveted/ bolted and welded.	8	15
5	**Design of beam:** Columns and columns based welded and riveted column bases- moment resistant connection - semi rigid connection- design of supports.	5	10
6	**Design of steel industrial sheds**. Wind Design.	8	10

7	Introduction inelastic action and plastic hinges application of PD and LRFD.	3	10

Evaluation Details:

<table>
<tr><th colspan="6">Total Marks=150</th></tr>
<tr><td colspan="4">Internal Marks= 50</td><td colspan="2">External Marks=100</td></tr>
<tr><td colspan="3">Theory=30</td><td>Sessional=20</td><td>Theory=70</td><td>Sessional=30</td></tr>
<tr><td>Mid Term =20</td><td>Attendance =5</td><td>Class Test/ Assignment =5</td><td></td><td>End Semester Exam= 70**</td><td></td></tr>
</table>

****** The End Term Comprehensive examination will be held at the end of semester. The mandatory requirement of 75% attendance in all theory classes is to be met for being eligible to appear in this component.

Approval authority of this document:

Designation	Name	Signature
Course Coordinator	Mr. Ahsan Rabbani	
H.O.D	Mr. S.S Choudhary	

Details of Assignments:

Sl. No.	Assignment No.	Types of assignment	Date of submission	Checked (Yes/No)	Remarks
1	Assignment 1	Subjective			
2	Assignment 2	Subjective			
3	Assignment 3	Subjective			