

Structural Design and Foundation Design

ACBA6018

Seismic Design &
Structural Analysis & Design of a Small Office Building

Timothy Issac Mills

Student Number: 2206144

Submission Date: 01/05/2023

Contents

1) Seismic Design of Buildings 1a) Introduction	4
1b) Seismic Design Codes	5
1c) Structural Systems for Seismic Design	5
Moment-Resisting Frames (MRF) System	6
Braced Frame System	6
Shear Wall Systems	7
Dual System	7
Base Isolation System	8
1d) Case Study – Turkey & Syria Earthquake 2023	9
1e) Conclusion	10
1f) References	10
2) Structural Analysis & Design of a Small Office Building	12
2a) Introduction	12
2b) Proposed Design Plans of Office Block	13
2c) Structural Analysis of Member A	15
2d) Structural Analysis of Member B	19
2e) Structural Analysis of Member C	23
2f) Structural Analysis of Member D	27
2g) Structural Analysis of Member E	30
2h) Structural Analysis of Column A	34
2i) Structural Analysis of Base Plate - Column A	38
2j) Structural Analysis of Connection A – Member C to Column A	41
3) Appendix A – Member A – UB254x146x31	45
4) Appendix B – Member A – UB305x102x28	47
5) Appendix C – Member A – UB254x102x28 - Check	49
6) Appendix D – Member B – UB305x165x40	51
7) Appendix E – Member B – UB254x146x43	53
8) Appendix F – Member B – UB305x127x37 - Check	55
9) Appendix G - Member C - UB533x312x151	57
10) Appendix H - Member C - UB610x305x149	59
11) Appendix I - Member C - UB533x312x182	61
12) Appendix J - Member C - UB610x305x179 - Check	63
13) Appendix K - Member D - UB610x305x179 - Check	65
14) Appendix L - Member E - UB178x102x19	67
15) Appendix M - Member E - UB203x102x23	69
16) Appendix N - Member E - UB254x102x22 - Check	71
17) Appendix O - Column A - UC254x254x107	73

18) Appendix P - Column A - UC305x305x97	75
19) Appendix Q - Column A - UC305x305x118 - Check.....	77

1) Seismic Design of Buildings

1a) Introduction

Seismic activity is the occurrence of earthquakes, developing in a sudden release of energy from the earth's crust causing ground movement & vibration. These seismic events can have a profound impact on both natural and manmade structures, including buildings, bridges, slopes, and other infrastructure.

During such events, structures are subjected to increased vertical and horizontal forces, which must be carefully considered during the design process to ensure the building's ability to withstand seismic forces. Horizontal forces pose the greatest risk to structures due to the sway and twisting motions they endure, which can severely compromise the structures stability. The failure of a structure during a seismic event can lead to catastrophic consequences, it is essential for civil engineers, structural engineers, and architects to account for seismic activity in their designs. This includes ensuring that beams, columns, and foundations are designed in compliance with relevant seismic design codes.

This research aims to explore the considerations involved in the seismic design of both structural and architectural aspects. It will examine various design codes; review case studies of structural failures caused by seismic activity and highlight successful seismic design strategies that have benefited both structural integrity and architectural functionality.

Although the UK is not classed as a high seismic area it is susceptible to small seismic activity. Below is a map to show the seismic activity across the UK, with the highest seismic area being in Wales.

While the UK is not considered a high seismic region, it is susceptible to minor seismic activity. The map below illustrates seismic activity across the country, highlighting Wales as the area with the highest recorded seismic activity.

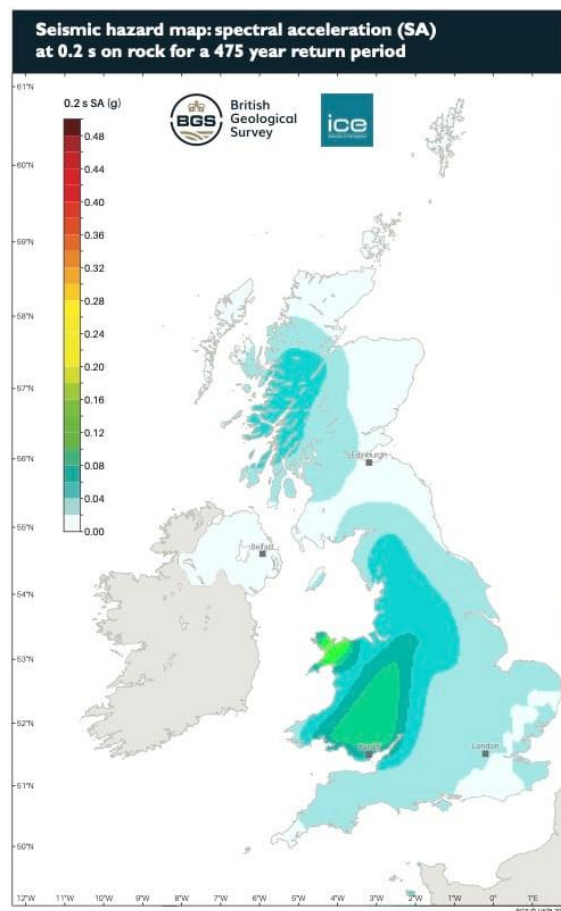


Image 1 - ICE - Seismic Map UK (Baptie et al., 2020)

1b) Seismic Design Codes

Seismic design is governed by British Standard BS EN 1998 (Eurocode 8) – Design of Structures for Earthquake Resistance. This set of design codes offers a detailed guideline to engineers in creating buildings and infrastructure that can withstand the forces generated by seismic.

BS EN 1998 (Eurocode 8) outlines the essential requirements for designing structures to resist both the horizontal and vertical forces typically experienced during an earthquake. These codes cover various aspects, including material strength, structural layout, and safety measures, ensuring that all potential risks are considered during the planning and construction phases.

By following these design codes, engineers can effectively design and lessen the effects of seismic forces, improving the overall resilience and safety of buildings. Furthermore, adherence to seismic design codes helps to regulate practices across the industry, consistency and reliability in the design of earthquake-resistant structures. This ensures that buildings are not only functional but also safe in regions prone to seismic activity.

“Providing specific building-related requirements for various structural materials (including concrete, steel, timber and masonry), it details the fundamental requirements and other significant design and safety issues related to the base isolation of structures, particularly buildings” (BSI Group, 2025)

BS EN 1998 (Eurocode 8) – *Design of structures for earthquake resistance* is broken down into 6 sections, these are as follows:-

BS EN 1998-1 (EC8 – Part 1) - *General rules, seismic actions, and rules for buildings*

BS EN 1998-2 (EC8 – Part 2) - *Bridges*

BS EN 1998-3 (EC8 – Part 3) - *Strengthening and retrofitting of buildings*

BS EN 1998-4 (EC8 – Part 4) - *Silos, tanks, and pipelines*

BS EN 1998-5 (EC8 – Part 5) - *Foundations, retaining structures, and geotechnical aspects.*

BS EN 1998-6 (EC8 – Part 6) - *Towers, masts, and chimneys.*

1c) Structural Systems for Seismic Design

During seismic design, selecting a suitable structural system is essential for ensuring a building's stability during an earthquake. There are several structural systems used for seismic design these are: -

- Moment-Resisting Frames System
- Braced Frame System
- Shear Wall System
- Dual System
- Base Isolation System
- Diaphragm Systems
- Mass Damper System

Frame systems consist of interconnected beams and columns, which work together to resist both vertical and horizontal forces, allowing for flexibility during seismic events. Wall systems, on the other hand, rely on shear walls to provide resistance to lateral forces, offering increased stiffness and strength to the structure. Braced Frame systems combine the benefits of both frames and walls, utilizing diagonal braces to enhance the building's resistance to lateral loads while maintaining flexibility and stability.

Each of these systems has distinct advantages and is chosen based on factors such as building height, design requirements, and the severity of seismic activity in the area. The right choice of structural system is essential for optimizing a building's performance during an earthquake.

Moment-Resisting Frames (MRF) System

Moment-resisting frames are a design where the connections between beams and column are designed with moment (fixed) connections, which allows the structure to act in unison, allowing the structure to bend while maintaining its overall stability. This system is commonly used in mid-to-high rise buildings, commercial buildings and office complexes.

Some of the key features of using MRF system include: -

- High Ductility - Allowing large deformation without structurally failing.
- Architecturally flexible – Allows for fewer wall & braces to create larger open spaces.

Some of the limitations and disadvantages of the MRF include: -

- Typically, heavier members required to maintain strength and ductility.

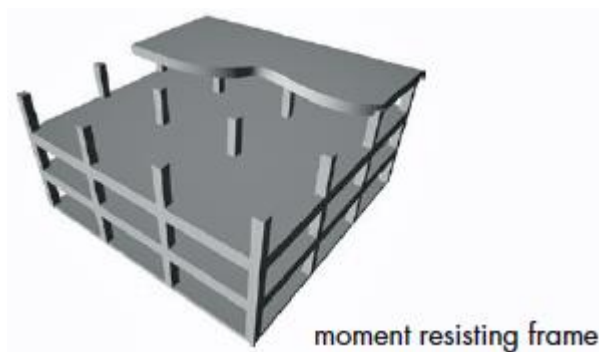


Image 2 – Moment Resisting Frame (*Hazard-specific building-resilience considerations (2025)*)

Braced Frame System

Braced Frame System resists against seismic loads through diagonal bracing members. These bracing members work by forming triangulated frames, which transfer lateral loads to the column which transfer the loads to the ground, reducing the demand on the primary members by getting the structure to work in unison and transferring the loads to the ground. This system is commonly used in mid-rise buildings, commercial buildings and retrofit applications.

There are two main types of braced frame system, these are:-

- Concentrically Braced Frames (CBF) where each brace member meets at a single point this making it efficient and stiff but less ductile.
- Eccentrically Braced Frames (EBF) where the brace members are offset from the beam/column joint, creating a link that allows for controlled deformation and high energy dissipation.

Some of the key features of using braced frame systems include: -

- Efficient load paths – Aids in axial load distribution and often lighter than MRF.
- Aids in Seismic & Wind loads – Suitable for two different load applications
- Retrofit – Bracing members can be added to existing structure to improve

Some of the limitations and disadvantages of the MRF include: -

- Complex analysis - Irregular geometry can complicate seismic modelling due to the load distribution throughout the structure
- Architectural interference – Obstruction for architectural items such as windows, doors or open space layouts.

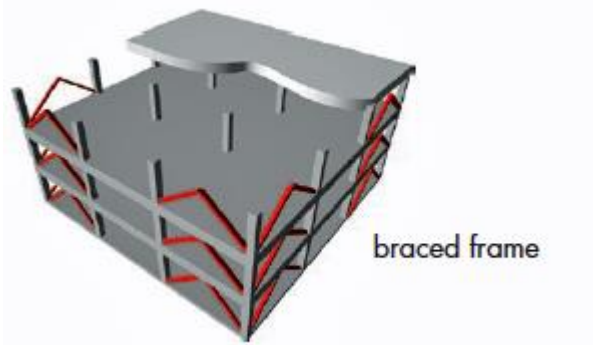


Image 3 – Braced Frame System (*Hazard-specific building-resilience considerations (2025)*)

Shear Wall Systems

A Shear Wall System resists seismic loads by using vertical structural walls that act like deep beams that transfer the loads to the foundations by resisting shear forces, bending, and overturning moments. Shear walls are typically built surrounding a stair well, elevator or the perimeter of the building. This system is commonly used in high rise buildings/residential towers, critical buildings such as schools and hospitals.

Some of the key features of using MRF system include: -

- Suitable for large load & lateral resistance – Can carry different load combinations simultaneously.
- Architectural Benefit – Can be designed to have the shear walls as the core of the structure with the main structure surrounding it.

Some of the limitations and disadvantages of the MRF include: -

- Architectural interference – Large walls may limit the design flexibility & amount of windows/openings.
- Increased weight compared to other systems – Increased mass to the structure, which can increase base shear during seismic activity.

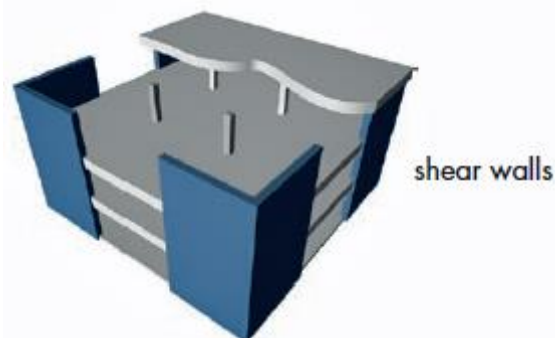


Image 4 – Shear Wall Systems (*Hazard-specific building-resilience considerations (2025)*)

Dual System

A Dual System is a combination of moment-resisting frame system & either shear walls or braced frames system. This system merges the benefits of both systems of moment frames by utilising the ductility and rigidity and strength from the shear walls or bracing elements. This system is commonly used in mid-to-high rise buildings & structures in high seismic areas.

Some of the key features of using MRF system include: -

- Balanced system – with the combination of a ductility and stiffness through both systems.
- Architectural Benefit – Can be suitable for complex design layouts.

Some of the limitations and disadvantages of the MRF include: -

- Engineering difficulty – Analysis ensuring the co-ordination between two different systems.
- Architectural limitation – space requirements for the shear walls and bracing can have limiting factors for architectural requirements such as open spaces and window/door openings.

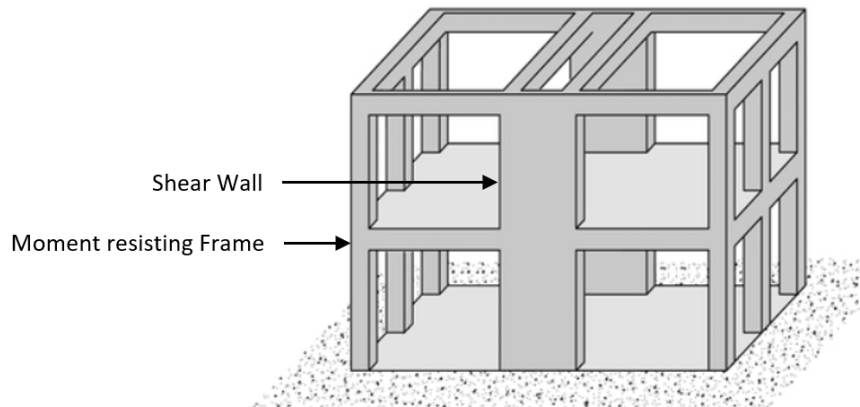


Image 5 – Dual System – Moment Resisting & Shear Wall (*Allen, L. et al. (2008)*)

Base Isolation System

Base Isolation System separates the structure from its foundations by using isolation dampeners. During an earthquake, the isolators absorb and deflect seismic energy, reducing the energy transferred to the superstructure. The common types of Isolators are high damping rubber bearings, friction pendulum system and lead rubber bearings.

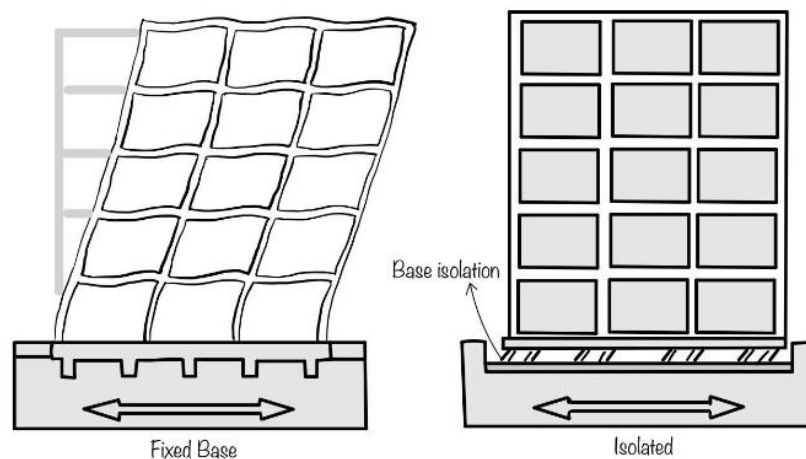


Image 5 – Dual System – Moment Resisting & Shear Wall (*Sivraj (2023)*)

1d) Case Study – Turkey & Syria Earthquake 2023

A recent disaster that occurred in Turkey and Syria, highlighted the significant impact that seismic activity can play a critical role in a structure failing. Turkey was subjected to a 7.8-magnitude earthquake which was shortly followed by a second 7.5-magnitude earthquake and many other aftershocks in February 2023, reports are estimating that at least 6,400 buildings have been destroyed across the two nations, with many more badly damaged. (Johnson, T.-2023).

The damage caused by the earthquake has been substantial which studies have forecasted the initial amount of damage is between £28.8bn and £41.2bn. (Johnson, T.-2023) There isn't just a financial impact to seismic failure, data following the disaster stated more than 9,000 deaths and 52,900 injured.



Image 6 – Turkey Earthquake Disaster 2023 (*The Independent* (2023))

Earthquake Engineering Field Investigation Team (EEFIT) at University of Cambridge undertook a longitudinal study report following the disaster, from their studies they investigated the shortfalls that caused so many building failures. The main takeaways were as follows: -

1) Non-compliance and regulatory failures.

- Profit driven construction culture with reduction in material to cut costs.
- Inadequate inspections during construction.

2) Issues within the reinforced concrete structures

- Poor concrete quality and inadequate detailing showing lack of reinforcement.
- Newly Built reinforced buildings showed several design and construction flaws
- Signs of cracking and brittle failure on non-structural elements.

3) Un-engineered/Self-Built Additions

- Many buildings showed signs of unpermitted extensions & rooftop additions, which resulted in a disruption in the balance and structural load paths.
- Ground failures such as settlement, tilting, and subsidence were evident in some locations.

EEFIT (2024)

The 2023 Turkey & Syria earthquake disaster has exposed the vulnerabilities that is caused within the built environment when seismic resilience is compromised. Although the presence of modern building codes, non-compliance, poor construction practices, and regulatory failures led to the collapse of thousands of structures. EEFIT highlighted critical issues driven by cost-cutting and insufficient oversight, which ultimately contributed to the catastrophic human and economic losses cause by the seismic activity. The disaster is a reminder of the importance of enforcing building standards and integrating seismic design into all phases of construction.

1e) Conclusion

Seismic analysis is a critical part of the design of buildings and infrastructures, when considering the unpredictable nature and severity of earthquakes. The design of a building is heavily reliant on the location of the structure; by designing under the compliance with current codes such as BS EN 1998 (Eurocode 8) ensures that the structure is designed to withstand the seismic load requirements for the specific location.

Although designing structures to withstand loads is critical it is even more critical to ensure that the design of the building is constructed accordingly as shown in the results from the Turkey and Syria Earthquake in 2023. Despite having access to modern standards, failure in implementation and compliance can result in catastrophic failures and extensive loss of life.

This report shows various seismic design systems including moment-resisting frames, braced frames, shear walls, dual systems, and base isolation and shows the engineering principles essential for reducing seismic risks. Each system presents unique benefits and limitations, and their suitability depends on structural requirements, site conditions, and seismic exposure.

Although the UK is considered a region of low seismic activity, minor tremors are still possible. As urban environments continue to evolve, it remains essential that civil and structural engineers integrate seismic considerations into their projects, ensuring safety and building performance even in regions where seismic risks are perceived to be low. When designing it is important to consider the future safeguarding of the structure for any change in climate/activity.

1f) References

- 1) Aktaş, Y. D., So, E., Johnson, C., Dönmez, K., Özden, A. T., Vatteri, A. P., O’Kane, A., Kalkan, A., Andonov, A., Verrucci, E., Çabuk, E., Opabola, E., Malcıoğlu, F. S., Marko, H. P., Giardina, G., Madabhushi, G., Triantafyllou, I., Byun, J. E., Jones, J. N., Asinari, M., Free, M., Bashein, M., Bektaş, N., Adamidis, O., Gözenoğlu, Ö., Milillo, P., Dede, Ş., Boulton, S. J., Açıkgöz, S., Gökçe, T., Efeoğlu, T., Tetik, T., Novelli, V., Gonnuru, P., Voelker, B., Tavakkoli, A., Macchiarulo, V., Gutierrez-Urzu, F., Freddi, F. & Rossetto, T. (2024). The Türkiye Earthquake Sequence of February 2023: A Longitudinal Study Report by EEFIT. Earthquake Engineering Field Investigation Team (EEFIT), Institution of Structural Engineers (IStructE). (Accessed: 12 April 2025).
- 2) Allen, L. et al. (2008) *Dual frame-wall system, Glossary for gem taxonomy*. Available at: <https://taxonomy.openquake.org/terms/dual-frame-wall-system-ldual> (Accessed: 19 April 2025).
- 3) Baptie, B. et al. (2020) *Updated seismic hazard maps for the UK | institution of civil engineers (ICE), Updated seismic hazard maps for the UK*. Available at: <https://www.ice.org.uk/news-views-insights/inside-infrastructure/updated-seismic-hazard-maps-for-the-uk> (Accessed: 10 April 2025).
- 4) *Hazard-specific building-resilience considerations* (2025) WBDG. Available at: <https://www.wbdg.org/resources/hazard-specific-building-resilience-considerations> (Accessed: 15 April 2025).
- 5) Johnson, T. (2023) *Infrastructure damage highlights severe impact of Turkey and Syria earthquakes, New Civil Engineer*. Available at: <https://www.newcivilengineer.com/latest/infrastructure-damage-highlights-severe-impact-of-turkey-and-syria-earthquakes-13-02-2023/> (Accessed: 19 April 2025).

- 6) Sivraj (2023) What are the types of base isolation devices, eigenplus. Edited by M. Krishnan and K. Patel. Available at: <https://www.eigenplus.com/what-are-the-types-of-base-isolation-devices/> (Accessed: 19 April 2025).
- 7) *Watch live: Aerial views show damage in Turkey after earthquakes* (2023) *The Independent*. Available at: <https://www.independent.co.uk/news/world/europe/turkey-earthquake-damage-aerial-footage-b2277975.html> (Accessed: 20 April 2025).

2) Structural Analysis & Design of a Small Office Building

2a) Introduction

The project involves the structural analysis for a design of a typical floor layout within a five-storey building. The structural layout consists of a reinforced concrete slab supported by a system of universal steel beams. The objective is to ensure that the selected structural components can safely support all applied loads while complying with relevant design codes and standards.

The floor system is subject to the following characteristic design loads:

- Dead Load - including slab self-weight, steelwork, finishes, ceilings, partitions, services, and fire protection = 6.0 kN/m^2
- Imposed Load (live load) = 4.0 kN/m^2

The total uniformly distributed load acting on the floor is therefore 10.0 kN/m^2 .

All structural steel used will be designed to grade S275 steel. The design will be done in accordance with British Standards 5950.

The purpose of the analysis is to determine the size and arrangement of universal beams required to support the proposed reinforced concrete slab, while aiming for an economical design that minimizes overall project costs and avoids unnecessary oversizing.

The proposed office layout from the architect is shown below:-

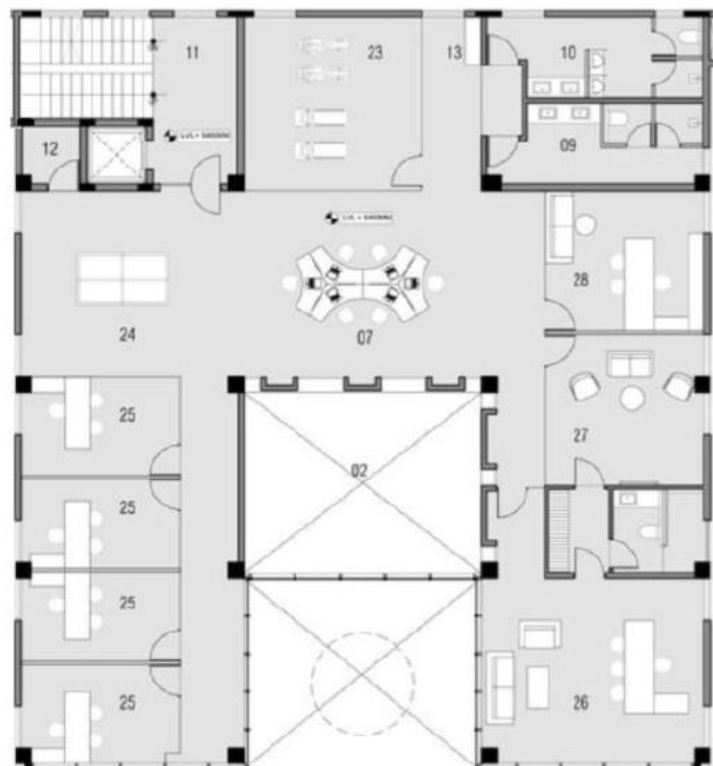


Image 1 - Proposed layout at each level

2b) Proposed Design Plans of Office Block

Confirmed member sizes (Refer to section 2c - 2h for analysis):-

Member A = Universal Beam - 254x102x28

Member B = Universal Beam - 305x217x37

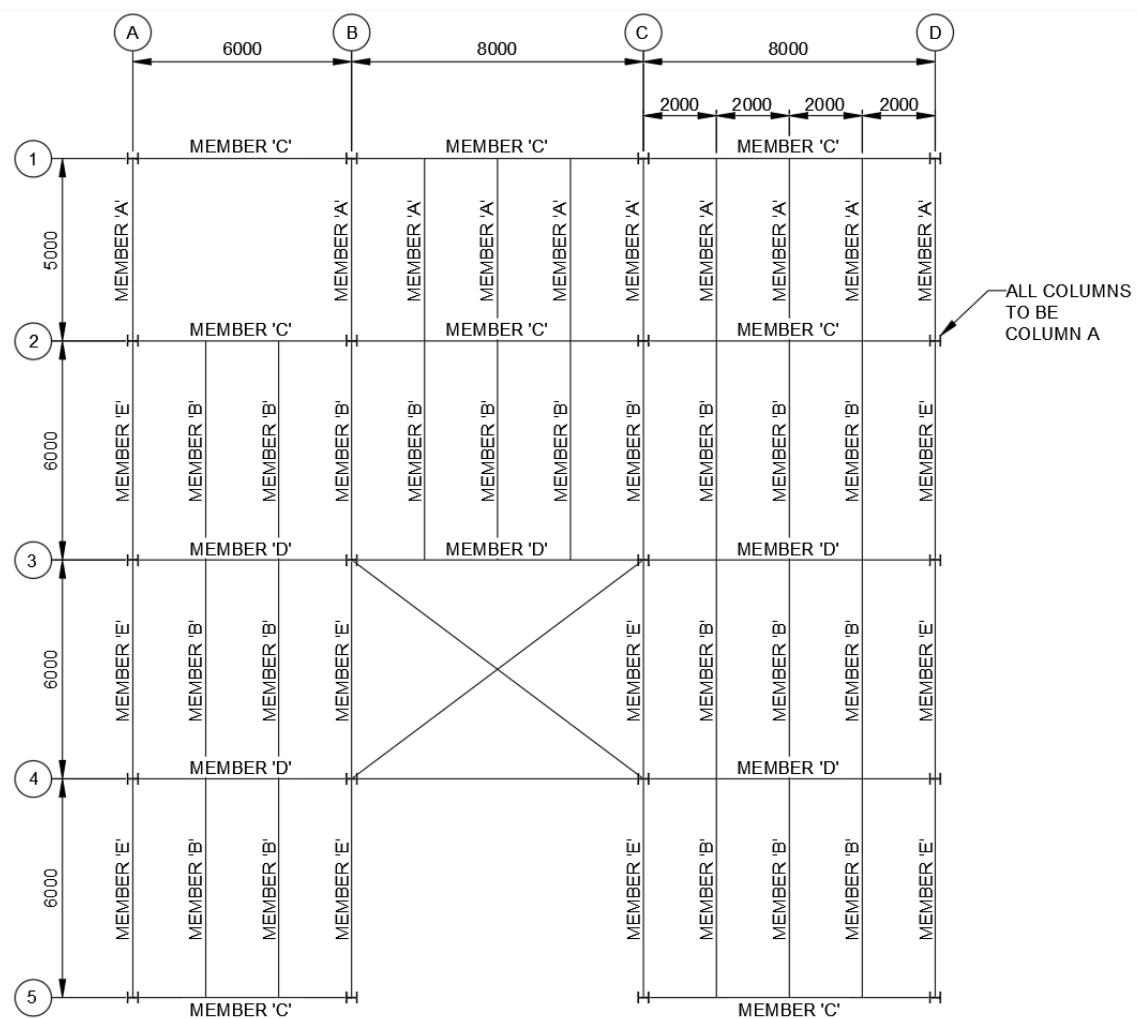
Member C = Universal Beam - 610x305x179

Member D = Universal Beam - 610x305x179

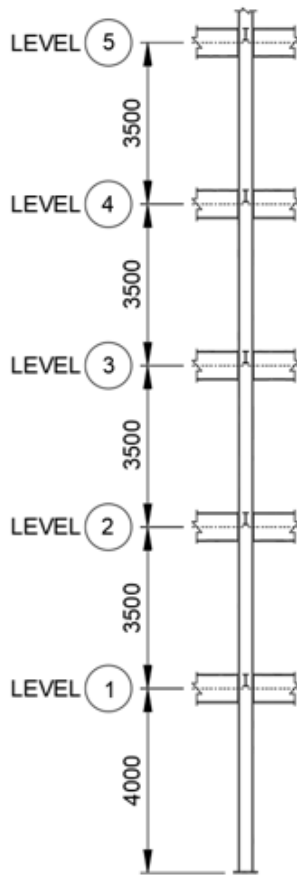
Member E = Universal Beam - 254x102x22

Column A = Universal Column - 305x305

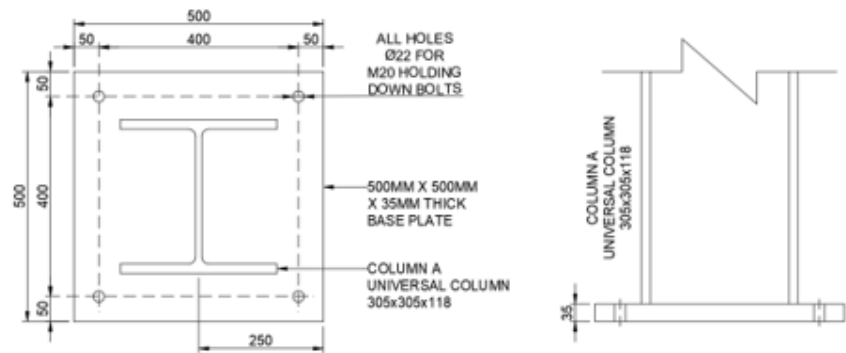
Member layout on plan at Level 1 / 2 / 3 / 4 / 5



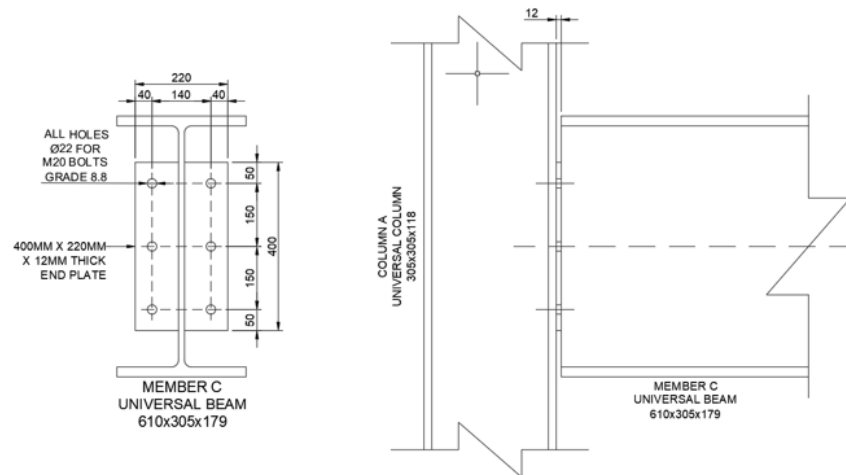
Section View of Column A



Base Plate – Details

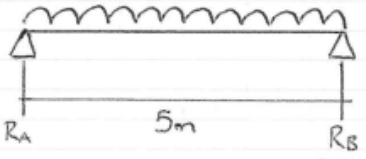


Connection A – Details

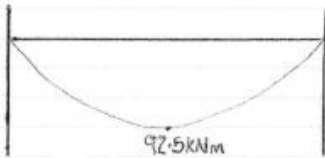
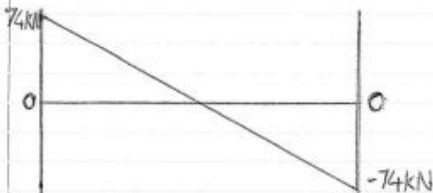


2c) Structural Analysis of Member A

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member A	Sheet	1 of 5

Reference	Calculation	Output
	Load acting on Secondary Member :- Dead Load (g_k) = 6 kN/m^2 Imposed Load (q_k) = 4 kN/m^2 Area acting on Secondary Member = $5 \text{ m} \times 2 \text{ m} = 10 \text{ m}^2$ Total Dead Load (g_k) = $6 \text{ kN/m}^2 \times 2 \text{ m}$ $= 12 \text{ kN/m}$ $g_k = 12 \text{ kN/m}$ Total Imposed Load (q_k) = $4 \text{ kN/m}^2 \times 2 \text{ m}$ $= 8 \text{ kN/m}$ $q_k = 8 \text{ kN/m}$  Calculate Ultimate Load (w) $w = 1.4 g_k + 1.6 q_k$ $w = 1.4 \times 12 \text{ kN/m} + 1.6 \times 8 \text{ kN/m}$ $w = 16.8 \text{ kN/m} + 12.8 \text{ kN/m}$ $w = 29.6 \text{ kN/m}$ $w = 29.6 \text{ kN/m}$	

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member A	Sheet	2 of 5

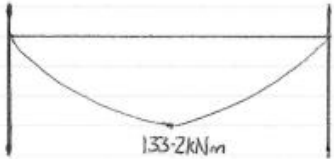
Reference	Calculation	Output
	Design of Moment & Shear $M = \frac{wl^2}{8} = \frac{29.6 \text{ kN/m} \times 5 \text{ m}^2}{8} = \frac{740}{8}$ $M = 92.5 \text{ kNm}$  Maximum Shear at R_A & R_B $R_A = R_B = \frac{wl}{2} = \frac{29.6 \text{ kN/m} \times 5 \text{ m}}{2} = \frac{148}{2}$ $R_A = R_B = 74 \text{ kN}$  Initial Member Selection Steel Grade (p_y) = 275 N/mm^2 $M_c = p_y S_x \Rightarrow S_x \geq \frac{M}{p_y} = S_x \geq \frac{92.5 \text{ kNm}}{275 \text{ N/mm}^2}$ $S_x \geq \frac{92.5 \times 10^6}{275} = S_x \geq 336,363.63 \text{ mm}^3$ $S_x \geq 336.36 \text{ cm}^3$	$M = 92.5 \text{ kNm}$ $R_A = R_B = 74 \text{ kN}$

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member A	Sheet	3 of 5

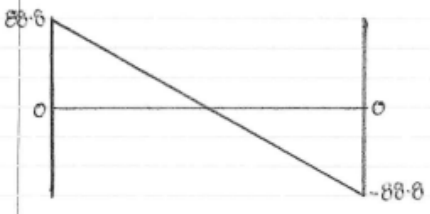
Reference	Calculation	Output																								
BS EN 10025	Initial Member Selection																									
	<table border="1"> <thead> <tr> <th></th> <th>S_x</th> <th>$\frac{b}{T}$</th> <th>$\frac{d}{T}$</th> <th>T</th> <th></th> </tr> </thead> <tbody> <tr> <td>UB 254 x 102 x 28</td> <td>353 cm³</td> <td>4.04</td> <td>35.7</td> <td>10.0 mm</td> <td>Appendix C</td> </tr> <tr> <td>UB 254 x 146 x 31</td> <td>393 cm³</td> <td>7.26</td> <td>36.5</td> <td>8.6 mm</td> <td>Appendix B</td> </tr> <tr> <td>UB 305 x 102 x 28</td> <td>403 cm³</td> <td>4.58</td> <td>46.0</td> <td>7.8 mm</td> <td>Appendix A</td> </tr> </tbody> </table>		S_x	$\frac{b}{T}$	$\frac{d}{T}$	T		UB 254 x 102 x 28	353 cm ³	4.04	35.7	10.0 mm	Appendix C	UB 254 x 146 x 31	393 cm ³	7.26	36.5	8.6 mm	Appendix B	UB 305 x 102 x 28	403 cm ³	4.58	46.0	7.8 mm	Appendix A	
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UB 254 x 146 x 31	393 cm ³	7.26	36.5	8.6 mm	Appendix B																					
UB 305 x 102 x 28	403 cm ³	4.58	46.0	7.8 mm	Appendix A																					
	Member Selected = UB 254 x 102 x 28																									
	Strength Classification																									
Table 9 BS 5950	T = 10 mm (< 16 mm, so $p_y = 275$ N/mm ²)	$p_y = 275$ N/mm ²																								
Table 11 BS 5950	$\epsilon = \sqrt{\frac{275}{p_y}} = \epsilon = \sqrt{\frac{275}{275}} = \epsilon = 1$	$\epsilon = 1$																								
	Section Classification																									
Table 11 BS 5950	$\frac{b}{T} = 4.04 < 9\epsilon = 9 = \text{flange is Plastic}$																									
	$\frac{d}{T} = 35.7 < 80\epsilon = 80 = \text{Web is Plastic}$																									
	UB 254 x 102 x 28 Section is Class 1 Plastic																									
	Shear Strength																									
	As $\frac{d}{T} = 35.7 < 70\epsilon = 70$, Shear Buckling is not considered																									
	Shear Capacity of Section (P_v)																									
	$P_v = 0.6 p_y A_v = 0.6 p_y t D \quad P_v = 0.6 \times 275 \times 6.3 \times 260.4$																									
	$P_v = 270.7 \times 10^3 \text{ N} = P_v = 270.7 \text{ kN}$	$P_v = 270.7 \text{ kN}$																								

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member A	Sheet	4 of 5
Reference	Calculation			Output
	Check $F_v < 0.6 P_v$ $74\text{kN} < 0.6 \times 270.7$ $74\text{kN} < 162.42\text{kN} \quad = \text{Low Shear Load}$ Bending Moment Moment Capacity of Section (M_c) $M_c = P_y S_x = 275 \times 353 \times 10^3 = M_c = 97.1 \times 10^6 \text{ N.mm}$ $M_c = 97.1\text{kNm}$ Extra Moment due to Self Weight (M_{sw}) $M_{sw} = \frac{1.4 (28 \times 10)^2}{8} \quad M_{sw} = 1.23\text{kNm}$ Total Moment (M_t) $M_t = M + M_{sw} = M_t = 92.5 + 1.23$ $M_t = 93.73\text{kNm}$ Check $M_t < M_c$ $93.73 < 97.1 \quad = \text{OK}$ Deflection Check (Δ_{max}) $\Delta_{max} = \frac{5WL^3}{384EI}$ $W = wL = 8\text{kN/m} \times 5\text{m} = 40\text{kN}$ $E = 205\text{kN/mm}^2 = 205 \times 10^6 \text{ kN/m}^2$ $I(I_x) = 4000\text{cm}^4 = 40 \times 10^{-6} \text{ m}^4$ $\Delta_{max} = \frac{5 \times 40 \times 5^3}{384 \times 205 \times 10^6 \times 40 \times 10^{-6}} = \Delta_{max} = 0.00794\text{m}$ $\Delta_{max} = 7.94\text{mm}$			$M_c = 97.1\text{kNm}$ $M_{sw} = 1.23\text{kNm}$ $M_t = 93.73\text{kNm}$ $\Delta_{max} = 7.94\text{mm}$

2d) Structural Analysis of Member B

	Project		Name	Date
	InSite Project No.		By	T M
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member B	Sheet	1 of 4
Reference	Calculation		Output	
	Load acting on Secondary Member:- Area acting on Member:- $6m \times 2m$ $= 12m^2$ Total Dead Load (g_k) = $6kN/m^2 \times 2m$ $= 12kN/m$ Total Imposed Load (q_k) = $4kN/m^2 \times 2m$ $= 8kN/m$ Ultimate Load (w) = $1.4g_k + 1.6q_k$ $= 1.4 \times 12 + 1.6 \times 8$ $= 29.6kN/m$		 $g_k = 12kN/m$ $q_k = 8kN/m$ $w = 29.6kN/m$	
	Design Moment & Shear Maximum Moment at Midspan (M) $M = \frac{wl^2}{8} = M = \frac{29.6kN/m \times 6m^2}{8} = M = \frac{1065.6}{8}$ $M = 133.2kNm$		$M = 133.2kNm$	
				
	Maximum Shear at R_A & R_B $R_A = R_B = \frac{wl}{2} = \frac{29.6 \times 6}{2} = \frac{177.6}{2}$ $R_A = R_B = 88.8kN$		$R_A = R_B = 88.8kN$	

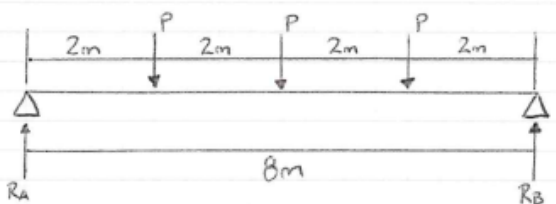
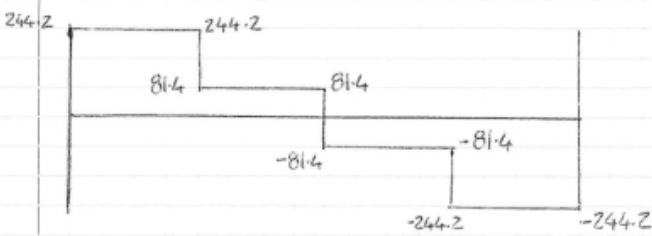
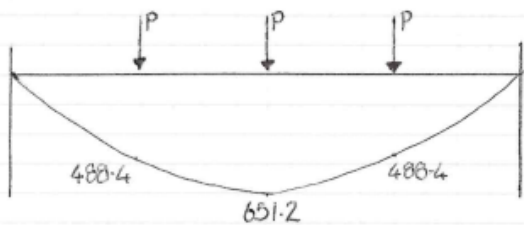
	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member B	Sheet	2 of 4

Reference	Calculation	Output																								
	 Initial Member Selection Steel Grade (p_y) = 275 N/mm² $M_c = p_y S_x \Rightarrow S_x \geq \frac{M}{p_y} = S_x \geq \frac{133.2 \text{ kN}}{275 \text{ N/mm}^2}$ $S_x \geq \frac{133.2 \times 10^6}{275} = S_x \geq 484.36 \text{ cm}^3$ Initial Member Selection <table border="1"> <thead> <tr> <th></th> <th>S_x</th> <th>$\frac{b}{T}$</th> <th>$\frac{d}{T}$</th> <th>T</th> <th></th> </tr> </thead> <tbody> <tr> <td>UB 254x146x43</td> <td>566</td> <td>4.92</td> <td>30.4</td> <td>12.7</td> <td>Appendix E</td> </tr> <tr> <td>UB 305x127x37</td> <td>539</td> <td>4.60</td> <td>37.4</td> <td>10.7</td> <td>Selected</td> </tr> <tr> <td>UB 305x165x40</td> <td>623</td> <td>6.92</td> <td>44.2</td> <td>10.2</td> <td>Appendix F - Check Appendix D</td> </tr> </tbody> </table> Strength Classification Table 6 BS 5950 $T = 10.7 \text{ mm} (< 16 \text{ mm, so } p_y = 275 \text{ N/mm}^2) = E = 1$ $E = 1$ Section Classification Table 11 BS 5950 $\frac{b}{T} = 4.60 < 9E = 9 = \text{flange is Plastic}$ $\frac{d}{T} = 37.4 < 80E = 80 = \text{Web is Plastic}$ UB 305x127x37 is Class 1 Plastic Shear Strength As $\frac{d}{T} = 37.4 < 70E = 70$, Shear Buckling Not Considered		S_x	$\frac{b}{T}$	$\frac{d}{T}$	T		UB 254x146x43	566	4.92	30.4	12.7	Appendix E	UB 305x127x37	539	4.60	37.4	10.7	Selected	UB 305x165x40	623	6.92	44.2	10.2	Appendix F - Check Appendix D	
	S_x	$\frac{b}{T}$	$\frac{d}{T}$	T																						
UB 254x146x43	566	4.92	30.4	12.7	Appendix E																					
UB 305x127x37	539	4.60	37.4	10.7	Selected																					
UB 305x165x40	623	6.92	44.2	10.2	Appendix F - Check Appendix D																					

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member B	Sheet	4 of 4

Reference	Calculation	Output																
Table 8 BS 5950	Calculate Deflection Limit Beams Carrying Plaster = $\text{Span} / 360$ $= 6 \times 10^3 / 360$ $= 16.67 \text{mm}$ Check $\Delta_{\text{max}} < \text{Deflection Limit}$ $9.18 \text{mm} < 16.67 \text{mm} = \text{OK}$ UB 305x127x37 is a Suitable Section <table border="1"> <thead> <tr> <th></th> <th>Design Limit</th> <th>Provided</th> <th>Utilisation %</th> </tr> </thead> <tbody> <tr> <td>Shear</td> <td>88.8kN</td> <td>213.96kN</td> <td>41.50 %</td> </tr> <tr> <td>Bending</td> <td>135.53kNm</td> <td>148.23kNm</td> <td>91.43 %</td> </tr> <tr> <td>Deflection</td> <td>9.18mm</td> <td>16.67mm</td> <td>55.01 %</td> </tr> </tbody> </table>		Design Limit	Provided	Utilisation %	Shear	88.8kN	213.96kN	41.50 %	Bending	135.53kNm	148.23kNm	91.43 %	Deflection	9.18mm	16.67mm	55.01 %	$\delta = 16.67 \text{mm}$
	Design Limit	Provided	Utilisation %															
Shear	88.8kN	213.96kN	41.50 %															
Bending	135.53kNm	148.23kNm	91.43 %															
Deflection	9.18mm	16.67mm	55.01 %															

2e) Structural Analysis of Member C

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member C	Sheet	1 of 4
Reference	Calculation		Output	
	Loads acting on Member C  $P = \text{Reaction from Member A \& Member B}$ $P = 74\text{ kN} + 88.8\text{ kN} = P = 162.8\text{ kN}$ $P = 162.8\text{ kN}$ Design Shear & Moment Maximum Shear $R_A = R_B = \frac{3P}{2} = \frac{3 \times 162.8}{2}$ $R_A = R_B = 244.2\text{ kN}$ $R_A = R_B = 244.2\text{ kN}$  Maximum Moment at Mid-Span (M) $M = R\left(\frac{L}{2}\right) - P\left(\frac{L}{2} - a\right) = M = 244.2\left(\frac{8}{2}\right) - 162.8\left(\frac{8}{2} - 2\right)$ $M = 976.8 - 325.6 = M = 651.2\text{ kNm}$ $M = 651.2\text{ kNm}$ 			

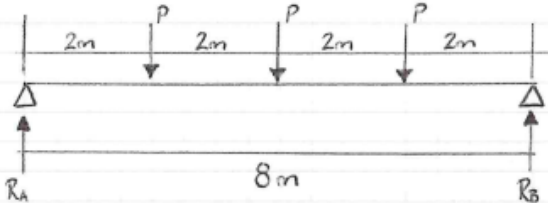
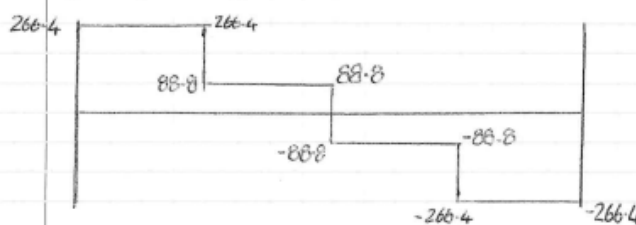
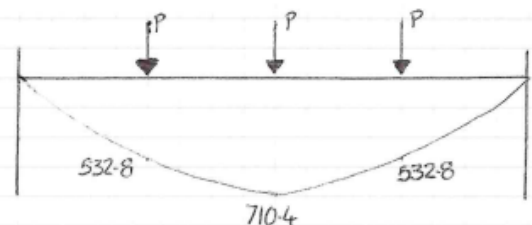
	Project		Name	Date		
	InSite Project No.		By	T.M		
	Client Project No.		Chk'd			
	Document No.		App'd			
	Calculation Title	Member C	Sheet	2 of 4		
Reference	Calculation			Output		
BS EN 10025	Member Selection					
	S_x	T	$\frac{b}{T}$	$\frac{d}{T}$		
	UB 533x312x151	4150	20.3	6.75	37.5	Appendix G
	UB 610x305x149	4590	19.7	6.6	45.8	Appendix H
	UB 533x312x182	5040	24.4	5.61	31.3	Appendix I
	UB 610x305x179	5550	23.6	5.51	38.3	Appendix J - Check
	Member Selected = UB 610x305x179					
	Strength Classification					
Table 9 BS 5950	$T = 23.6$ (16mm - 40mm, so $P_y = 265$)					$P_y = 265 \text{ N/mm}^2$
Table 11 BS 5950	$E = \frac{\sqrt{275}}{P_y} = E = \frac{\sqrt{275}}{265} = E = 1.02$					$E = 1.02$
	Section Classification					
Table 11 BS 5950	$\frac{b}{T} = 5.51 < 9E = 9.17 = \text{flange is Plastic}$					
	$\frac{d}{T} = 38.3 < 80E = 81.6 = \text{Web is Plastic}$					
	UB 610x305x179 is Class 1 Plastic					
	Shear Strength					
	As $\frac{d}{T} = 38.3 < 70E = 71.4$, Shear Buckling Not Considered					
	Shear Capacity of Section (P_v)					
	$P_v = 0.6 P_y t D = P_v = 0.6 \times 265 \times 14.1 \times 620.2$					
	$P_v = 1390.43 \text{ kN}$					$P_v = 1390.43 \text{ kN}$
	Check $F_v < 0.6 P_v$					
	$244.2 \text{ kN} < 834.26 \text{ kN} = \text{Low Shear Load}$					

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member C	Sheet	3 of 4
Reference	Calculation		Output	
Table 13 BS 5950	Both flanges free to rotate on plan $L_E = 1.0 L_T$ $L_E = 8m$ Buckling Resistance ($M_x \leq M_b$) $M_x = M + M_{sw}$ $M_{sw} = \frac{1.4 (179 \times 10)^8^2}{8} = M_{sw} = \frac{160,384}{8}$ $M_{sw} = 20.05 kNm$ $M_x = M + M_{sw} = M_x = 651.2 kNm + 20.05 kNm$ $M_x = 671.25 kNm$		$M_{sw} = 20.05 kNm$ $M_x = 671.25 kNm$	
Table 20 BS 5950	$M_b = p_b S_x$ for Class 1 Plastic p_b is obtained from Table 20, BS 5950: λ is given by = $\sqrt{b_w} (L_E / r_y) = \sqrt{1.0} \times \frac{8 \times 10^3}{7.07 \times 10} = 113.15$ $\frac{D}{T} = \frac{620.2}{23.6} = 26.28$ Using Interpolation from Table 20, BS 5950 (2) 5275, Thickness 16-40) $p_b = 145.17 N/mm^2$ $M_b = p_b S_x = M_b = 145.17 \times 5550 \times 10^3 = M_b = 805.69 \times 10^6$ $M_b = 805.69 kNm$ Check $M_x \leq M_b$ $671.25 \leq 805.69 = OK$		$p_b = 145.17 N/mm^2$	

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member C	Sheet	4 of 4

Reference	Calculation	Output																
	Deflection Check (Δ_{max}) $\Delta_{max} = \frac{5WL^3}{384EI}$ $W = 3P = 3 \times 162.8 = 488.4 \text{ kN}$ $E = 205 \text{ kN/mm}^2 = 205 \times 10^6 \text{ N/m}^2$ $I(x_x) = 153000 \text{ cm}^4 = 1530 \times 10^{-6} \text{ m}^4$ $\Delta_{max} = \frac{5 \times 488.4 \times 8^3}{384 \times 205 \times 10^6 \times 1530 \times 10^{-6}} = \Delta_{max} = 0.0104 \text{ m}$ $\Delta_{max} = 10.4 \text{ mm}$ $\Delta_{max} = 10.4 \text{ mm}$ Calculate Deflection Limit Table 8 BS 5950 Beams carrying plaster = Span / 360 $= 8 \times 10^3 / 360$ $= 22.22 \text{ mm}$ $\delta = 22.22 \text{ mm}$ Check $\Delta_{max} < \text{Deflection Limit}$ $10.4 \text{ mm} < 22.22 \text{ mm}$ UB610x305x179 is a suitable Section																	
	<table border="1"> <thead> <tr> <th></th> <th>Design Limit</th> <th>Provided</th> <th>Utilisation %</th> </tr> </thead> <tbody> <tr> <td>Shear</td> <td>244.2 kN</td> <td>834.26 kN</td> <td>29.27 %</td> </tr> <tr> <td>Bending</td> <td>671.25 kNm</td> <td>805.69 kNm</td> <td>83.31 %</td> </tr> <tr> <td>Deflection</td> <td>10.4 mm</td> <td>22.22 mm</td> <td>46.80 %</td> </tr> </tbody> </table>		Design Limit	Provided	Utilisation %	Shear	244.2 kN	834.26 kN	29.27 %	Bending	671.25 kNm	805.69 kNm	83.31 %	Deflection	10.4 mm	22.22 mm	46.80 %	
	Design Limit	Provided	Utilisation %															
Shear	244.2 kN	834.26 kN	29.27 %															
Bending	671.25 kNm	805.69 kNm	83.31 %															
Deflection	10.4 mm	22.22 mm	46.80 %															

2f) Structural Analysis of Member D

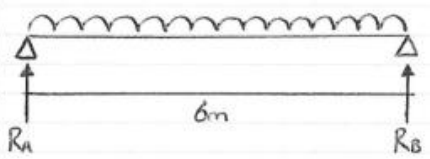
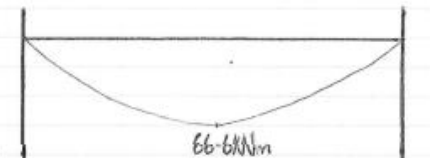
	Project		Name	Date
	InSite Project No.		By	TH
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member D	Sheet	1 of 3
Reference	Calculation		Output	
	Loads acting on Member D  $P = 2 \times \text{Reaction from Member B}$ $P = 2 \times 88.8 \text{ kN} \quad P = 177.6 \text{ kN} \quad P = 177.6 \text{ kN}$ Design Moment & Shear Maximum Shear $R_A = R_B = \frac{3P}{2} = \frac{3 \times 177.6 \text{ kN}}{2}$ $R_A = R_B = 266.4 \text{ kN} \quad R_A = R_B = 266.4 \text{ kN}$  Maximum Moment at Mid-Span (M) $M = R\left(\frac{L}{2}\right) - P\left(\frac{L}{2} - a\right) = M = 266.4 \text{ kN} \left(\frac{8}{2}\right) - 177.6 \left(\frac{8}{2} - 2\right)$ $M = 1065.6 - 355.2 = M = 710.4 \text{ kNm} \quad M = 710.4 \text{ kNm}$ 			

	Project		Name	Date								
	InSite Project No.		By	TM								
	Client Project No.		Chk'd									
	Document No.		App'd									
	Calculation Title	Member D	Sheet	2 of 3								
Reference	Calculation		Output									
Table 9 BS 5950 Table 11	Member Selection											
		<table><tr><th>S_x</th><th>T</th><th>$\frac{b}{T}$</th><th>$\frac{d}{T}$</th></tr><tr><td>5550</td><td>23.6</td><td>5.51</td><td>38.3</td></tr></table>	S_x	T	$\frac{b}{T}$	$\frac{d}{T}$	5550	23.6	5.51	38.3	Appendix K -Check	
	S_x	T	$\frac{b}{T}$	$\frac{d}{T}$								
	5550	23.6	5.51	38.3								
	Member Selected = UB 610x305x179											
	The same member used for Member C, so											
	Strength Classification = $P_y = 265\text{N/mm}^2$		$P_y = 265\text{N/mm}^2$									
	Section Classification = Class 1 Plastic											
	Shear Strength = Not Considered ($<70\epsilon$)											
	Shear Capacity = $P_v = 1390.43\text{kN}$		$P_v = 1390.43\text{kN}$									
Buckling Resistance = $M_b = 805.69\text{kNm}$		$M_b = 805.69\text{kNm}$										
Effective Length the Same as Member C = $L_E = 8\text{m}$		$L_E = 8\text{m}$										
Deflection Limit (δ) = 22mm		$\delta = 22\text{mm}$										
Check Shear $F_v < 0.6P_v$												
$266.4\text{kN} < 834.26\text{kN} = \text{Low Shear Load}$												
Buckling Resistance ($M_x \leq M_b$)												
$M_x = M + M_{sw} = M_x = 710.4 + 20.05 = M_x = 730.45$ $M_x = 730.45\text{kNm}$												
$M_{sw} = \text{Taken from Member C (20.05kNm)}$												
Check $M_x \leq M_b = 730.45\text{kNm} \leq 805.69\text{kNm} = \text{OK}$												

	Project		Name	Date																
	InSite Project No.		By	TM																
	Client Project No.		Chk'd																	
	Document No.		App'd																	
	Calculation Title	Member D	Sheet	3 of 3																
Reference	Calculation			Output																
Deflection Check (Δ_{max}) $\Delta_{max} = \frac{5WL^3}{384EI}$ $W = 3P = 3 \times 177.6 \text{ kN} = 532.8 \text{ kN}$ $E = 205 \times 10^6 \text{ kN/m}$ $I(x) = 1530 \times 10^{-6} \text{ m}^4$ $\Delta_{max} = \frac{5 \times 532.8 \times 8^3}{384 \times 205 \times 10^6 \times 1530 \times 10^{-6}} = \Delta_{max} = 0.01132 \text{ m}$ $\Delta_{max} = 11.32 \text{ mm}$ $\Delta_{max} = 11.32 \text{ mm}$ Check $\Delta_{max} < \text{Deflection Limit } (\delta)$ $11.32 \text{ mm} < 22.22 \text{ mm} = \text{OK}$ UB 610 x 305 x 179 is a suitable section <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th>Design Limit</th> <th>Provided</th> <th>Utilisation %</th> </tr> </thead> <tbody> <tr> <td>Shear</td> <td>266.4 kN</td> <td>834.26 kN</td> <td>31.93 %</td> </tr> <tr> <td>Bending</td> <td>730.45 kNm</td> <td>805.69 kNm</td> <td>90.66 %</td> </tr> <tr> <td>Deflection</td> <td>11.32 mm</td> <td>22.22 mm</td> <td>50.95 %</td> </tr> </tbody> </table>						Design Limit	Provided	Utilisation %	Shear	266.4 kN	834.26 kN	31.93 %	Bending	730.45 kNm	805.69 kNm	90.66 %	Deflection	11.32 mm	22.22 mm	50.95 %
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Shear	266.4 kN	834.26 kN	31.93 %																	
Bending	730.45 kNm	805.69 kNm	90.66 %																	
Deflection	11.32 mm	22.22 mm	50.95 %																	

2g) Structural Analysis of Member E

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member E	Sheet	1 of 4

Reference	Calculation	Output
	Loads acting on Member E Dead Load (g_k) = 6 kN/m^2 Imposed Load (q_k) = 4 kN/m^2 Area acting on Member E = $6 \text{ m} \times 1 \text{ m} = 6 \text{ m}^2$ Total Dead Load (g_k) = $6 \text{ kN/m}^2 \times 1 \text{ m}$ $= 6 \text{ kN/m}$ $g_k = 6 \text{ kN/m}$ Total Imposed Load (q_k) = $4 \text{ kN/m}^2 \times 1 \text{ m}$ $= 4 \text{ kN/m}$ $q_k = 4 \text{ kN/m}$  Calculate Ultimate Load (W) $W = 1.4 g_k + 1.6 q_k$ $W = 1.4 \times 6 + 1.6 \times 4 = W = 8.4 + 6.4$ $W = 14.8 \text{ kN/m}$ $W = 14.8 \text{ kN/m}$ Design of Moment & Shear $M = \frac{w l^2}{8} = M = \frac{14.8 \times 6^2}{8} = M = \frac{532.8}{8}$ $M = 66.6 \text{ kNm}$ $M = 66.6 \text{ kNm}$ 	

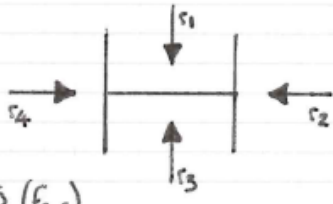
	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member E	Sheet	2 of 4

Reference	Calculation	Output																						
	Maximum Shear at R_A & $R_B = \frac{wl}{2}$ $R_A = R_B = \frac{14.8 \times 6}{2} = R_A = R_B = \frac{88.8}{2}$ $R_A = R_B = 44.4 \text{ kN}$  Initial Member Selection Steel Grade (P_y) = 275 N/mm^2 $M_c = P_y S_x \Rightarrow S_x \geq \frac{M}{P_y} = S_x \geq \frac{66.6 \text{ kNm}}{275 \text{ N/mm}^2}$ $S_x \geq \frac{66.6 \times 10^6}{275} = S_x = 242.18 \text{ cm}^3$ BS EN 10025 Initial Member Selection <table border="1"> <thead> <tr> <th></th> <th>S_x</th> <th>T</th> <th>$\frac{b}{T}$</th> <th>$\frac{d}{T}$</th> <th></th> </tr> </thead> <tbody> <tr> <td>UB 254x102x22</td> <td>259</td> <td>6.8</td> <td>5.93</td> <td>39.5</td> <td rowspan="3">Appendix - N - Check Appendix - M Appendix - L</td> </tr> <tr> <td>UB 203x102x23</td> <td>234</td> <td>9.3</td> <td>4.37</td> <td>31.4</td> </tr> <tr> <td>UB 178x102x19</td> <td>171</td> <td>7.9</td> <td>5.14</td> <td>30.6</td> </tr> </tbody> </table> Member Selected = UB 254x102x22 Strength Classification Table 9 BS 5950 Table 11 $T = 6.8 \text{ mm} (< 16 \text{ mm, so } P_y = 275 \text{ N/mm}^2)$ $E = \frac{\sqrt{275}}{P_y} = E = \frac{\sqrt{275}}{275} = E = 1$ $P_y = 275 \text{ N/mm}^2$ $E = 1$		S_x	T	$\frac{b}{T}$	$\frac{d}{T}$		UB 254x102x22	259	6.8	5.93	39.5	Appendix - N - Check Appendix - M Appendix - L	UB 203x102x23	234	9.3	4.37	31.4	UB 178x102x19	171	7.9	5.14	30.6	
	S_x	T	$\frac{b}{T}$	$\frac{d}{T}$																				
UB 254x102x22	259	6.8	5.93	39.5	Appendix - N - Check Appendix - M Appendix - L																			
UB 203x102x23	234	9.3	4.37	31.4																				
UB 178x102x19	171	7.9	5.14	30.6																				

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Member E	Sheet	3 of 4
Reference	Calculation		Output	
Table 11 BS 5950	Section Classification $\frac{b}{t} = 5.93 < 9\epsilon = 9 = \text{flange is plastic}$ $\frac{d}{t} = 39.5 < 80\epsilon = 80 = \text{Web is plastic}$ UB 254x102x22 is Class 1 Plastic Shear Strength As $\frac{d}{t} = 39.5 < 70\epsilon = 70$, Shear Buckling is not considered Shear Capacity of Section (P_v) $P_v = 0.6 p_y t D = P_v = 0.6 \times 275 \times 5.7 \times 254$ $P_v = 238.89 \times 10^3 \text{ N/mm} \quad P_v = 238.89 \text{ kN} \quad P_v = 238.89 \text{ kN}$ Check $F_v < 0.6 P_v = 44.4 \text{ kN} < 0.6 \times 238.89 \text{ kN}$ $= \text{OK} \quad = 44.4 \text{ kN} < 143.33 \text{ kN} = \text{OK}$ Bending Moment Moment Capacity of Section (M_c) $M_c = p_y S_x = M_c = 275 \times 259 \times 10^3 = M_c = 71.23 \text{ kNm} \quad M_c = 71.23 \text{ kNm}$ Extra Moment due to Self Weight (M_{sw}) $M_{sw} = \frac{1.4 (22 \times 10) 6^2}{8} = M_{sw} = 1.39 \text{ kNm} \quad M_{sw} = 1.39 \text{ kNm}$ Total Moment (M_t) $M_t = M + M_{sw} = M_t = 66.6 \text{ kNm} + 1.39 \text{ kNm}$ $M_t = 67.99 \text{ kNm} \quad M_t = 67.99 \text{ kNm}$ Check $M_t < M_c$ $67.99 \text{ kNm} < 71.23 \text{ kNm} = \text{OK}$			

	Project		Name	Date																
	InSite Project No.		By	TM																
	Client Project No.		Chk'd																	
	Document No.		App'd																	
	Calculation Title	Member E	Sheet	4 of 4																
Reference	Calculation		Output																	
	Deflection Check (Δ_{max}) $\Delta_{max} = \frac{5WL^3}{384EI}$ $W = wL = 4kN/m \times 6m = 24kN$ $E = 205 \times 10^6 kN/m^2$ $I(x_x) = 2840cm^4 = 28.4 \times 10^{-6} m^4$ $\Delta_{max} = \frac{5 \times 24 \times 6^3}{384 \times 205 \times 10^6 \times 28.4 \times 10^{-6}} = \Delta_{max} = 0.01159$ $\Delta_{max} = 11.6mm$ $\Delta_{max} = 11.6mm$ Calculate Deflection Limit Table 8 BS 5950 Beams Carrying Plaster = $\frac{Span}{360}$ = $\frac{6 \times 10^3}{360}$ = 16.67mm $\delta = 16.67mm$ Check $\Delta_{max} < \text{Deflection Limit } (\delta)$ 11.6mm < 16.67mm = OK UB 254x102x22 is a suitable section <table border="1" style="width: 100%;"> <thead> <tr> <th></th> <th>Design Limit</th> <th>Provided</th> <th>Utilisation %</th> </tr> </thead> <tbody> <tr> <td>Shear</td> <td>44.4kN</td> <td>143.33kN</td> <td>30.98%</td> </tr> <tr> <td>Bending</td> <td>67.99kNm</td> <td>71.23kNm</td> <td>95.45%</td> </tr> <tr> <td>Deflection</td> <td>11.6mm</td> <td>16.67mm</td> <td>69.59%</td> </tr> </tbody> </table>			Design Limit	Provided	Utilisation %	Shear	44.4kN	143.33kN	30.98%	Bending	67.99kNm	71.23kNm	95.45%	Deflection	11.6mm	16.67mm	69.59%		
	Design Limit	Provided	Utilisation %																	
Shear	44.4kN	143.33kN	30.98%																	
Bending	67.99kNm	71.23kNm	95.45%																	
Deflection	11.6mm	16.67mm	69.59%																	

2h) Structural Analysis of Column A

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Column A (Gridline C2)	Sheet	1 of 4
Reference	Calculation			Output
	Loads applied to Column A at Level 1			
	reaction from Member A: $r_1 = 74\text{kN}$			
	" " " $C = r_2 = 244.2\text{kN}$			
	" " " $B = r_3 = 88.8\text{kN}$			
	" " " $C = r_4 = 244.2\text{kN}$			
				
	Axial Load from Level 2 - Level 5 (f_{2-5})			
	$= (r_1 + r_2 + r_3 + r_4) \times 4 =$			
	$= (74 + 244.2 + 88.8 + 244.2) \times 4 = 651.2 \times 4 = 2604.8\text{kN}$			
	Total Axial Load (f_c) = $f_{2-5} + r_1 + r_2 + r_3 + r_4$			
	$f_c = 2604.8\text{kN} + 74 + 244.2 + 88.8 + 244.2$			
	$f_c = 3256\text{kN}$			$f_c = 3256\text{kN}$
	Member Selection			
	UC 254x254x107	Z_y 458	S_x 1480	A_g 136
	UC 305x305x97	479	1590	123
	UC 305x305x118	589	1960	150
				Appendix O
				Appendix P
				← Selected Appendix Q
	Member Selected = 305x305x118 UC			
	Section Classification			
Table 11 BS 5950	$T = 18.7\text{mm} \quad (16\text{mm} < 40\text{mm}, \text{ so } p_y = 265\text{N/mm}^2)$			$p_y = 265\text{N/mm}^2$
Table 11 BS 5950	$E = \sqrt{\frac{275}{p_y}} = E = \sqrt{\frac{275}{265}} = E = 1.02$			$E = 1.02$

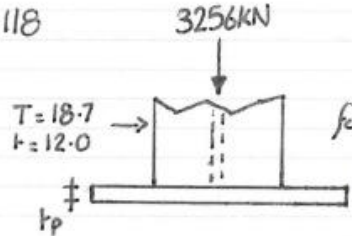
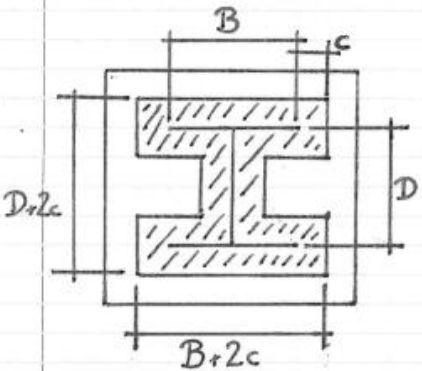
		Project		Name	Date
		InSite Project No.		By	TM
		Client Project No.		Chk'd	
		Document No.		App'd	
		Calculation Title	Column A (Gridline C2)	Sheet	2 of 4
Reference	Calculation			Output	
Table 11 BS 5950	$\frac{b}{T} = 7.09 < 9E = 9.18 = \text{flange is Plastic}$ $\frac{d}{t} = 20.6 < \frac{80E}{1+r_1} = \frac{81.6}{2} = 40.8 = \text{Web is Plastic}$ $r_1 = \frac{f_c}{d \cdot p_{yw}} = r_1 = \frac{3256 \times 10^3}{307.4 \times 12 \times 265} = r_1 = 3.33$ but $-1 < r_1 < 1$, so $r_1 = 3.33 > 1 = r_1 = 1$			$r_1 = 1$	
	UC 305x305x118 is Class 1 Plastic Effective Length (L_E) About x-x axis : $L_{Ex} = 0.85L = L_{Ex} = 0.85 \times 4000$ $L_{Ex} = 3400\text{mm}$			$L_{Ex} = 3400\text{mm}$	
Table 22 BS 5950	About y-y axis : $L_{Ey} = 1L = L_{Ey} = 1 \times 4000$ $L_{Ey} = 4000\text{mm}$			$L_{Ey} = 4000\text{mm}$	
Table 23 BS 5950	Compressive Strength (P_c) x-x axis obtained from table 24(b) y-y axis obtained from table 24(c) Slenderness : $\lambda_x = L_{Ex}/r_x = 3.4 \times 10^3 / 13.6 \times 10$ $\lambda_x = 25$ $\lambda_y = L_{Ey}/r_y = 4 \times 10^3 / 7.77 \times 10$ $\lambda_y = 51.48$				
	Using interpolation from Table 24(b) : $P_{cx} = 254 \text{ N/mm}^2$ Table 24(c) : $P_{cy} = 208.78 \text{ N/mm}^2$				

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Column A (Gridline C2)	Sheet	3 of 4
Reference	Calculation			Output
	Critical Compressive Strength (P_c) = 254 N/mm^2 Check Compression Resistance = $F_c \leq P_c$ ($P_c = A_g P_c$) $P_c = 150 \times 10^2 \times 254 \times 10^{-3} = P_c = 3810 \text{ kN}$ $P_c = 3810 \text{ kN}$ $F_c = 3256 \text{ kN} \leq P_c = 3810 \text{ kN} = \text{OK}$ Nominal Moment due to eccentricity Because the moment is equal on both side of the flanges $M_{2x} = 0$ Total Moment at level 1 from beams connected to the web $M_{2y} = (R_1 - R_3) (l/2 + 100) = (74 - 88.8) (12/2 + 100) \times 10^{-3}$ $M_{2y} = 1.63 \text{ kNm}$ Moment Distribution Stiffness of Column between Level 0 to Level 1 = $EI/4$ Level 1 to Level 2 = $EI/3.5$ ratio of Stiffness Value = $\frac{4}{3.5}$ there fore $\frac{3.5}{7.5}$ of M_{2x} and M_{2y} transfered to Column 0-1 So, $M_{x(0-1)} = \frac{3.5}{7.5} M_{2x} = \frac{3.5}{7.5} \times 0 = 0 \text{ kNm}$ $M_{y(0-1)} = \frac{3.5}{7.5} M_{2y} = \frac{3.5}{7.5} \times 1.63 = M_{y(0-1)} = 0.76 \text{ kNm}$ $M_{y(0-1)} = 0.76 \text{ kNm}$			

	Project		Name	Date
	InSite Project No.		By	T.M
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Column A (Gridline C2)	Sheet	4 of 4
Reference	Calculation		Output	
Table 16 BS 5950	Buckling Resistance			
	$\lambda_{LT} = 0.5 L / r_y = 0.5 \times 4 \times 10^3 / 7.77 \times 10$			
	$\lambda_{LT} = 25.74$			
	$P_b = 265 \text{ N/mm}^2$		$P_b = 265 \text{ N/mm}^2$	
	Buckling Resistance Moment Capacity of Column (M_{bs})			
	$M_{bs} = P_b S_x = M_{bs} = 265 \times 1960 \times 10^{-3}$			
	$M_{bs} = 519.4 \text{ kN.m}$		$M_{bs} = 519.4 \text{ kN.m}$	
	Check Buckling (Combined Axial Force & Moment)			
	$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} \leq 1.0$			
	$\frac{3256}{3810} + \frac{0}{519.4} + \frac{0.76}{265 \times 589 \times 10^{-3}} = 0.85 + 0 + 0.004$			
	$= 0.86 \leq 1 = \text{OK}$			
	UC 305x305x118 in S275 steel is a suitable Section			
		Design Limit	Provided	Utilisation %
	Compressive Resistance	3256 kN	3810 kN	85.46 %
	Buckling Resistance	0.86	1	86 %
	Column Flange Thicker than Connection Plate so assumed Sufficient.			

2i) Structural Analysis of Base Plate - Column A

	Project		Name	Date
	InSite Project No.		By	T/M
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Base Plate - 305x305x118 - Column A	Sheet	1 of 3

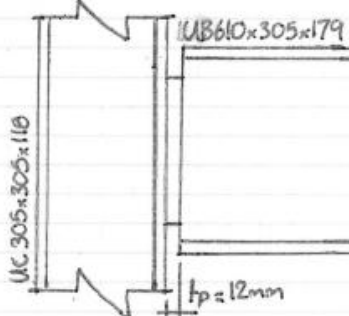
Reference	Calculation	Output
	Column Size = 305x305x118 Compression Characteristic Strength = $f_{cu} = 30 \text{ N/mm}^2$ Steel Grade = 5275 Effective Area, $A_{eff} \geq \frac{\text{Axial Load}}{\text{bearing Strength}}$ $A_{eff} = \frac{3256 \times 10^3}{0.6 \times 30}$ $A_{eff} = 1.81 \times 10^5 \text{ mm}^2$  $f_{cu} = 30 \text{ N/mm}^2$  $D = 314.5 \text{ mm}$ $B = 307.4 \text{ mm}$ $t = 12.0 \text{ mm}$ $T = 18.7 \text{ mm}$ $A_{eff} = (D+2c)(B+2c) - 2 \left[\{D-2(c-T)\} \left\{ \frac{B-t}{2} \right\} \right] =$ $A_{eff} = 4c^2 + (2D+4B-2t)c + (2BT+Dt-2Tt) =$ $A_{eff} = 4c^2 + (2 \times 314.5 + 4 \times 307.4 - 2 \times 12.0)c + (2 \times 307.4 \times 18.7 + 314.5 \times 12.0 - 2 \times 18.7 \times 12.0)$ $A_{eff} = 4c^2 + 1,834.6c + 14,821.96$ Substituting $A_{eff} = 1.81 \times 10^5$ $1.81 \times 10^5 = 4c^2 + 1834.6c + 14,821.96 =$ $4c^2 + 1834.6c - 166,178.04 = 0$	$A_{eff} = 1.81 \times 10^5 \text{ mm}^2$

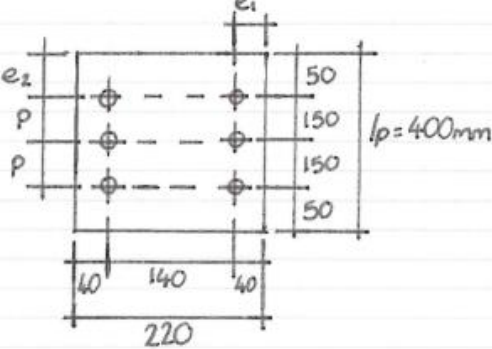
	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Base Plate - 305x305x118 - Column A	Sheet	2 of 3
Reference	Calculation		Output	
	Solve for c = $c = \frac{-b \pm \sqrt{b^2 - 4ad}}{2a}$ a = 4 b = 1834.6 d = -166178.04 $c = \frac{-1834.6 \pm \sqrt{1834.6^2 - 4 \times 4 \times -166178.04}}{2 \times 4}$ $c = \frac{-1834.6 \pm \sqrt{3,363,556 + 2,658,848.64}}{8}$ $c = \frac{-1834.6 + 2454.06}{8}$ $c = \frac{619.46}{8}$ $c = 77.44 \text{ mm}$		c = 77.44 mm	
	Minimum Length of Baseplate = $D + 2c = 314.5 \text{ mm} + 2 \times 77.44$ $L_{\min} = 314.5 + 154.88$ $L_{\min} = 469.38 \text{ mm}$		Lmin = 469.38 mm	
	Minimum Width of Baseplate = $B + 2c = 307.4 + 2 \times 77.44$ $W_{\min} = 307.4 + 154.88$ $W_{\min} = 462.28 \text{ mm}$		Wmin = 462.28 mm	
	Provide a 500 mm x 500 mm baseplate in grade S275		Lprov = 500 mm	
	Check $A_{\text{provided}} > A_{\text{eff}}$ $500 \times 500 > 1.81 \times 10^5$ $2.50 \times 10^5 > 1.81 \times 10^5 = \text{OK}$		Wprov = 500 mm	

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Base Plate - 305x305x118 - Column A	Sheet	3 of 3
Reference	Calculation			Output
	Thickness of Base Plate (t_p) Assume Thickness = $16 < t_p \leq 40\text{mm}$ So design strength (P_{yp}) = 265N/mm^2 $t_p = c \left(\sqrt{\frac{3w}{P_{yp}}} \right) \quad w = 0.6 f_{cu}$ $t_p = 77.44 \left(\sqrt{\frac{3 \times 0.6 \times 30}{265}} \right)$ $t_p = 77.44 \left(\sqrt{\frac{54}{265}} \right)$ $t_p = 77.44 \times 0.45$ $t_p = 34.85\text{mm} \quad \text{Provide Minimum: } 35\text{mm} \quad t_p = 35\text{mm}$ Check: Base Plate thickness (t_p) > Column Flange thickness (T) = $35\text{mm} > 18.7\text{mm}$ Provide a Base Plate 500x500x35mm (Utilisation = 99.57%) Would recommend increasing plate thickness to 40mm to future proof the structure (Utilisation = 87.13%)			

2j) Structural Analysis of Connection A – Member C to Column A

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Connection A	Sheet	1 of 4

Reference	Calculation	Output
	End Plate Connection	$E = 1$
	Steel Grade (P_y) = 275N/mm ² ($= E = 1$)	$P_y = 275\text{N/mm}^2$
	Bolts(d_b) = M20, grade 8.8	$d_b = 20\text{mm}$
	Bolt Hole Diameter (D_h) = 22mm	$D_h = 22\text{mm}$
	Plate Thickness (t_p) = 12mm	$t_p = 12\text{mm}$
	Bolt Spacing	
	Minimum Pitch of Bolt (P_{min}) = $2.5d_b = 2.5 \times 20 = 50\text{mm}$	$P_{min} = 50\text{mm}$
	Maximum Pitch of Bolt (P_{max}) = $14t_p = 14 \times 12 = 168\text{mm}$	$P_{max} = 168\text{mm}$
	Therefore, $P_{min} \leq P \leq P_{max} = 50\text{mm} \leq P \leq 168\text{mm}$	
	Edge / End Distance	
	Minimum Edge/End Distance ($e_1 - e_2 \min$) = $1.4D_h = 1.4 \times 22\text{mm} = 30.8\text{mm}$	$e_1 - e_2 \min = 30.8\text{mm}$
	Maximum Edge/End Distance ($e_1 - e_2 \max$) = $11t_p E = 11 \times 12 \times 1 = 132\text{mm}$	$e_1 - e_2 \max = 132\text{mm}$
	Therefore, $e_1 - e_2 \min \leq e_1 - e_2 \leq e_1 - e_2 \max = 30.8\text{mm} \leq e_1 - e_2 \leq 132\text{mm}$	
	Connection Details	
	Column = UC 305x305x118	
	↳ Backmark = 140mm	
	↳ Flange Thickness (T) = 18.7	
	Beam = UB 610x305x179	
	↳ Depth = 620.2mm	
	↳ Web Thickness (t_w) = 14.1mm	

	Project		Name	Date
	InSite Project No.		By	TM
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Connection A	Sheet	2 of 4
Reference	Calculation		Output	
	Proposed Connection 		$P = 150\text{mm}$ $e_1 = 40\text{mm}$ $e_2 = 50\text{mm}$ $l_p = 400\text{mm}$	
	Bolt Spacing = $P_{\min} \leq P \leq P_{\max} = 50\text{mm} \leq 150\text{mm} \leq 168\text{mm}$ = OK			
	Edge/End Distance = $e_1 - e_2 \min \leq e_1 - e_2 \leq e_1 - e_2 \max$ $= 30.8\text{mm} \leq 40 \& 50 \leq 132\text{mm}$ = OK			
	Bolt Group Strength			
	Shear Capacity of Bolt Group			
	6no. 20mm, grade 8.8 in Single Shear			
	From Bolt Capacities table			
	Single - Shear Value for 20mm diameter (P_s) = 91.9kN		$P_s = 91.9\text{kN}$	
	Shear Capacity of bolt group = $6P_s = 6 \times 91.9\text{kN}$ = 551.4kN		Shear Capacity = 551.4kN	
	Bearing Capacity			
	From Bolt Capacities table			
	For 20mm bolt and 12mm Plate (Minimum of P_{bb} & P_{bs}) = 110kN			
	Bearing Capacity of Connection = $6P_{bs} = 6 \times 110\text{kN}$ = 660kN		Bearing Capacity = 660kN	

	Project		Name	Date
	InSite Project No.		By	T.M
	Client Project No.		Chk'd	
	Document No.		App'd	
	Calculation Title	Connection A	Sheet	4 of 4
Reference	Calculation		Output	
	Shear Strength of Beam Web at End Plate (P_{vb}) = $0.6 P_g A_v$ Where $A_v = 0.9 t_w l_p = A_v = 0.9 \times 14.1 \times 400$ $A_v = 5076 \text{ mm}^2$ $A_v = 5076 \text{ mm}^2$ $P_{vb} = 0.6 \times 275 \times 5076 = P_{vb} = 837,450 \text{ N}$ $P_{vb} = 837.45 \text{ kN}$ Shear Strength of Beam Web = 837.45 kN			
	Summary of Results			
	Shear Capacity of Bolt Group	551.4 kN		
	Bearing Capacity of Connection	660.0 kN		
	Shear Capacity of End Plate assuming two failure Planes	1190.4 kN		
	Shear Strength of Weld	785.3 kN		
	Shear Strength of beam web at the end plate	837.45 kN		
	Strength of Connection is controlled by the Shear Capacity of Bolt Group which is 551.4 kN Check Connection Capacity > Shear from Member D 551.4 kN > 266.4 kN Connection is suitable with a Utilisation of 48.31%			

3) Appendix A – Member A – UB254x146x31

Member Size

305x102x

28

D =	308.7	mm
T =	8.8	mm
t =	6	mm
b/T =	4.58	
d/t =	46	
ry (Radius of Gyration) =	2.08	
Sx (Plastic Modulus) =	403	cm ³
Ix (Second Moment of Area) =	53.7	m ⁴
		5370 cm ⁴
Beam Length =	5	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	148	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 148 kNm

Point Load = 0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb = 74.00 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 92.5 kN

Bending Moment for Point Load = 0 kN

Total Bending Moment (M) = 92.5 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon = 1.00$

b/T < 9 $\epsilon = 4.58 < 9.00$

d/t < 80 $\epsilon = 46 < 80.00$

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon = 46 < 70.00$

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D$$

$$305.61 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v =$$

$$74.00 \text{ kN}$$

$$0.6 P_v =$$

$$183.37 \text{ kN}$$

Low Shear Load

$$\text{Shear Capacity Utilisation} =$$

$$40.36\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x$$

$$110.83 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8$$

$$1.23 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw}$$

$$93.73$$

$$\text{Check } M_t < M_c$$

$$M_t$$

$$93.73 \text{ kN}$$

$$M_c$$

$$110.83 \text{ kN}$$

Section Suitable for
Moment Capacity

$$\text{Moment Capacity Utilisation} =$$

$$84.57\%$$

Deflection Check

$$E =$$

$$205$$

$$\Delta_{max} =$$

$$0.00296 \text{ m}$$

$$\Delta_{max} =$$

$$2.96 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 > \Delta_{max}$$

$$\text{SPAN}/360 =$$

$$13.89 \text{ mm}$$

$$\Delta_{max} =$$

$$3.0 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} =$$

$$21.29\%$$

4) Appendix B – Member A – UB305x102x28

Member Size

254x146x

31

D =	251.4	mm
T =	8.6	mm
t =	6	mm
b/T =	7.26	
d/t =	36.5	
ry (Radius of Gyration) =	3.36	
Sx (Plastic Modulus) =	393	cm ³
Ix (Second Moment of Area) =	44.1	m ⁴
		4410 cm ⁴
Beam Length =	5	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	148	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 148 kNm

Point Load = 0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb = 74.00 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 92.5 kN

Bending Moment for Point Load = 0 kN

Total Bending Moment (M) = 92.5 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\epsilon = 1.00$

b/T < 9 $\epsilon = 7.26 < 9.00$

d/t < 80 $\epsilon = 36.5 < 80.00$

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon = 36.5 < 70.00$

Shear Buckling is not
considered

Shear capacity of Section (Pv)

Pv=0.6 Pyt D 248.89 kN

Check Fv < 0.6Pv

Fv = 74.00 kN
0.6Pv = 149.33 kN

Low Shear Load

Shear Capacity Utilisation = 49.55%

Moment Capacity of Section (Mc)

Mc= pySx 108.08 kNm

Additional Moment to Self Weight (Msw)

Msw = $1.4wl^2/8$ 1.36 kNm

Total Moment (Mt)

Mt = M+Msw 93.86

Check MT < Mc

Mt 93.86 kN
Mc 108.08 kN

Section Suitable for
Moment Capacity

Moment Capacity Utilisation = 86.84%

Deflection Check

E = 205
 Δ_{max} = 0.00360 m
 Δ_{max} = 3.60 mm

Deflection Check = SPAN/360 > Δ_{max}

SPAN/360 = 13.89 mm
 Δ_{max} = 3.6 mm

SELECTION SUITABLE

Deflection Utilisation = 25.93%

5) Appendix C – Member A – UB254x102x28 - Check

Member Size 254x102x **28**

D =	260.4	mm
T =	10	mm
t =	6.3	mm
b/T =	4.04	
d/t =	35.7	
ry (Radius of Gyration) =	2.22	
Sx (Plastic Modulus) =	353	cm ³
Ix (Second Moment of Area) =	40	m ⁴
		4000 cm ⁴
Beam Length =	5	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	148	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 148 kNm
 Point Load = 0.00 kNm
 No. of Point load = **0**
 Total Shear = Ra = Rb = **74.00** kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 92.5 kN
 Bending Moment for Point Load = 0 kN
 Point Load Distance from Mid Point to 1st Point Load
 a = **0**
 Total Bending Moment (M) = **92.5** kN

From Table 11, BS5950 = $\varepsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275 T = 16 - 40MM Py = 265

$\varepsilon =$ 1.00

b/T < 9 ε = 4.04 < 9.00
 d/t < 80 ε = 35.7 < 80.00

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 ε = 35.7 < 70.00

Shear Buckling is not considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 270.69 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 74.00 \text{ kN}$$
$$0.6 P_v = 162.41 \text{ kN}$$

Low Shear Load

$$\text{Shear Capacity Utilisation} = 45.56\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 97.08 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 1.23 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 93.73$$

$$\text{Check } M_T < M_c$$

$$M_t = 93.73 \text{ kN}$$
$$M_c = 97.08 \text{ kN}$$

Section Suitable for
Moment Capacity

$$\text{Moment Capacity Utilisation} = 96.55\%$$

Deflection Check

$$E = 205$$
$$\Delta_{max} = 0.00397 \text{ m}$$
$$\Delta_{max} = 3.97 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 > \Delta_{max}$$

$$\text{SPAN}/360 = 13.89 \text{ mm}$$
$$\Delta_{max} = 4.0 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 28.58\%$$

6) Appendix D – Member B – UB305x165x40

Member Size

305x165x

40

D =	303.4	mm
T =	10.2	mm
t =	6	mm
b/T =	6.92	
d/t =	44.2	
ry (Radius of Gyration) =	12.9	
Sx (Plastic Modulus) =	623	cm ³
Ix (Second Moment of Area) =	85	m ⁴
		8500 cm ⁴
Beam Length =	6	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	177.6	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load =

177.6 kNm

Point Load =

0.00 kNm

No. of Point load =

0

Total Shear = Ra = Rb =

88.80 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load =

133.2 kN

Bending Moment for Point Load =

0 kN

Total Bending Moment (M) =

133.2 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\epsilon =$

1.00

b/T < 9 ϵ =

6.92

<

9.00

Flange is plastic

d/t < 80 ϵ =

44.2

<

80.00

Flange is plastic

Shear Strength

d/t < 70 ϵ =

44.2

<

70.00

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D$$

$$300.37 \text{ kN}$$

Check $F_v < 0.6 P_v$

$$F_v =$$

$$88.80 \text{ kN}$$

$$0.6 P_v =$$

$$180.22 \text{ kN}$$

Low Shear Load

Shear Capacity Utilisation =

$$49.27\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x$$

$$171.33 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8$$

$$2.52 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw}$$

$$135.72$$

Check $M_t < M_c$

$$M_t$$

$$135.72 \text{ kN}$$

$$M_c$$

$$171.33 \text{ kN}$$

Section Suitable for
Moment Capacity

Moment Capacity Utilisation =

$$79.22\%$$

Deflection Check

$$E =$$

$$205$$

$$\Delta_{max} =$$

$$0.00387 \text{ m}$$

$$\Delta_{max} =$$

$$3.87 \text{ mm}$$

Deflection Check = $SPAN/360 > \Delta_{max}$

$$SPAN/360 =$$

$$16.67 \text{ mm}$$

$$\Delta_{max} =$$

$$3.9 \text{ mm}$$

SELECTION SUITABLE

Deflection Utilisation =

$$23.24\%$$

7) Appendix E – Member B – UB254x146x43

Member Size

254x146x43

D =	259.6	mm
T =	12.7	mm
t =	7.2	mm
b/T =	4.92	
d/t =	30.4	
ry (Radius of Gyration) =	10.9	
Sx (Plastic Modulus) =	566	cm ³
Ix (Second Moment of Area) =	65.4	m ⁴
		6540 cm ⁴
Beam Length =	6	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	177.6	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 177.6 kNm

Point Load = 0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb = 88.80 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 133.2 kN

Bending Moment for Point Load = 0 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

Total Bending Moment (M) = 133.2 kN

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\epsilon = 1.00$

b/T < 9 $\epsilon = 4.92 < 9.00$

d/t < 80 $\epsilon = 30.4 < 80.00$

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon = 30.4 < 70.00$

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 308.40 \text{ kN}$$

Check $F_v < 0.6 P_v$

$$F_v = 88.80 \text{ kN}$$
$$0.6 P_v = 185.04 \text{ kN}$$

Low Shear Load

Shear Capacity Utilisation = 47.99%

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 155.65 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 2.71 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 135.91$$

Check $M_T < M_c$

$$M_t = 135.91 \text{ kN}$$
$$M_c = 155.65 \text{ kN}$$

**Section Suitable for
Moment Capacity**

Moment Capacity Utilisation = 87.32%

Deflection Check

$$E = 205$$
$$\Delta_{max} = 0.00503 \text{ m}$$
$$\Delta_{max} = 5.03 \text{ mm}$$

Deflection Check = $SPAN/360 > \Delta_{max}$

$$SPAN/360 = 16.67 \text{ mm}$$
$$\Delta_{max} = 5.0 \text{ mm}$$

SELECTION SUITABLE

Deflection Utilisation = 30.21%

8) Appendix F – Member B – UB305x127x37 - Check

Member Size

305x127x

37

D =	304.4	mm
T =	10.7	mm
t =	7.1	mm
b/T =	4.6	
d/t =	37.4	
ry (Radius of Gyration) =	12.3	
Sx (Plastic Modulus) =	539	cm ³
Ix (Second Moment of Area) =	71.7	m ⁴
		7170 cm ⁴
Beam Length =	6	m
Continuous Load	29.6	kN/m
Each Point Load	0	kN
Total Load on Beam =	177.6	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 177.6 kNm

Point Load = 0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb = 88.80 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 133.2 kN

Bending Moment for Point Load = 0 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

Total Bending Moment (M) = 133.2 kN

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\epsilon = 1.00$

b/T < 9 $\epsilon = 4.6 < 9.00$

d/t < 80 $\epsilon = 37.4 < 80.00$

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon = 37.4 < 70.00$

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 356.60 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 88.80 \text{ kN}$$
$$0.6 P_v = 213.96 \text{ kN}$$

Low Shear Load

$$\text{Shear Capacity Utilisation} = 41.50\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 148.23 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 2.33 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 135.53$$

$$\text{Check } M_t < M_c$$

$$M_t = 135.53 \text{ kN}$$
$$M_c = 148.23 \text{ kN}$$

Section Suitable for
Moment Capacity

$$\text{Moment Capacity Utilisation} = 91.44\%$$

Deflection Check

$$E = 205$$
$$\Delta_{max} = 0.00459 \text{ m}$$
$$\Delta_{max} = 4.59 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 > \Delta_{max}$$

$$\text{SPAN}/360 = 16.67 \text{ mm}$$
$$\Delta_{max} = 4.6 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 27.55\%$$

9) Appendix G - Member C - UB533x312x151

Member Size

533x312x

151

D =	542.5	mm
T =	20.3	mm
t =	12.7	mm
b/T =	6.75	
d/t =	37.5	
ry (Radius of Gyration) =	7.32	
Sx (Plastic Modulus) =	4150	cm ³
Ix (Second Moment of Area) =	1010	101000 cm ⁴
Beam Length	8	m
Continuous Load	0	kN/m
Each Point Load	162.8	kN
Total Load on Beam	488.4	kN

Maximum Shear at Ra & Rb

Continuous load =

0 kNm

Point Load =

244.20 kNm

No. of Point load =

3

Total Shear = Ra = Rb =

244.20 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load =

0 kN

Bending Moment for Point Load =

651.2 kN

Distance from Mid
Point to 1st Point Load

(m)

a =

2

Total Bending Moment =

651.2 kN

From Table 11, BS5950 = $\varepsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\varepsilon =$

1.02

b/T < 9 $\varepsilon =$

6.75

<

9.17

d/t < 80 $\varepsilon =$

37.5

<

81.50

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\varepsilon =$

20.3

<

71.31

Shear Buckling is not considered

Shear capacity of Section (P_v)

$$P_v = 0.6 P_y t D = 1095.47 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 244.20 \text{ kN}$$

$$0.6 P_v = 657.28 \text{ kN}$$

Low Shear Load

$$\text{Shear Utilisation} = 37.15\%$$

Effective Length From Table 13, BS5950 - Both Flanges free to rotate on plan

$$L_E = 1.0 L L T = 8 \text{ m}$$

$$M_{sw} = 16.91 \text{ kNm}$$

$$M_x = 668.11 \text{ kNm}$$

$$M_b = p_b S_x \quad \text{For Plastic Section}$$

p_b obtained from Table 20, BS 5950:-

It is Given by:-

$$\sqrt{B_w (L_E / r_y)} = 109.29$$

$$D/T = 26.72$$

From Table 20, BS5950 (1)

$$P_b = 151.85 \text{ N/mm}^2$$

Interpolate if needed See Next Tab
"Table 20, BS 5950"

$$M_b = p_b S_x = 630.17 \text{ kNm}$$

$$\text{Check } M_x < M_b$$

$$M_x = 668.11$$

$$M_b = 630.17$$

Member Fails Under Lateral
Torsional Buckling

$$\text{Bending Utilisation} = 106.02\%$$

Deflection Check

$$E = 205$$

$$\Delta_{max} = 0.01573 \text{ m}$$

$$\Delta_{max} = 15.73 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 < \Delta_{max}$$

$$\text{SPAN}/360 = 22.22 \text{ mm}$$

$$\Delta_{max} = 15.73 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 70.77\%$$

10) Appendix H - Member C - UB610x305x149

Member Size

610x305x149

D =	612.4	mm
T =	19.7	mm
t =	11.8	mm
b/T =	6.6	
d/t =	45.8	
r_y (Radius of Gyration) =	7	
S_x (Plastic Modulus) =	4590	cm ³
I_x (Second Moment of Area) =	1260	cm ⁴
Beam Length	8	m
Continuous Load	0	kN/m
Each Point Load	162.8	kN
Total Load on Beam	488.4	kN

Maximum Shear at Ra & Rb

Continuous load =

0 kNm

Point Load =

244.20 kNm

No. of Point load =

3

Total Shear = Ra = Rb =

244.20 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load =

0 kN

Bending Moment for Point Load =

651.2 kN

Distance from Mid
Point to 1st Point Load
(m)

a =

2

Total Bending Moment =

651.2 kN

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon =$

1.02

b/T < 9 ϵ =

6.6

<

9.17

d/t < 80 ϵ =

45.8

<

81.50

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 ϵ =

19.7

<

71.31

Shear Buckling is not considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D = 1148.98 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 244.20 \text{ kN}$$

$$0.6 P_v = 689.39 \text{ kN}$$

Low Shear Load

$$\text{Shear Utilisation} = 35.42\%$$

Effective Length From Table 13, BS5950 - Both Flanges free to rotate on plan

$$L_E = 1.0 L T = 8 \text{ m}$$

$$M_{sw} = 16.69 \text{ kNm}$$

$$M_x = 667.89 \text{ kNm}$$

$$M_b = p_b S_x \quad \text{For Plastic Section}$$

p_b obtained from Table 20, BS 5950:-

It is Given by:-

$$\sqrt{b_w (L_E / r_y)} = 114.29$$

$$D/T = 31.09$$

From Table 20, BS5950 (1)

$$p_b = 139.71 \text{ N/mm}^2$$

Interpolate if needed See Next Tab
"Table 20, BS 5950"

$$M_b = p_b S_x = 641.29 \text{ kNm}$$

$$\text{Check } M_x < M_b$$

$$M_x = 667.89$$

$$M_b = 641.29$$

Member Fails Under Lateral
Torsional Buckling

$$\text{Bending Utilisation} = 104.15\%$$

Deflection Check

$$E = 205$$

$$\Delta_{max} = 0.01261 \text{ m}$$

$$\Delta_{max} = 12.61 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 < \Delta_{max}$$

$$\text{SPAN}/360 = 22.22 \text{ mm}$$

$$\Delta_{max} = 12.61 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 56.72\%$$

11) Appendix I - Member C - UB533x312x182

Member Size

533x312x

182

D =	550.7	mm
T =	24.4	mm
t =	15.2	mm
b/T =	5.61	
d/t =	31.3	
ry (Radius of Gyration) =	7.4	
Sx (Plastic Modulus) =	5040	cm ³
Ix (Second Moment of Area) =	1230	
		123000 cm ⁴
Beam Length	8	m
Continuous Load	0	kN/m
Each Point Load	162.8	kN
Total Load on Beam	488.4	kN

Maximum Shear at Ra & Rb

Continuous load = 0 kNm

Point Load = 244.20 kNm

No. of Point load = 3

Total Shear = Ra = Rb = 244.20 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 0 kNm

Bending Moment for Point Load = 651.2 kNm

Distance from Mid
Point to 1st Point Load
(m)

a = 2

Total Bending Moment = 651.2 kNm

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon = 1.02$

b/T < 9 $\epsilon = 5.61 < 9.17$

d/t < 80 $\epsilon = 31.3 < 81.50$

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon = 24.4 < 71.31$

Shear Buckling is not considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D = 1330.93 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 244.20 \text{ kN}$$

$$0.6 P_v = 798.56 \text{ kN}$$

Low Shear Load

$$\text{Shear Utilisation} = 30.58\%$$

Effective Length From Table 13, BS5950 - Both Flanges free to rotate on plan

$$L_E = 1.0 L_T = 8 \text{ m}$$

$$M_{sw} = 20.38 \text{ kNm}$$

$$M_x = 671.58 \text{ kNm}$$

$$M_b = p_b S_x \quad \text{For Plastic Section}$$

p_b obtained from Table 20, BS 5950:- It is Given by:-

$$\sqrt{B_w (L_E/r_y)} = 108.11$$

$$D/T = 22.57$$

From Table 20, BS5950 (1)

$$p_b = 163.43 \text{ N/mm}^2$$

Interpolate if needed See Next Tab
"Table 20, BS 5950"

$$M_b = p_b S_x = 823.70 \text{ kNm}$$

$$\text{Check } M_x < M_b$$

$$M_x = 671.58$$

$$M_b = 823.70$$

Section Suitable

$$\text{Bending Utilisation} = 81.53\%$$

Deflection Check

$$E = 205$$

$$\Delta_{max} = 0.01291 \text{ m}$$

$$\Delta_{max} = 12.91 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 < \Delta_{max}$$

$$\text{SPAN}/360 = 22.22 \text{ mm}$$

$$\Delta_{max} = 12.91 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 58.11\%$$

12) Appendix J - Member C - UB610x305x179 - Check

Member Size

610x305x179

D =	620.2	mm
T =	23.6	mm
t =	14.1	mm
b/T =	5.51	
d/t =	38.3	
ry (Radius of Gyration) =	7.07	
Sx (Plastic Modulus) =	5550	cm ³
Ix (Second Moment of Area) =	1530	153000 cm ⁴
Beam Length	8	m
Continuous Load	0	kN/m
Each Point Load	162.8	kN
Total Load on Beam	488.4	kN

Maximum Shear at Ra & Rb

Continuous load = 0 kNm

Point Load = 244.20 kNm

No. of Point load = 3

Total Shear = Ra = Rb = 244.20 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 0 kN

Bending Moment for Point Load = 651.2 kN

Distance from Mid Point to 1st Point Load (m)

a = 2

Total Bending Moment = 651.2 kN

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon = 1.02$

b/T < 9 ϵ = 5.51 < 9.17

d/t < 80 ϵ = 38.3 < 81.50

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 ϵ = 23.6 < 71.31

Shear Buckling is not considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D = 1390.43 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 244.20 \text{ kN}$$

$$0.6 P_v = 834.26 \text{ kN}$$

Low Shear Load

$$\text{Shear Utilisation} = 29.27\%$$

Effective Length From Table 13, BS5950 - Both Flanges free to rotate on plan

$$L_E = 1.0 L L T = 8 \text{ m}$$

$$M_{sw} = 20.05 \text{ kNm}$$

$$M_x = 671.25 \text{ kNm}$$

$$M_b = p_b S_x \quad \text{For Plastic Section}$$

p_b obtained from Table 20, BS 5950:- It is Given by:-

$$\sqrt{B_w (L_E / r_y)} = 113.15$$

$$D/T = 26.28$$

From Table 20, BS5950 (1)

$$p_b = 148.17 \text{ N/mm}^2$$

Interpolate if needed See Next Tab
"Table 20, BS 5950"

$$M_b = p_b S_x = 822.33 \text{ kNm}$$

$$\text{Check } M_x < M_b$$

$$M_x = 671.25$$

$$M_b = 822.33$$

Section Suitable

$$\text{Bending Utilisation} = 81.63\%$$

Deflection Check

$$E = 205$$

$$\Delta_{max} = 0.01038 \text{ m}$$

$$\Delta_{max} = 10.38 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 < \Delta_{max}$$

$$\text{SPAN}/360 = 22.22 \text{ mm}$$

$$\Delta_{max} = 10.38 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 46.71\%$$

13) Appendix K - Member D - UB610x305x179 - Check

Member Size

610x305x179

D =	620.2	mm	
T =	23.6	mm	
t =	14.1	mm	
b/T =	5.51		
d/t =	38.3		
ry (Radius of Gyration) =	7.07		
Sx (Plastic Modulus) =	5550	cm ³	
Ix (Second Moment of Area) =	1530		153000 cm ⁴
Beam Length	8	m	
Continuous Load	0	kN/m	
Each Point Load	177.6	kN	
Total Load on Beam	532.8	kN	

Maximum Shear at Ra & Rb

Continuous load = 0 kNm

Point Load = 266.40 kNm

No. of Point load = 3

Total Shear = Ra = Rb = 266.40 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 0 kN

Bending Moment for Point Load = 710.4 kN

Distance from Mid Point to 1st Point Load (m)

a = 2

Total Bending Moment = 710.4 kN

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon = 1.02$

b/T < 9 ϵ = 5.51 < 9.17

d/t < 80 ϵ = 38.3 < 81.50

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 ϵ = 23.6 < 71.31

Shear Buckling is not considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D = 1390.43 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 266.40 \text{ kN}$$

$$0.6 P_v = 834.26 \text{ kN}$$

Low Shear Load

$$\text{Shear Utilisation} = 31.93\%$$

Effective Length From Table 13, BS5950 - Both Flanges free to rotate on plan

$$L_E = 1.0 L L T = 8 \text{ m}$$

$$M_{sw} = 20.05 \text{ kNm}$$

$$M_x = 730.45 \text{ kNm}$$

$$M_b = p_b S_x \quad \text{For Plastic Section}$$

p_b obtained from Table 20, BS 5950:- It is Given by:-

$$\sqrt{B_w (L_E / r_y)} = 113.15$$

$$D/T = 26.28$$

From Table 20, BS5950 (1)

$$p_b = 148.17 \text{ N/mm}^2$$

Interpolate if needed See Next Tab
"Table 20, BS 5950"

$$M_b = p_b S_x = 822.33 \text{ kNm}$$

$$\text{Check } M_x < M_b$$

$$M_x = 730.45$$

$$M_b = 822.33$$

Section Suitable

$$\text{Bending Utilisation} = 88.83\%$$

Deflection Check

$$E = 205$$

$$\Delta_{max} = 0.01132 \text{ m}$$

$$\Delta_{max} = 11.32 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 < \Delta_{max}$$

$$\text{SPAN}/360 = 22.22 \text{ mm}$$

$$\Delta_{max} = 11.32 \text{ mm}$$

SELECTION SUITABLE

$$\text{Deflection Utilisation} = 50.96\%$$

14) Appendix L - Member E - UB178x102x19

Member Size

178x102x

19

D =	177.8	mm
T =	7.9	mm
t =	4.8	mm
b/T =	5.14	
d/t =	30.6	
ry (Radius of Gyration) =	7.48	
Sx (Plastic Modulus) =	171	cm ³
Ix (Second Moment of Area) =	13.6	m ⁴
		1360 cm ⁴
Beam Length =	6	m
Continuous Load	14.8	kN/m
Each Point Load	0	kN
Total Load on Beam =	88.8	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load =

88.8 kNm

Point Load =

0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb =

44.40 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load =

66.6 kN

Bending Moment for Point Load =

0 kN

Point Load Distance
from Mid Point to
1st Point Load
a = 0

Total Bending Moment (M) =

66.6 kN

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\epsilon =$

1.00

b/T < 9 $\epsilon =$

5.14

<

9.00

d/t < 80 $\epsilon =$

30.6

<

80.00

Flange is plastic

Flange is plastic

Shear Strength

d/t < 70 $\epsilon =$

30.6

<

70.00

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 140.82 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$F_v = 44.40 \text{ kN}$$
$$0.6 P_v = 84.49 \text{ kN}$$

Low Shear Load

$$\text{Shear Capacity Utilisation} = 52.55\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 47.03 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 1.20 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 67.80$$

$$\text{Check } M_t < M_c$$

$$M_t = 67.80 \text{ kN}$$
$$M_c = 47.03 \text{ kN}$$

Fails In Moment Capacity

$$\text{Moment Capacity Utilisation} = 144.17\%$$

Deflection Check

$$E = 205$$
$$\Delta_{max} = 0.02421 \text{ m}$$
$$\Delta_{max} = 24.21 \text{ mm}$$

$$\text{Deflection Check} = \text{SPAN}/360 > \Delta_{max}$$

$$\text{SPAN}/360 = 16.67 \text{ mm}$$
$$\Delta_{max} = 24.2 \text{ mm}$$

Fails in Deflection

$$\text{Deflection Utilisation} = 145.27\%$$

15) Appendix M - Member E - UB203x102x23

Member Size

203x102x

23

D =	203.2	mm
T =	9.3	mm
t =	5.4	mm
b/T =	4.37	
d/t =	31.4	
ry (Radius of Gyration) =	8.46	
Sx (Plastic Modulus) =	234	cm ³
Ix (Second Moment of Area) =	21	m ⁴
		2100 cm ⁴
Beam Length =	6	m
Continuous Load	14.8	kN/m
Each Point Load	0	kN
Total Load on Beam =	88.8	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load =

88.8 kNm

Point Load =

0.00 kNm

No. of Point load = 0

Total Shear = Ra = Rb =

44.40 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load =

66.6 kN

Bending Moment for Point Load =

0 kN

Point Load Distance
from Mid Point to
1st Point Load

a = 0

Total Bending Moment (M) =

66.6 kN

From Table 11, BS5950 = $\epsilon = \sqrt{\text{Grade}/P_y}$

T = <16MM $P_y = 275$

T = 16 - 40MM $P_y = 265$

$\epsilon =$

1.00

b/T < 9 $\epsilon =$

4.37 < 9.00

Flange is plastic

d/t < 80 $\epsilon =$

31.4 < 80.00

Flange is plastic

Shear Strength

d/t < 70 $\epsilon =$

31.4 < 70.00

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 181.05 \text{ kN}$$

Check $F_v < 0.6 P_v$

$$F_v = 44.40 \text{ kN}$$
$$0.6 P_v = 108.63 \text{ kN}$$

Low Shear Load

Shear Capacity Utilisation = 40.87%

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 64.35 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 1.45 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 68.05$$

Check $M_t < M_c$

$$M_t = 68.05 \text{ kN}$$
$$M_c = 64.35 \text{ kN}$$

Fails In Moment Capacity

Moment Capacity Utilisation = 105.75%

Deflection Check

$$E = 205$$
$$\Delta_{max} = 0.01568 \text{ m}$$
$$\Delta_{max} = 15.68 \text{ mm}$$

Deflection Check = $SPAN/360 > \Delta_{max}$

$$SPAN/360 = 16.67 \text{ mm}$$
$$\Delta_{max} = 15.7 \text{ mm}$$

Selection Suitable

Deflection Utilisation = 94.08%

16) Appendix N - Member E - UB254x102x22 - Check

Member Size

254x102x22

D =	254	mm
T =	6.8	mm
t =	5.7	
b/T =	5.93	
d/t =	39.5	
r_y (Radius of Gyration) =	10.1	
S_x (Plastic Modulus) =	259	cm ³
I_x (Second Moment of Area) =	28.4	m ⁴
		2840 cm ⁴
Beam Length =	6	m
Continuous Load	14.8	kN/m
Each Point Load	0	kN
Total Load on Beam =	88.8	kN
Unfactored Imposed Load	4	kN/m

Maximum Shear at Ra & Rb

Continuous load = 88.8 kNm

Point Load = 0.00 kNm

No. of Point load = 0

Total Shear = $R_a = R_b =$ 44.40 kNm

Maximum Bending Moment at Mid Point

Bending Moment for Continuous Load = 66.6 kNm

Bending Moment for Point Load = 0 kNm

Total Bending Moment (M) = 66.6 kNm

Point Load Distance
from Mid Point to
1st Point Load
a = 0

From Table 11, BS5950 = $\epsilon = \sqrt{Grade/Py}$

T = <16MM $Py = 275$

T = 16 - 40MM $Py = 265$

$\epsilon =$ 1.00

$b/T < 9\epsilon =$ 5.93 < 9.00

$d/t < 80\epsilon =$ 39.5 < 80.00

Flange is plastic

Flange is plastic

Shear Strength

$d/t < 70\epsilon =$ 39.5 < 70.00

Shear Buckling is not
considered

Shear capacity of Section (Pv)

$$P_v = 0.6 P_y t D \quad 238.89 \text{ kN}$$

$$\text{Check } F_v < 0.6 P_v$$

$$\begin{aligned} F_v &= 44.40 \text{ kN} \\ 0.6 P_v &= 143.33 \text{ kN} \end{aligned}$$

Low Shear Load

$$\text{Shear Capacity Utilisation} = 30.98\%$$

Moment Capacity of Section (Mc)

$$M_c = p_y S_x \quad 71.23 \text{ kNm}$$

Additional Moment to Self Weight (Msw)

$$M_{sw} = 1.4 w l^2 / 8 \quad 1.39 \text{ kNm}$$

Total Moment (Mt)

$$M_t = M + M_{sw} \quad 67.99$$

$$\text{Check } M_t < M_c$$

$$\begin{aligned} M_t &= 67.99 \text{ kN} \\ M_c &= 71.23 \text{ kN} \end{aligned}$$

Section Suitable for
Moment Capacity

$$\text{Moment Capacity Utilisation} = 95.45\%$$

Deflection Check

$$\begin{aligned} E &= 205 \\ \Delta_{max} &= 0.01159 \text{ m} \\ \Delta_{max} &= 11.59 \text{ mm} \end{aligned}$$

$$\text{Deflection Check} = \text{SPAN}/360 > \Delta_{max}$$

$$\begin{aligned} \text{SPAN}/360 &= 16.67 \text{ mm} \\ \Delta_{max} &= 11.6 \text{ mm} \end{aligned}$$

Selection Suitable

$$\text{Deflection Utilisation} = 69.56\%$$

17) Appendix O - Column A - UC254x254x107

Member Size

254x254x107

D =	266.7	mm
d =	258.8	mm
T =	20.5	mm
t =	12.8	mm
b/T =	5.38	mm
d/t =	15.6	mm
rx =	11.30	cm
ry =	6.59	cm
Zy (Elastic modulus) =	458	cm ³
Sx (Plastic modulus) =	1480	cm ³
Ag (Area of Section) =	136	cm ²
E =	205	kN/mm ²
Column Length Above	3.5	m
Column Length Below	4	m
Total Reaction on (x-x)	162.8	kN
Total Reaction on (y-y)	488.4	kN
Total Axial Load from above levels (Fh1/h2)	2604.8	kN

Number of Levels Above =	4
Reaction 1 (x-x) =	74
Reaction 2 (y-y) =	244.2
Reaction 3 (x-x) =	88.8
Reaction 4 (y-y) =	244.2

Calculate (Fc) = FH1/H2 + r1 + r2 + r3 + r4

Fc = 3256 kN

From Table 11, BS 5950 = $\varepsilon = \sqrt{275/P_y}$

T = <16MM Py = 275 T = 16 - 40MM Py = 265

$\varepsilon = 1.02$

10 $\varepsilon = 10.19$

b/T < 9 $\varepsilon = 5.38 < 9.17$

d/t < 80 $\varepsilon/1+r1 = 15.6 < 40.75$

Flange is plastic

Flange is plastic

r1 = Fc/ d t Pyw = 3.71 If r1 > 1 then use = 1

Effective Length

From Table 22, BS 5950

About x-x axis: Lex = 0.85L = 3400 mm

About y-y axis: Ley = 1L = 4000 mm

Compressive Strength (Pc)

From Table 23, BS 5950

$$\lambda = L_{ex}/r_x = 30.09 \quad P_{cx} = 248.91 \text{ N/mm}^2$$

$$\lambda = L_{ey}/r_y = 60.70 \quad P_{cy} = 194.95 \text{ N/mm}^2$$

$$\text{Critical Compressive Strength (pc)} = 248.91 \text{ N/mm}^2$$

$$P_c = A_g p_c = 3385.20$$

Check Compressive Resistance = $F_c \leq P_c$

$$F_c \leq P_c \quad 3256 < 3385.20$$

Compressive Resistance is adequate

$$\text{Compressive Resistance Utilisation} = 96.18\%$$

Nominal Moment due to Eccentricity

$$M_{2x} = (R_2 - R_4) (t/2 + 100) = 0.00 \text{ kNm}$$

$$M_{2y} = (R_1 - R_3) (t/2 + 100) = 1.60 \text{ kNm}$$

Moment Distribution

$$M_{x(2-1)} = h_2 / (h_1 + h_2) \times M_{2x} = 0.00 \text{ kNm}$$

$$M_{y(2-1)} = h_2 / (h_1 + h_2) \times M_{2y} = 0.74 \text{ kNm}$$

Buckling Resistance

$$\lambda_{LT} = 0.5L/r_y = 30.35$$

Refer to Table 16, BS 5950

$$P_b = 265.00$$

Buckling resistance moment capacity of column ($M_{bs} = P_b S_x$)

$$M_{bs} = P_b S_x = 392.2$$

Check Buckling (Combined Axial Force & Moment)

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} = 0.97 \leq 1$$

Ok

$$\text{Buckling Utilisation} = 96.80\%$$

18) Appendix P - Column A - UC305x305x97

Member Size

305x305x97

D =	307.9	mm
d =	305.3	mm
T =	15.4	mm
t =	9.9	mm
b/T =	8.6	mm
d/t =	24.9	mm
rx =	13.40	cm
ry =	7.69	cm
Zy (Elastic modulus) =	479	cm ³
Sx (Plastic modulus) =	1590	cm ³
Ag (Area of Section) =	123	cm ²
E =	205	kN/mm ²
Column Length Above	3.5	m
Column Length Below	4	m
Total Reaction on (x-x)	162.8	kN
Total Reaction on (y-y)	488.4	kN
Total Axial Load from above levels (Fh1/h2)	2604.8	kN

Number of Levels	
Above =	4
Reaction 1 (x-x) =	74
Reaction 2 (y-y) =	244.2
Reaction 3 (x-x) =	88.8
Reaction 4 (y-y) =	244.2

Calculate (Fc) = FH1/H2 + r1 + r2 + r3 + r4

Fc = 3256 kN

From Table 11, BS 5950 = $\varepsilon = \sqrt{275/P_y}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\varepsilon = 1.00$

10 $\varepsilon = 10.00$

b/T < 9 $\varepsilon = 8.6 < 9.00$
d/t < 80 $\varepsilon/1+r1 = 24.9 < 40.00$

Flange is plastic

Flange is plastic

r1 = Fc / d t Pyw = 3.92 If r1 > 1 then use = 1

Effective Length

From Table 22, BS 5950

About x-x axis: Lex = 0.85L = 3400 mm
About y-y axis: Ley = 1L = 4000 mm

Compressive Strength (Pc)

From Table 23, BS 5950

$$\lambda = L_{ex}/r_x = 25.37 \quad P_{cx} = 253.63 \text{ N/mm}^2$$

$$\lambda = L_{ey}/r_y = 52.02 \quad P_{cy} = 207.98 \text{ N/mm}^2$$

$$\text{Critical Compressive Strength (pc)} = 253.63 \text{ N/mm}^2$$

$$P_c = A_g p_c = 3119.61$$

Check Compressive Resistance = $F_c \leq P_c$

$$F_c \leq P_c \quad 3256 < 3119.61$$

Fails Under Compressive Resistance

$$\text{Compressive Resistance Utilisation} = 104.37\%$$

Nominal Moment due to Eccentricity

$$M_{2x} = (R_2 - R_4) (t/2 + 100) = 0.00 \text{ kNm}$$

$$M_{2y} = (R_1 - R_3) (t/2 + 100) = 1.66 \text{ kNm}$$

Moment Distribution

$$M_{x(2-1)} = h_2 / (h_1 + h_2) \times M_{2x} = 0.00 \text{ kNm}$$

$$M_{y(2-1)} = h_2 / (h_1 + h_2) \times M_{2y} = 0.78 \text{ kNm}$$

Buckling Resistance

$$\lambda_{LT} = 0.5L/r_y = 26.01$$

Refer to Table 16, BS 5950

$$P_b = 265.00$$

Buckling resistance moment capacity of column ($M_{bs} = P_b S_x$)

$$M_{bs} = P_b S_x = 421.35$$

Check Buckling (Combined Axial Force & Moment)

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} = 1.05 \leq 1$$

Fails Under Buckling from Combined Axial Force & Moment

$$\text{Buckling Utilisation} = 104.96\%$$

19) Appendix Q - Column A - UC305x305x118 - Check

Member Size

305x305x

118

D =	314.5	mm
d =	307.4	mm
T =	18.7	mm
t =	12	mm
b/T =	7.09	mm
d/t =	20.6	mm
rx =	13.60	cm
ry =	7.77	cm
Zy (Elastic modulus) =	589	cm ³
Sx (Plastic modulus) =	1960	cm ³
Ag (Area of Section) =	150	cm ²
E =	205	kN/mm ²
Column Length Above	3.5	m
Column Length Below	4	m
Total Reaction on (x-x)	162.8	kN
Total Reaction on (y-y)	488.4	kN
Total Axial Load from above levels (Fh1/h2)	2604.8	kN

Number of Levels Above =	4
Reaction 1 (x-x) =	74
Reaction 2 (y-y) =	244.2
Reaction 3 (x-x) =	88.8
Reaction 4 (y-y) =	244.2

Calculate (Fc) = FH1/H2 + r1 + r2 + r3 + r4

Fc = 3256 kN

From Table 11, BS 5950 = $\varepsilon = \sqrt{275/P_y}$

T = <16MM Py = 275

T = 16 - 40MM Py = 265

$\varepsilon = 1.02$

10 $\varepsilon = 10.19$

b/T < 9 $\varepsilon = 7.09 < 9.17$

d/t < 80 $\varepsilon/1+r1 = 20.6 < 40.75$

Flange is plastic

Flange is plastic

r1 = Fc / d t Pyw = 3.33 If r1 > 1 then use = 1

Effective Length

From Table 22, BS 5950

About x-x axis: Lex = 0.85L = 3400 mm

About y-y axis: Ley = 1L = 4000 mm

Compressive Strength (Pc)

From Table 23, BS 5950

$$\lambda = L_{ex}/r_x = 25.00 \quad P_{cx} = 254.00 \text{ N/mm}^2$$

$$\lambda = L_{ey}/r_y = 51.48 \quad P_{cy} = 208.78 \text{ N/mm}^2$$

$$\text{Critical Compressive Strength (pc)} = 254.00 \text{ N/mm}^2$$

$$P_c = A_g p_c = 3810.00$$

Check Compressive Resistance = $F_c \leq P_c$

$$F_c \leq P_c \quad 3256 < 3810.00$$

Compressive Resistance is adequate

$$\text{Compressive Resistance Utilisation} = 85.46\%$$

Nominal Moment due to Eccentricity

$$M_{2x} = (R_2 - R_4) (t/2 + 100) = 0.00 \text{ kNm}$$

$$M_{2y} = (R_1 - R_3) (t/2 + 100) = 1.63 \text{ kNm}$$

Moment Distribution

$$M_{x(2-1)} = h_2 / (h_1 + h_2) \times M_{2x} = 0.00 \text{ kNm}$$

$$M_{y(2-1)} = h_2 / (h_1 + h_2) \times M_{2y} = 0.76 \text{ kNm}$$

Buckling Resistance

$$\lambda_{LT} = 0.5L/r_y = 25.74$$

Refer to Table 16, BS 5950

$$P_b = 265.00$$

Buckling resistance moment capacity of column ($M_{bs} = P_b S_x$)

$$M_{bs} = P_b S_x = 519.4$$

Check Buckling (Combined Axial Force & Moment)

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} = 0.86 \leq 1$$

Ok

$$\text{Buckling Utilisation} = 85.95\%$$