MINISTERO
DELLE INFRASTRUTTURE
E DELLA MOBILITÀ SOSTENIBILI

E.N.A.C
ENTE NAZIONALE per
L'AVIAZIONE CIVILE

Committente Principale

Toscana Aeroporti

AEROPORTO INTERNAZIONALE DI FIRENZE – "AMERIGO VESPUCCI"

Opera

MASTERPLAN AEROPORTUALE 2035
INTERVENTI DI ADEGUAMENTO E OTTIMIZZAZIONE DELLE AREE TERMINALI

Titolo Documento

RELAZIONE DI CALCOLO STRUTTURE TERMINAL

Fase di Progetto

PROGETTO DI FATTIBILITÀ TECNICO-ECONOMICA

FASE	REV	DATA EMISSIONE	SCALA	CODICE FILE
PF	00	Ottobre 2022	N/A	FLR-NT-ST-RC-NA-08_Rel Tec Calc Strutture
				NOME FILE Rel Tec Calc Strutture

			Digitally signed by James O'Callaghan Date: 2022.06.20 16:13:50 +01'00'		
00	10/2022	Prima Emissione	J. O'Callaghan	Arch. C. Grouman	Ing. L. Tenerani
REV	DATA	DESCRIZIONE	REDATTO	VERIFICATO	APPROVATO

COMMITTENTE PRINCIPALE	GRUPPO DI PROGETTAZIONE	SUPPORTI SPECIALISTICI	
Toscana Aeroporti **ACCOUNTABLE MANAGER** Dott. Vittorio Fanti	Toscana Aeroporti engineering **DIRETTORE TECNICO** Ing. Lorenzo Tenerani	**SUPPORTO PROGETTAZIONE ARCHITETTONICA** **ACI ENGINEERING S.A.** RAFAEL VIÑOLY ARCHITECTS PC	
	RESPONSABILE INTEGRAZIONE PRESTAZIONI SPECIALISTICHE Ing. Lorenzo Tenerani Ordine degli Ingegneri di Massa Carrara N°631	**SUPPORTO STRUTTURALE** Eckersley O'Callaghan	**SUPPORTO IMPIANTI IDRICI E IDRAULICI** TEKNE
	PROGETTISTA ARCHITETTURA E STRUTTURE Arch. David Perri Ordine degli Architetti di Lucca N°1157	**SUPPORTO IMPIANTI TERMOMECCANICI** me engineers	**SUPPORTO IMPIANTI ELETTRICI E SPECIALI** TECHNION S.r.l.
POST HOLDER DI AREA **POST HOLDER PROGETTAZIONE AD INTERIM** Dott. Vittorio Fanti	**PROGETTISTA SPECIALISTICO IMPIANTI** Ing. Andrea Bonciani Ordine degli Ingegneri di Firenze N°4150	**SUPPORTO IDROLOGIA E IDRAULICA** HYDROGEO INGEGNERIA	**SUPPORTO PROGETTO APRON** M.C.P. Marradi Consulting Partners Infrastructures Engineering, Technology & Consulting
POST HOLDER MANUTENZIONE Ing. Nicola D'Ippolito	**PROGETTISTA SPECIALISTICO APRON** Ing. Lorenzo Tenerani Ordine degli Ingegneri di Massa Carrara N°631	**GEOLOGIA** TERRA & OPERE S.R.L. SOCIETÀ D'INGEGNERIA CONSULENZE GEOLOGICHE	**ARCHEOLOGIA** ARS ARCHEO SISTEMI
POST HOLDER AREA DI MOVIMENTO Geom. Luca Ermini	**PROGETTISTA VIABILITA' E OPERE ESTERNE** Arch. David Perri Ordine degli Architetti di Lucca N°1157	**SUPPORTO SISTEMA BHS** TECNOGECO	**CANTIERIZZAZIONE E SICUREZZA** architecna engineering

Relazione Di Calcolo Strutture Terminal

Indice

Introduzione 6
Sostenibilità 7
Concept della struttura 8
Tetto 8
Sovra-struttura 12
Edifici Laterali 15
Sottostruttura 16
Fondazioni 17
Criteri di progettazione 18
Codici e Regole di progettazione 18
Software e Licenze 18
Carichi 19
Peso proprio 19
Permanenti (non strutturali) 19
Variabili 22
Neve 22
Vento 24
Temperatura 31
Sismica 33
Combinazioni di carico 36
Fattore parziale 36
Fattore di azione variabile di accompagnamento 37
Condizioni del terreno 38

Criteri di prestazione 39
Vita utile di progetto 39
Classe d'uso 39
Materiali 39
Carpenteria metallica 39
Calcestruzzo 41
Tendini di precompressione 41
Fattori parziali 41
Deflessioni 42
Esercizio 42
Sismica 42
Vibrazioni 43
Vibrazioni indotte dal calpestio 43
Considerazioni chiave per il cliente 44
Vibrazioni indotte degli impianti 44
Riassunto del Design Strutturale 45
Piano terra 45
Disposizione e dimensionamento tipici 45
Plinti di fondazione 46
Trave di cemento armato 47
Soletta in cemento armato 48
Colonna in cemento armato 49
Primo piano 52
Disposizione e dimensionamento tipici 52
Trave prefabbricata 53
Travi di sostegno per le travi prefabbricate da ponte 53

Piano intermedio 55
Definizione del Modello 55
Deflessioni 56
Comportamento Vibrazionale 57
Tetto Piano Intermedio 58
Ponte di imbarco 58
Definizione del modello 58
Analisi Dinamica 60
Ponte 1 61
Ponte 2 65
Ponte 3 65
Deflessioni 65
Dimensionamento finale 66
Tetto principale 67
Definizione del modello 67
Presupposti del Modello 67
Costruibilità 71
Giunti di Movimento 72
Casi e Combinazioni di carico 76
Analisi Modale 78
Analisi Sismica 78
Trave Principale 80
Carico Assiale 80
Momento Flettente 80
Deflessioni 81
Trave di trasferimento 82

Carico Assiale 82
Momento Flettente 82
Deflessioni 83
Mega colonna 84
Carico Assiale 84
Momento Flettente 85
Deflessioni 86
Edifici Laterali 87
Edificio Ovest: 87
Primo Piano 87
Tetto 88
Edificio Est: 89
Terminal tetto a guscio 90
Appendice A: Sistema Informativo Geologico del Sottosuolo 91
Appendice B: Gamma di tubi in acciaio 92
Appendice C: Calcoli del piano terra 93
Appendice D: Calcoli del primo piano 94
Appendice E: Calcoli del piano intermedio 95
Appendice F: Calcoli ponte d'imbarco 96
Appendice G: Calcoli del tetto del Terminal a guscio 97
Appendice H: Calcoli del tetto principale 98

Introduzione

Questa relazione descrive il progetto di ingegneria strutturale intrapreso da Eckersley O'Callaghan (EOC) in collaborazione con Rafael Viñoly Architects (RVA) per il progetto dell'aeroporto di Firenze.

Il progetto consiste in un nuovo edificio del terminal dell'aeroporto esistente, con dimensioni in pianta che si estendono per circa 570 m in direzione longitudinale e 265 m in direzione trasversale. L'edificio è formato da un piano terra, una parte minore di primo piano e un tetto tipo "tettoia". Il tetto sostiene un vigneto funzionante e si inclina lievemente dal livello del suolo fino a circa 30m sopra il livello della pista; questo è sostenuto da colonne tipo "albero".

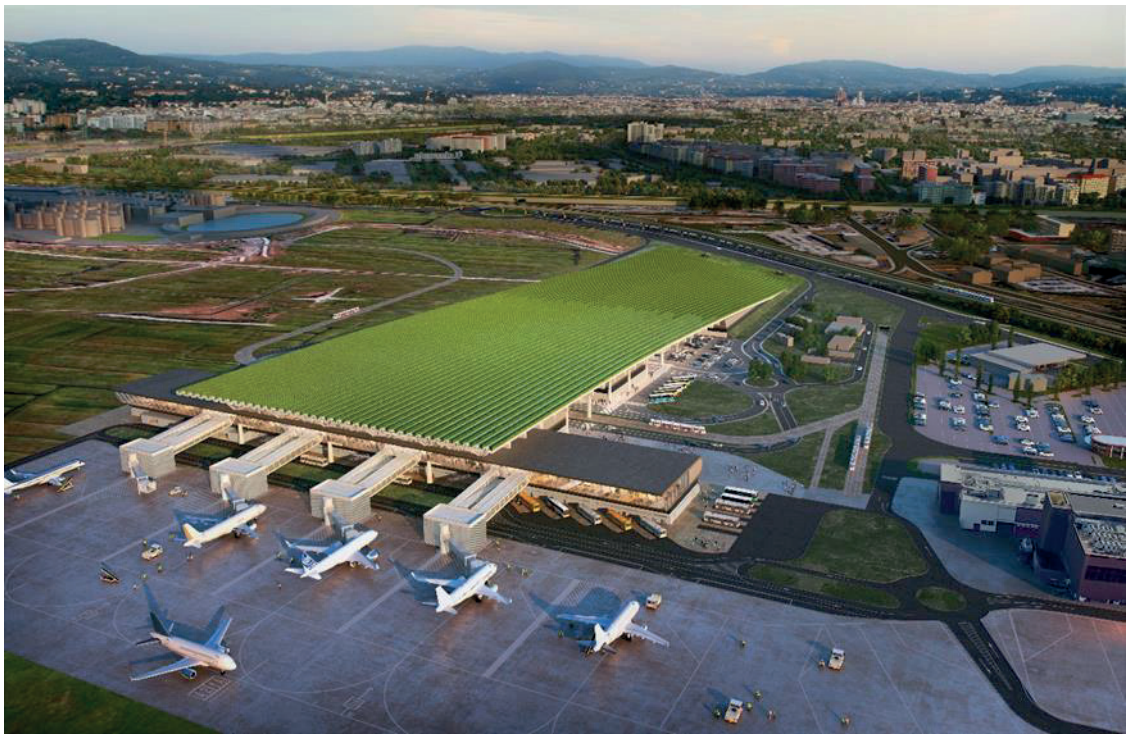


Figura 1: Render architettonico del progetto

Questa relazione deve essere letta insieme alla relazione concettuale di Rafael Viñoly Architects e alla relazione di ingegneria di facciata di EOC. Le ipotesi sul design e sui criteri di prestazione sono fornite con l'obiettivo che il cliente e gli altri membri del team di progettazione possano commentarle e confermarle prima che inizi la fase successiva del progetto.

Alcune informazioni sono state fornite per permettere al cliente di produrre un piano dei costi. Tutte le informazioni devono essere considerate preliminari in questa fase del progetto e a questo scopo dovrebbe essere previsto un design contingente per quanto riguarda il piano dei costi.

Sostenibilità

All'interno dei vincoli del progetto è prevista l'implementazione di strategie di progettazione sostenibile per ridurre il contenuto di carbonio struttura, tra cui:

- ottimizzazione strutturale dettagliata delle membrature, evitando un approccio unico per tutti
- revisione dettagliata dei requisiti di carico, in particolare per la struttura del tetto, per evitare qualsiasi sovra progettazione
- la specificazione del materiale di base con un basso contenuto di carbonio

Per quanto riguarda i materiali, verrà specificato quanto segue:

- carpenteria con più del 60% di contenuto riciclato, prodotta con il metodo del forno elettrico ad arco
- acciaio di rinforzo con un contenuto di acciaio riciclato superiore al 90%, prodotto con il metodo del forno elettrico ad arco
- Il calcestruzzo avrà un contenuto di sostituzione del cemento GGBS (Ground Granulated Blast-furnace Slag- Scorie d'altoforno macinate granulate) tra il 50% e il 70%, o l'uso di calcestruzzo senza cemento (ad esempio Cemfree sviluppato da DB Group, o Earth Friend Concrete sviluppato da Wagners)
- Il calcestruzzo avrà un contenuto di aggregati riciclati maggiore del 20%.

Concept della struttura

Le seguenti sezioni descrivono la progettazione dei tipici elementi strutturali presenti nell'edificio.

Tetto

Il tetto ha le dimensioni di 128 x 276m in pianta e comprende un sistema di colonne ad "albero" che sostengono una serie di travi lineari prefabbricate in cemento della "vigna" e i lucernari intermedi in vetro. Le colonne-albero sono formate da una sezione circolare del tronco e da quattro rami inclinati, ogni ramo è formato da un palo a sezione rettangolare che si assottiglia verso la sommità. I rami sono collegati da dei tiranti in acciaio. Fare riferimento alla Figura 2 per un diagramma del sistema di base.

Gli alberi sono posti secondo una griglia di distanza di 24 m in entrambe le direzioni ortogonali, con la distanza dei rami di 12 m. La sezione della colonna del tronco è alta tra i 3.5 m e i 14.2m ed è formata da una sezione di calcestruzzo prefabbricato, con la sezione ottagonale esterna di cemento di circa 2,5m di diametro massimo che si assottiglia verso la cima. Si prevede che le colonne del tronco saranno prefabbricate per ottenere una finitura in calcestruzzo a vista di alta qualità. Gli elementi dei rami formano una piramide capovolta the sostiene la trave di trasferimento del tetto. La loro sezione è di 700mmx1000mm alla base e 500x700 mm al punto di sostegno della trave. I rami sono legati in direzione longitudinale alla loro sommità da sezioni tubolari in acciaio di circa 244 mm di diametro e sostengono una trave trapezoidale in calcestruzzo armato in direzione trasversale, che agisce anche per legare i rami. Al di sopra di ciò, il tetto è formato da una rete di travi prefabbricate post-tensionate, posizionate ortogonalmente. Le travi primarie del vigneto si estendono in direzione N-S, mentre delle travi cave trapezoidali poste a 12 m c/c trasferiscono il loro carico alla Mega Colonna. Le travi del vigneto sono tipicamente 2.8 m di larghezza per 1.35 m di profondità, mentre le travi trapezoidali sono profonde 2.0m e larghe 700mm alla base e 1200mm alla loro sommità. Per garantire la stabilità strutturale, le travi devono essere continue e rigidamente connesse. Ciò ridurrebbe la quantità di momento flettente residuo, trasferito ai rami delle Mega Colonne, dovuto all'eccentricità tra il vigneto e la sommità della piramide.

È stata eseguita un'analisi della sostenibilità sulla struttura del tetto considerando i seguenti parametri: carico del suolo, distanza tra le colonne e angolo di inclinazione dei puntoni. Prima è stata esaminata l'implicazione della riduzione del diametro e della quantità di materiale dei puntoni diagonali per i rami.

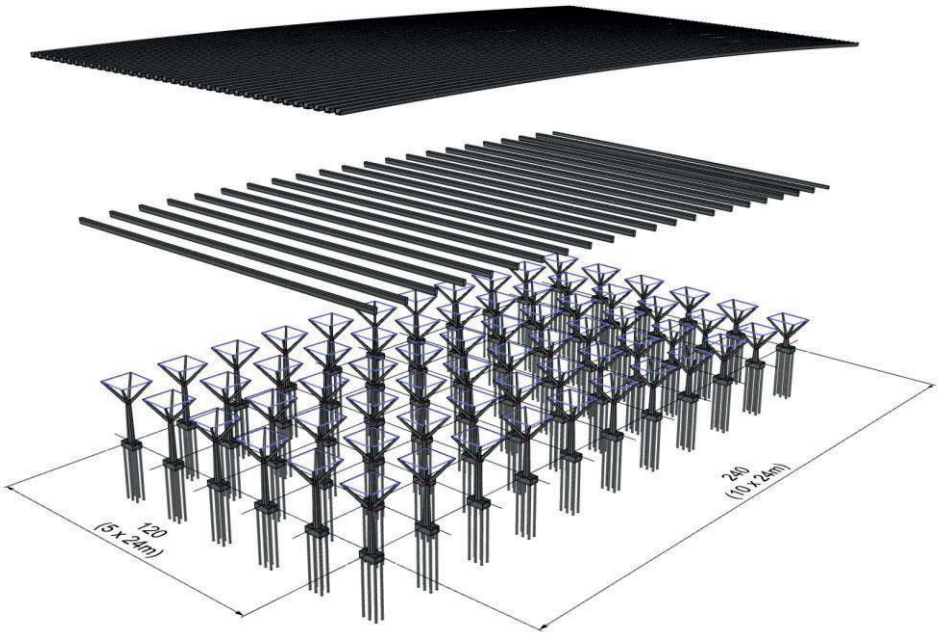
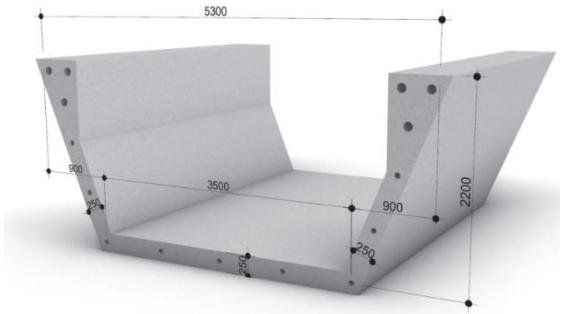


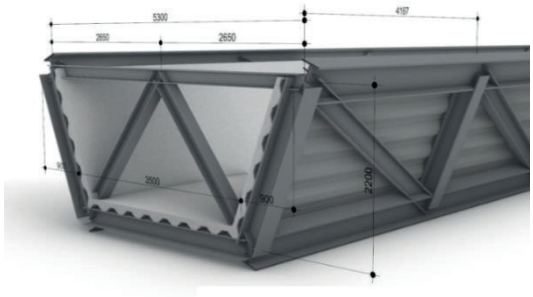
Figura 2:Esploso della struttura del tetto

Figura 3: sistema di base

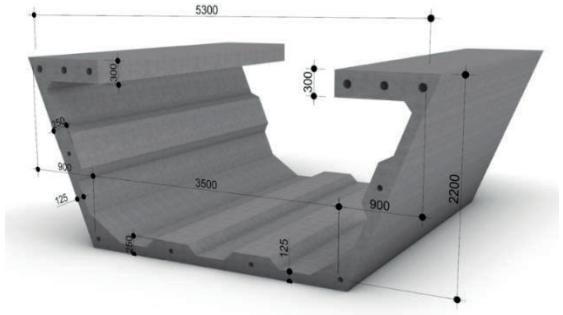
Sono state studiate un certo numero di opzioni per la sua costruzione, tra cui: calcestruzzo prefabbricato; calcestruzzo ad altissime prestazioni (UHPC); composito acciaio e calcestruzzo; e completamente in acciaio. Fare riferimento alla figura 3. Attualmente il calcestruzzo prefabbricato è la soluzione migliore dal punto di vista dei costi, della robustezza e della non necessità di un rivestimento della superficie inferiore esposta.



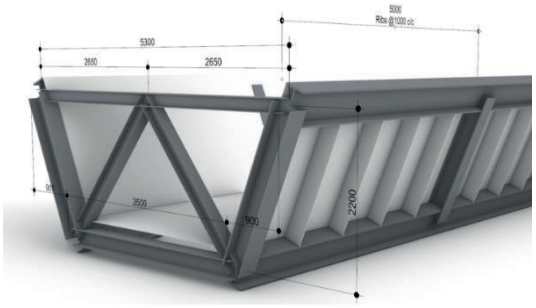
Opzione A: Trave in Calcestruzzo armato Post-tesa



Opzione C: Traliccio in acciaio & pannello metallico



Opzione B: Trave in Calcestruzzo armato Post-teso UHPC



Opzione D: Traliccio Vierendeel in acciaio con piastra di copertura in acciaio

Figura 4: Forme di costruzione considerate per le travi del vigneto

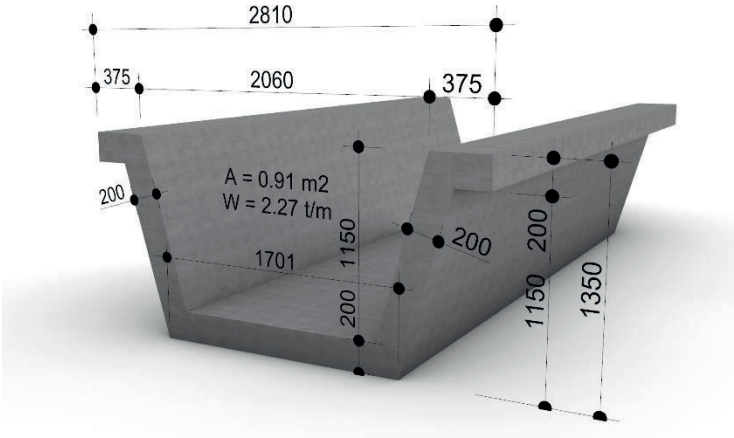


Figura 5: Design corrente della trave del vigneto: Trave in Calcestruzzo Armato (CA) precompressa

Inizialmente le travi del vigneto sono state considerate come una singola sezione di 5.0 metri di larghezza, anche se questo sarebbe molto difficile da trasportare e potrebbe essere richiesto di essere gettato in loco. La posa in opera richiederebbe una grande gru in grado di sollevarle la struttura in acciaio del tetto. Per questo motivo, le dimensioni della trave del vigneto sono state ridotte a 2.80x1.35 m al fine di ridurre il peso proprio e consentire la prefabbricazione. Tipicamente la trave dovrebbe avere delle pareti laterali spesse 200mm e pesare circa 2.3t per metro lineare, quindi circa 27.2t per una sezione lunga 12.0m. Ulteriori misure potrebbero consistere nel dividere la trave in diversi componenti che potrebbero essere assemblati in loco (giunti bagnati in situ).

Il calcestruzzo è una soluzione pesante per le travi del vigneto e quindi aumenta i requisiti della struttura portante e delle fondazioni, sebbene il carico del suolo ed imposto siano anch'essi molto alti, con il peso proprio della trave del vigneto che è solo circa il 20-25% del carico complessivo del tetto. Il tetto è attualmente progettato per sostenere da 300 a massimo 500 mm di profondità del suolo per la coltivazione delle viti. Questo è ovviamente un carico molto alto (circa 2.000 kg/m2) e se fosse possibile ridurlo si potrebbe ottenere un significativo risparmio sui costi. Le opzioni da considerare sono la riduzione della profondità complessiva, la riduzione della larghezza del terreno, avere fioriere discrete evitando così una striscia continua di terreno o una combinazione di tutte queste opzioni.

Le travi prefabbricate saranno gettate in sezioni diritte lunghe 12.0 m che saranno sfaccettate al profilo curvo del tetto. Si prevede che le singole sezioni saranno post-tese insieme formando una sezione continua. Le sezioni prefabbricate saranno imbullonate in loco alla sezione trapezoidale in acciaio tramite sezioni di supporto in acciaio fuso.

Dato che i carichi propri e portati della struttura sono estremamente elevati, anche le deflessioni associate sono molto alte, a questo scopo proponiamo di pre-curvare ed impostare la struttura per compensare queste deflessioni evitando la necessità di irrigidire la struttura per ridurle.

Al fine di mitigare le azioni del carico termico sulla struttura del tetto, i giunti di movimento saranno integrati sia nella vigna che nelle travi di trasferimento. La densità e la disposizione dei giunti di movimento influiscono sul comportamento strutturale, soprattutto in condizioni sismiche; è stato condotto uno studio di sensibilità per valutare le implicazioni di tutte le opzioni considerate.

I risultati di questo studio suggeriscono che le opzioni con un minor numero di giunti di movimento funzionano meglio di altre e, pertanto, la nostra raccomandazione sarà quella di optare per una disposizione con 2 giunti di movimento solo attraverso le travi di vigna, allineate con le linee termiche dell'edificio, lavorando in combinazione con una serie di rilasci ortogonali in cima alle mega colonne per prevenire il trasferimento del momento flettente residuo. Tale disposizione presuppone che, tra i giunti di movimento, le travi di vigna e di trasferimento forniscano un certo livello di azione diaframmatica. Per questo motivo, le connessioni tra le due travi e con le mega colonne dovranno essere esaminate in dettaglio nella fase successiva.

Sovra-struttura

La sovrastruttura è tipicamente progettata in gran parte come prefabbricato in calcestruzzo per consentire una rapida costruzione e raggiungere un elevato standard di qualità. Si prevede che un certo numero di componenti prefabbricati possa essere costituito da elementi standard del catalogo del produttore, e nella fase successiva del progetto si cercheranno fornitori locali. Le travi di trasferimento speciali, come le travi a sbalzo a nord o la trave di trasferimento che sostiene le travi prefabbricate del ponte, includono il post-tensionamento per soddisfare i requisiti di progetto e limitare la loro profondità complessiva. Tali travi sono state progettate come sovrasupporto continuo (continuità del momento flettente) e saranno in situ.

Il pavimento del livello 1 è costituito da travi prefabbricate e in calcestruzzo gettato in opera in entrambe le direzioni ortogonali, con profondità tipica di 1.000 mm e fino a 1.500 mm, con una luce di 12 m tra le colonne. Le travi di trasferimento che sostengono le travi del ponte sulle corsie di traffico, a causa della loro luce (24 m) e della loro maggiore area di affluenza, saranno molto più profonde, da 2,20 a Queste travi sostengono tavole prefabbricate in calcestruzzo precompresso profonde circa 400 mm, che saranno unite da una soletta di copertura in calcestruzzo armato di 50 mm. La parte centrale del livello 1, invece, ha una configurazione particolare, descritta nei paragrafi seguenti.

Le colonne che sostengono solo i piani sono sezioni circolari costruite in calcestruzzo prefabbricato, tipicamente tra 800 e 1.000 mm. Le colonne sono tipicamente a sbalzo per garantire la stabilità complessiva.

La sezione centrale sarà costruita con travi a Y standard prefabbricate e precompresse a catalogo, profonde da 900 a 1.100 mm circa, e una soletta di chiusura di circa 100 mm gettata in opera. Queste saranno sostenute da travi in cemento armato, che si estendono da nord a sud, gettate in opera con una profondità di circa 1.500 mm per la sezione chiusa dell'edificio.

Invece, nella sezione centrale "a ponte", che si estende per 48 metri sopra la strada e i parcheggi, le travi di sostegno che si estendono da est a ovest saranno post-tese. La trave centrale avrà una profondità massima di 2.700 mm - esclusa la soletta di rivestimento - mentre le travi di sostegno laterali centrali avranno una profondità di circa 2.000 mm. Per il collegamento tra le travi dell'impalcato e le travi di sostegno sono state prese in considerazione diverse opzioni, tra cui l'incasso delle prime nelle seconde e l'uso di staffe. È necessario effettuare uno studio più approfondito per determinare l'opzione migliore.

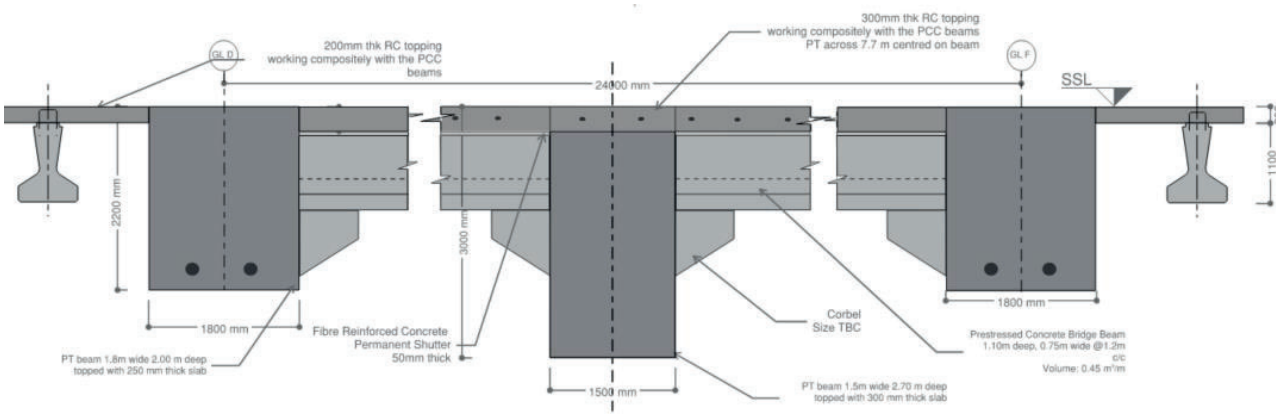


Figura 6: Travi di trasferimento in situ tra le linee di griglia D e F

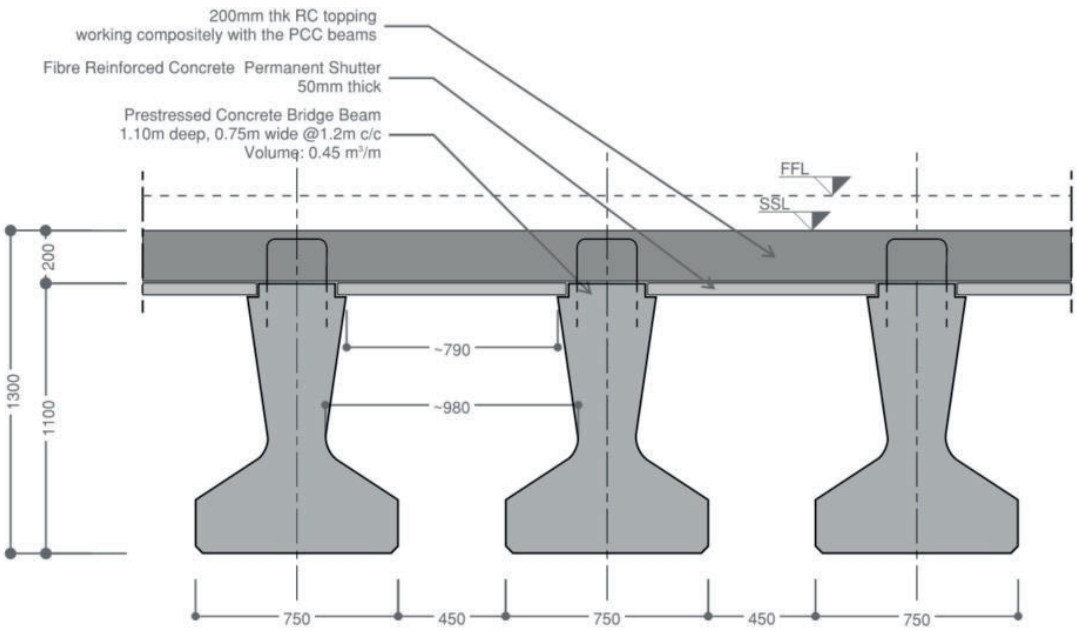


Figura 7: Travi da ponte prefabbricate - Sezione tipica

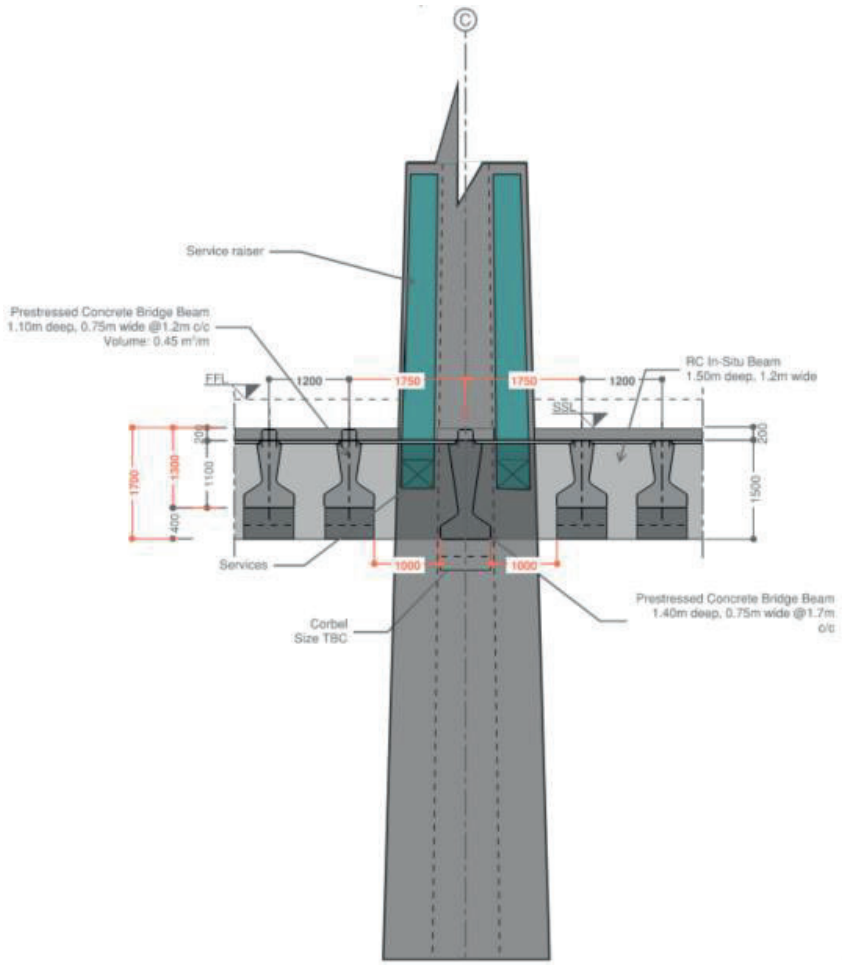


Figura 8: Tipica disposizione della trave a ponte prefabbricata davanti alla mega colonna con condotti di ventilazione

Edifici Laterali

I due edifici laterali accostati nella parte Nord dell'edificio hanno le stesse dimensioni complessive, ma mentre l'edificio Ovest ha una griglia regolare 12x12m l'edificio Est presenta luci variabili fino a 18 metri. Entrambe sono costituiti da un piano terra, un mezzanino, un primo piano ed un tetto non accessibile se non per manutenzione.

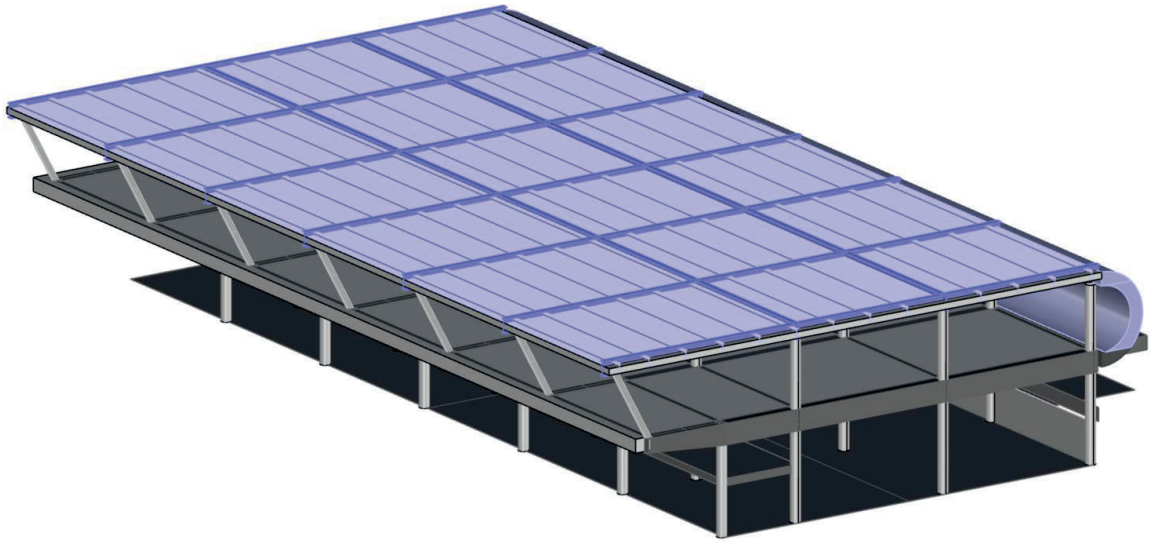


Figura 9: Rendering dell'edificio laterale Ovest

In entrambe gli edifici il tetto è costituito da una griglia in acciaio che sostiene una lastra metallica di copertura, scelta in quanto il tetto, non sarà accessibile se non per manutenzione. Le travi del tetto sono sostenute dalle colonne che salgono dal pianterreno e sulla facciata anteriore da dei pali inclinati, che poggiano su una mensola di luce 6 metri. I pali poggiano direttamente sulle travi primarie in calcestruzzo armato post-teso; ciò consente di affusolare le travi in corrispondenza della facciata e minimizzare la profondità della trave di legamento, per un gradevole effetto architettonico. Il solaio del primo piano è costituito da solette con nucleo cavo di 400 mm ovunque, eccetto la mensola, dove la soletta si è ridotta a 250 mm con una copertura di 50 mm.

Sottostruttura

Si prevede che la soletta del piano terra sarà in cemento armato gettato in opera, sostenuto su travi di terra tra le fondazioni. Uno strato di impermeabilizzazione sarà applicato alla parte inferiore della soletta.

L'area del seminterrato sarà un tradizionale muro di contenimento in cemento armato dello spessore di 300 - 400 mm, costruito da uno scavo battuto. Uno strato di impermeabilizzazione rivestirà la superficie esterna.

Il terrapieno che porta al tetto è di costruzione artificiale, formato da terra battuta che include il materiale di scavo del terminal. Dove si collega al tetto ci saranno un edificio per la cantina e un locale tecnico, questi saranno costruiti in cemento armato con un muro di contenimento irrigidito con muri di contrafforte esterni a causa dell'altezza del terreno trattenuto. Ci sarà un giunto di movimento strutturale tra la struttura in calcestruzzo del terrapieno e la struttura del tetto in acciaio e calcestruzzo, in modo che ognuno agisca come edificio indipendente.

Si prevede che non ci sarà più di 1 m di terreno sul tetto, per mantenere il carico del suolo al minimo. Se non c'è l'esigenza di uno spazio interno così alto, allora si può usare un vuoto leggero sul tetto per compensare i livelli da una lastra piatta. Uno strato di impermeabilizzazione sarà applicato alla parte inferiore della soletta.

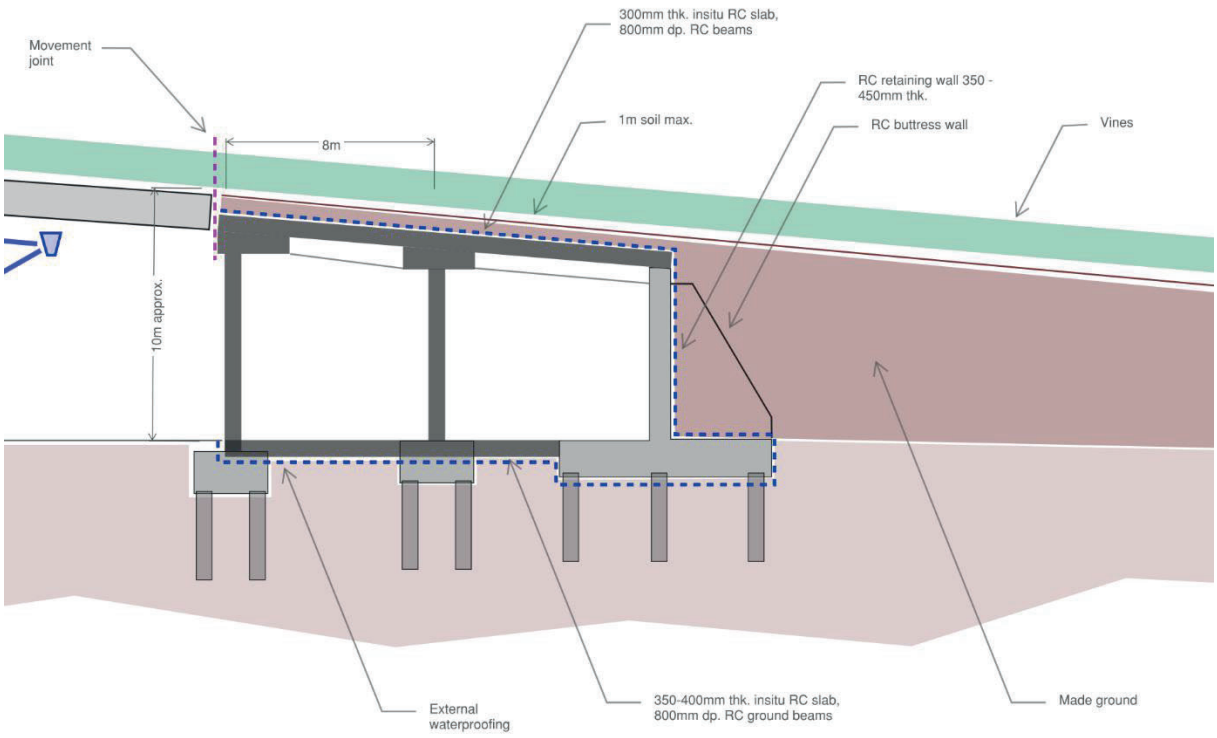


Figura 10:- Vigneto e locale tecnico all'interno del terrapieno

Fondazioni

Le fondazioni sono state considerate come pali trivellati in cemento armato, che saranno collocati in gruppi da 2 fino a 6 sotto un cappello per pali in cemento armato. Attualmente non disponiamo di informazioni dettagliate sull'indagine geotecnica del sito; quindi, si è intrapresa una progettazione dettagliata dei pali, anche se si prevede che i pali avranno un diametro di 900 mm a causa dell'elevato carico del tetto.

Criteri di progettazione

Codici e Regole di progettazione

Per lo sviluppo della progettazione sono stati utilizzate le seguenti regole e i corrispondenti documenti di rilevanza nazionale.

NTC-2018	NORME TECNICHE PER LE COSTRUZIONI, 2018
UNI-EN-1990	Criteri generali di progettazione strutturale
UNI EN 1991-1-1	Azioni sulle strutture Parte 1-1: Azioni generali – Densità, pesi propri, sovraccarichi per edifici
UNI EN 1991-1-3	Azioni sulle costruzioni Parte 1-3: Carichi della neve
UNI-EN 1991-1-4	Azioni sulle costruzioni Parte 1-4: Azioni in generale – Azioni del vento
UNI-EN 1991-1-5	Azioni sulle costruzioni Parte 1-5: Azioni in generale – Azioni Termiche
UNI EN 1992-1-1	Eurocodice 2: Progettazione di strutture di calcestruzzo Parte 1-1: Regole generali e regole per gli edifici
UNI-EN-1993-1-1	Eurocodice 3: Progettazione delle strutture in acciaio Parte 1-1: Regole generali e regole per gli edifici.
UNI-EN-1994-1-1	Eurocodice 4: Progettazione delle strutture composte acciaio-calcestruzzo Parte 1-1: Regole generali e regole per gli edifici
UNI-EN-1997-1	Eurocodice 7: Progettazione geotecnica Parte 1: Regole generali
UNI-EN-1998-1	Progetto delle strutture in zona sismica. Parte 1- Regole generali, azione sismica e regole per gli edifici

Software e Licenze

- Autodesk Revit 2019: 3D model, 2D drawings
- Autodesk ROBOT Structural Analysis Professional 2019: Modello di analisi globale, progettazione carpenteria metallica, progettazione in cemento armato.
- Tekla Structural Designer: Modello di analisi globale, progettazione di carpenteria metallica
- Hilti Profis Anchor: connessioni acciaio calcestruzzo
- Tekla Tedds 2018: altri calcoli

Carichi

Peso proprio

Per il dimensionamento sono state utilizzate le seguenti densità:

- Calcestruzzo $\rho_c = 25.0\, kN/m^3$
- Acciaio $\rho_s = 78.5\, kN/m^3$
- Vetro $\rho_g = 25.0\, kN/m^3$

Permanenti (non strutturali)

Sono stati assunti i seguenti pesi proprio:

Tetto:

- **Copertura verde estesa** (risultante su trave prefabbricata)
  - Suolo, 20kN/m^3, 300mm deep, 0.62 m3/m — 12.40 kN/m
  - Strato filtrante, 18 kN/m3, 10mm deep, 0.21 m3/m — ~3.80 kN/m
  - Strato di deposito geo-cellulare, 150mm deep, 10.2 kN/m2 — 1.50 kN/m
  - Alleggerimenti in polistirolo e Impermeabilizzazione 0.70 kN/m3, 1.34 m3/m — ~1.00 kN/m
  - Finiture e servizi, 1.00 kN/m2 (pannelli metallici leggeri e intelaiatura secondaria) — 4.00 kN/m
  - Lucernario, 2.00 kN/m2, 1.20 m wide (incluso il vetro e l'intelaiatura secondaria) — 2.40 kN/m

    TOTALE — **25.00 kN/m**
- **Portinnesto non-fruttifero** (risultante su trave prefabbricata)
  - Suolo, 20kN/m^3 , 600mm deep, ~1.24 m3/m — 24.80 kN/m
  - Strato filtrante, 18 kN/m3, 10mm deep, 0.21 m3/m — ~3.80 kN/m
  - Strato di deposito geo-cellulare, 150mm deep, 10.2 kN/m2 — 1.50 kN/m
  - Alleggerimenti in polistirolo e Impermeabilizzazione 0.70 kN/m3, ~0.72 m3/m — 0.50 kN/m
  - Rivestimento, 1.00 kN/m2 (pannelli metallici leggeri e intelaiatura secondaria) — 4.00 kN/m
  - Lucernario, 2.00 kN/m2, 1.20 m wide (incluso il vetro e l'intelaiatura secondaria) — 2.40 kN/m

    TOTALE — **37.00 kN/m**

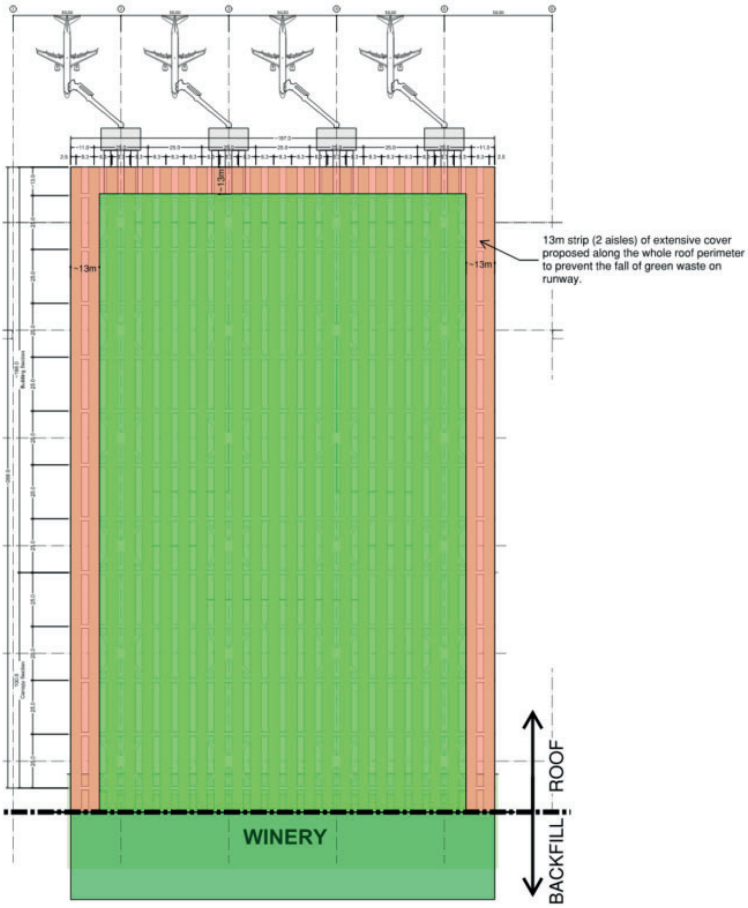


Figura 11: Tipologia di portainnesto e carichi

Legenda	Descrizione	Tipo di accesso	Tipica copertura del suolo [mm]	Tipico carico del suolo [kg/m2]	Carico variabile [kg/m2]
	Vasta vegetazione	Solo manutenzione	300	60	60
	Portinnesto non-a frutto	Solo manutenzione	600	1000	60
	Portinnesto a frutto	Completamente accessibile	>1000	>2000	500

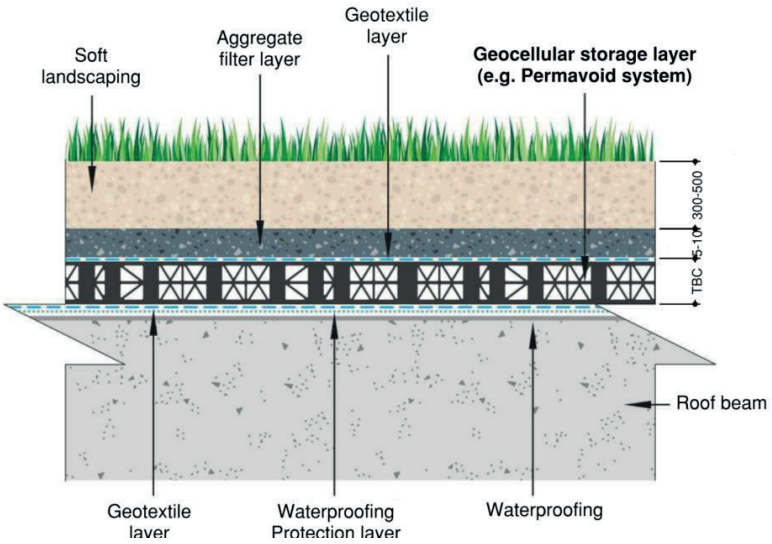


Figura 12: Sistema di impermeabilizzazione proposto per il tetto

- **Tetto Edifici Laterali**
  - Rivestimento 1.00 kN/m2
    (pannelli metallici leggeri e intelaiatura secondaria)
  - Alleggerimenti in polistirolo e Impermeabilizzazione ~1.00 kN/m2
    0.70 kN/m3, 1.34 m3/m
  - Soffitti 1.00 kN/m2
  - Impianti (MEP) 0.50 kN/m2
  - HVAC Unit 0.50 kN/m2

- **Pavimenti (tipici):**
  - Finitura 1.20 kN/m2
    (40mm di pietra + 20mm di calcestruzzo alleggerito)
  - Massetto (75mm) 1.50 kN/m2
  - Impianti (MEP) 0.50 kN/m2
  - Soffitti 1.00 kN/m2
  - **TOTALE** **4.20 kN/m2**

- **Strada, area pavimentata esterna (tipici):**
  - ipotizzata ghiaia di 1,50 m a 20 kN/m3 TBC 30.0 kN/m2

Variabili

- **Tetto (manutenzione solo) 0.60 kN/m2**
- **Carico della folla 5.00 kN/m2**
- **Traffico (Modello di carico 1, come da EN 1991-2) 9.00 kN/m2**
  300 kN/asse

Neve

Carichi da neve caratteristici

Altitudine del sito (sopra livello del mare) [m]	$\boldsymbol{a_s = 40m}$
Coefficiente di Esposizione [-]	$C_e = 1.0$
Coefficiente Termico [-]	$C_t = 1.0$
Valore caratteristico del carico della neve sul suolo	$\boldsymbol{q_{sk} = 1.00\ kN/m^2}$

Coefficiente della forma del tetto

Disposizione senza deriva

Coefficiente per la forma del tetto - vigneto [-]	$\boldsymbol{\mu_1 = 0.80}$
Coefficiente per la forma del tetto - lucernario [-]	$\mu_1 = 0.80$

Disposizione alla deriva

Coefficiente di forma del tetto- vigneto [-]	$\boldsymbol{\mu_2 = 0.95}$
Coefficiente di forma del tetto-lucernario[-]	$\mu_3 = 2.00$
Coefficiente di forma del tetto in rapporto al vigneto [-]	$\alpha = 1.83$

Carichi da neve eccezionali sul suolo

Il caso di azioni di neve eccezionali, come descritto nel EN 1991-1-3:2003, non é appllicabile in Italia.

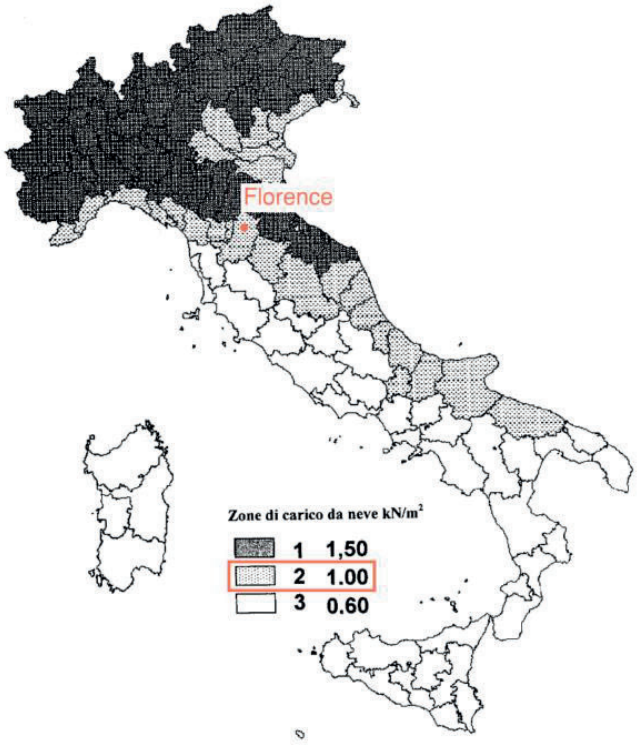


Figura 12 - Valore caratteristico del carico neve al suolo (sk), UNI EN 1991-1-3:2003

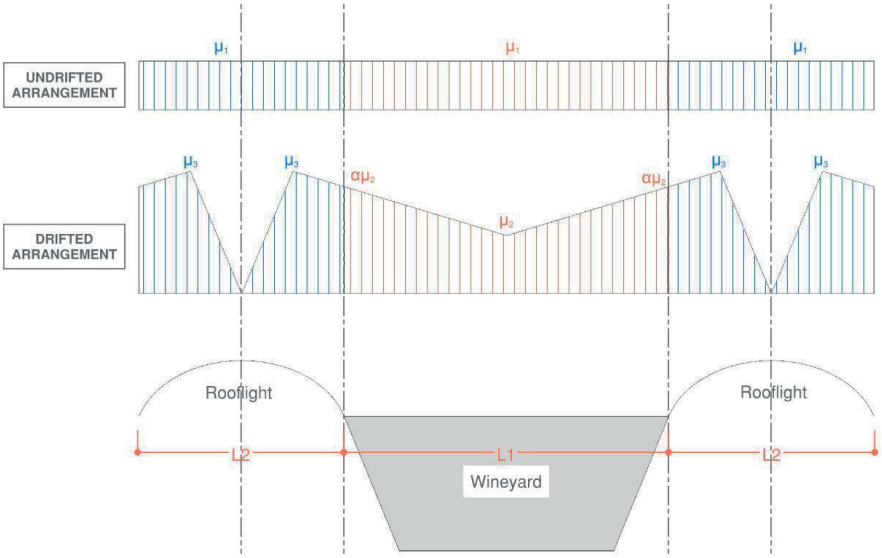


Figura 13 - Disposizione del carico di neve

Vento

In conformità con la norma EN 1991 Parte 4, il seguente carico del vento è stato considerato in base ai criteri seguenti.

Zona di vento	3
Categoria del terreno	II
Altitudine del situ (dal livello del mare)	40m
Altitudine dal suolo	30m
Lunghezza della rugosità	0.05m
Valore fondamentale della velocità di base del vento	27m/s
Pressione di picco della velocità	1.4kPa

Figura 14 - Mappa della zona del vento. Fig. N.A.1 da UNI-EN 1991-1-4

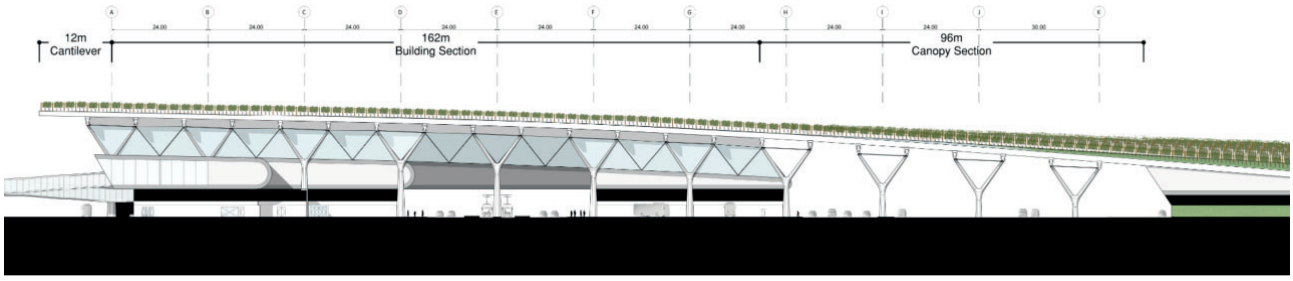


Figura 15 - Dimensioni globali dell'edificio

Coefficiente di pressione esterna – Sezione dell'edificio

BS EN 1991-1-4:2005+A1:2010
EN 1991-1-4:2005+A1:2010 (E)

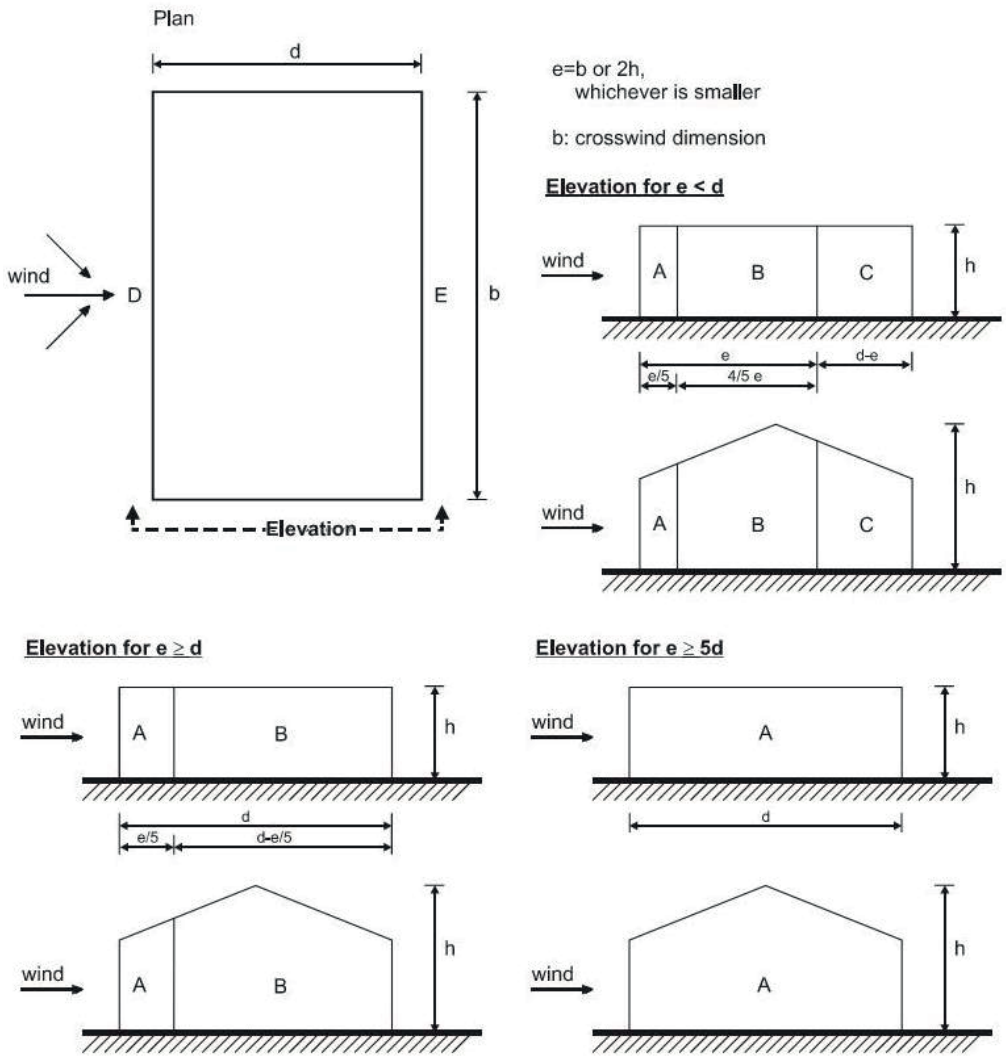


Figure 7.5 — Key for vertical walls

NOTE 1 The values of $c_{pe,10}$ and $c_{pe,1}$ may be given in the National Annex. The recommended values are given in Table 7.1, depending on the ratio *h*/*d*. For intermediate values of *h*/*d*, linear interpolation may be applied. The values of Table 7.1 also apply to walls of buildings with inclined roofs, such as duopitch and monopitch roofs.

Table 7.1 — Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

NOTE 2 For buildings with $h/d > 5$, the total wind loading may be based on the provisions given in 7.6 to 7.8 and 7.9.2.

(3) In cases where the wind force on building structures is determined by application of the pressure coefficients c_{pe} on windward and leeward side (zones D and E) of the building simultaneously, the lack of correlation of wind pressures between the windward and leeward side may have to be taken into account.

NOTE The lack of correlation of wind pressures between the windward and leeward side may be considered as follows. For buildings with $h/d \geq 5$ the resulting force is multiplied by 1. For buildings with $h/d \leq 1$, the resulting force is multiplied by 0,85. For intermediate values of h/d, linear interpolation may be applied.

BS EN 1991-1-4:2005+A1:2010
EN 1991-1-4:2005+A1:2010 (E)

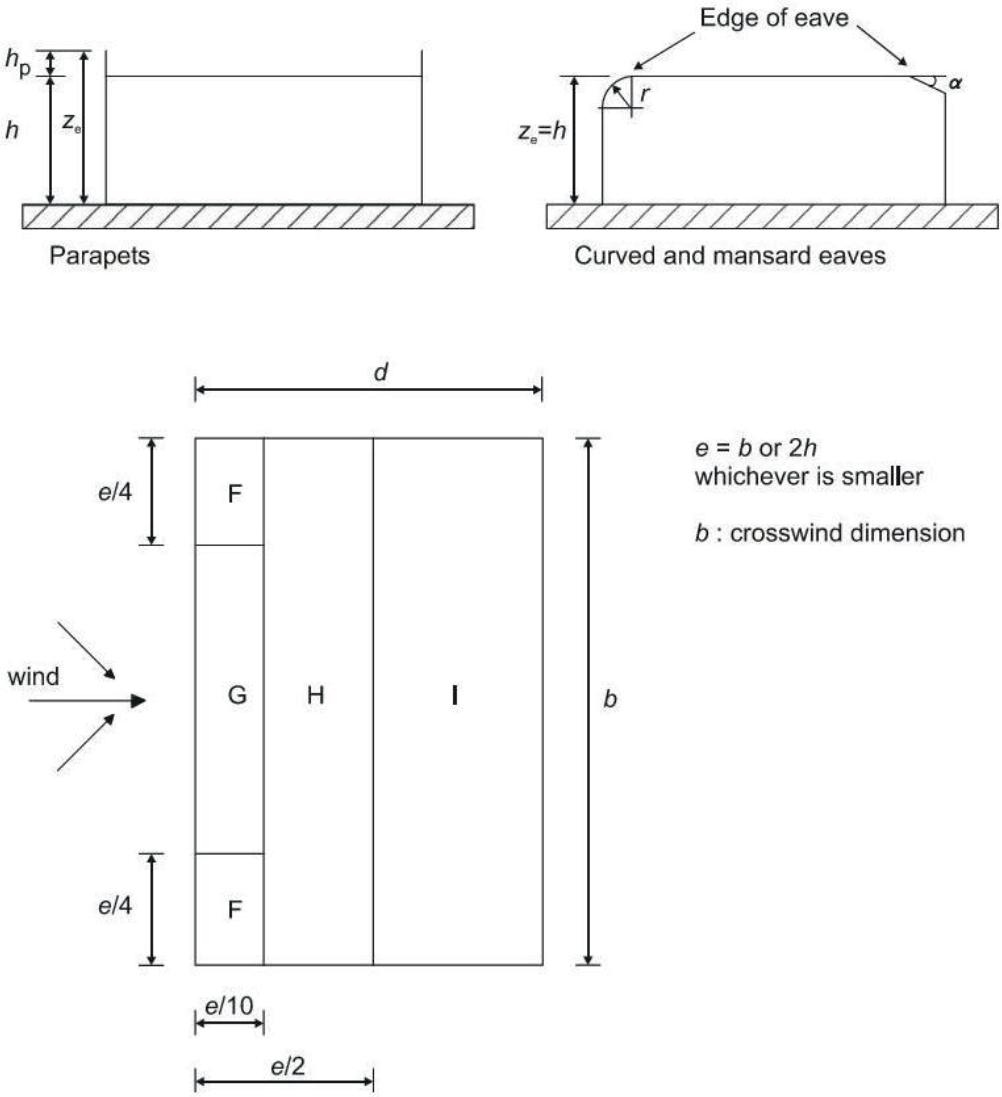


Figure 7.6 — Key for flat roofs

A1 Table 7.2 — Recommended values of external pressure coefficients for flat roofs A1

Roof type		Zone F		Zone G		Zone H		Zone I	
		$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
Sharp eaves		-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	+0,2	
								-0,2	
With Parapets	h_p/h=0,025	-1,6	-2,2	-1,1	-1,8	-0,7	-1,2	+0,2	
								-0,2	
	h_p/h=0,05	-1,4	-2,0	-0,9	-1,6	-0,7	-1,2	+0,2	
								-0,2	
	h_p/h=0,10	-1,2	-1,8	-0,8	-1,4	-0,7	-1,2	+0,2	
								-0,2	
Curved Eaves	r/h = 0,05	-1,0	-1,5	-1,2	-1,8	-0,4		+0,2	
								-0,2	
	r/h = 0,10	-0,7	-1,2	-0,8	-1,4	-0,3		+0,2	
								-0,2	
	r/h = 0,20	-0,5	-0,8	-0,5	-0,8	-0,3		+0,2	
								-0,2	
Mansard Eaves	α = 30°	-1,0	-1,5	-1,0	-1,5	-0,3		+0,2	
								-0,2	
	α = 45°	-1,2	-1,8	-1,3	-1,9	-0,4		+0,2	
								-0,2	
	α = 60°	-1,3	-1,9	-1,3	-1,9	-0,5		+0,2	
								-0,2	

NOTE 1 For roofs with parapets or curved eaves, linear interpolation may be used for intermediate values of h_p/h and r/h.

NOTE 2 For roofs with mansard eaves, linear interpolation between α = 30°, 45° and α = 60° may be used. For α > 60° linear interpolation between the values for α = 60° and the values for flat roofs with sharp eaves may be used.

NOTE 3 A1 In Zone I, where positive and negative values are given, both values should be considered. A1

NOTE 4 For the mansard eave itself, the external pressure coefficients are given in Table 7.4a "External pressure coefficients for duopitch roofs: wind direction 0° ", Zone F and G, depending on the pitch angle of the mansard eave.

NOTE 5 For the curved eave itself, the external pressure coefficients are given by linear interpolation along the curve, between values on the wall and on the roof.

A1 NOTE 6 For mansard eaves with horizontal dimension less than $e/10$,the values for sharp eaves should be used. For the definition of e see Figure 7.6. A1

Coefficiente di pressione esterna – Sezione del baldacchino

BS EN 1991-1-4:2005+A1:2010
EN 1991-1-4:2005+A1:2010 (E)

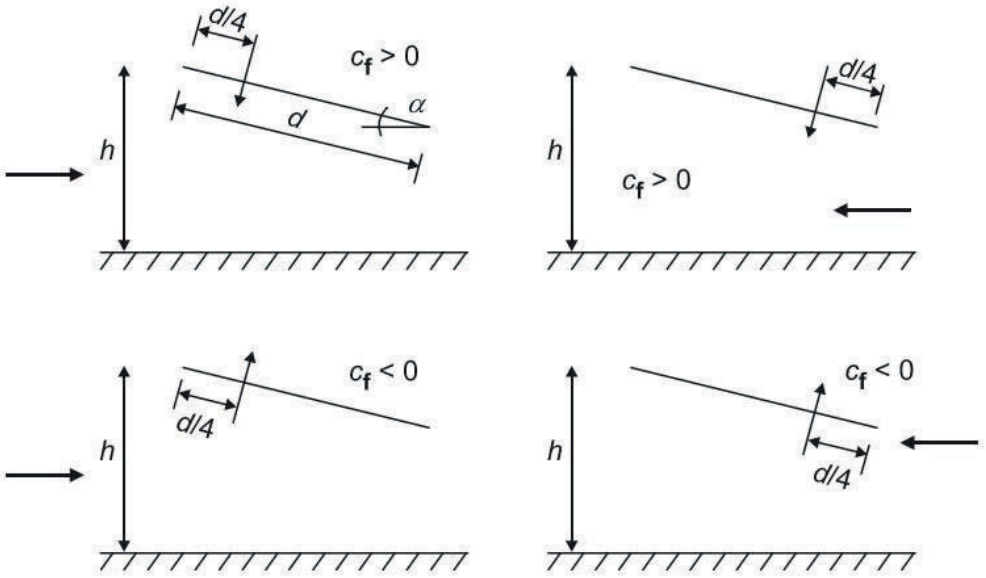


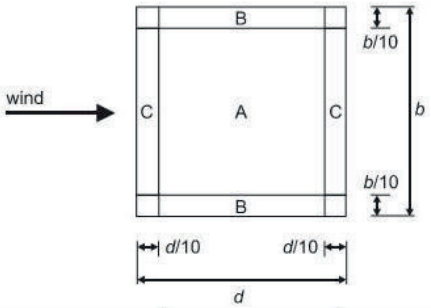
Figure 7.16 — Location of the centre of force for monopitch canopies

BS EN 1991-1-4:2005+A1:2010
EN 1991-1-4:2005+A1:2010 (E)

Table 7.6 — $c_{p,net}$ and c_f values for monopitch canopies

Net Pressure coefficients $c_{p,net}$

Key plan



Roof angle α	Blockage φ	Overall Force *Coefficients* c_f	Zone A	Zone B	Zone C
0°	Maximum all φ	+ 0,2	+ 0,5	+ 1,8	+ 1,1
	Minimum $\varphi = 0$	- 0,5	- 0,6	- 1,3	- 1,4
	Minimum $\varphi = 1$	- 1,3	- 1,5	- 1,8	- 2,2
5°	Maximum all φ	+ 0,4	+ 0,8	+ 2,1	+ 1,3
	Minimum $\varphi = 0$	- 0,7	- 1,1	- 1,7	- 1,8
	Minimum $\varphi = 1$	- 1,4	- 1,6	- 2,2	- 2,5
10°	Maximum all φ	+ 0,5	+ 1,2	+ 2,4	+ 1,6
	Minimum $\varphi = 0$	- 0,9	- 1,5	- 2,0	- 2,1
	Minimum $\varphi = 1$	- 1,4	AC2 - 1,6 AC2	- 2,6	- 2,7
15°	Maximum all φ	+ 0,7	+ 1,4	+ 2,7	+ 1,8
	Minimum $\varphi = 0$	- 1,1	- 1,8	- 2,4	- 2,5
	Minimum $\varphi = 1$	- 1,4	- 1,6	- 2,9	- 3,0
20°	Maximum all φ	+ 0,8	+ 1,7	+ 2,9	+ 2,1
	Minimum $\varphi = 0$	- 1,3	- 2,2	- 2,8	- 2,9
	Minimum $\varphi = 1$	- 1,4	- 1,6	- 2,9	- 3,0
25°	Maximum all φ	+ 1,0	+ 2,0	+ 3,1	+ 2,3
	Minimum $\varphi = 0$	- 1,6	- 2,6	- 3,2	- 3,2
	Minimum $\varphi = 1$	- 1,4	- 1,5	- 2,5	- 2,8
30°	Maximum all φ	+ 1,2	+ 2,2	+ 3,2	+ 2,4
	Minimum $\varphi = 0$	- 1,8	- 3,0	- 3,8	- 3,6
	Minimum $\varphi = 1$	- 1,4	- 1,5	- 2,2	- 2,7

NOTE + values indicate a net downward acting wind action

- values represent a net upward acting wind action

Temperatura

Carico termico definito secondo UNI EN 1991-1-5:2005.

Struttura fuori terra

Variazione uniforme

Temperatura iniziale T_0 [°C]	**+15**
Ambiente Interno	
Estate T_1 [°C]	+20
Inverno T_2 [°C]	+25
Ambiente Esterno	
Estate T_{max} [°C] 1)	
Ombreggiato	+45
Esposto verso Nord-Est	+45 / +47 / +49
Esposto verso Sud-Ovest	+63 / +75 / +87
Inverno T_{min} [°C]	-15
Variazione Caratteristica della Temperatura	
Ambiente Interno $\Delta T_{u,i}$ [°C]	**0 to +10**
Ambiente Esterno $\Delta T_{u,o}$ [°C] 1)	
Ombreggiato	**-30 to +30**
Verso Nord-Est	**-30 to +30 / +32 / +34**
Verso Sud-Ovest	**-30 to +48 / +60 / +72**

1): Assorbibilità della superficie : chiaro chiaro (0,5) / chiaro colorato (0,7) / scuro (0.9)

Sotto la struttura del suolo

Variazione uniforme

Temperatura Iniziale T_0 [°C]	**Si riferisce a Struttura fuori terra**
Ambiente Interno	Si riferisce a Struttura fuori terra
Ambiente Esterno	
Estate T_{max} [°C] 2)	8 / 5
Inverno T_{min} [°C] 2)	-5 / -3
Variazione Caratteristica della temperatura	
Ambiente Interno $\Delta T_{u,i}$ [°C]	Si riferisce a Struttura fuori terra
Ambiente Esterno $\Delta T_{u,o}$ [°C] 2)	**-20 to -7**

1): Profondità sotto il livello del suolo: meno di 1.0m / più di 1.0m

Coefficiente di espansione lineare

I seguenti coefficienti sono utilizzati per la modellazione e i calcoli in conformità alla tabella C.1 della norma EN 1991-5:2003.

Materiale	**Coefficiente Termico** α_T $[x10^6/°C]$
Acciaio strutturale	12.0
Calcestruzzo normale e ad alta resistenza	10.0
Calcestruzzo ad altissime prestazioni (UHPC)	*TBC*
Calcestruzzo leggero	7.0
Rinforzo in acciaio passivo e attivo del calcestruzzo	10.0
Legno, lungo le fibre	5.0
Legno, trasversale	*TBC (30-70)*

Sismica

In accordo con la EN 1998 sono stati considerati i seguenti criteri sismici.

Parametri principali

Tipo di terreno	**C (assunzione)**
Fattore di comportamento	2.5
Classe di importanza	4 (importanza vitale)

Nota: il terreno di tipo C corrisponde a "depositi profondi di sabbia da densa a mediamente densa, ghiaia o argilla rigida con spessore da alcune decine a molte centinaia di metri". Questo deve essere confermato nella prossima fase del progetto con un'indagine geotecnica specifica del sito.

Periodo di ritorno e picco di accelerazione del suolo

I parametri di progettazione si basano sulla Tabella 1 degli Allegati A e B al Decreto del Ministro delle Infrastrutture del 14 gennaio 2008, pubblicato nel S.O. alla Gazzetta Ufficiale del 4 febbraio 2008, n. 29, ed eventuali successivi aggiornamenti (ID 20059, LON 11.275, LAT 43.764).

I valori dei parametri sono stati interpolati in base alla formula [2] del documento citato. Per periodi di ritorno superiori a 2475 anni, il valore corrispondente a 2475 anni è stato definito in conformità con "Allegato alle norme tecniche per le costruzioni: Pericolosità sismica", pagina 2.

	Periodo di ritorno T_R [1] [anno]	**picco di accelerazione a_g [m/s²]**	**F_o [-]**	**T_C [s]**
Stato limite ultimo				
Prevenzione del collasso (SLC)	3899 (2475 considerato)	2.14	2.42	0.320
Sicurezza della vita (SLV)	1898	1.97	2.41	0.317
Stato limite di danno				
Occupazione immediata (SLD)	200	0.91	2.51	0.290
Operativo (SLO)	120	0.73	2.61	0.285

Metodo di Analisi

Gli effetti sismici e gli effetti delle altre azioni incluse nella situazione di progetto sismico saranno determinati sulla base del **comportamento lineare-elastico** della struttura utilizzando l'analisi dello **spettro di risposta modale** in accordo con la UNI-EN-1998-1.

Classe di Duttilitá

La progettazione sismica della struttura mira a garantire una sufficiente capacità di dissipazione dell'energia della struttura attraverso un comportamento globale duttile con sufficiente affidabilità. Sulla base della materialità e della geometria della struttura, per la progettazione dell'intera struttura, cioè tetto principale, terminale e cantina si assume la Classe di Duttilitá Media (CDM) - **Medium Ductility Class (DCM)** -

Fattore di comportamento

Considerando che l'analisi EF sarà limitata al comportamento lineare-elastico, il fattore di comportamento q è considerato per catturare la duttilità della struttura, cioè la sua capacità di dissipare l'energia. Il fattore di comportamento viene applicato solo per ridurre le forze sismiche di progetto, ma non le deflessioni sismiche di progetto.

Il valore di progetto del fattore di comportamento è stato determinato in base al materiale, alla forma della struttura resistente laterale e alla classe di duttilità (vedi sopra). In prima approssimazione, la struttura del tetto può essere riferita come un sistema a telaio a un piano. La particolare geometria della colonna risulta in una significativa variazione di rigidità in elevazione, quindi il fattore di comportamento è stato ridotto di conseguenza. Essendo la struttura del tetto molto regolare in pianta, si presume che lo stesso fattore di comportamento sarà usato in entrambe le direzioni X e Y. Pertanto, il seguente fattore di comportamento è stato assunto per il progetto:

$$\boldsymbol{q = 2.50}$$

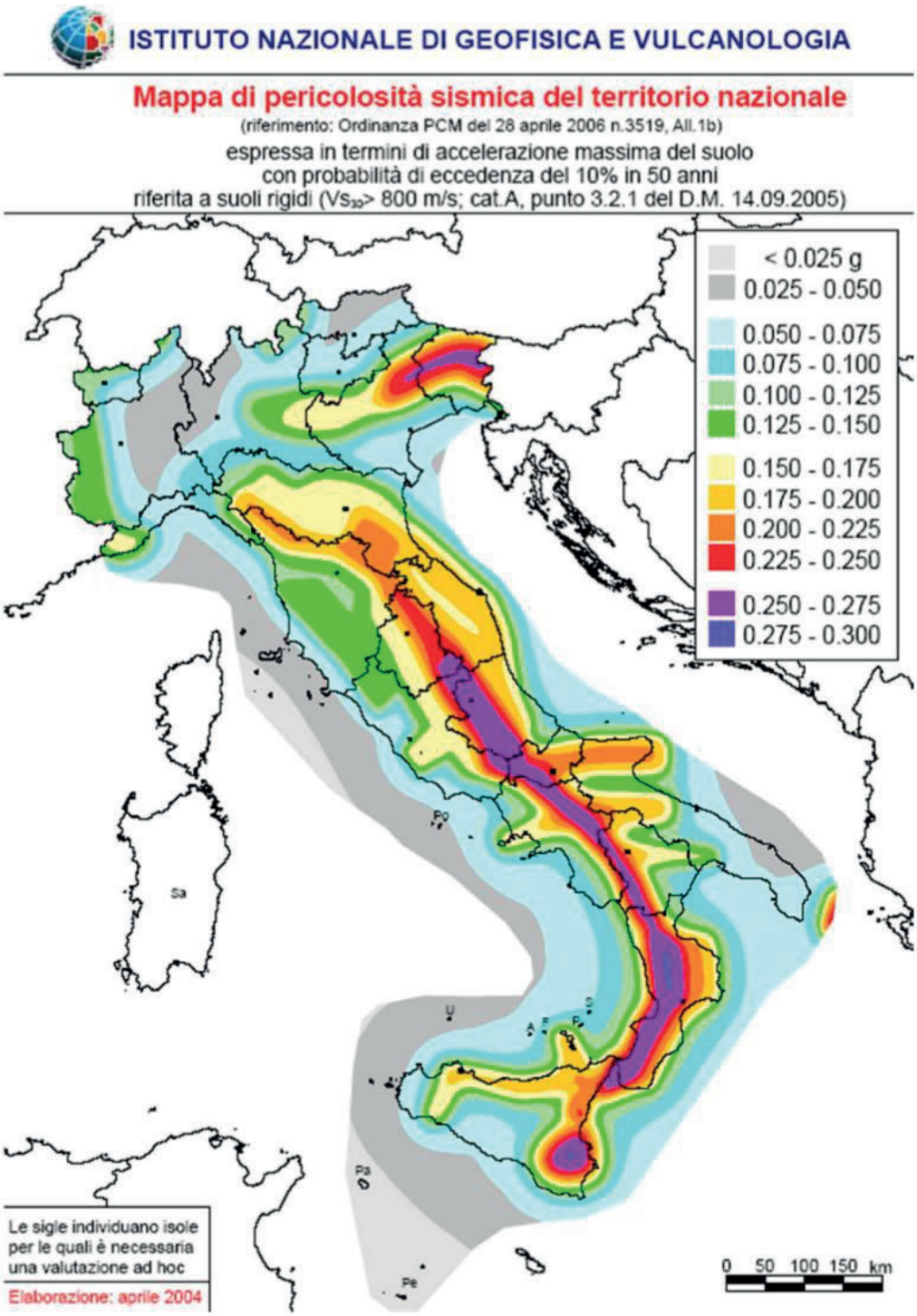


Figura 13: mappa di pericolosità sismica del territorio nazionale

Combinazioni di carico

Fattore parziale

Limit state	Permanent actions (G_k)	Leading variable action ($Q_{k,1}$)	Accompanying variable actions ($Q_{k,i}$)	Reference
a) Equilibrium (EQU) Set A				
	$\gamma_{G,sup}$ ($\gamma_{G,inf}$)[a]	$\gamma_{Q,1}$	$\gamma_{Q,i}\psi'_{0,i}$	BS EN 1990 Table A1.2(A) (Set A)
UK values	1.10 (0.9)[a]	1.50 (0.0)[a]	$1.50\psi'_{0,i}$ (0.0)[a]	NA to BS EN 1990
b) Strength at ULS (STR/GEO) not involving geotechnical actions Set B				
Either				
Exp. (6.10)	γ_G	γ_Q	$\psi'_{0,i}\gamma_Q$	BS EN 1990 Exp. (6.10) & Table A1.2(B)
UK values	1.35 (1.0)[a]	1.5	$1.5\psi'_0$	NA to BS EN 1990
or worst case of				
Exp. (6.10a)	γ_G	$\psi'_{0,1}\gamma_Q$	$\psi'_{0,i}\gamma_Q$	BS EN 1990 Exp. (6.10a) & Table A1.2(B)
UK values	1.35 (1.0)[a]	$1.5\psi'_0$	$1.5\psi'_0$	NA to BS EN 1990
and				
Exp. (6.10b)	$\xi\gamma_G$	γ_Q	$\psi'_{0,i}\gamma_Q$	BS EN 1990 Exp. (6.10b) & Table A1.2(B)
UK values	0.925 x 1.35 = 1.25 (1.0)[a]	1.5	$1.5\psi'_0$	NA to BS EN 1990
c) Strength at ULS with geotechnical actions (STR/GEO)				
Worst case of				
Set B	γ_{G1}	γ_{Q1}		BS EN 1990 Table A1.2(B)
UK values	1.35 (1.0)[a]	1.5 (0.0)[a]		NA to BS EN 1990
or				
Set C	γ_{G2}	γ_{Q2}		BS EN 1990 Table A1.2(C)
UK values	1.0	1.3		NA to BS EN 1990
d) Serviceability				
Characteristic	1.00	1.00	$1.00\psi'_{0,i}$	BS EN 1990 Table A1.4
Frequent	1.00	$1.00\psi'_{1,1}$	$1.00\psi'_{2,i}$	
Quasi-permanent	1.00	$1.00\psi'_{2,1}$	$1.00\psi'_{2,i}$	
e) Accidental design situations				
Exp. (6.11a)	$\gamma_{G,sup}$ or ($\gamma_{G,inf}$)	$\psi'_{1,1}$[b]	$\psi'_{2,i}$	BS EN 1990 Exp. (6.11a)
UK values	1.00	$\psi'_{1,1}$[b]	$\psi'_{2,i}$	NA to BS EN 1990

Key
a Value if favourable
b Leading *accidental* action (A_d)

Notes
The values of ψ' are given in Table 2.1.
Where the variation between $G_{k,sup}$ and $G_{k,inf}$ is not great, say < 10%, G_k is taken to represent permanent action.
Geotechnical actions given in the table are based on Design Approach 1 in Clause A1.3.1(5) of BS EN 1990, which is recommended in the National Annex for BS EN 1990.

Figura 14: Fattori parziali (γF) da utilizzare nella verifica degli stati limite in situazioni di progetto persistenti e transitorie.

Fattore di azione variabile di accompagnamento

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area vehicle weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area 30 kN < vehicle weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs[a]	0.7	0.0	0.0
Snow loads where altitude ≤ 1000 m above sea level[a]	0.5	0.2	0.0
Wind loads[a]	0.5	0.2	0.0
Temperature effects (non-fire)[a]	0.6	0.5	0.0

Note
The numerical values given above are in accordance with BS EN 1990 and its UK National Annex

Key
a See also BS EN 1991

Figura 18 - Valori di i fattori

Integrazione per la combinazione Set B (ULS-STR)

L'espressione (6.10) è sempre uguale o più conservativa della meno favorevole delle espressioni (6.10a) e (6.10b). Rispetto all'uso dell'espressione (6.10), l'uso dell'espressione (6.10a) o (6.10b) porta a un indice di affidabilità più coerente tra materiali leggeri e pesanti**.** L'espressione (6.10b) si applica normalmente quando le azioni permanenti non sono maggiori di 4,50 volte le azioni variabili, ad eccezione dei carichi di stoccaggio (Figura 18, categoria E). La Figura 19 dà un'indicazione più precisa sul limite dell'apparecchio tra (6.10a) e (6.10b).

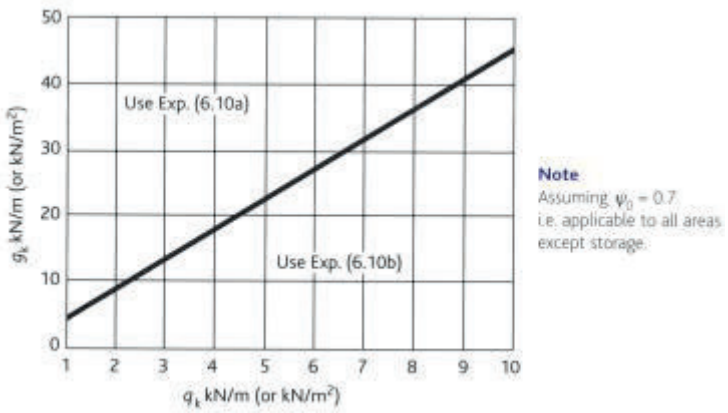


Figura 19 - Quando usare l'Esp. (6.10a) o Esp. (6.10b)

Condizioni del terreno

In una fase successiva saranno eseguite opportune indagini per identificare le condizioni del suolo. In questa prima fase, le condizioni del sottosuolo sono stimate utilizzando il Sistema Informativo Geologico del Sottosuolo di Firenze (https://sigs.comune.fi.it/).

Il database di Firenze include i rilievi esistenti sul sito. Sono stati conservati e analizzati 7 log dei pozzi, ovvero 296 / 297 / 828 / 830 / 837 / 1707 / 1709. La loro rispettiva gamma si estende da 5 a 50m di profondità. Il log della stratigrafia dei 7 pozzi è stato concatenato ed è presentato di seguito. Per maggiori dettagli, fare riferimento all'Appendice A.

Descrizione	Compattazione	Profondità iniziale [m]	Profondità finale [m]
Suolo terroso	-	0.0	0.5
Limo argilloso	Medio Compatto	0.5	5.0
Limo con modesto legante argilloso con formazioni calcaree	Compatto	5.0	10.0
Miscela di ghiaia, sabbia con elementi smussati e modesto limo argilloso	Compatto	10.0	15.0
Limo argilloso con sabbia e tracce di calcare	Compatto	15.0	20.0
Miscela di sabbia, ghiaia, ghiaia e limo argilloso	Compatto	20.0	25.0
Limo argilloso e calce con sabbia e ghiaia	Compatto	25.0	30.0
Limo argilloso e calce con sabbia e ghiaia	Compatto	30.0	35.0
Limo argilloso con inclusioni di calcare	Compatto	35.0	40.0
Miscela di sabbia, ghiaia e limo argilloso	Compatto	40.0	45.0
Calce argillosa con diffusi piccoli nuclei di CaCO3	Compatto	45.0	50.0

Figura 20 - Estratto del Florence Geotechnical Information System, che elenca 7 indagini disponibili esistenti

Criteri di prestazione

Vita utile di progetto

In conformità con la norma NTC 2018, si ritiene che l'edificio abbia una vita utile di progetto di 100 anni.

Classe d'uso

In conformità con la norma NTC 2018, è stata determinata la seguente classe di conseguenze.

Classe d'uso: *Classe IV*

Materiali

Carpenteria metallica

Grado d'acciaio

Sono stati considerati i seguenti tipi di acciaio:

- Acciaio ad alta resistenza S460M secondo la norma EN 10025-4.

- Resistenza minima allo snervamento: 460Mpa (spessore ≤16mm) - 400Mpa (spessore 80-100mm)
- Resistenza alla trazione finale: 540-720Mpa (spessore ≤16mm) - 500-680Mpa (spessore 80-100mm)

Durabilità

Ambiente	Categoria di corrosione
Interno	C2
Esterno	C3

Calcestruzzo

Grado calcestruzzo

Sono stati considerati i seguenti tipi di calcestruzzo:

- Calcestruzzo gettato in opera- Tipico C40/50
- Calcestruzzo prefabbricato - Tipico C60/70
- Calcestruzzo prefabbricato - Colonne ad albero C80/90

Durabilità

Ambiente	Classe di esposizione	Rivestimento min. [mm]
Interno - protetto ed esposto	XC1	30
Esterno - protetto	XC3, XD3, XF4	40
Esterno - esposto	XC4, XD3, XF4	40
Esterno – esposto al suolo	XC2, XD3, XF4, XA1	50
Pilastri	XC2, XA1	75

Tendini di precompressione

Le seguenti caratteristiche del materiale sono state considerate in accordo con la letteratura dei produttori e la prEN 10138-3.

- Grado: 1.770 Mpa
- Diametro nominale: 15,7 mm
- Carico di rottura garantito: 265 kN

Fattori parziali

Design situation	γ_c – concrete	γ_s – reinforcing steel
ULS – Persistent and transient	1.50	1.15
Accidental – Non-fire	1.20	1.00
Accidental – Fire	1.00	1.00
SLS	1.00	1.00

Figura 21 - Fattore parziale alla resistenza di progetto del calcestruzzo e dell'acciaio

resistance of cross-sections $\gamma_{M0} = 1.00$
resistance of member to instability $\gamma_{M1} = 1.0$
resistance of cross-section to fracture $\gamma_{M2} = 1.10$

Figura 22 - Fattore parziale alla resistenza di progetto della sezione d'acciaio

Deflessioni

I seguenti limiti di deflessione sono stati considerati per i carichi imposti in conformità con i valori indicati nell' EC0. Durante la prossima fase del progetto questi dovranno essere rivisti ulteriormente in relazione ai requisiti del design del rivestimento e alle selezioni del sistema.

Esercizio

Deflessioni orizzontali

	Massime deformazioni ammissibili Δw_h
SLE - quasi-permanente, Trave semplice	Luce / 250
SLE- quasi-permanente, Trave a sbalzo	Luce / 180
SLE- Solo carico dinamico, Trave semplice	Luce / 500
SLE- Solo carico dinamico, Trave a sbalzo	Luce / 360

Le raccomandazioni del Centre for Window and Cladding Technology (CWCT) sono state riviste anche per gli elementi strutturali che sostengono le facciate e i rivestimenti.

Deflessioni verticali

	Massime deformazioni ammissibili Δw_v
SLE - quasi-permanente, Trave semplice	height / 300

Sismica

Limiti di deviazioni interpiano

Per le verifiche sismiche SLS, si assume che gli elementi non strutturali siano fissati in modo da non interferire con le deformazioni strutturali.

$$d_r \leq \frac{h}{k \times v}$$

Con:
d_r : *deriva interpiano di progetto*
v : *fattore di riduzione associato al requisito di limitazione del danno*
h : *altezza del piano*

Stato limite di danno - k	**100**
Occupazione immediata (SLD) – v	0.40
Operativo (SLO) - v	0.40

Vibrazioni

In generale, la valutazione delle vibrazioni dei pavimenti riguarda principalmente il comfort degli utenti, anche se le vibrazioni possono causare danni minori come le crepe e possono colpire le attrezzature sensibili.

Vibrazioni indotte dal calpestio

I limiti di progettazione per le vibrazioni sono tipicamente espressi come un fattore di risposta (RF), che è il rapporto tra l'ampiezza effettiva delle vibrazioni e quella del livello minimo che è tipicamente percepibile da un essere umano. Un fattore di risposta inferiore a 1 indica vibrazioni che non possono essere tipicamente percepite.

In accordo con l'attuale pratica standard del settore e con riferimento alla Steel Construction Institute (SCI) Design Guide P354, i seguenti limiti di RF sono proposti al fine di assicurare una "bassa probabilità" di commenti negativi da parte degli utenti e prevenire danni alla struttura.

Area	RF max. ammissibile
Solaio	8
Scale (aree di circolazione primaria / aree di uso leggero)	24 / 32

I fattori considerati nella valutazione di un appropriato RF della vibrazione indotta includono:

- La frequenza con cui la vibrazione si verifica - È più probabile che vibrazioni frequenti provochino un commento negativo per l'utente rispetto a vibrazioni della stessa entità che si verificano raramente.
- La posizione dei percorsi pedonali - I percorsi vicino ai supporti e alle parti più rigide della struttura risulteranno in una risposta minore rispetto a percorsi simili al centro di una campata del pavimento.

Considerazioni chiave per il cliente

I limiti delle vibrazioni ai piani si basano sul raggiungimento di una bassa probabilità di un riscontro negativo da parte degli utenti. Le vibrazioni delle strutture non possono sempre essere completamente evitate senza impatti significativi sul design strutturale e potenzialmente sul layout architettonico. Si dovrebbe considerare quanto segue:

- L'aumento del traffico pedonale in particolari momenti della giornata può portare a un aumento temporaneo delle vibrazioni percepite;
- Gruppi di persone che camminano insieme, in particolare persone che camminano in sincronia tra loro, possono causare una maggiore risposta alle vibrazioni;
- La percezione delle vibrazioni è molto soggettiva e ha effetti diversi su persone diverse, alcune persone inevitabilmente risponderanno più negativamente di altre alle vibrazioni;

Queste considerazioni dovrebbero essere discusse e i limiti di progettazione concordati all'inizio della fase 3.

Vibrazioni indotte degli impianti

Si presume che qualunque impianto o rete di servizi che causa vibrazioni sarà sostenuta su supporti isolanti che impediscono alle vibrazioni di essere trasmesse al resto della struttura.

Riassunto del Design Strutturale

Piano terra

Il riassunto del progetto della colonna è fornito qui sotto. Il pacchetto di calcoli dettagliato si trova nell'appendice C.

Disposizione e dimensionamento tipici

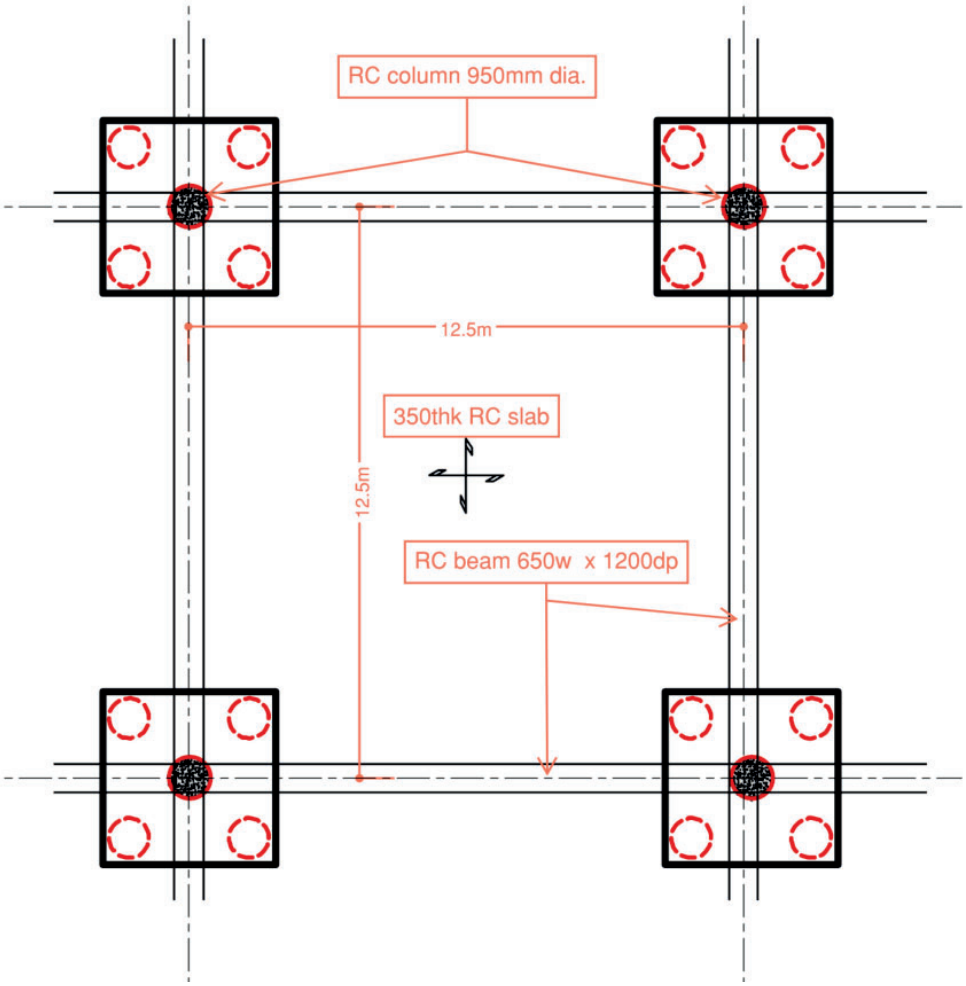
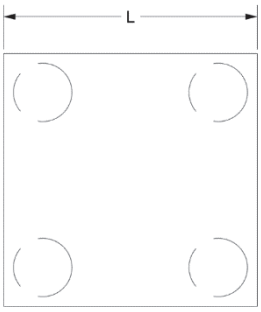
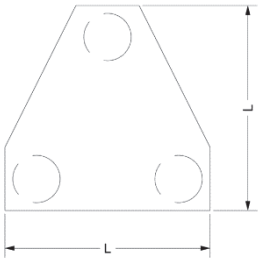
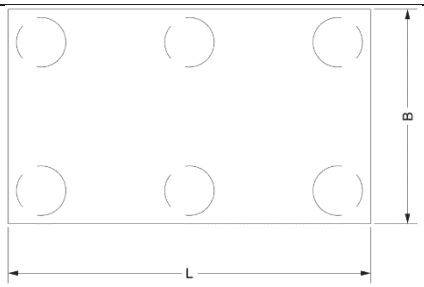
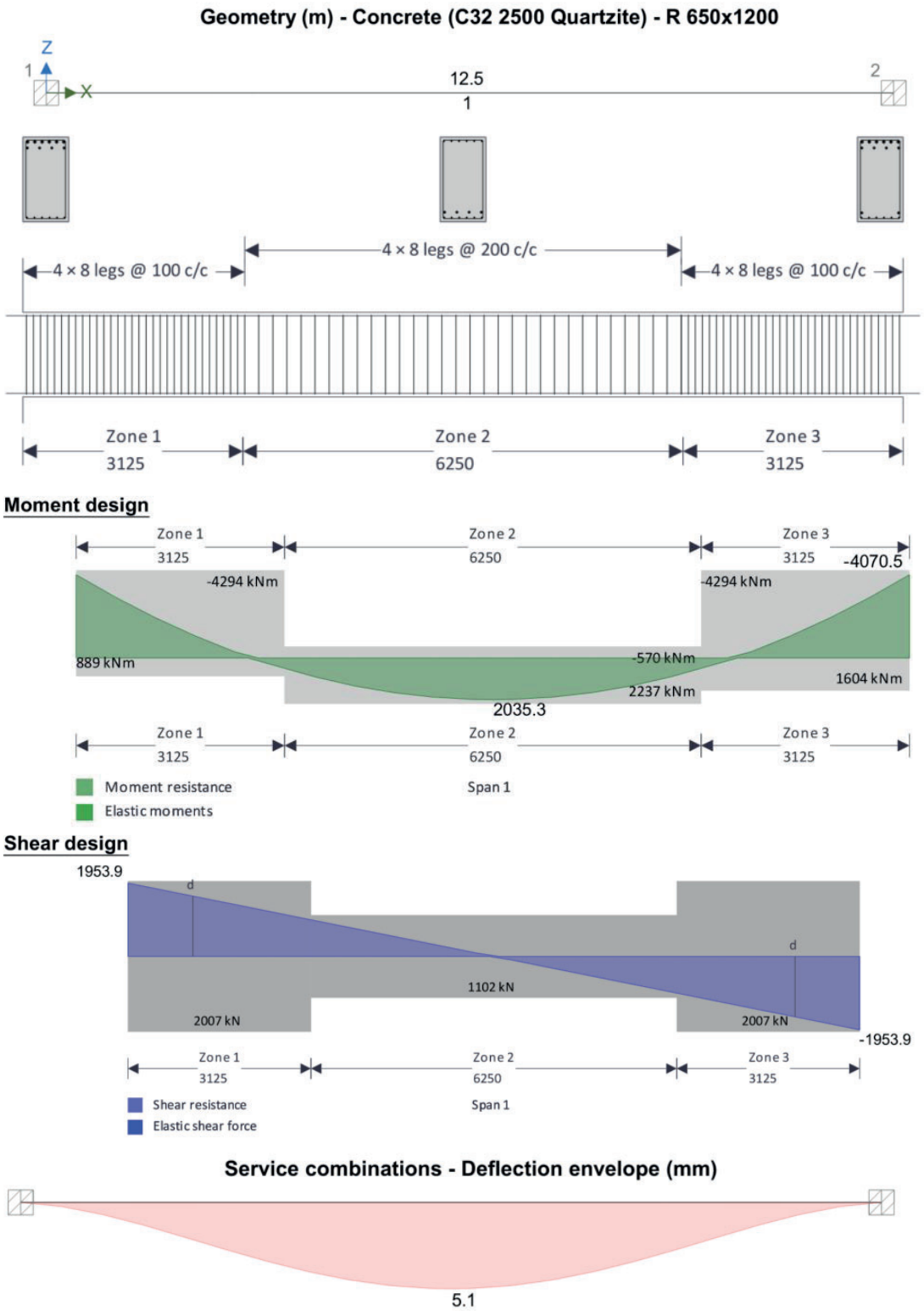


Figura 15: Progetto delle colonne

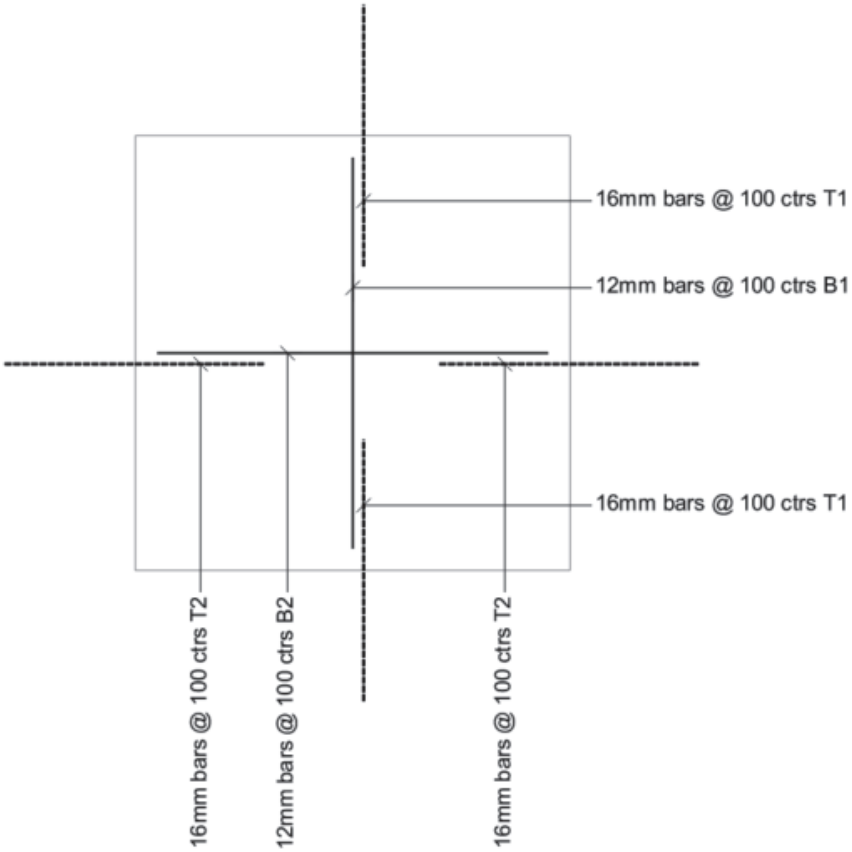
Plinti di fondazione

	TIPO A 1no. Dia. 900mm palo in CA (Calcestruzzo Armato) Spessore = 1200mm L = 1200mm	54 No. (54 No. Pali)
	TIPO B 2no. Dia. 900mm palo in CA Spessore = 1500mm L = 3900mm B = 1200mm	59 No. (118 No. Pali)
	*TIPO C* 4no. Dia. 900mm palo in CA Spessore = 2200mm L = 3900mm L = 1200mm	71 No. (284 No. Pali)
	*TIPO D* 3no. Dia. 900mm palo in CA Spessore = 2200mm L = 3900mm	12 No. (36 No. Pali)
	*TIPO E* 6no. Dia. 900mm palo in CA Spessore = 2200mm L = 6600mm B = 3900mm	66 No. (396 No. Pali)
	TIPO F 2no. Dia. 900mm palo in CA Spessore = 1500mm L = 2700mm *B = 900mm*	56 No. (112 No. Pali)

Trave di cemento armato



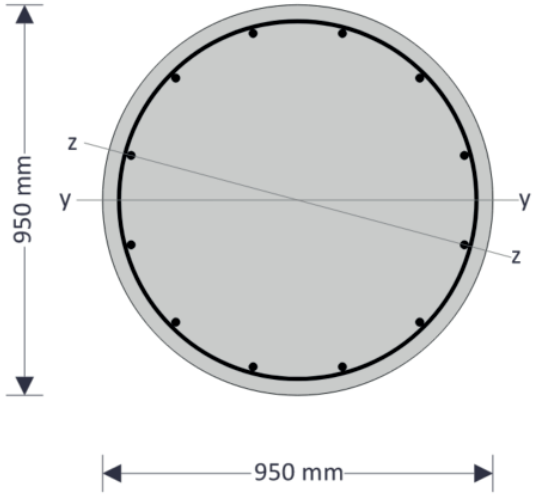
Soletta in cemento armato



Colonna in cemento armato

Colonna angolo

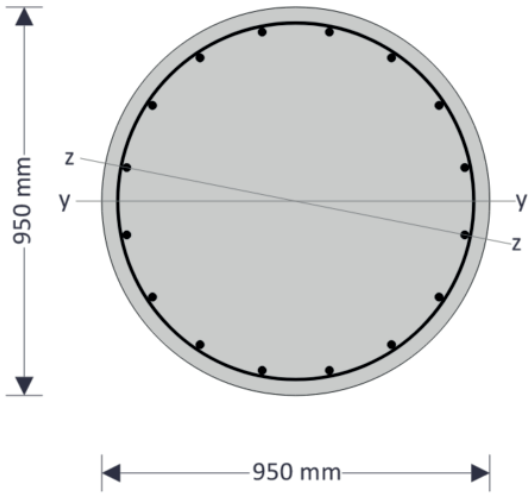
Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1096.78	186.29	0.17	PASS



12 no. 20 mm diameter longitudinal bars
10 mm diameter links
Max link spacing 400 mm generally, 240 mm for
950 mm above and below slab/beam and at laps

Colonna di bordo

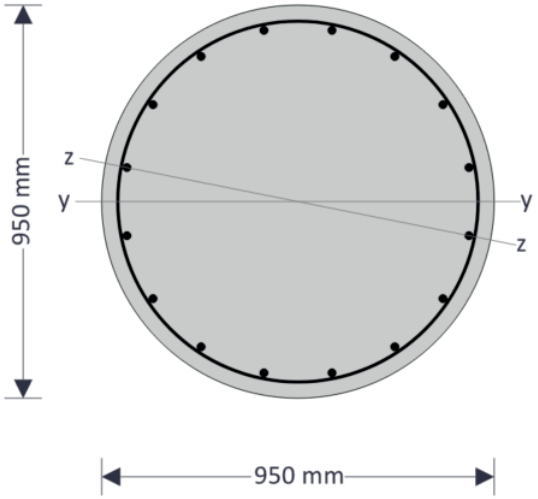
Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1486.32	335.17	0.23	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for
950 mm above and below slab/beam and at laps

Colonna centrale

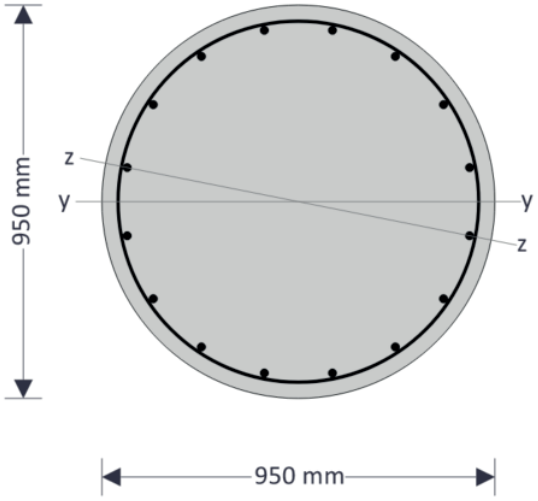
Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1604.34	1340.33	0.84	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for
950 mm above and below slab/beam and at laps

Colonna centrale con carichi di supporto

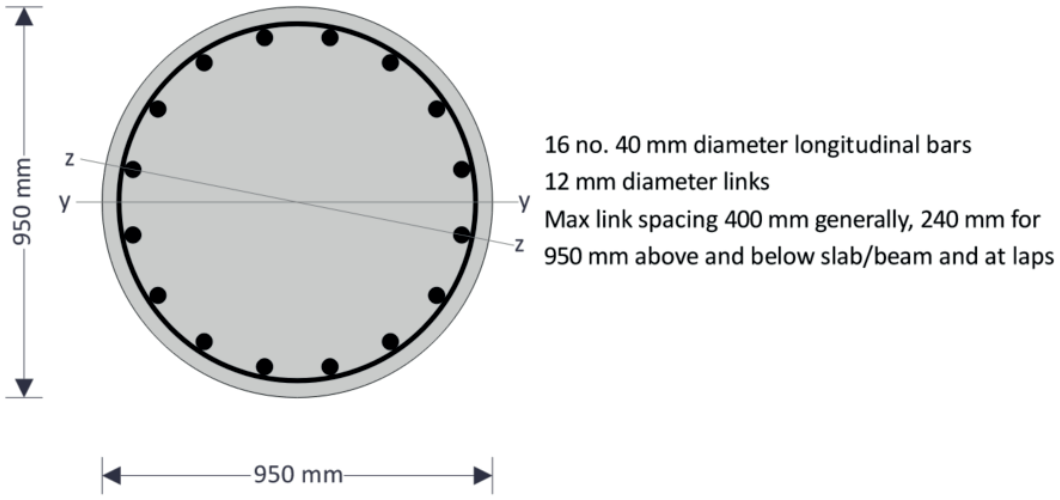
Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1569.39	1451.77	0.93	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for
950 mm above and below slab/beam and at laps

Colonna centrale con carichi di supporto di una luce di 24m

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	2383.97	2309.28	0.97	PASS



Primo piano

Il riassunto del progetto è fornito di seguito. Il pacchetto di calcoli dettagliato può essere trovato nell'appendice D.

Disposizione e dimensionamento tipici

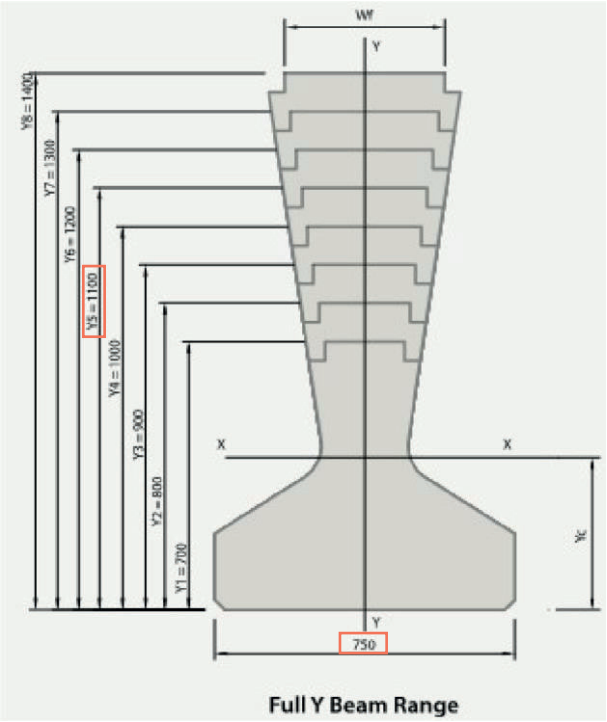
BEAM SCHEDULE	
	PRESTRESSED BEAM Y5 1100dp
	RC BEAM 500w x 1000dp
	RC BEAM 1200w x 1000dp
	PT RC BEAM 1000w x 1500dp
	RC BEAM 1000w x 1500dp
	RC BEAM 1000w x 1000dp
	RC BEAM 1200w x 1500dp
	PT RC BEAM 1800w x 2000dp
	PT RC BEAM 1500w x 2700dp

HOLLOWCORE PLANKS & PRESTRESSED BRIDGE BEAMS TO BE PREFABBRICATED
ALL OTHER BEAMS TO BE CAST IN SITU

Elenco delle travi

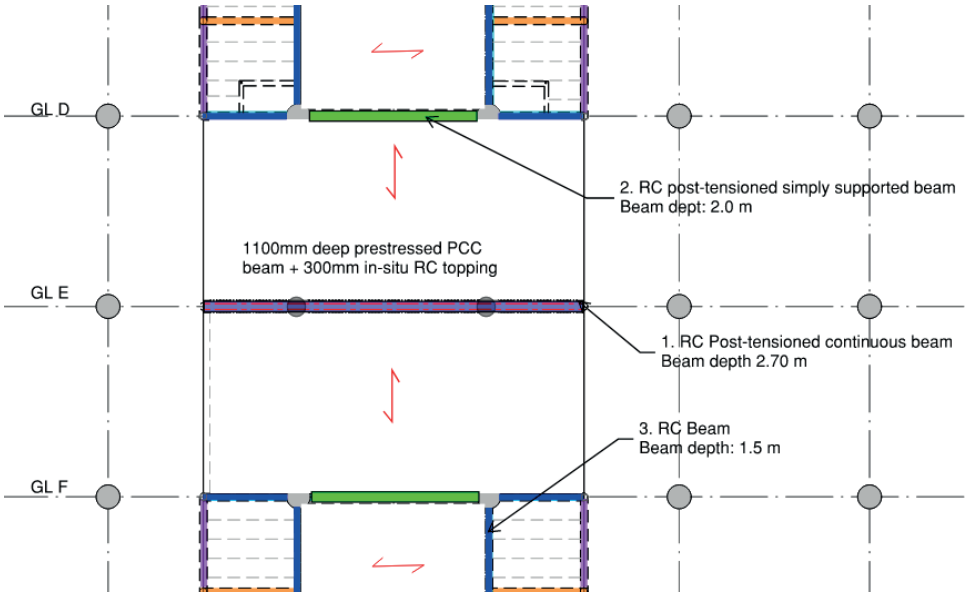
Trave prefabbricata

La trave nella figura è stata selezionata da un catalogo standard per sezioni di travi da ponte, secondo il criterio della luce necessaria e del carico supportabile.

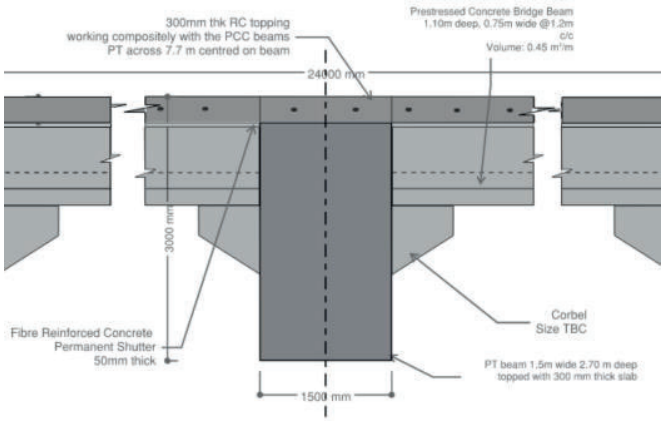


Travi di sostegno per le travi prefabbricate da ponte

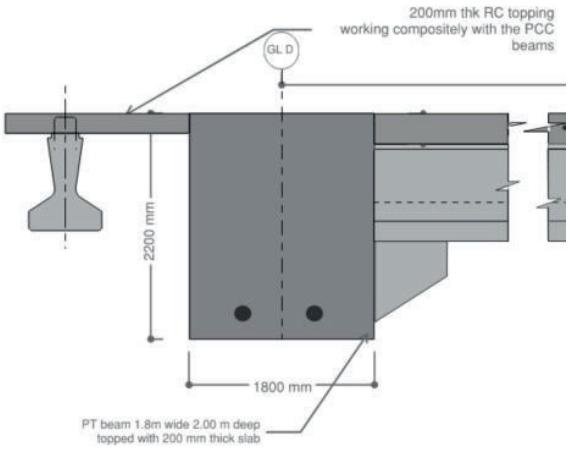
Per adattarsi a una serie di layout preservando un efficiente uso di materiale, tre tipologie travi di sostegno sono state progettate a seconda delle condizioni di supporto ammissibili e dei carichi previsti.



1. Trave centrale (arancione): è la trave più oberata di carichi in quanto ha una larghezza tributaria di 24 m. É supportata da due Megacolonne centrali, la trave avrà un grado di continuità attraverso la soletta di 300 mm, ma per conservatività è stata progettata come due mensole di 12 metri e la parte semplicemente supportata di 24 metri. La trave necessita di post-tensionamento.



2. Travi laterali (in verde): anche queste necessitano di post-tensionamento ed hanno una luce di 24 m, tuttavia si è dato priorità ad avere una trave meno profonda e più larga per non ridurre l'altezza dedita ai servizi. Si presume che non necessitano di una connessione rigida.



3. Travi Livello 1 al coperto (blu): queste travi hanno una luce di 12 metri e supportano in parte le travi da ponte ed in parte il solaio ordinario. Saranno pesantemente rinforzate ma un design preliminare indica che possono essere semplicemente supportate dalle colonne.

Piano intermedio

Il riassunto del progetto è fornito di seguito. Il pacchetto di calcoli dettagliato può essere trovato nell'Appendice E.

Definizione del Modello

Parte del solaio è stata modellata nel modello EF per comprendere il comportamento del solaio composito. Ai fini dei controlli di vibrazione, i supporti sono stati considerati fissi, poiché sotto la forza di eccitazione, la rigidità delle connessioni può essere considerata come connessioni fisse.

L'immagine sotto mostra la parte del solaio modellata e comprende il solaio lungo 24 m senza supporti intermedi.

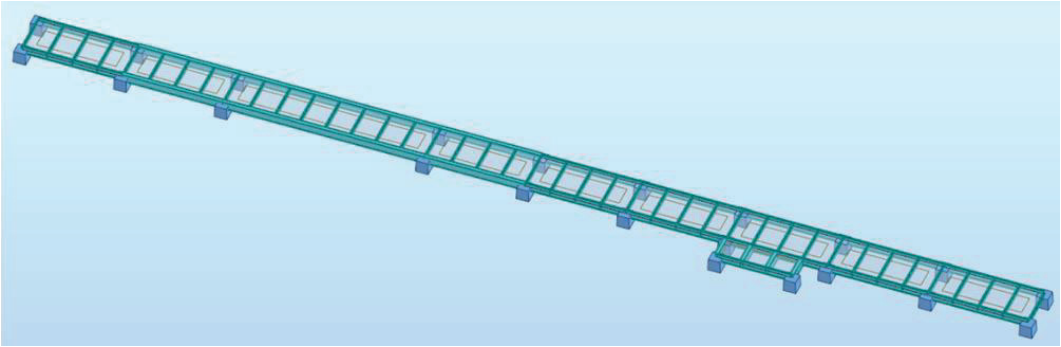


Figura 16: Modello di analisi del ponte del mezzanino

Parte del solaio è stata modellata nel modello EF per comprendere il comportamento del solaio composito. Ai fini dei controlli di vibrazione, i supporti sono stati considerati fissi, poiché sotto la forza di eccitazione, la rigidità delle connessioni può essere considerata come connessioni fisse.

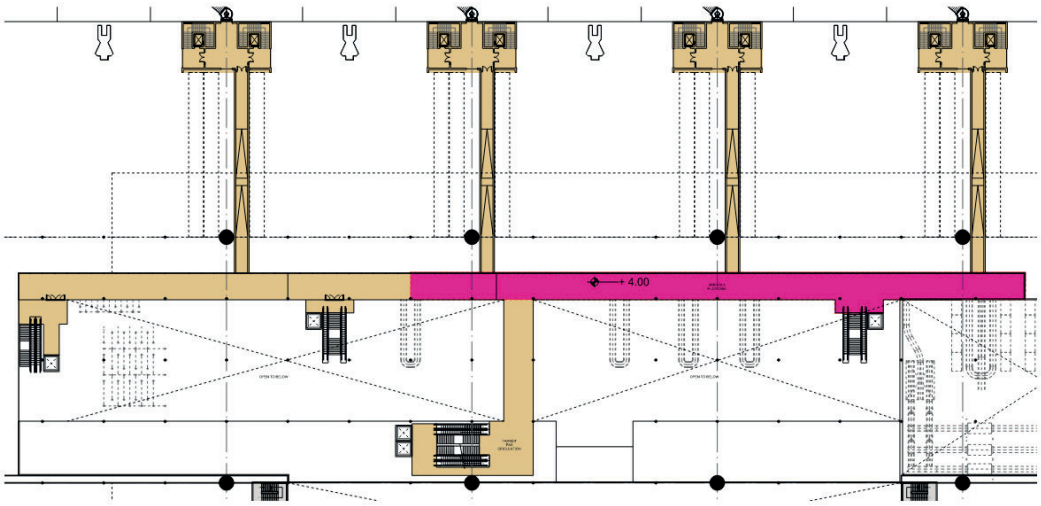


Figura 17: Schizzo raffigurante la parte del ponte modellata e comprende il pavimento lungo 24 m senza alcun supporto intermedio.

Deflessioni

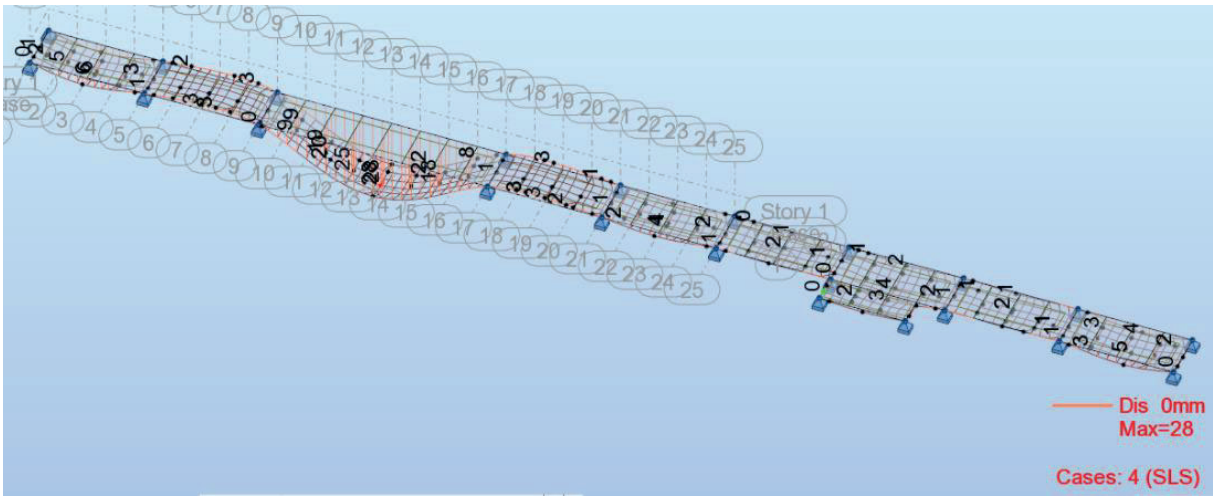
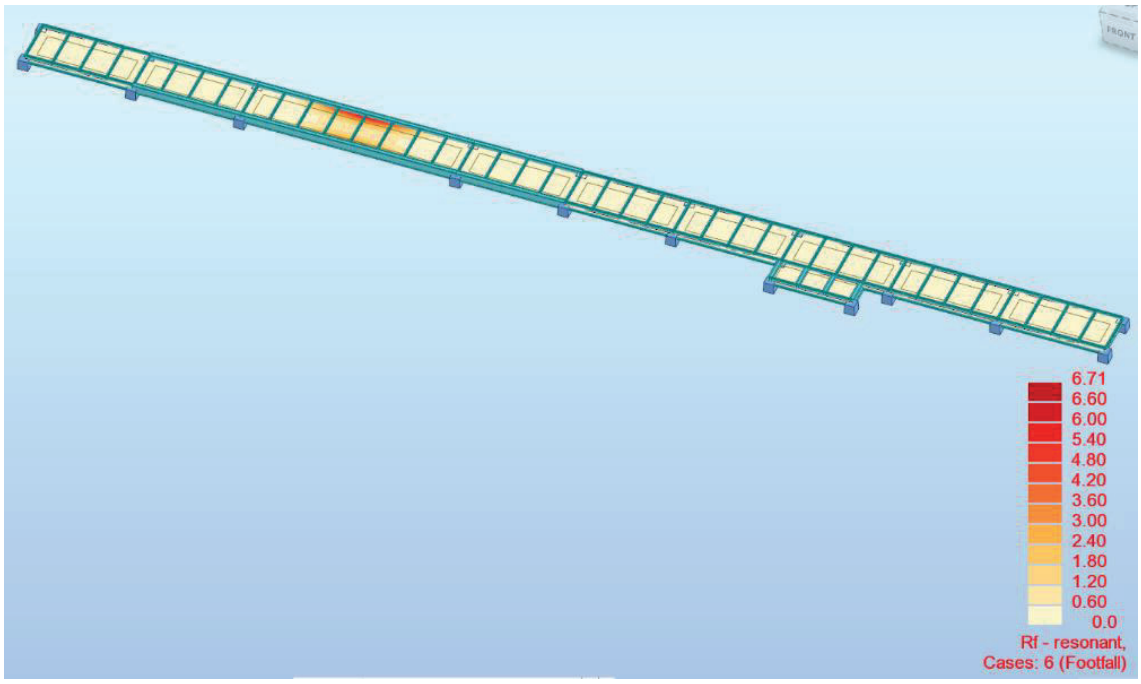


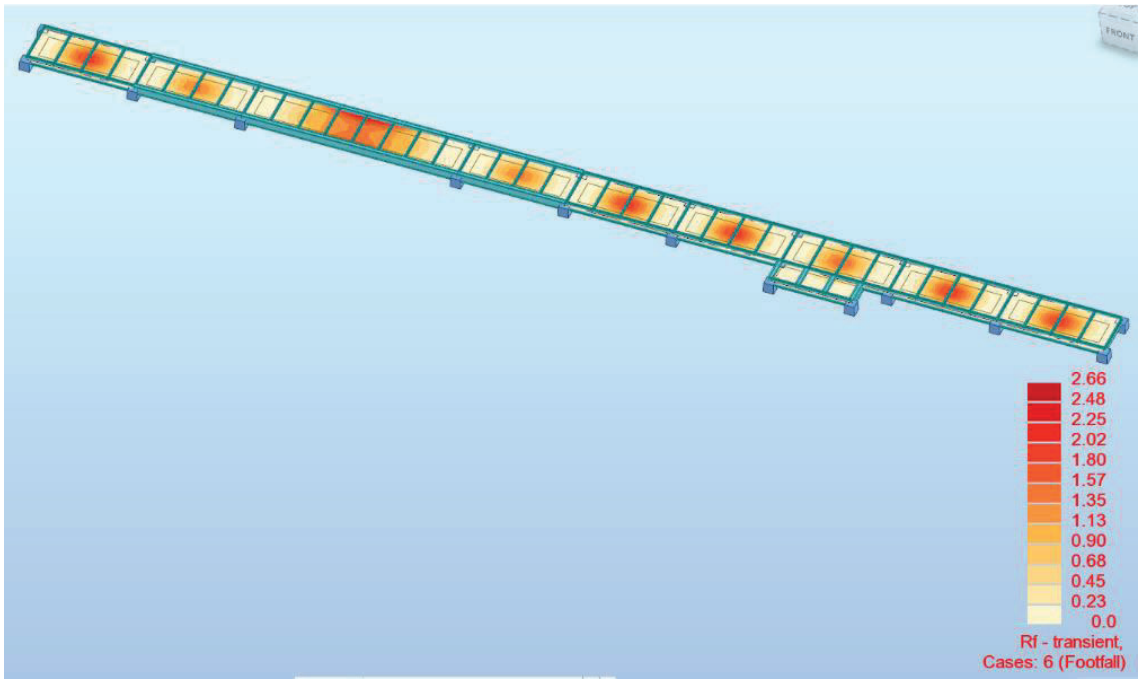
Figura 18: Deflessioni

Comportamento Vibrazionale

Risposta di Risonanza



Risposta ai Transienti



Tetto Piano Intermedio

Posteriormente agli edifici laterali si trova una struttura a telaio a portale di un piano, il cui tetto arriva al livello del mezzanino ed è composto, come il tetto degli edifici laterali, da travi d'acciaio che sostengono un solaio metallico.

Questa struttura, oltre ad essere un'estensione del terminal, è stata progettata per supportare le unità di HVAC, le quali sono invece sospese dal primo piano nella parte centrale dell'edificio.

La tipologia di travi selezionata è indicata nella figura di seguito e comporta il rispetto del criterio L/360 a mezza spanna.

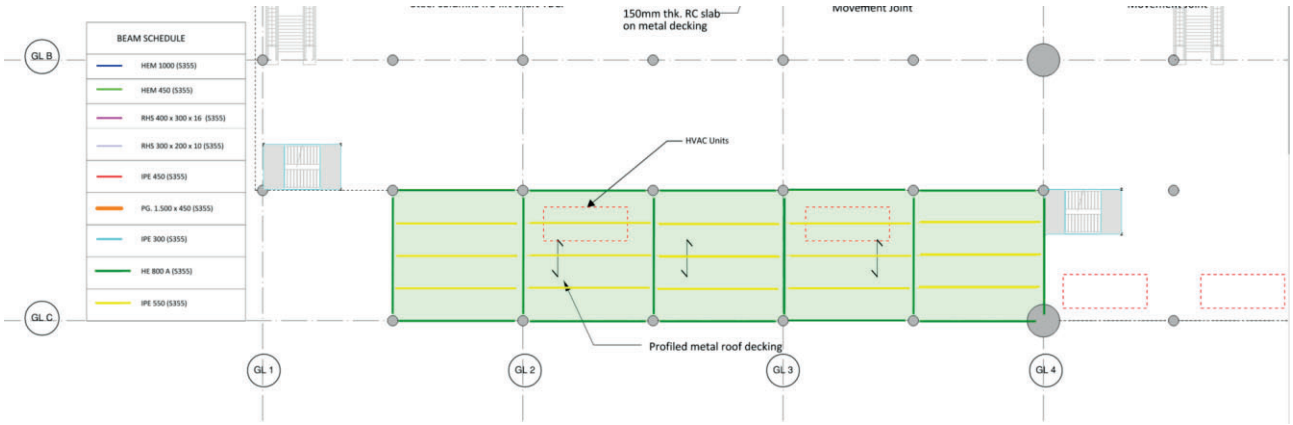


Figura 19: individuazione delle travi

Ponte di imbarco

Il riassunto della progettazione del ponte d'imbarco è fornito qui di seguito. Il pacchetto di calcoli dettagliato può essere trovato nell'Appendice F.

Definizione del modello

I ponti d'imbarco per gli aeroporti sono tipicamente costruiti come ponti a capriate in quanto è una soluzione efficace per lunghe campate che sono in grado di affrontare vibrazioni e deflessioni. Come tale, il ponte tipico per il nuovo terminal è stato modellato come un telaio a capriate. Il tipico sistema di ponti per il nuovo terminal dell'aeroporto di Firenze comprende tre ponti di varie dimensioni e lunghezze.

- Il ponte 1 è un ponte a campata doppia di lunghezza approssimativa di 33m.
- Il ponte 2 è un ponte a campata singola di lunghezza approssimativa di 39m.
- Il ponte 3 è un ponte a campata singola di lunghezza approssimativa di 33m.

Questa sezione riporta il comportamento del ponte sotto i requisiti di servizio e il comportamento dinamico sotto il calpestio dei pedoni. Si analizzano anche le dimensioni delle membrature strutturali.

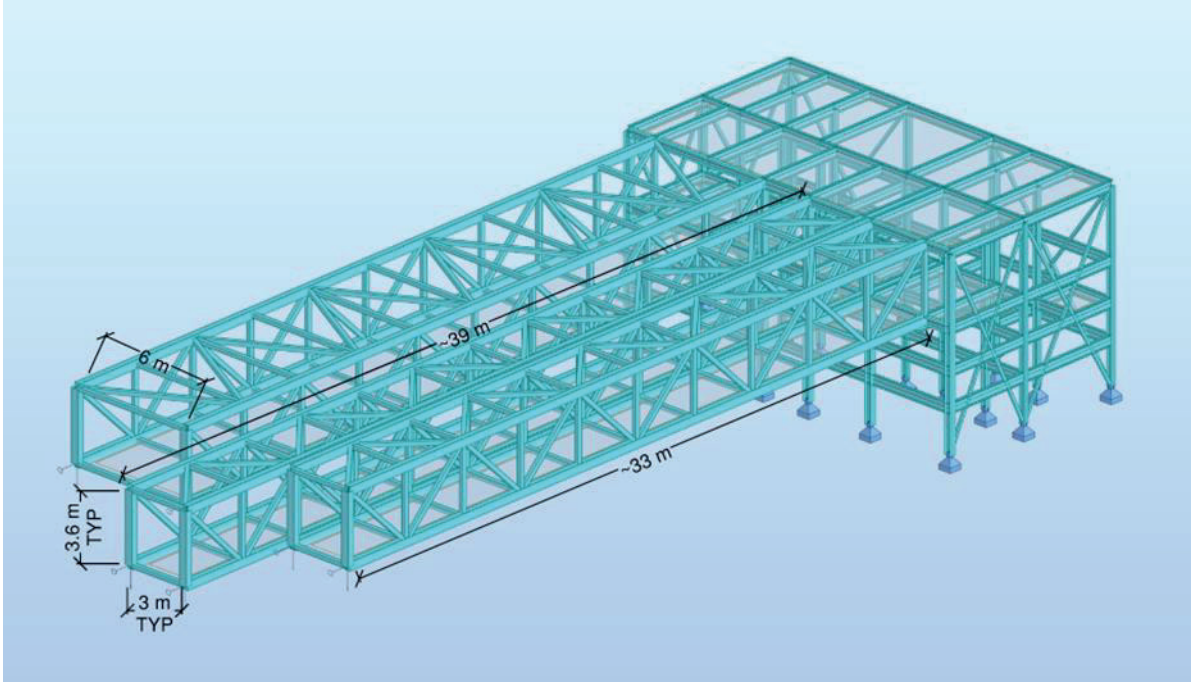


Figura 20: modello 3D dei ponti

Le condizioni di supporto all'estremità del salone di partenza dei ponti richiederanno probabilmente un supporto a rullo o fori asolati a causa di considerazioni sul carico sismico e del vento. Parte dei carichi derivanti dal peso e dall'uso dei ponti saranno sostenuti dalla profonda trave di bordo in calcestruzzo di 12,5 m che si estende tra le colonne circolari.

Analisi Dinamica

Le indicazioni per l'analisi dinamica di questi ponti sono state tratte dalle seguenti fonti:

- HIVoSS: *Design of Footbridges Guidline*
- CCIP 016: *A Design Guide for Footfall Induced Vibration of Structures*
- AISC DG11: *Floor Vibrations Due to Human Activity*
- SCI P354: *Design of Floors for Vibration: A New Approach*

HiVoSS delinea un metodo per progettare una passerella per soddisfare i criteri di comfort, mostrati nel seguente diagramma di flusso:

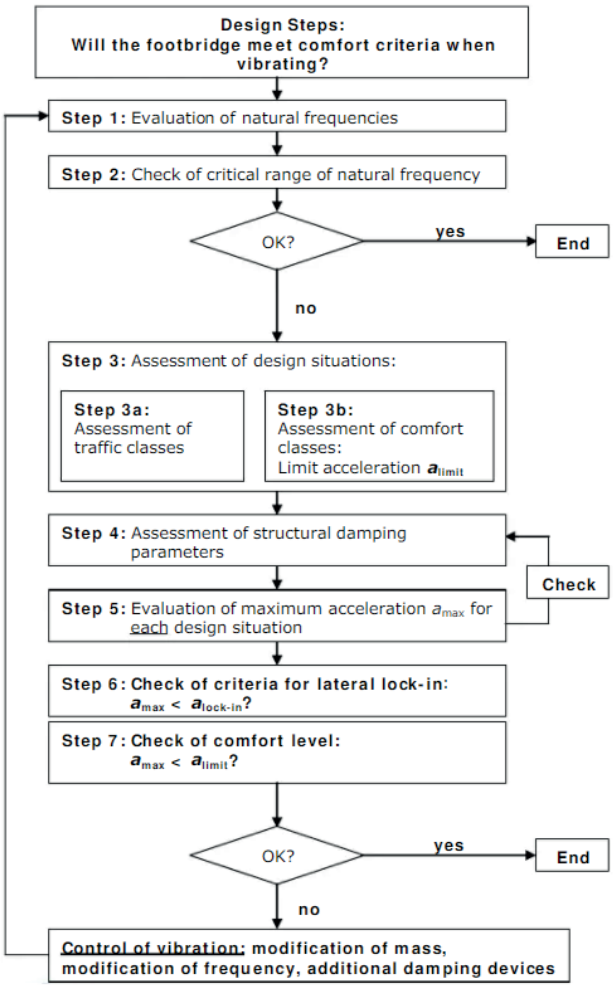


Figura 21:HiVoSS diagramma di flusso per il design delle passerelle secondo criteri di comfort

Per analizzare il comportamento dinamico dei ponti è stato necessario analizzare il comportamento dinamico di ogni ponte separatamente usando il metodo mostrato sopra. Il metodo degli elementi finiti è stato utilizzato per generare un'analisi modale che mostra le frequenze del ponte che causerebbero l'eccitazione. Di seguito è riportata una tabella riassuntiva dei parametri implementati per l'analisi del modello ad elementi finiti.

Tabella 0-1: Parametri di analisi dinamica del ponte

Parametro	Valore	Fonte
Rapporto di smorzamento	0.3%	HiVoSS
Gamma di frequenza di camminata	1.8 Hz to 2.2 Hz	SCI P354
Modulo d'elasticitá del Calcestruzzo	38 kN/mm2	SCI P354
Densitá del Calcestruzzo	2350 kg/m3	SCI P354

Ponte 1

Questo ponte a doppia campata ha una frequenza di 4.99Hz nella prima armonica che è una vibrazione verticale.

Dynamic Analysis Results - Case: 16 (Modal) Active modes: 1..50, CQC

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Cur.mas.UZ (%)	Total mass UX (kg)	Tota
16/ 1	4.99	0.20	0.00	0.25	78.96	0.00	0.25	78.96	97881.46	
16/ 2	8.94	0.11	0.08	0.25	78.96	0.08	0.00	0.00	97881.46	
16/ 3	9.72	0.10	0.08	0.29	78.97	0.00	0.04	0.01	97881.46	
16/ 4	11.62	0.09	0.08	0.29	85.43	0.00	0.00	6.47	97881.46	
16/ 5	12.08	0.08	55.93	0.29	85.44	55.85	0.00	0.00	97881.46	
16/ 6	13.06	0.08	55.93	0.29	85.44	0.00	0.00	0.01	97881.46	

Figura 22:Frequenze naturali del ponte a doppia campata

HiVoSS fornisce una guida sulla gamma critica di frequenze naturali per le passerelle.

4.2 Step 2: Check of critical range of natural frequencies

The critical ranges for natural frequencies f_i of footbridges with pedestrian excitation are:

- for vertical and longitudinal vibrations:

$$1{,}25\ \text{Hz} \leq f_i \leq 2{,}3\ \text{Hz}$$

- for lateral vibrations: $0{,}5\ \text{Hz} \leq f_i \leq 1{,}2\ \text{Hz}$

Footbridges with frequencies for vertical or longitudinal vibrations of

$$2{,}5\ \text{Hz} \leq f_i \leq 4{,}6\ \text{Hz}$$

might be excited to resonance by the 2nd harmonic of pedestrian loads [1]. In that case, the critical frequency range for vertical and longitudinal vibrations expands to:

$$1{,}25\text{Hz} \leq f_i \leq 4{,}6\text{Hz}$$

Lateral vibrations are not effected by the 2nd harmonic of pedestrian loads.

Note: A vertical vibration excitation by the second harmonic of pedestrian forces might take place. Until now there is no hint in the literature that onerous vibration of footbridges due to the second harmonic of pedestrians have occurred.

Figura 23:HiVoSS gamma critica di frequenze naturali

Il capitolo 4.2 della guida HiVoSS mostra che per le vibrazioni verticali l'intervallo critico per le frequenze naturali è 1,25Hz ≤ fi ≥ 4,6Hz. La più piccola frequenza naturale per il doppio ponte di 4,99Hz si trova al di fuori dell'intervallo critico, il che significa che nella sua configurazione attuale è improbabile che causi problemi vibrazionali alla struttura.

Il passo successivo è quello di valutare il ponte per il comfort. Per questo, è stata usata un'analisi del calpestio per calcolare l'accelerazione massima, alimit causata dal calpestio dei pedoni, così come il fattore di risposta, R.

La Tabella 4-2 nel documento guida HiVoSS descrive diverse classi di comfort a seconda della situazione di progettazione..

Table 4-2: Example of a specification of significant design situations

Design Situation	Description	Traffic Class (cf. 0)	Expected occurrence	Comfort Class (cf. 4.3.2)
1	inauguration of bridge	TC4	once in the lifetime	CL3
2	commuter traffic	TC2	daily	CL1
3	rambler at the weekend	TC1	weekly	CL2
⋮	⋮	⋮	⋮	⋮

HiVoSS esempio di situazioni per il design

La situazione di progetto più legata all'uso di un ponte d'imbarco in aeroporto è la situazione di progetto 2, che è descritta per richiedere una classe di comfort di CL1.

Table 4-4: Defined comfort classes with common acceleration ranges

Comfort class	Degree of comfort	Vertical a_{limit}	Lateral a_{limit}
CL 1	Maximum	< 0,50 m/s²	< 0,10 m/s²
CL 2	Medium	0,50 – 1,00 m/s²	0,10 – 0,30 m/s²
CL 3	Minimum	1,00 – 2,50 m/s²	0,30 – 0,80 m/s²
CL 4	Unacceptable discomfort	> 2,50 m/s²	> 0,80 m/s²

Note that the given acceleration ranges are just comfort criteria; lock-in criteria for horizontal vibrations are given in section 4.6.

HiVoSS classi di comfort

La Tabella 4-4 della guida HiVoSS mostra che il limite verticale per l'accelerazione è di 500mm/s^2 o 0,5m/ s^2. L'accelerazione massima di progetto per il Ponte 1, come mostrato di seguito, è di 102mm/sec^2 o 0,102m/ sec^2 che rientra nel limite raccomandato dal documento HiVoSS.

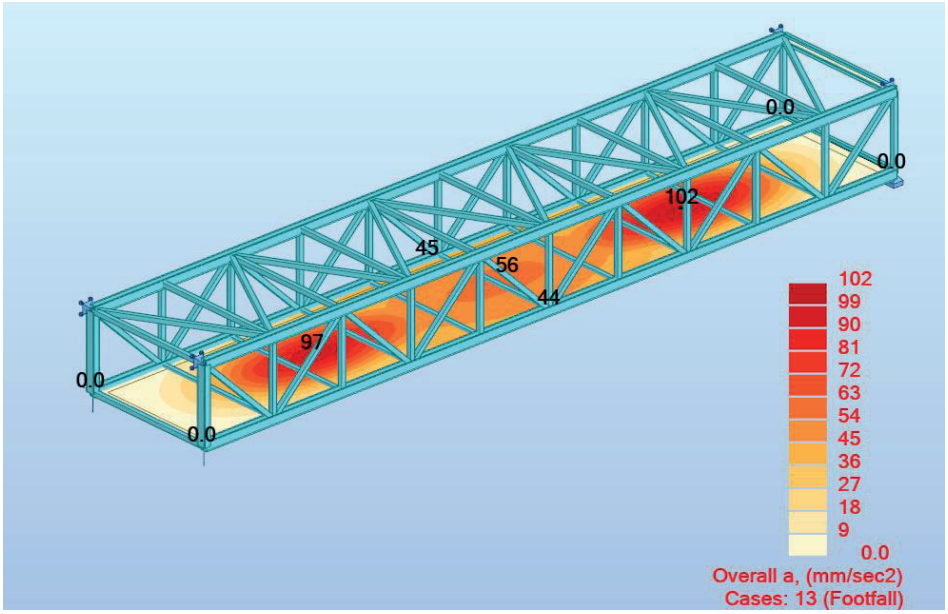


Figura 24: Ponte 1 Accelerazione massima

CCIP 016 fornisce indicazioni sulle prestazioni dinamiche accettabili per un ponte pedonale in base al fattore di risposta.

3.3.3 Performance targets for bridges, ramps and walkways

BS 5400[3] provides guidance on the acceptable levels of vertical vibration for footbridges. This is well founded for long span, low frequency (<3Hz) external footbridges. Though all footbridges should normally comply with these limits, the performance criteria given below are a refinement of these and are applicable for bridges of all spans and natural frequencies. They may also be applied to internal ramps and suspended walkways.

These criteria are for single person excitation at the most critical footfall rate.

- For external bridges generally $R < 64$.
- For indoor bridges generally $R < 32$.
- For indoor bridges that are not particularly lightweight, or are exposed, high in an atrium or heavily trafficked $R \sim< 24$.

Figura 25:CCIP 016 Limiti del Fattore di Risposta

Il fattore di risposta massimo per il ponte 1, come mostrato di seguito, è 20,35, che rientra nel limite accettabile di R<32 raccomandato da CCIP 016.

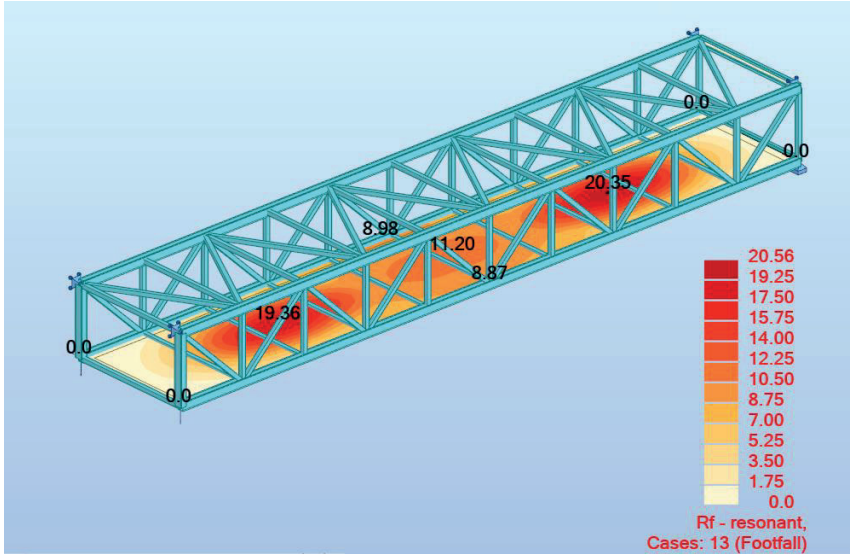


Figura 26: Ponte 1 Fattori di Risposta

Lo stesso metodo usato per il ponte 1 è stato utilizzato per analizzare il comportamento dinamico dei ponti 2 e 3. Di seguito è riportata una tabella riassuntiva dei risultati dell'analisi dinamica per i ponti da 1 a 3.

Tabella 0-2: Ponte 1 Risultati dell'analisi dinamica

Analisi Dinamica	Risultato	Limite Consigliato
Frequenza Naturale, f_i	4.99 Hz	$1.25Hz \leq f_i \geq 4.6Hz$
Accelerazione Massima, A_{limit}	0.102 m/s^2	$A_{limit} < 0.5m/s^2$
Fattore di Risposta, R	R = 20.35	R < 32

Ponte 2

Tabella 0-3: Ponte 2 Risultati dell'analisi dinamica

Analisi Dinamica	Risultato	Limite Consigliato
Frequenza Naturale, f_i	4.81 Hz	$1.25Hz \leq f_i \geq 4.6Hz$
Accelerazione Massima, A_{limit}	0.073 m/s^2	$A_{limit} < 0.5m/s^2$
Fattore di Risposta, R	R = 14.59	R < 32

Ponte 3

Tabella 0-4: Ponte 3 Risultati dell'analisi dinamica

Analisi Dinamica	Risultato	Limite Consigliato
Frequenza Naturale, f_i	5.64 Hz	$1.25Hz \leq f_i \geq 4.6Hz$
Accelerazione Massima, A_{limit}	0.035 m/s^2	$A_{limit} < 0.5m/s^2$
Fattore di Risposta, R	R = 7.07	R < 32

Deflessioni

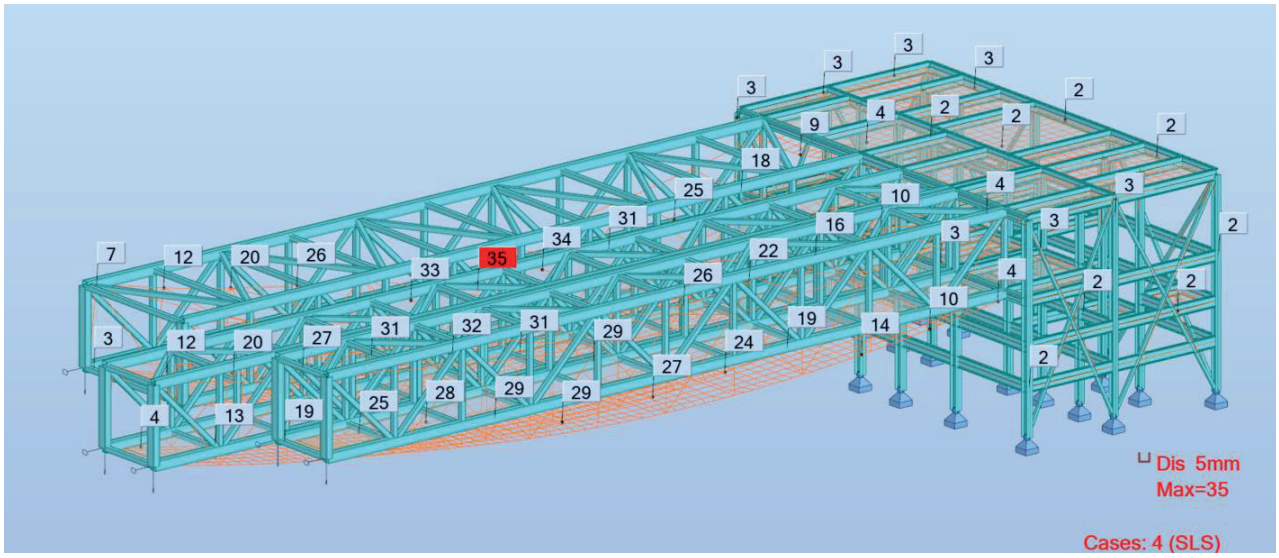


Figura 27: Deflessioni dei ponti

Con un limite di deflessione di L/250 = 132mm, la deflessione massima di 35mm dei ponti è entro limiti accettabili.

Dimensionamento finale

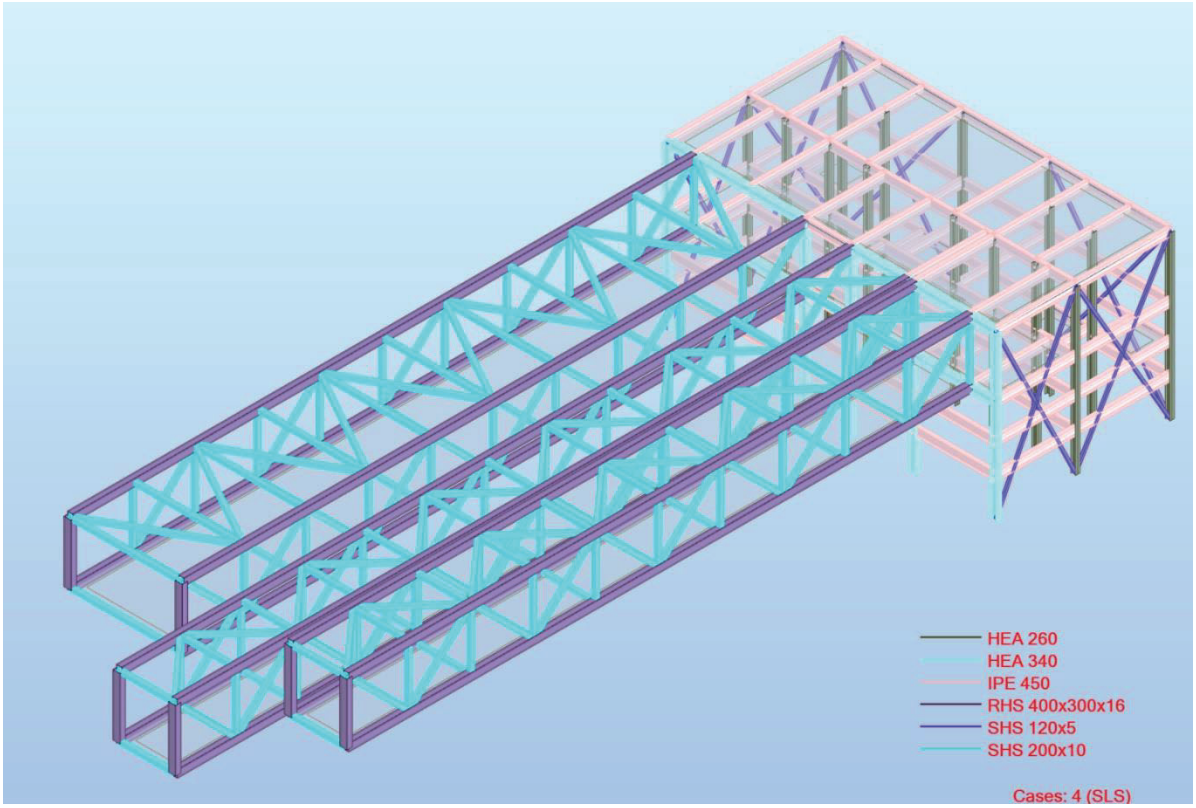


Figura 28: dimensionamento finale dei ponti

Tetto principale

Il riassunto del progetto è fornito di seguito. Il pacchetto di calcoli dettagliato può essere trovato nell'Appendice G.

Definizione del modello

Il riassunto del progetto è fornito di seguito. Il pacchetto di calcoli dettagliato può essere trovato nell'Appendice G. I dettagli dei calcoli sono forniti per gli elementi peggiori all'interno di ciascun Gruppo di Progetto. Il progetto è stato razionalizzato all'interno di un dato Gruppo di Progetto per evitare un eccesso di progettazione che porti a una sovrastima del tonnellaggio complessivo di acciaio. Questo significa che è stata cercata una conformità di progetto per il 90° percentile. Di conseguenza, si potrebbe osservare che alcuni controlli del Gruppo di Progetto non sono riusciti a raggiungere la conformità al codice, ma questo è stato accettato, comprendendo che queste membrature richiederanno una sezione più spessa, che sarà regolata nella fase di progettazione dettagliata.

Il tetto e altri elementi chiave come le Mega Colonne e la soletta di partenza sono modellati e analizzati in Robot Structural Analysis. I risultati del comportamento della struttura e delle forze sono riassunti di seguito.

Presupposti del Modello

Geometria

l modello di analisi è stato costruito per includere le travi principali del vigneto, le travi di trasferimento, le mega colonne e i rami, la soletta di partenza, il sistema di controventatura e il rivestimento. Le griglie riflettono la disposizione proposta dagli architetti, con campate di 24 metri tra le Mega Colonne.

Modello completo incluso – Travi prefabbricate delle vigne

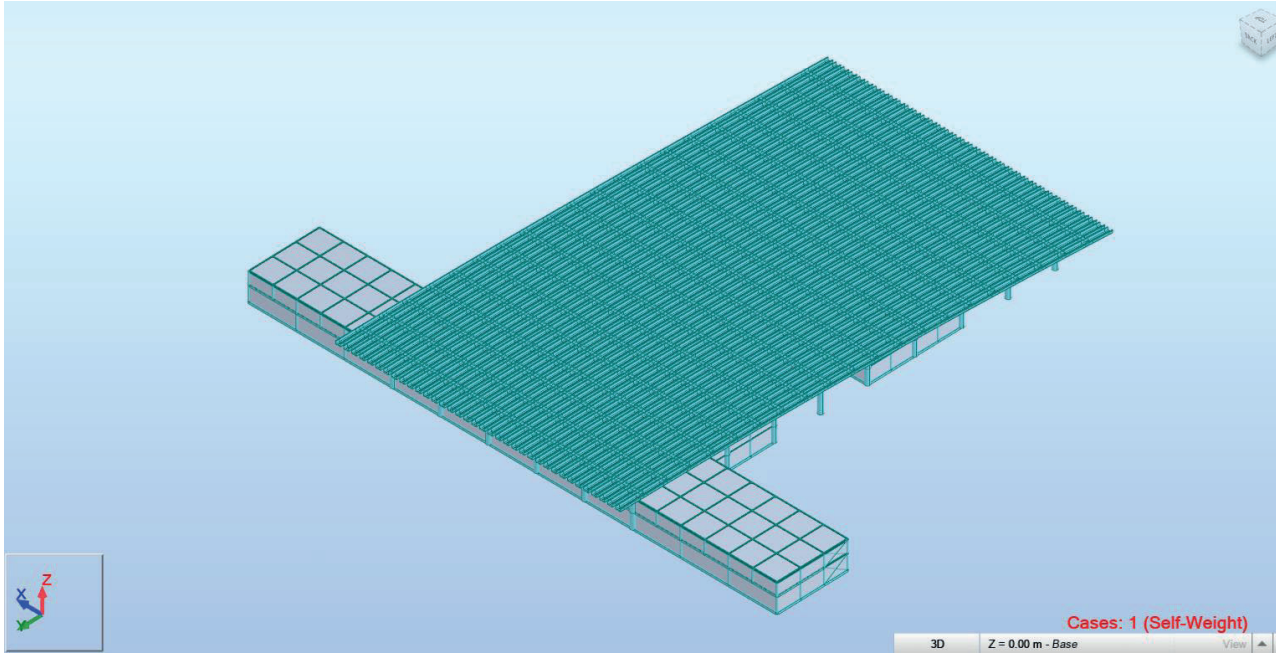


Figura 29: travi prefabbricate delle vigne

Mega colonne & Struttura secondaria del tetto

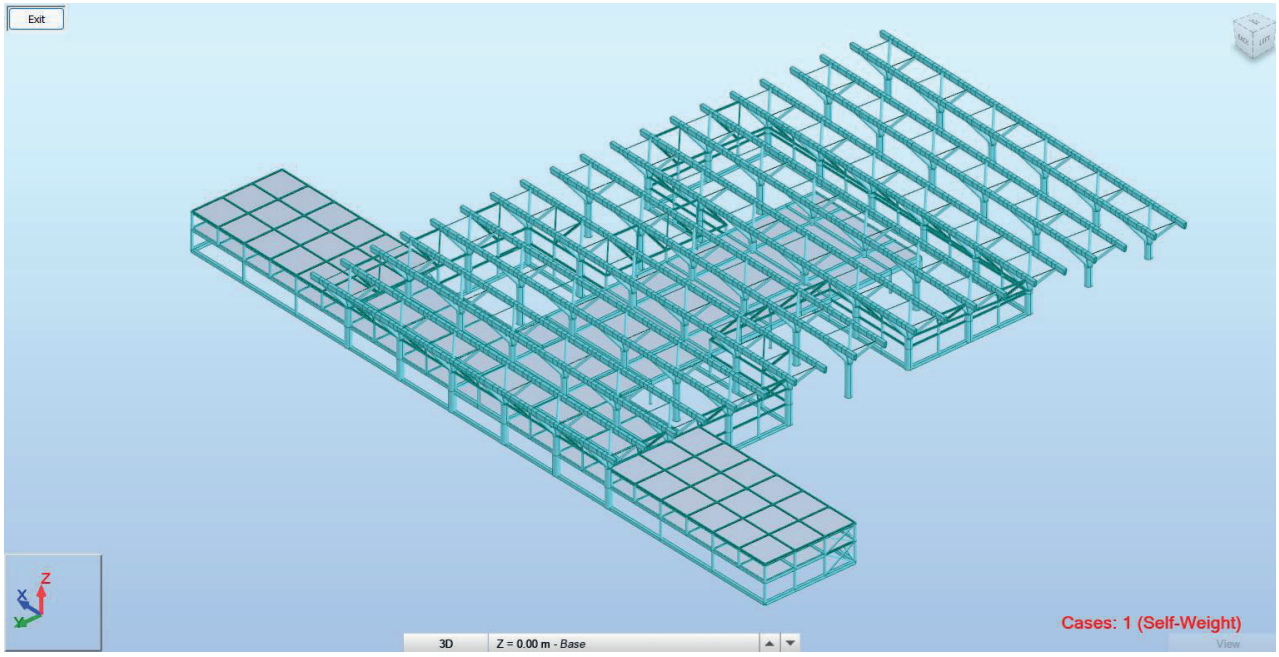


Figura 30. struttura secondaria del tetto

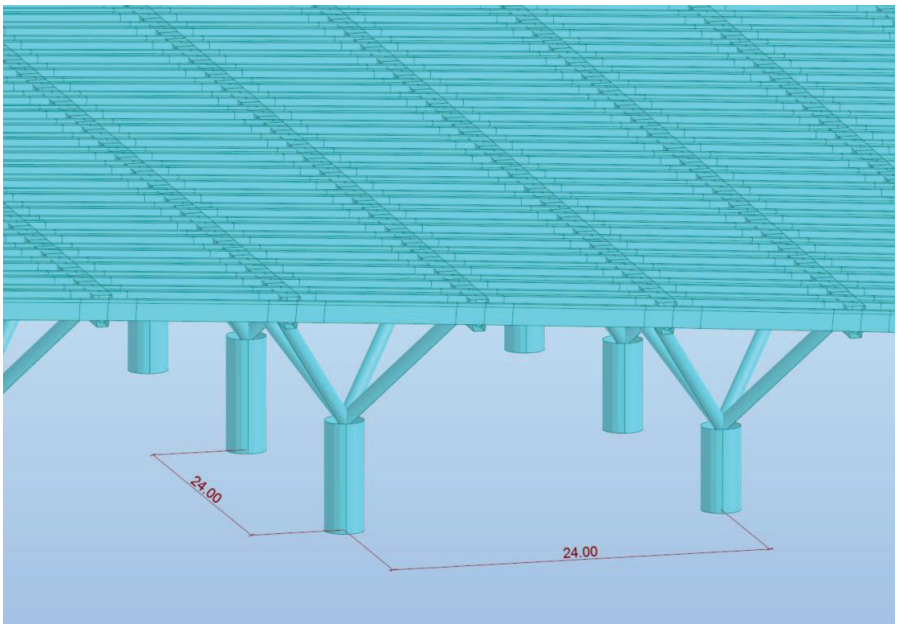


Figura 31: Mega colonne

Come mostrato nell'immagine qui sopra, per semplicità i tronchi delle Mega Colonne sono stati modellati come elementi cilindrici ma in realtà questi saranno progettati come colonne rastremate. Allo stesso modo, i rami sono stati originariamente modellati come elementi circolari, ma in realtà saranno costruiti come elementi rastremati rettangolari. Le dimensioni di questi elementi sono state ricavate da un progetto preliminare. Lo scopo principale di questo modello di analisi è quello di estrarre le forze che agiscono sui vari elementi e analizzare il comportamento della struttura in varie condizioni come sotto azioni del vento, sismiche e di temperatura. Le forze estratte da questo modello di analisi sono poi utilizzate per progettare e verificare separatamente le Mega colonne, i rami e le travi.

I pannelli della facciata sono stati modellati come elementi di rivestimento per aiutare l'analisi degli effetti dei carichi laterali del vento.

I livelli del mezzanino e del piano terra non sono stati modellati nel modello di analisi; i contributi di carico da questi piani siano stati considerati separatamente.

Gli edifici laterali a Nord-Ovest e Nord-Est sono modellati come se avessero tetti di cemento, la loro costruzione è prevista in acciaio.

Materiali

Tutti gli elementi sono modellati come calcestruzzo di C40 tranne le colonne e i rami. La tabella seguente fornisce un riassunto.

Elemento	Materiale
Travi del **vigneto**	C40
Travi di trasferimento	C40
Travi del solaio delle partenze	C40
Pilastro (0,950 m dia)	C60
Mega colonna (2.5 m dia)	C80
Rami d'albero	C80

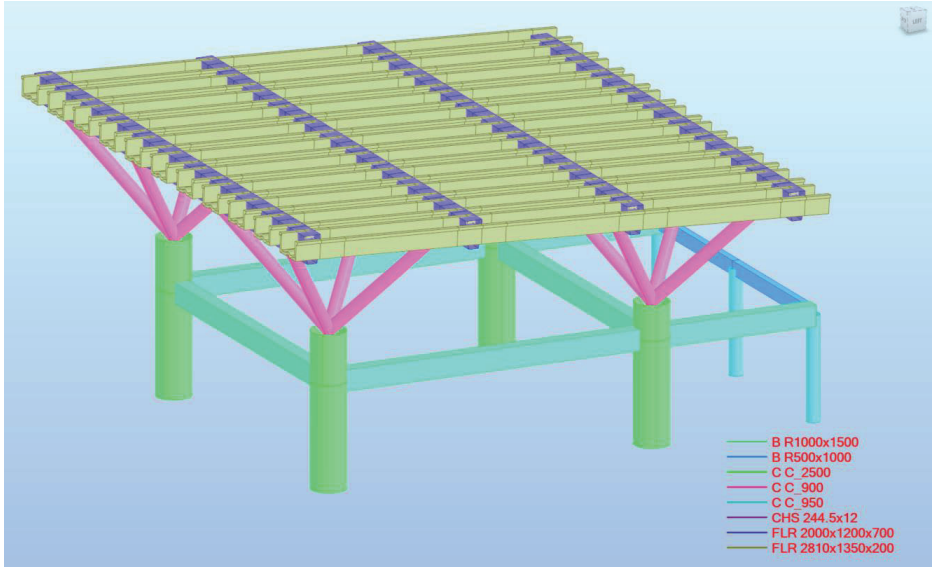


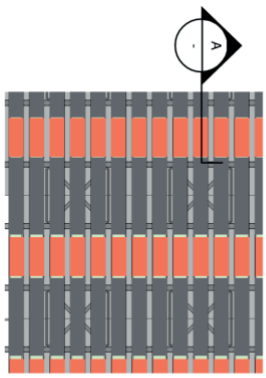
Figura 32: Modellazione degli elementi del tetto

Costruibilità

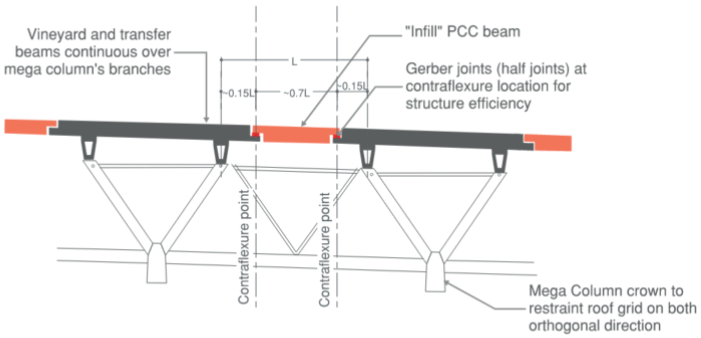
Alcune accortezze sono state implementate nel modello EF per tenere conto della sequenza di costruzione prevista del tetto: sia la trave del vigneto che la trave di trasferimento devono essere segmentati in elementi lunghi ~12m e poi sollevati in posizione e collegati agli elementi adiacenti.

Per massimizzare l'efficienza strutturale, i mezzi giunti sono stati impostati nei punti di controflessione (caso 1) eccetto per due casi particolari::

- Dove le travi hanno attraversato una linea termica, nel qual caso la giunzione è allineata alla linea termica (caso 2)
- Dove le travi sono a sbalzo e devono essere continue sulla campata posteriore (caso 3)

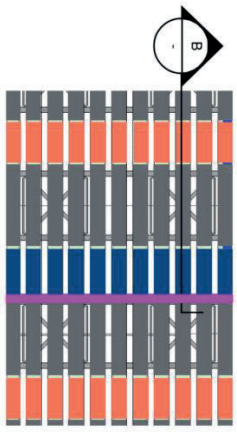


DETAIL 1 - TYPICAL CASE
NTS

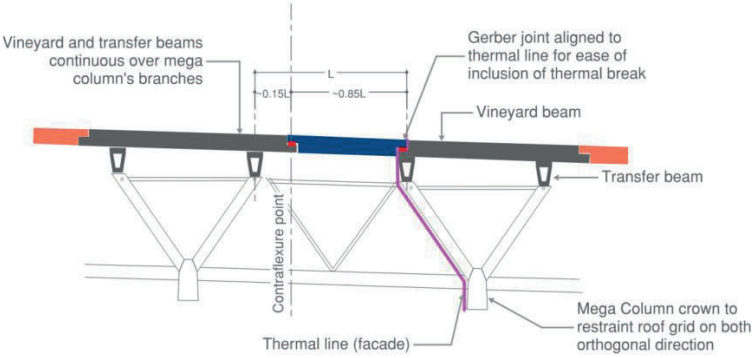


SECTION A - TYPICAL CASE
NTS

Caso 1

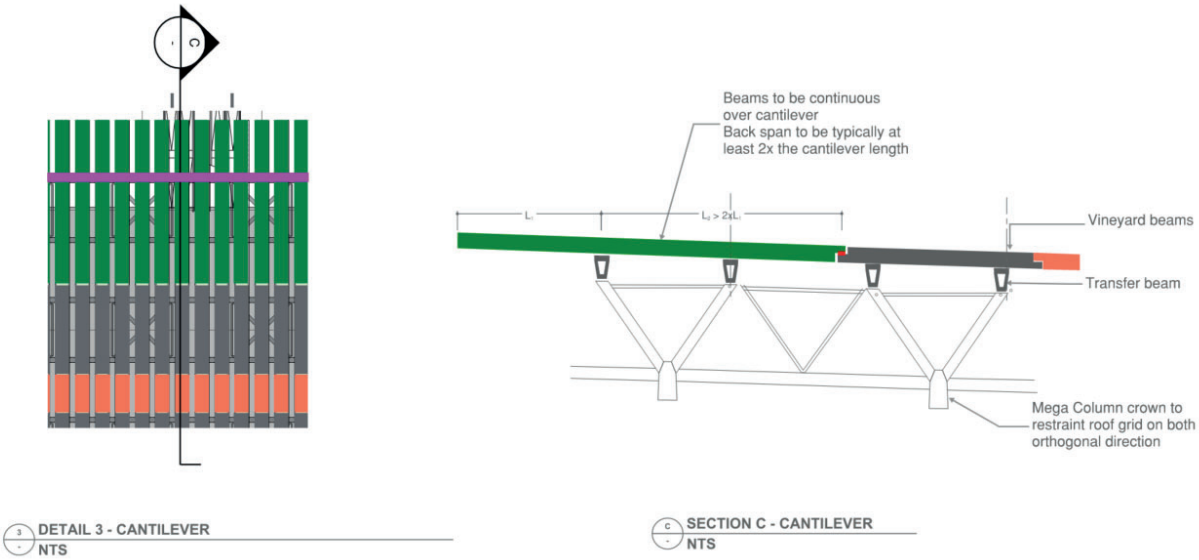


DETAIL 2 - THERMAL BREAK
NTS



SECTION B - THERMAL BREAK
NTS

Caso 2



Caso 3

Inizialmente, i giunti di costruzione sono stati modellati come mezzi giunti (semplicemente supportati, nessun trasferimento di momento flettente). Tuttavia, tali condizioni di vincolo hanno portato a spostamenti eccessivi, specialmente sotto azioni sismiche. È stato quindi scartato come strategia per la connessione delle travi nei punti di controflessione e si consiglia che al giunto tra le travi del tetto (per una cella standard) sia permesso di trasferire il momento da una trave all'altra, fornendo continuità.

Giunti di Movimento

Considerando le dimensioni complessive del tetto, l'implementazione dei giunti di movimento è necessaria per prevenire forze interne sproporzionate che potrebbero essere generate dai carichi termici in entrambe le travi del tetto ed in particolare nelle Mega Colonne.

Varie opzioni per il numero e le posizioni dei giunti di movimento (MJ) sono state valutate e gli annessi scenari di comportamento analizzati in accordo con la disposizione dei giunti di costruzione dettagliata sopra.

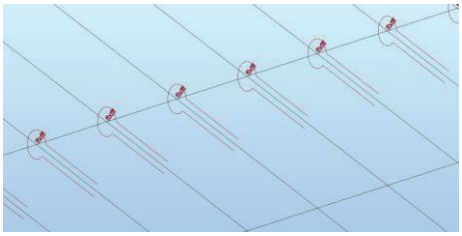
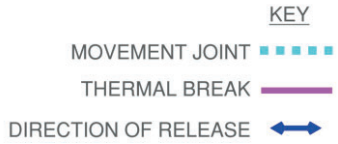


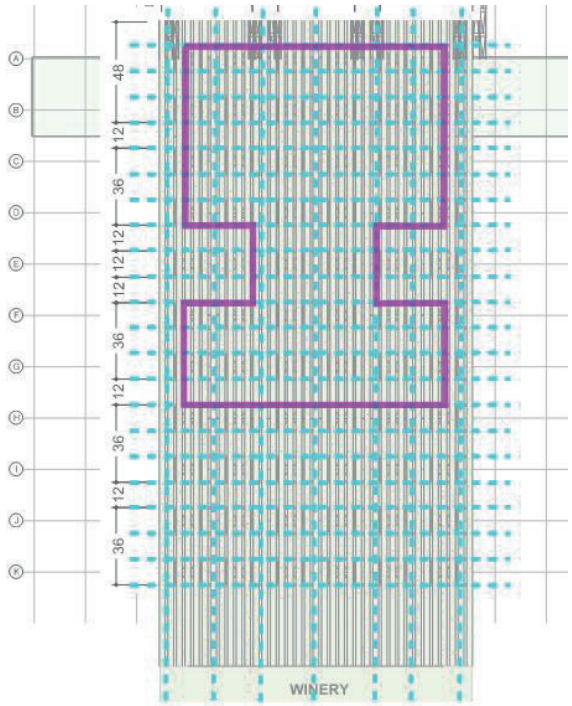
Figura 33: Giunti di movimento

Opzione 1

MJ ad ogni giunto di costruzione. Trave di trasferimento completamente fissata alle Mega Colonne in entrambe le direzioni (nessun rilascio).

Questa opzione è stata scartata in quanto non praticabile sia per la progettazione che per la costruzione e ha generato grandi spostamenti..



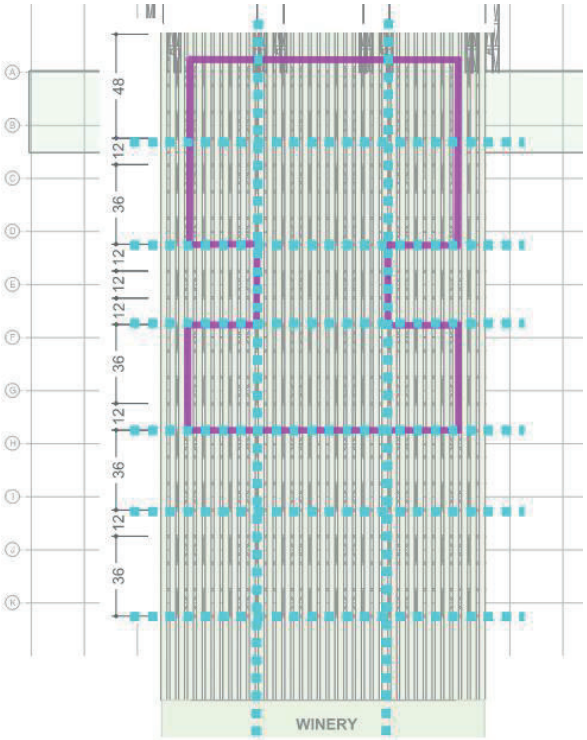


Opzione 2

2 MJs lungo la direzione X

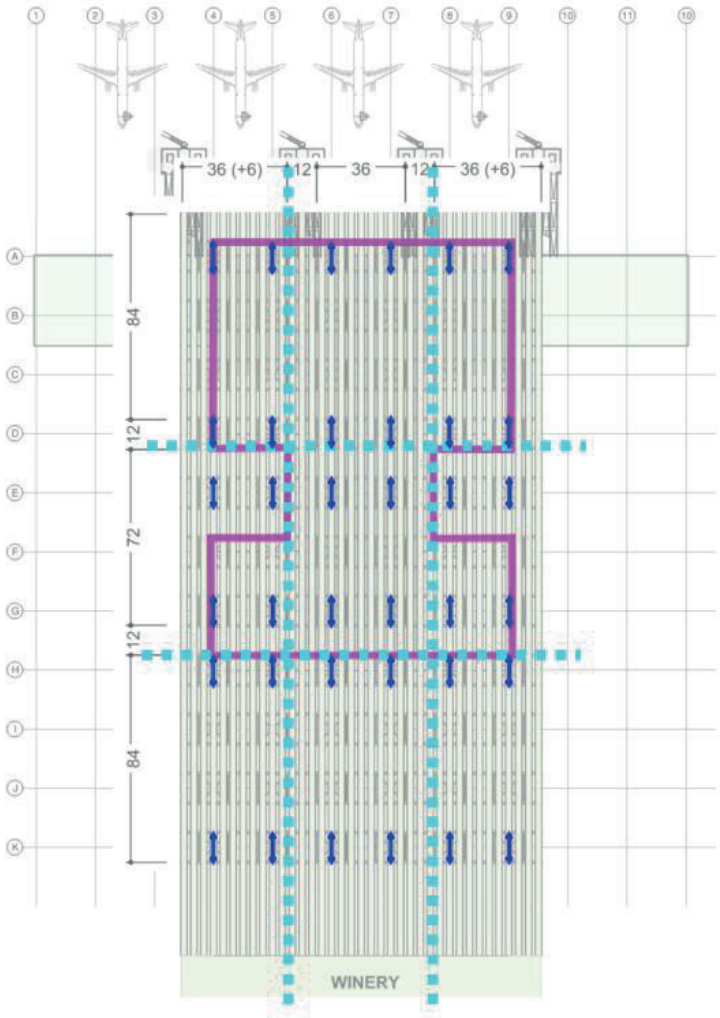
6 MJs lungo la direzione Y.

Questa opzione ha troppi giunti di movimento ed è stata scartata.



Le strategie dell'articolazione del movimento che sono state ritenute più idonee sono le opzioni 3 e 4, descritte in dettaglio di seguito.

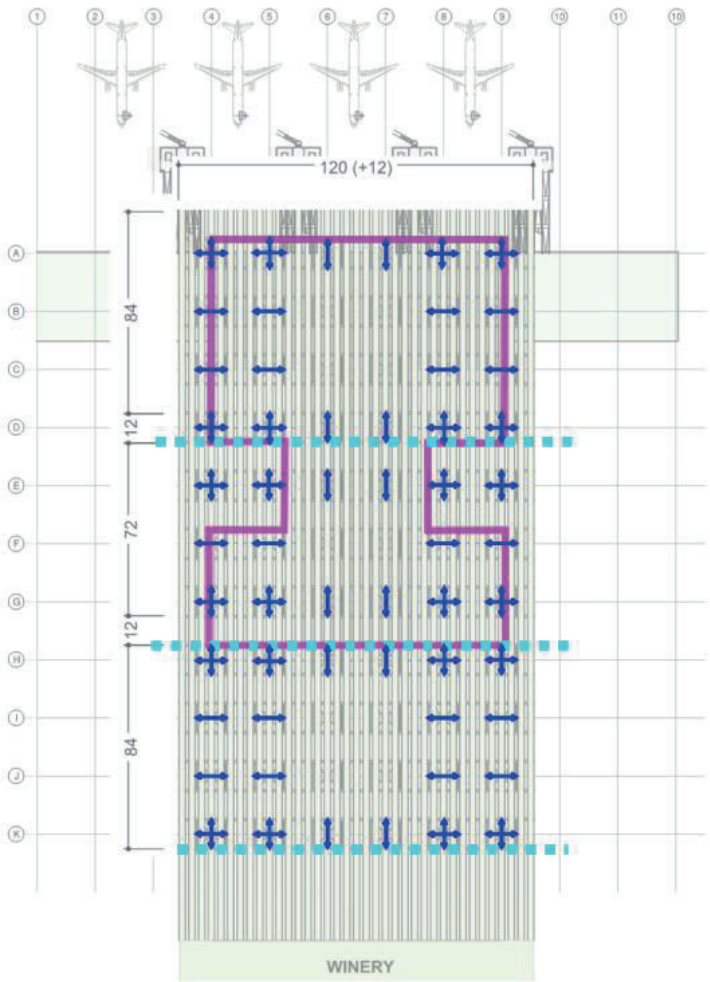
Opzione 3



Si è cercato di riprodurre lo schema della strategia dei giunti di movimento nella figura, rilasciando le travi del tetto in tutte le direzioni (tranne i collegamenti verticali e la rotazione sull'asse) in corrispondenza dei giunti di movimento (delineati in azzurro) e modificando i collegamenti rigidi sui rami delle colonne indicate per farli scorrere nella direzione Y. Un'ampia libertà di movimento nella direzione X è fornita dai giunti di movimento lungo la lunghezza dell'edificio.

Inoltre, gli elementi di facciata sono stati svincolati dalle Mega Colonne e dalla struttura del tetto per evitare una controventatura artificiale della struttura principale.

Opzione 4



Come mostrato nella figura per l'Opzione 4 del giunto di movimento, questa opzione mirava a dividere il tetto in tre celle invece di sei come nell'opzione 3, negando la necessità dei giunti di movimento lungo la direzione X, come era stato presentato nell'Opzione 3. Questo però significava che per alcune Mega Colonne era necessario introdurre ulteriori rilasci per permettere loro di scorrere nella direzione Y tramite collegamenti rigidi ai nodi superiori dei rami. Di conseguenza, sono state rilasciate più colonne rispetto all'Opzione 3. Per controllare la risposta agli effetti termici che causano troppe sollecitazioni sulle colonne al centro di ogni cella, alcune colonne sono state rilasciate sia in direzione X che Y.

Conclusione

In conclusione, l'opzione del giunto di movimento con il comportamento più favorevole è l'Opzione 4. Ciò è dovuto al fatto che le forze e le deformazioni sono migliori rispetto alle altre opzioni. Questi risultati sono mostrati di seguito.

Casi e Combinazioni di carico

La seguente tabella è una sintesi dei casi di carico (e della loro tipologia) che sono stati applicati nella definizione del modello di analisi.

Case	Description	Nature	Analysis Type
1	Peso proprio	Dead	Static - Linear
2	Copertura verde della trave del tetto 18.6 kN/m	Dead	Static - Linear
3	Portainnesto non fruttifero trave del tetto 30.6 kN/m	Dead	Static - Linear
4	Lucernario 2.4 kN/m	Dead	Static - Linear
5	Finiture del tetto e servizi 4.0 kN/m	Dead	Static - Linear
6	Lastra Partenze sovrapposta 4.20 kPa	Dead	Static - Linear
50	Tetto 0.60 kPa	Live	Static - Linear
51	Area Partenze, 5.00 kPa	Live	Static - Linear
64	Vento Est-Ovest	Wind	Static - Linear
65	Vento Ovest-Est	Wind	Static - Linear
66	Vento Nord-Sud	Wind	Static - Linear
67	Vento Sud-Nord	Wind	Static - Linear
70	Variazione Temperatura +	Temperature	Static - Linear
71	Variazione Temperatura -	Temperature	Static - Linear
80	Neve 0.80 kPa	Snow	Static - Linear
81	Neve accumulo 1.00 to 2.00 kPa	Snow	Static - Linear
203	Modale	Modal	Modal
204	Seismic EC 8 Direction_X	Seismic	Seismic-EC 8
205	Seismic EC 8 Direction_Y	Seismic	Seismic-EC 8
206	Seismic EC 8 Direction_Z	Seismic	Seismic-EC 8

La tabella seguente è un riassunto delle combinazioni di carico che sono state applicate nella definizione del modello di analisi.

Case	Description	Analysis Type
7	ULS	Linear Combination
8	ULS+	Seismic-EC 8
9	ULS-	Seismic-EC 8
10	SLS	Seismic-EC 8
11	SLS+	Seismic-EC 8
12	SLS-	Seismic-EC 8
13	SLS:CHR	Seismic-EC 8
14	SLS:CHR+	Seismic-EC 8
15	SLS:CHR-	Seismic-EC 8
16	SLS:QPR	Seismic-EC 8
17	SLS:QPR+	Seismic-EC 8
18	SLS:QPR-	Seismic-EC 8
19	ACC	Seismic-EC 8
20	ACC+	Seismic-EC 8
100	1.35 DL + 1.5 LL	Linear Combination
101	1.35 DL + 1.5 SL1 + 1.05 LL	Linear Combination
102	1.35 DL + 1.5 SL2 + 1.05 LL	Linear Combination
103	1.35 DL + 1.5 TLmax + 1.05 LL	Linear Combination
104	1.00 DL + 1.5 TLmax	Linear Combination
105	1.35 DL + 1.5 TLmin + 1.05 SL2	Linear Combination
106	1.00 DL + 1.5 TLmin	Linear Combination
107	1.00 DL + 1.50 WL East	Linear Combination
108	1.00 DL + 1.50 WL West	Linear Combination
109	1.35 DL + 1.5 TLmax + 1.05 WL East	Linear Combination
110	1.35 DL + 1.5 TLmax + 1.05 WL West	Linear Combination
200	1.0 DL	Linear Combination
207	Live	Linear Combination

Analisi Modale

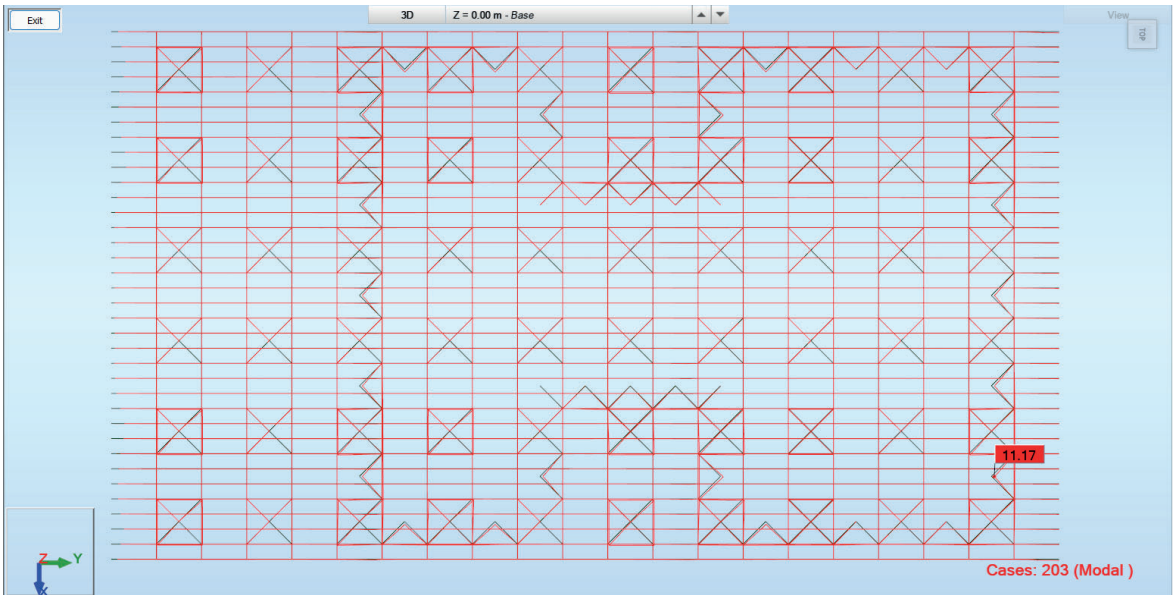


Figura 34: deflessione analisi modale

L'osservazione della deflessione sotto i risultati dell'analisi modale mostra che è entro i limiti.

Analisi Sismica

La direzione X della combinazione sismica mostra che a causa dei rilasci nella parte superiore delle colonne e del posizionamento dei giunti di movimento, lo spostamento massimo osservato è di 38 mm.

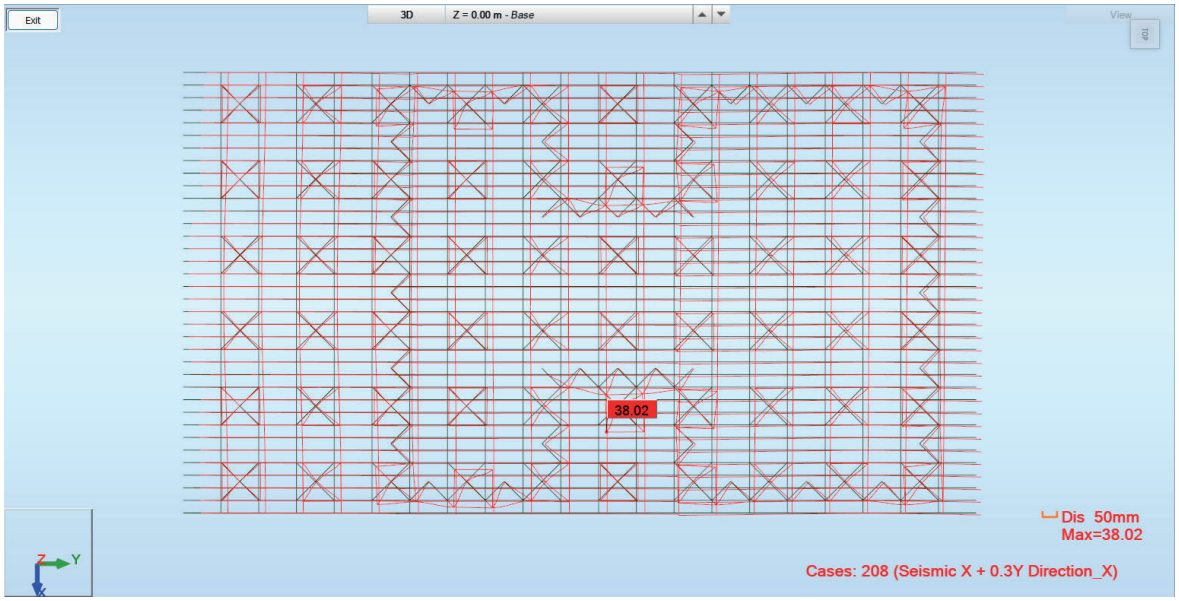
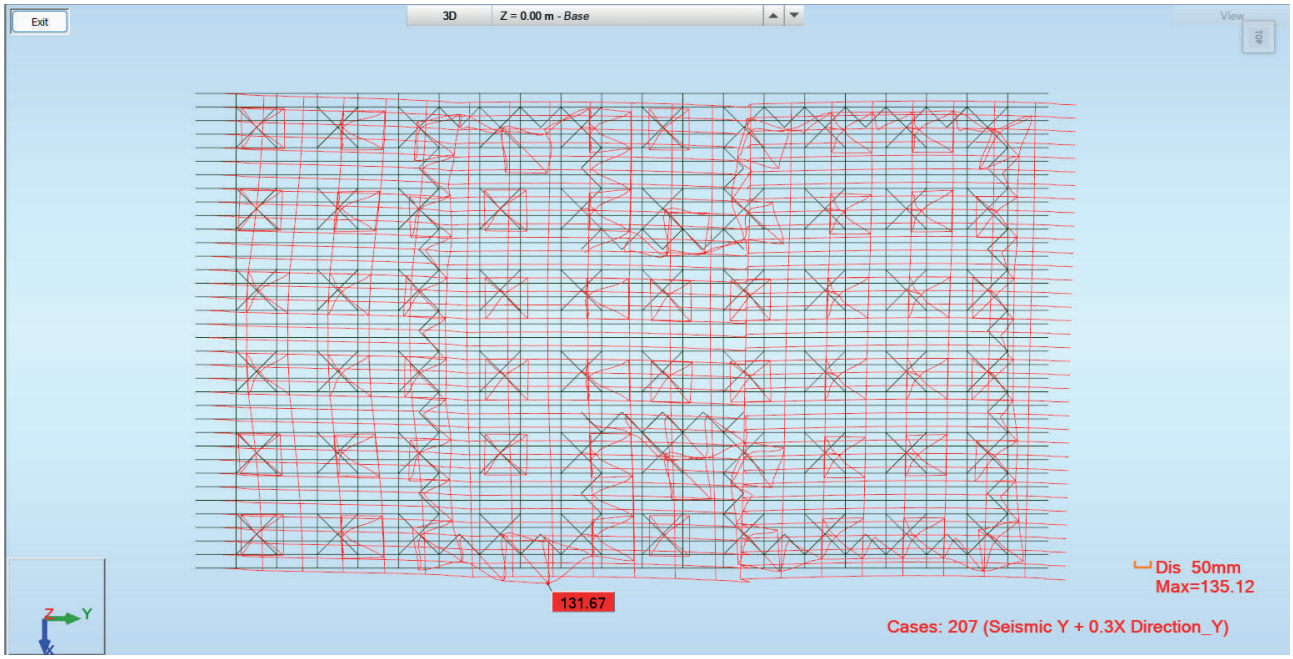


Figura 35: spostamenti direzione X combinazione sismica

La direzione Y sembra essere il caso critico nelle condizioni sismiche, con una deflessione di quasi 132 mm.



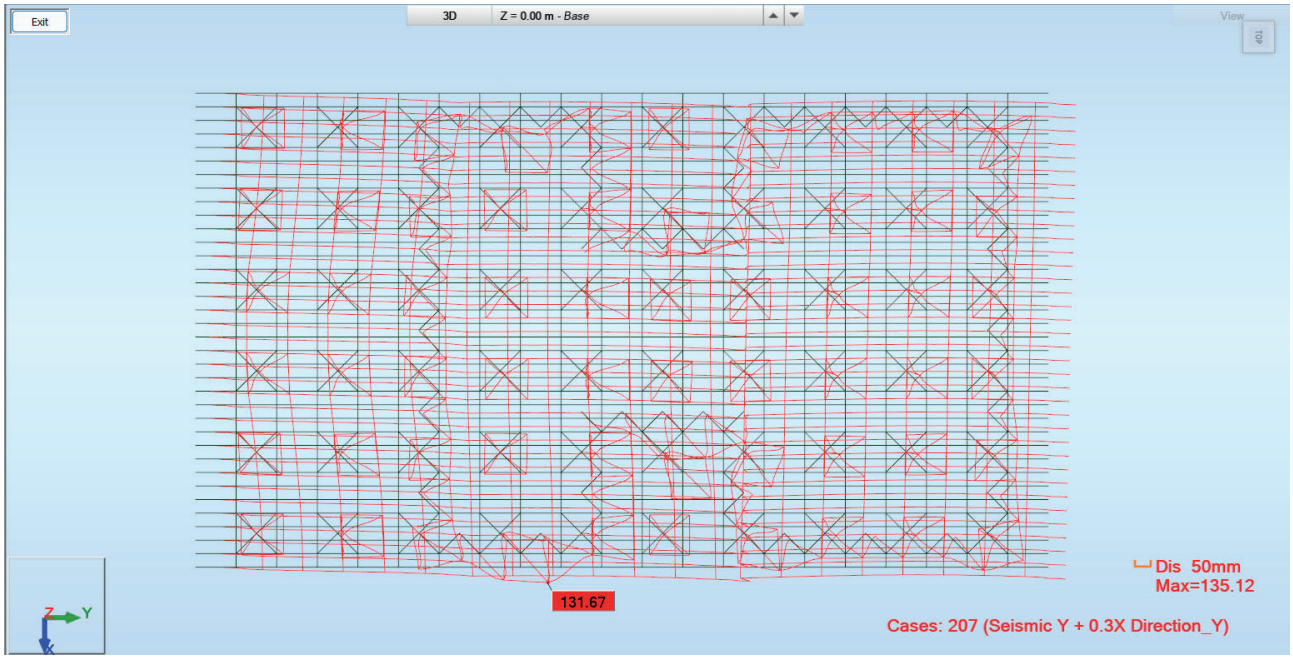
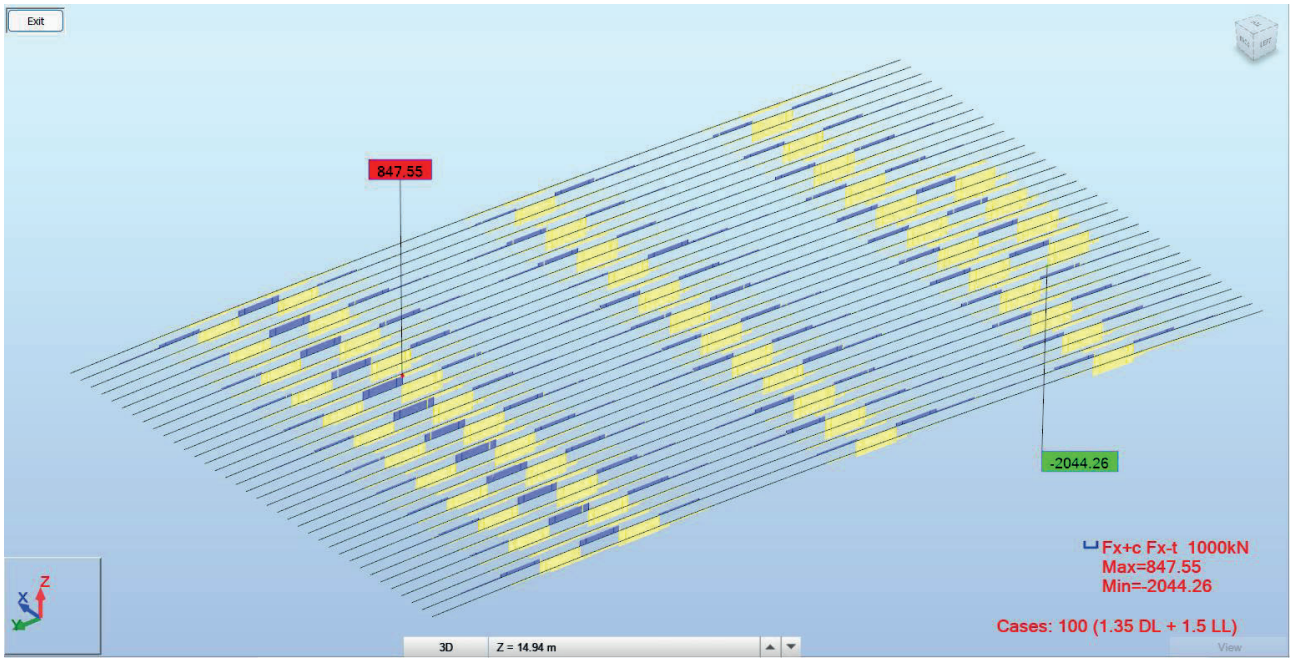


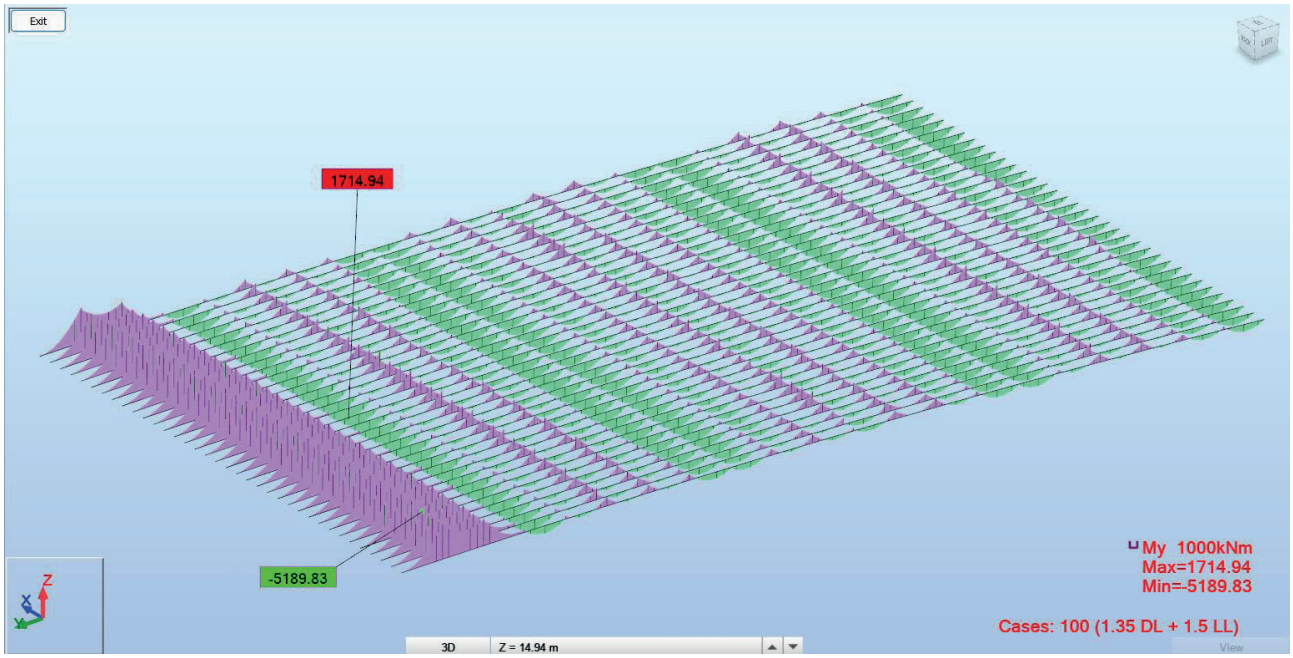
Figura 36: Deflessione direzione Y

Trave Principale

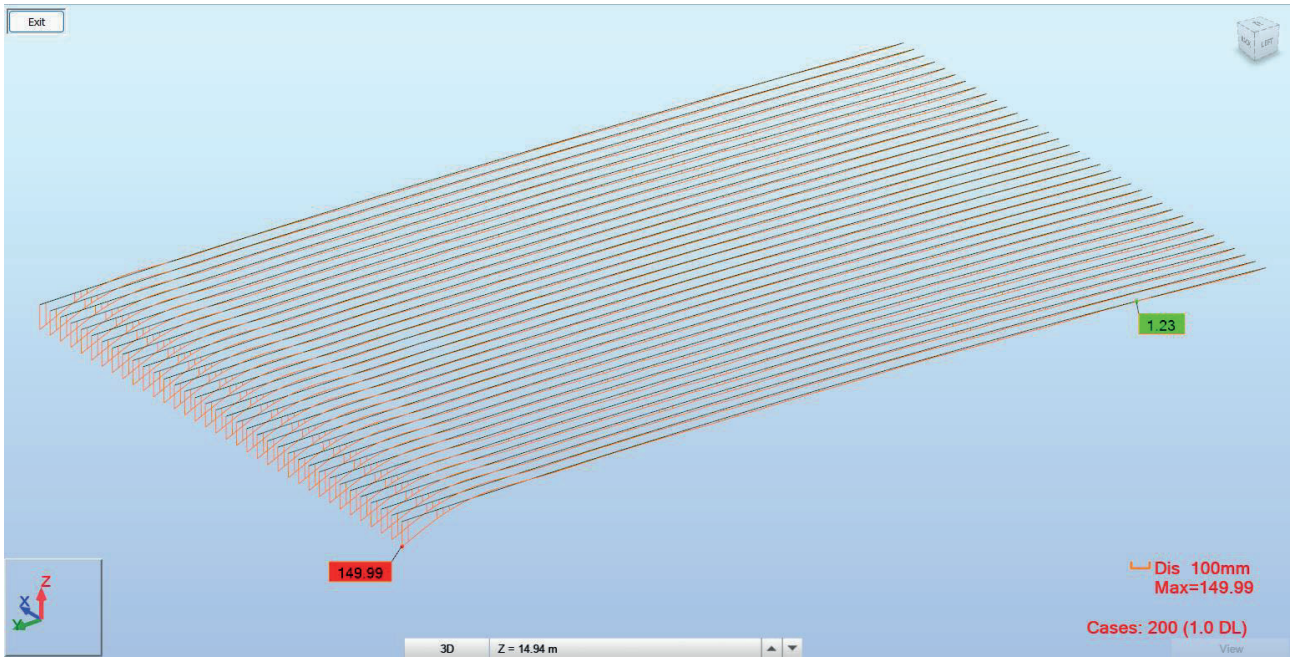
Carico Assiale

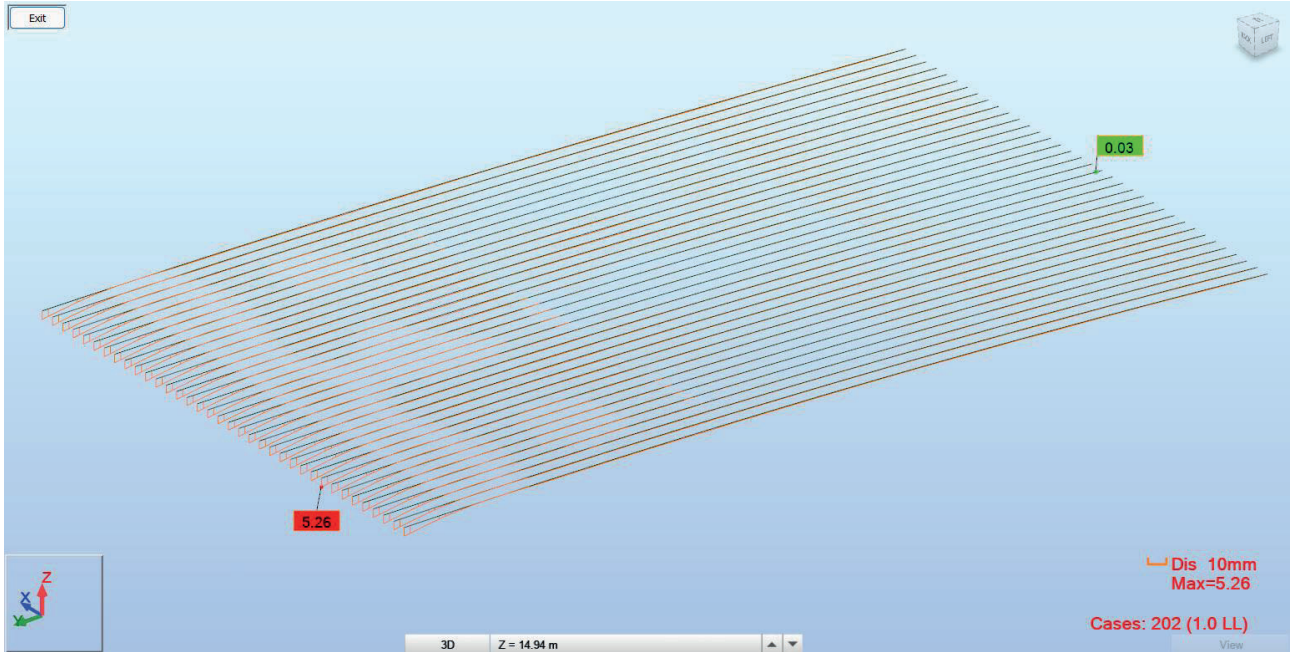


Momento Flettente



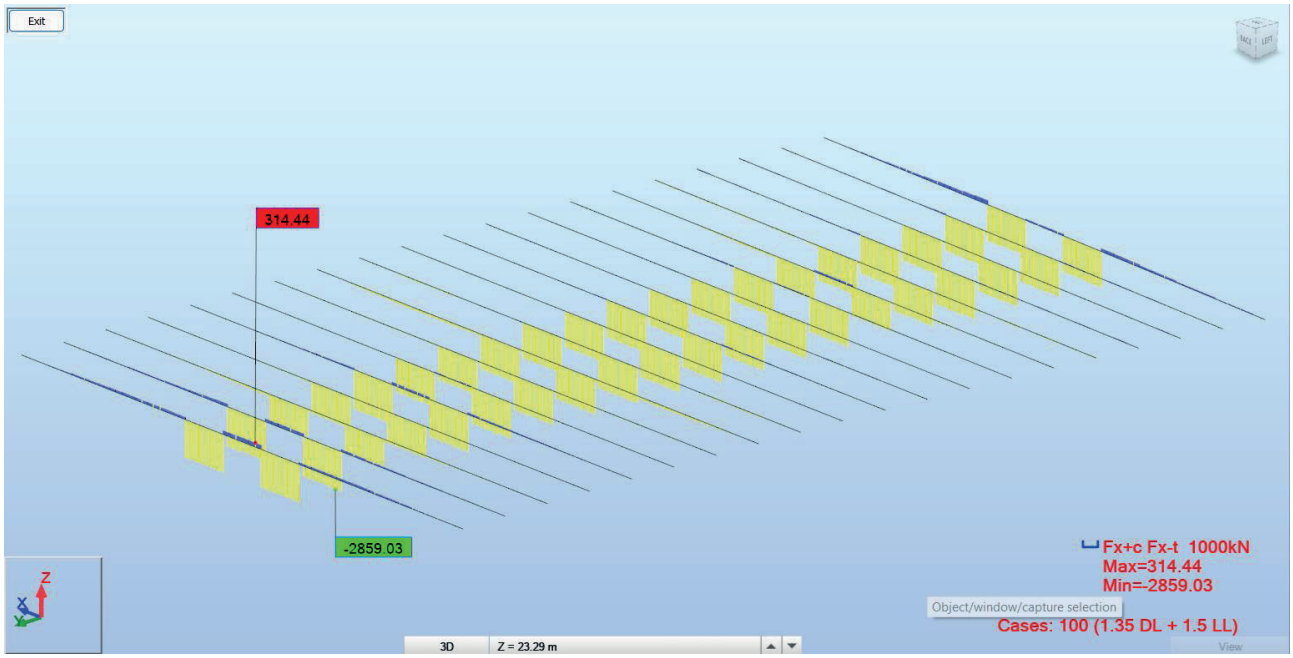
Deflessioni



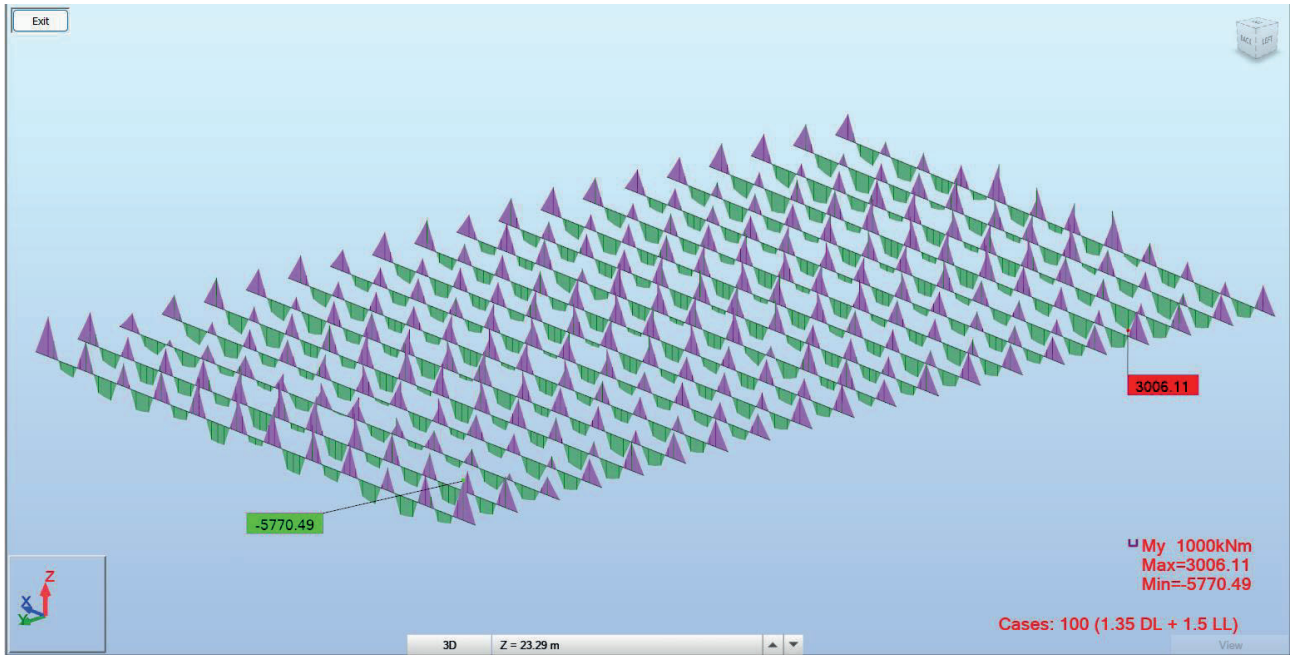


Trave di trasferimento

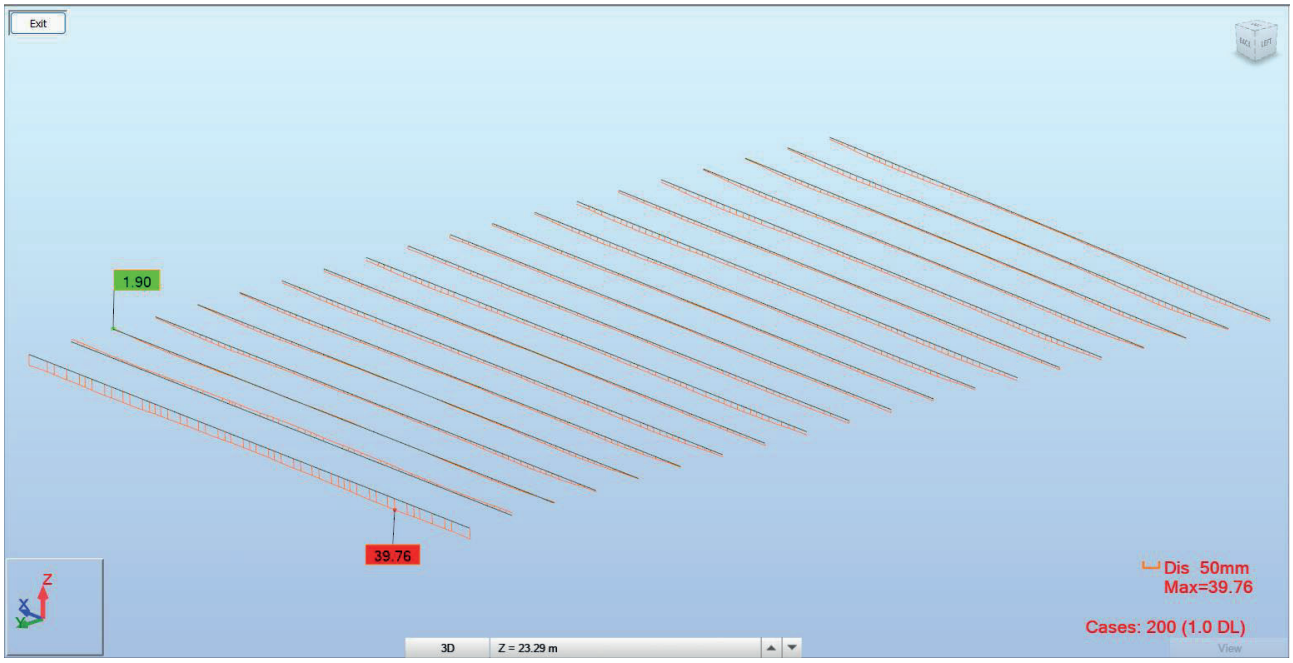
Carico Assiale

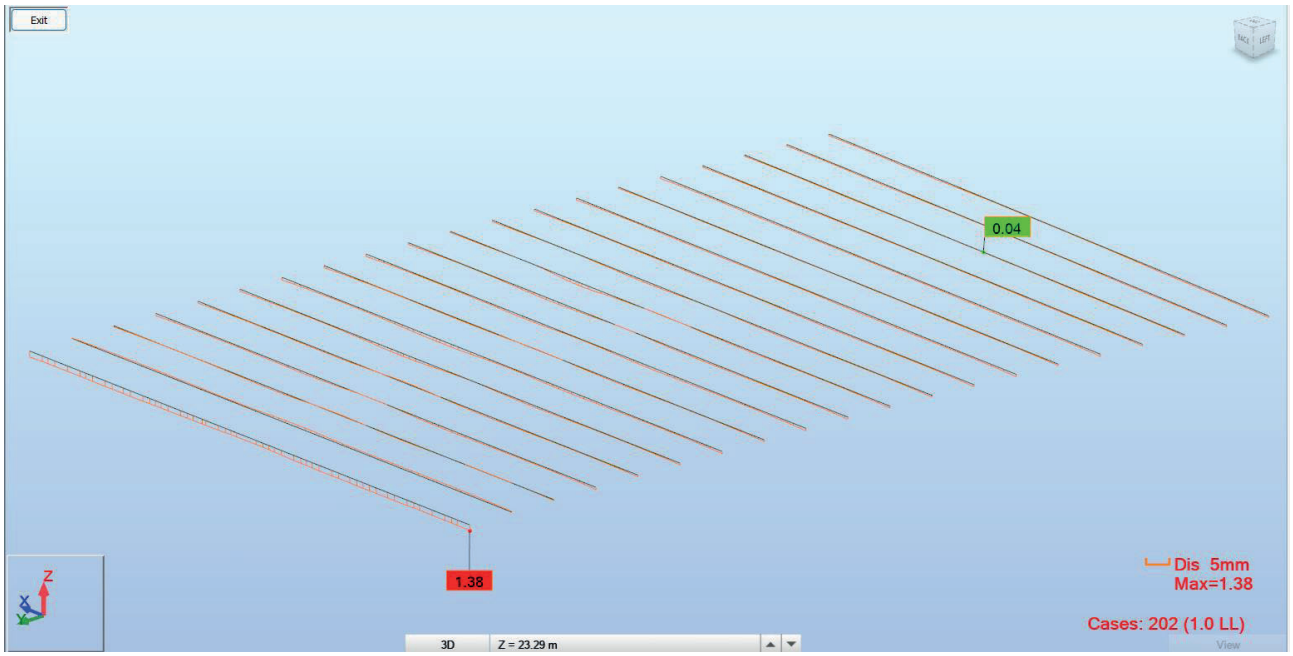


Momento Flettente



Deflessioni

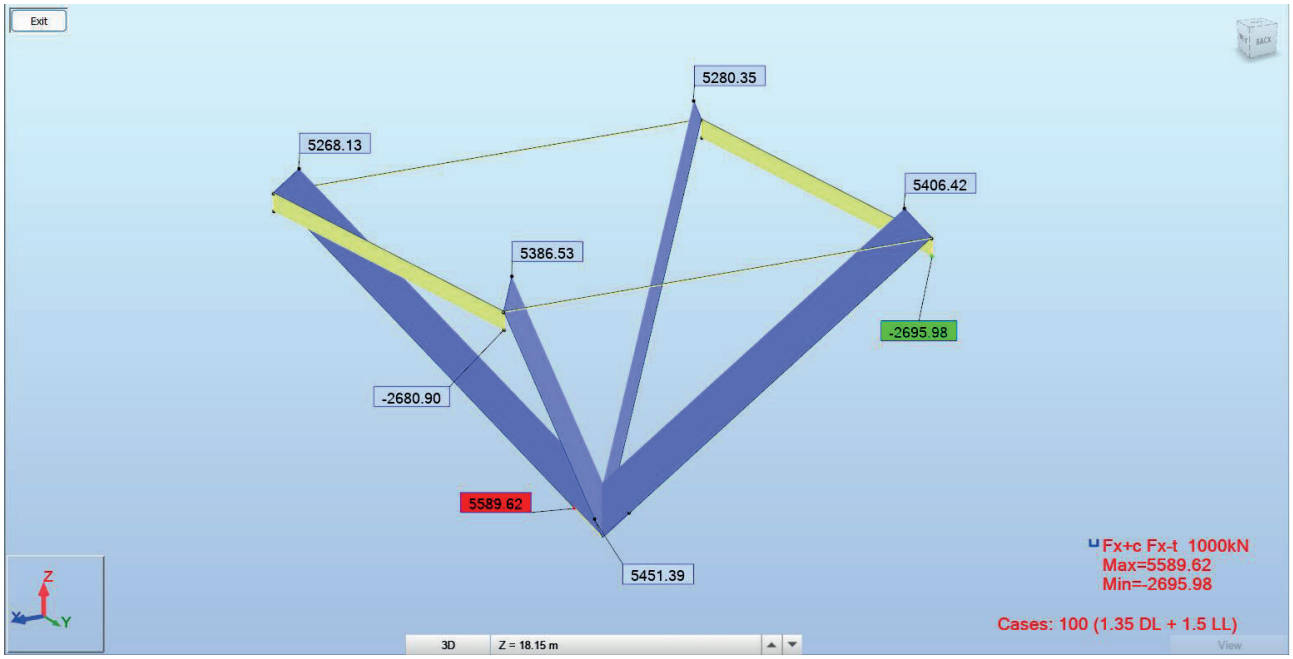




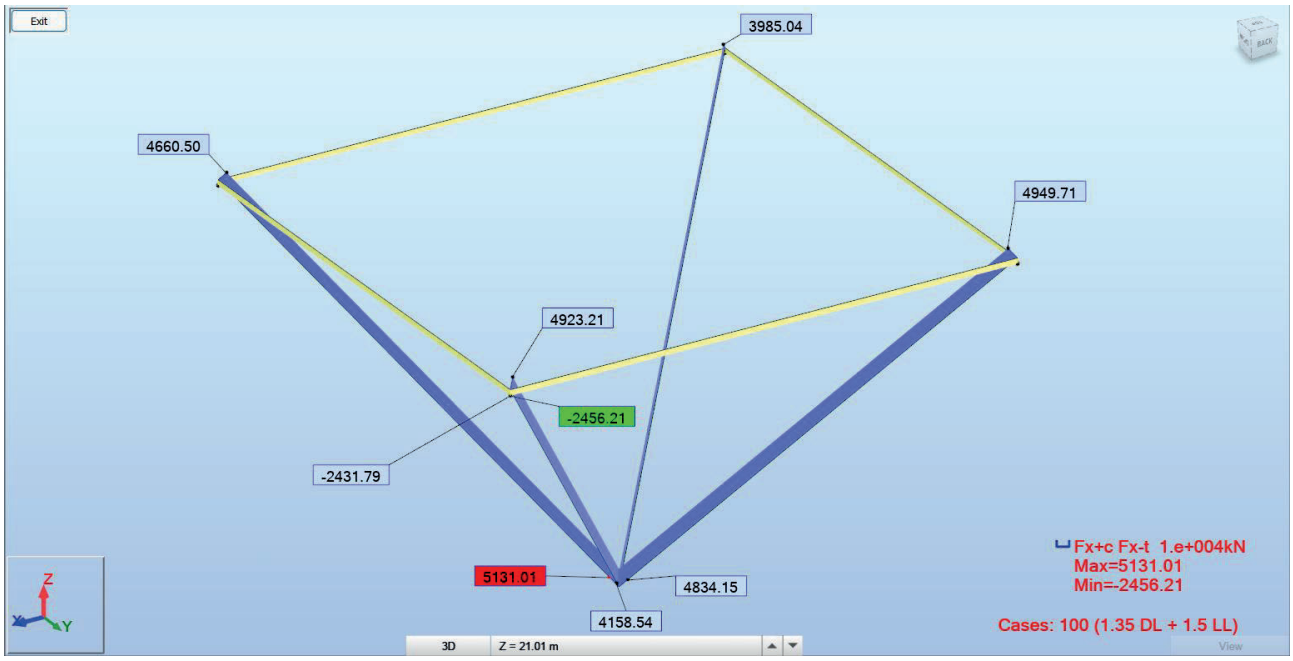
Mega colonna

Carico Assiale

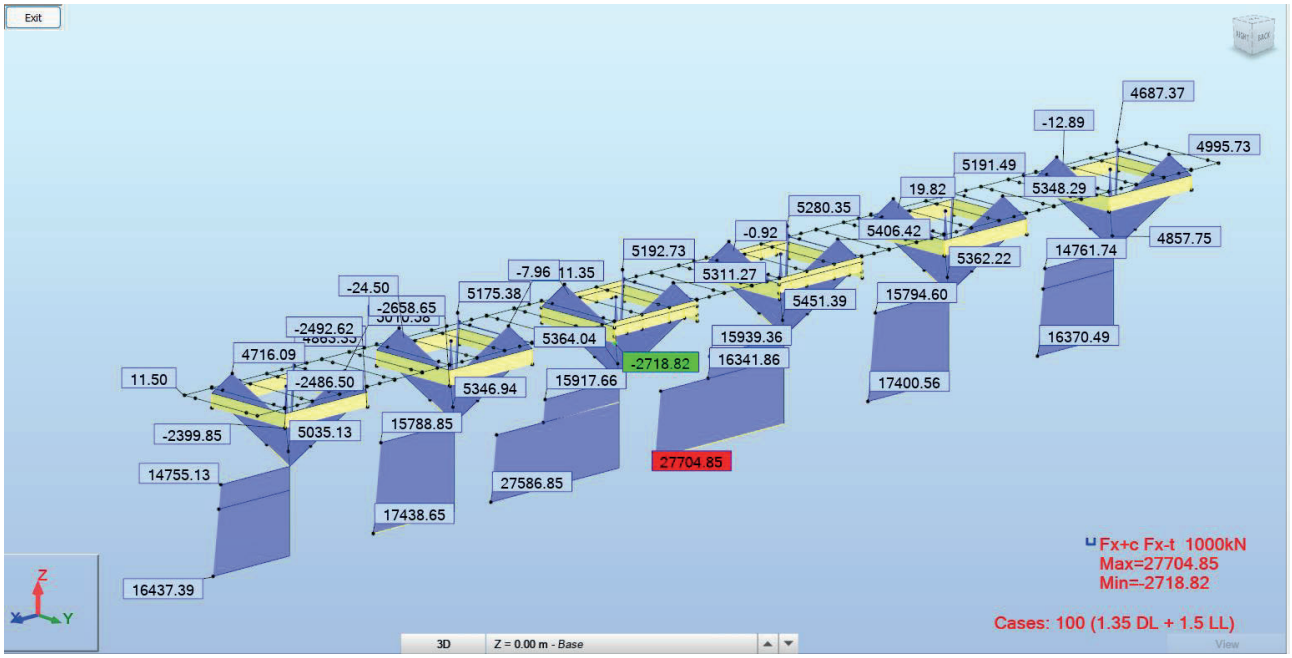
Colonna interna - Zoom sulla corona



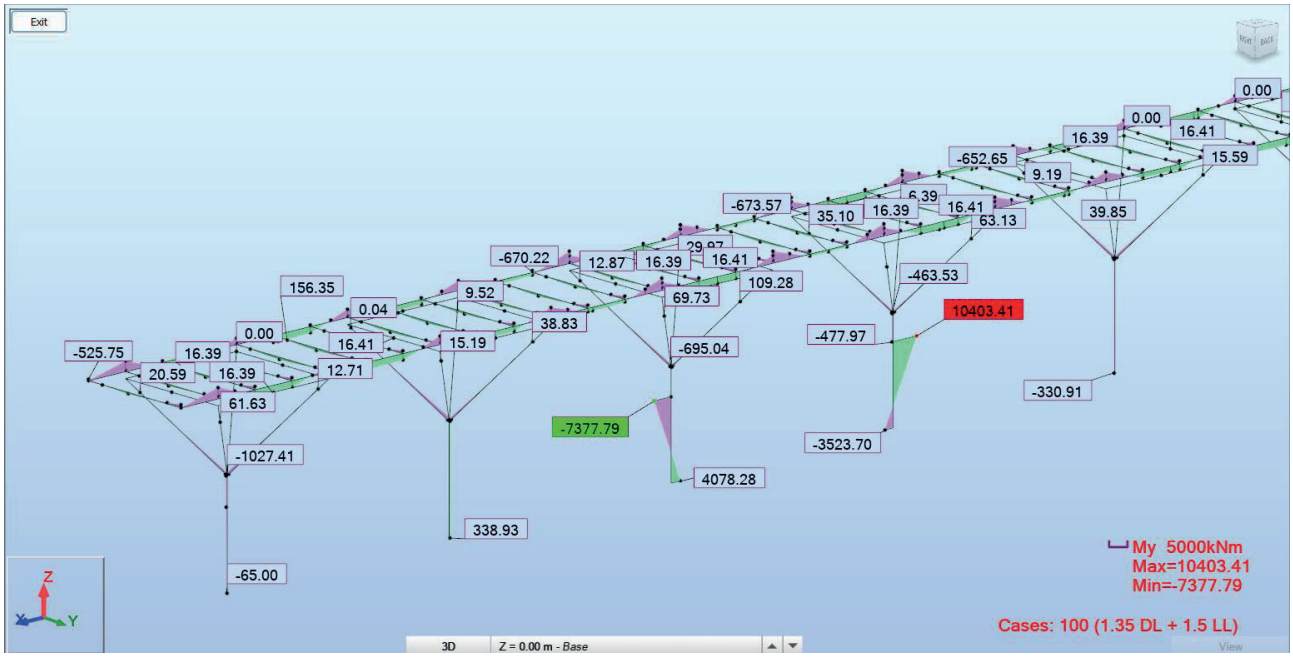
Colonna di bordo - Zoom sulla corona



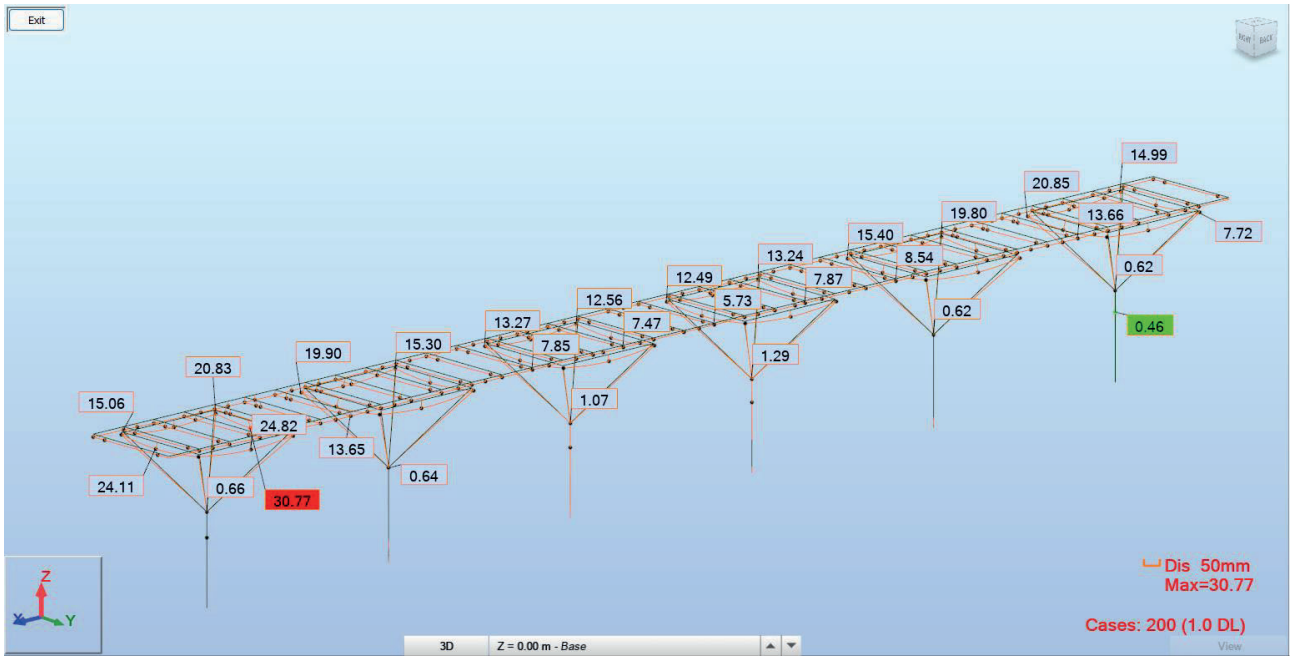
Campata tipica

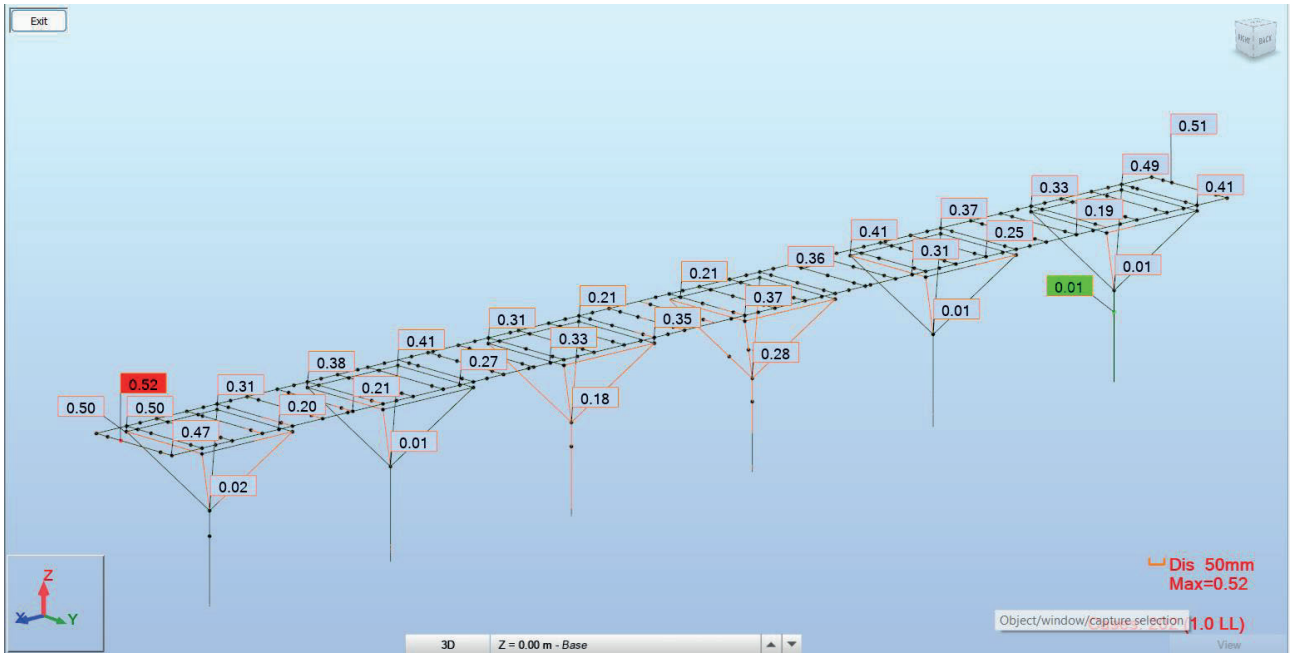


Momento Flettente



Deflessioni

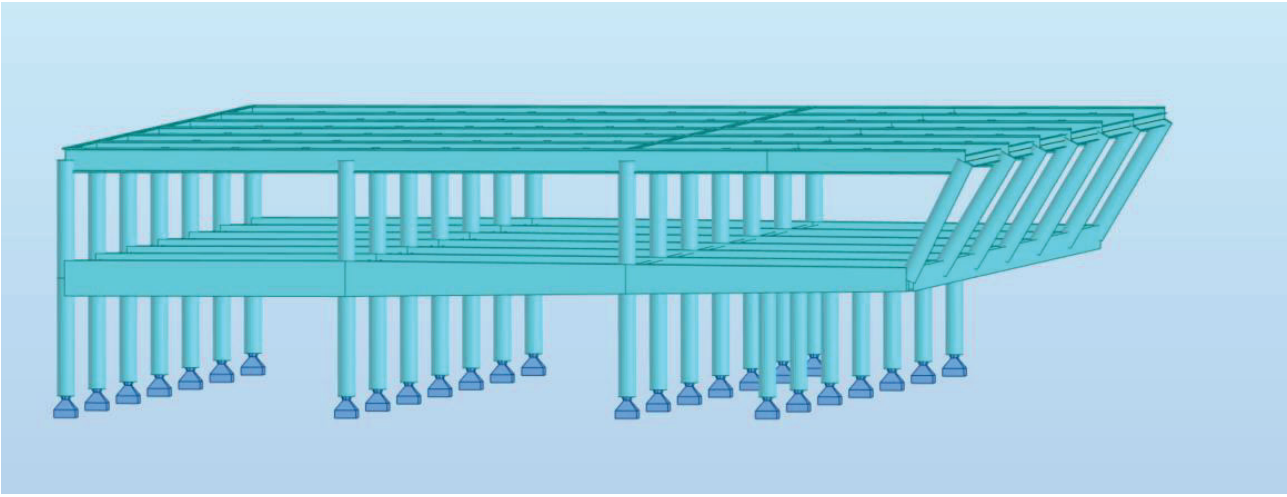


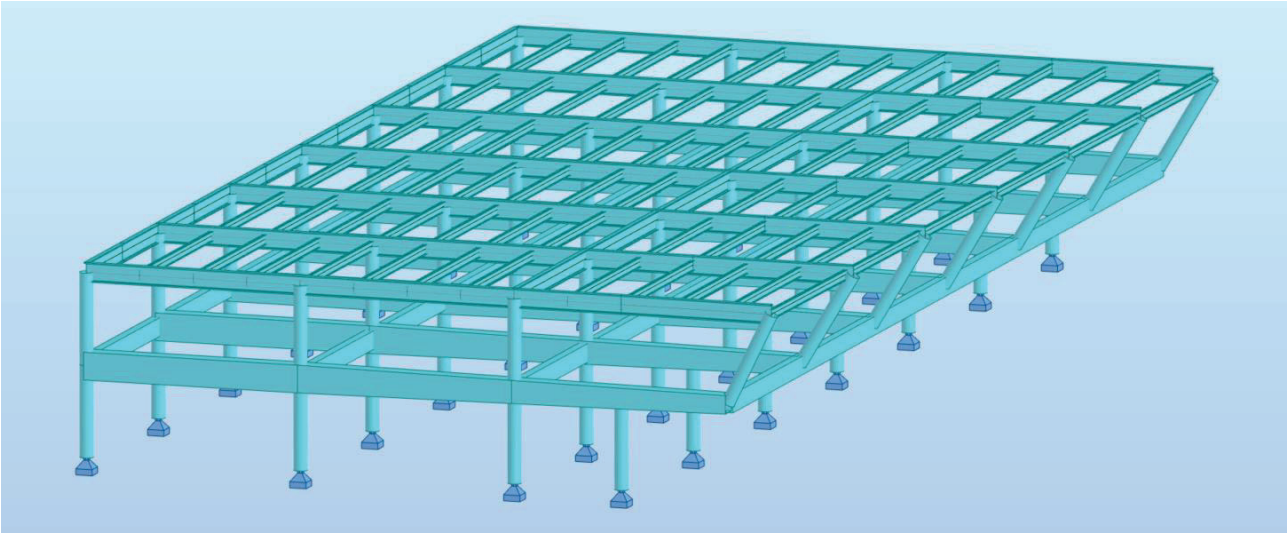


Edifici Laterali

Gli edifici laterali hanno dimensioni totali di 72x39 metri. Sono composti di piano terra, mezzanina, primo piano ed un tetto rivestito. Mentre la mezzanina ed il primo piano sono di calcestruzzo armato prefabbricato, il tetto è un telaio in acciaio. Gli edifici non sono simmetrici, infatti mentre l'edificio Ovest è composto da una griglia regolare 12x12, fino al tetto che si estende a Nord ulteriori 3 metri, l'edificio Est ha luci variabili.

Edificio Ovest:





Primo Piano

Il primo piano consiste in un telaio in cemento armato che supporta una lastra alveolare prefabbricata profonda 400 mm. Il ruolo delle travi secondarie in questo caso è principalmente di legamento con la struttura. Le travi primarie hanno dimensioni considerevoli, dovendo supportare una mensola di 6 metri; questa che dai pesi convenzionali del piano, anche dal carico proveniente dai pilastri diagonali che

sostengono il tetto. Le travi primarie sono continue fino all'ultima campata e pre-stressate o post-tese per fare fronte all'ingente carico.

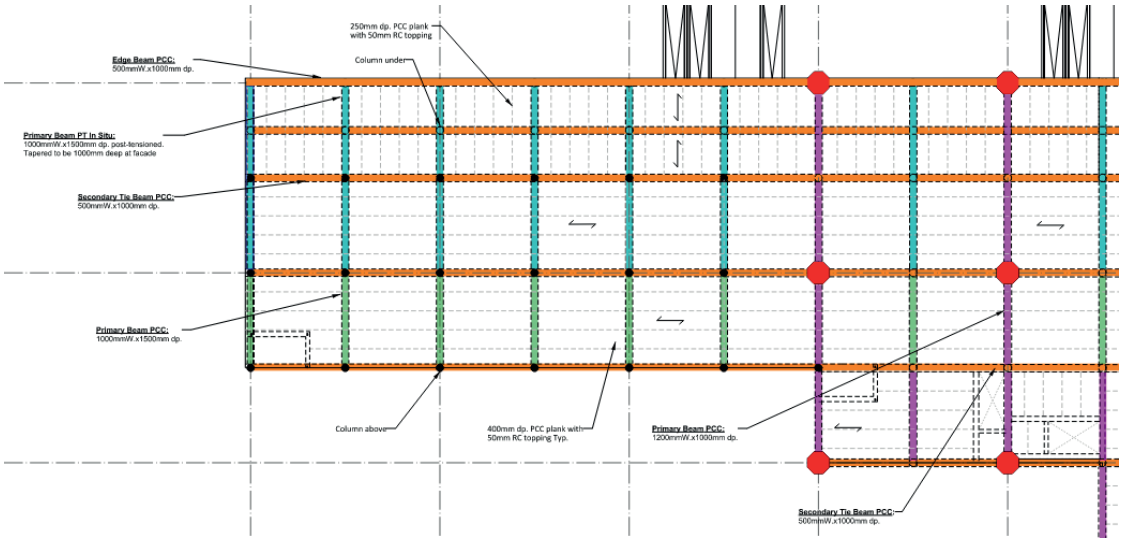


Figura 37: Travi primarie

Le travi posso essere affusolate così da essere profonde un metro sul fronte dell'edificio e 1500 mm nel punto di massimo sforzo, per permettere al post-tese di essere efficace.

Tetto

Il tetto è costituito da pannelli in metallo leggero sorretti dalle travi secondarie, parte di un telaio in metallo. I carichi secondo cui è stata fatta la progettazione includono i rivestimenti e i servizi per quanto riguarda l'azione permanente; l'azione variabile include solo il carico per la manutenzione, oltre al carico del vento e della neve, dettagliato nel paragrafo 4.

Il sistema del telaio consiste in travi secondarie (HE 500A) che poggiano su grosse travi primarie con una sezione prefabbricata (HE 800M). Queste travi hanno una luce di 15 metri, sul davanti dell'edificio poggiano su colonne diagonali. La trave è continua sul supporto centrale, per trarre vantaggio dai carichi applicati sulla campata posteriore, e bilanciare un'eventuale trazione del pilastro centrale.

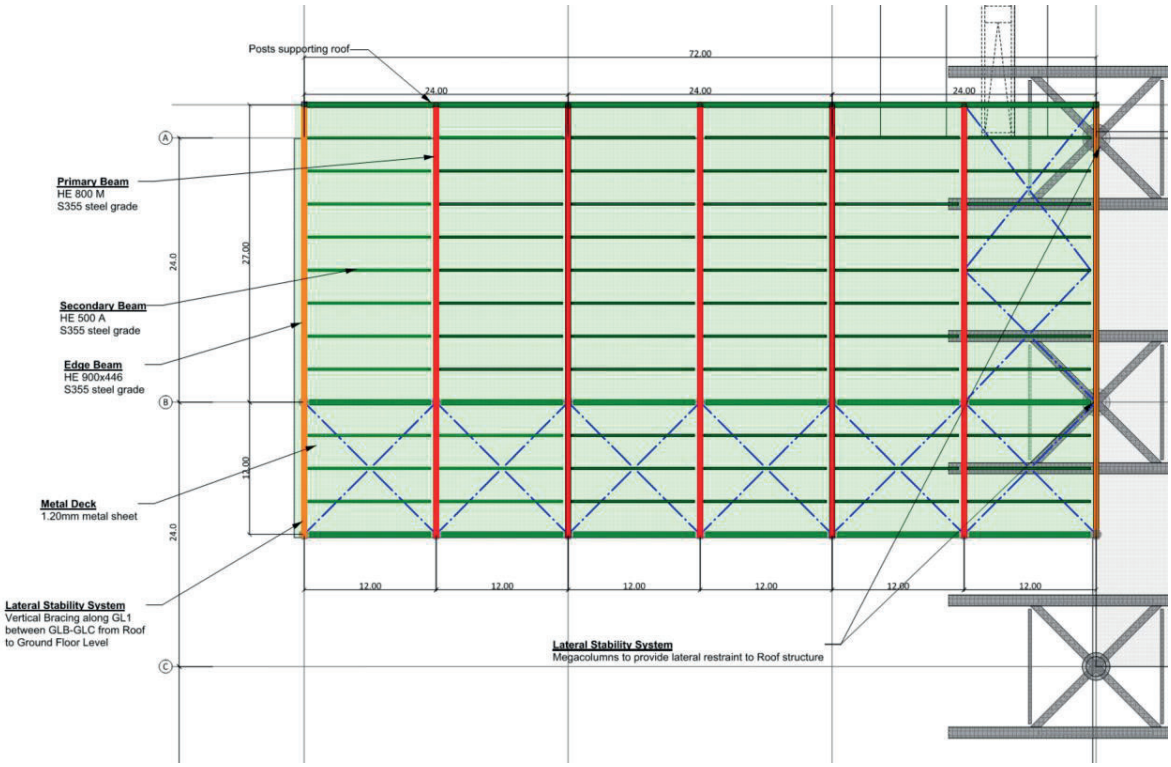


Figura 38: Sistema telaio del tetto

La stabilità laterale in direzione Est-Ovest fa affidamento sulle Mega Colonne, nelle quali il carico arriva per mezzo delle travi secondarie, e con l'aiuto dei rinforzi orizzontali (in blu nella figura di sopra). Per quanto riguarda la direzione Nord-Sud, oltre all'azione delle mega colonne, per evitare la torsione dell'edificio, si consiglia di inserire dei rinforzi dal piano terra al tetto come indicato in figura. Per via delle notevoli luci e dei supporti diagonali sul fronte della facciata, non è stato possibile progettare il telaio in modo che le cornici fungano da rinforzi nella direzione Nord-Sud.

Edificio Est:

L'edificio est comprende una disposizione della griglia leggermente diversa da quella dell'ala ovest. Di seguito riportata una sintesi della disposizione dell'edificio est. Le campate sono le stesse per il primo piano e il tetto dell'edificio est.

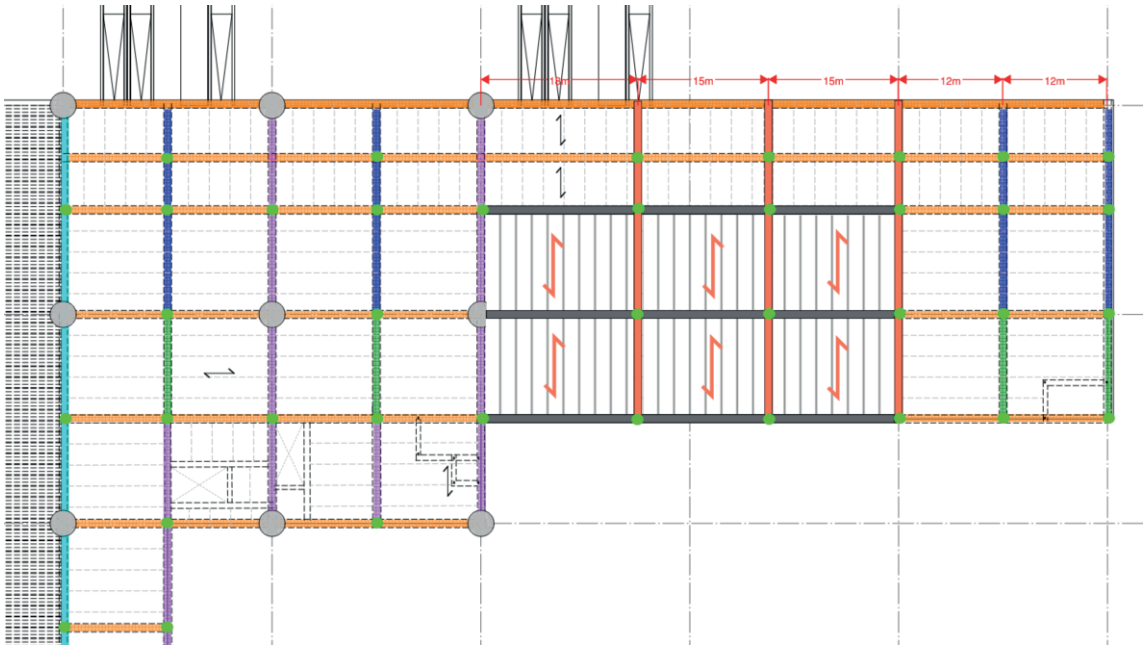


Figura 39: telaio edificio est

Nell'ala est, c'è un nastro per i bagagli che non funziona con una griglia di 12 metri x 12 metri. Questo crea la necessità di una griglia più grande, fino a 18m x 12m. Per questo motivo, le travi del primo piano in tamburato si appoggiano in direzione nord-sud sulle travi secondarie, per le campate di 15 m x 12 m e di 18 m x 12 m. Il progetto di questi elementi è riportato nell'Appendice D per le travi del primo piano nell'Appendice H per le travi del tetto.

Terminal tetto a guscio

Il pacchetto di calcoli dettagliato può essere trovato nell'Appendice G.

Appendice A: Sistema Informativo Geologico del Sottosuolo

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 296

Data inizio:
Data fine:
Cantiere: (PRG)
Località: Firenze
Quota p.c. (m): 37.74
Tipologia:

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Limo argilloso mediamente compatto a pigmentazione torbosa. Da 3,5 m sono inclusi minuti elementi calcarei	0.00	8.20	N
Ghiaia e ghiaietto in matrice limosa argillosa	8.20	10.20	N
Limo argilloso nocciola: livello plastico da 12,8 a 14,2 m	10.20	25.00	N
Ghiaia e ghiaietto in matrice sabbiosa limosa argillosa mediamente addensata	25.00	29.60	N
Limo argilloso compatto	29.60	30.00	N

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
Dr	a	lp	8.20
Dr	b	gs	10.20
Da	a	lp	25.00
Da	b	gs	29.60
FPT	c	ap	30.00

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 297

Data inizio:
Data fine:
Cantiere: (PRG)
Località: Firenze
Quota p.c. (m):
Tipologia:

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Limo argilloso a pigmentazione torbosa mediamente compatto con inclusi minuti frammenti calcarei	0.00	7.00	N
Limo argilloso compatto con inclusi minuti frammenti calcarei	7.00	8.20	N
Limo sabbioso con modesto legame argilloso e numerosi inclusi arrotondati e a spigoli vivi. Da 11 a 11,8 ghiaietto poco addensato	8.20	11.80	N
Limo argilloso mediamente compato con inclusi	11.80	13.00	N
Ghiaia e ghiaietto in matrice limosa argillosa	13.00	13.80	N
Limo argilloso con inclusi minuti elementi calcarei, da 16,4 a 17,2 passaggio di elementi litoidi arrotondati	13.80	27.20	N
Limo argilloso compatto talora minuti inclusi calcarei	27.20	30.00	N

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 828

Data inizio: 1984-11-12
Data fine:
Cantiere:
Località: Firenze
Quota p.c. (m): 40.75
Tipologia: Sondaggio

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Limo argilloso marrone a pgmentazione torbosa mediamente compatto, con inclusi minuti frammenti calcarei (diam max 3-4 mm).	0.00	7.00	N
Limo argilloso con striature grigio-azzurre ed inclusi minuti elementi calcarei.	7.00	8.20	N
Limo sabbioso marrone con modesto legante argilloso e numerosi inclusi arrotondati e a spigoli vivi (diam max 5-8 mm) compatto: da 11,00 a 11,80 ghiaietto poco addensato.	8.20	11.80	N
Limo argilloso ---------- con strature grigio-azzurre mediamente compatto con inclusi ghiaia e ghiaetto in matrice limo-argillosa.	11.80	15.35	N
Limo argilloso nocciola compatto con inclusi elementi calcarei; da 16,40 a 17,20 passaggio di elementi litoidi arrotondati (diam max 3-4 cm).	15.35	27.20	N
Limo argilloso grigio-verde compatto con scarse striature grigio-azzurre e talora minuti inclusi calcarei.	27.20	30.00	N

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
Dr	a	lp	8.20
Dr	a	ls	11.80
Dr	a	lp	13.80
Da	a	lp	27.20

FPT	c	ap	30.00

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 830

Data inizio:
Data fine: 1985-11-07
Cantiere:
Località: Firenze
Quota p.c. (m): 37.98
Tipologia: Sondaggio

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Ripoto a composizione limo argillosa con modesta coda sabbiosa. Compatto .	0.00	0.50	N
Limo argilloso marrone con noduletti di Mn e Fe .Compatto.	0.50	3.50	N
Limi argillosi con noduletti di Fe E Mn marroni:Mediamenyte compatti.	3.50	4.00	N
Limi argillosi marroni con concrezioni calcaree autigene diffuse.	4.00	4.50	N
Impasto argillo-limoso color marrone con ghiaia e ghiaietto.	4.50	5.00	N
Limi argillosi con livelli cementati a CaCo3 e diffusissime concrezioni (diam max 1-2 cm) calcaree autigene. Color nocciola molto compatto.	5.00	5.50	N
Limi argillosi con diffuse concrezioni calcaree autigene (diam max 1 cm). Compatti.	5.50	6.00	N
Limi argillosi variegati grigio e nocciola con diffuse concrezioni calcaree autigene (diam max 1 cm) con spalmature e noduletti di ossidi di Fe e Mn .Compatti. Da 9,50 a 10,00 mediamente compatti:	6.00	10.00	N
Limi argillosi inglobanti un raro e minuto ghiaietto con elementi arrotondati . Comuni i livelletti centimetrici di ghiaia,diffusi noduletti di ox di Fe e Mn.Colore nocciola.Compatti.	10.00	11.00	N
Ghiaia e sabbia con modesta coda argillosa.	11.00	11.15	N
Impasti di sabbia ghiaia, e limo.Nocciola.	11.15	11.50	N

Come da 10,00 a11,00 ma con abbondanti noduletti millimetrici di ox di Mn e Fe.	11.50	12.50	N
Limi argillosi prevalenti con clasti centimetrici subarrotondati e locali livelletti centimetrici ricchi di sabbia e ghiaia (il più potente va da 13,20 a 13,30).Variegati grigio e nocciola compatti.	12.50	14.00	N
Limi sabbiosi con modesta coda argillosa . Variegati grigio e nocciola .Addensati.	14.00	14.25	N
Impasto di sabbia ghiaietto ad elementi arrotondati e modesta coda limo argillosa.Presenza di sabbia manganesifera e noduletti millimetrici di Mn e Fe.	14.25	15.00	N
Limi argillosi con abbondante coda sabbiosa.Presenti livelletti cementati a CaCo3 : Variegati grigio e ocra. Compatti.	15.00	17.00	N
Sabbie limose con modesta coda argillosa e diffusi clasti centimetrici (diam max 3 cm) Subarrotondati arenacei e calcarei. Color nocciola prevalenti. Addensati.	17.00	18.00	N
Limi argillosi con livelletti cementati a CaCo3 e con concrezioni calcareee millimetriche con un raro ghiaietto arrotondato.Variegati grigio e nocciola.Compatti.	18.00	20.00	N
Impasto di sabbia,ghiaia ghiaietto e limo argilloso. Color nocciolo.	20.00	20.30	N
Limi argillosi e/o argille limose inglobanti diffusi noduletti millimetrici a CaCo3 autigeni.Compatti.	20.30	20.85	N
Come da 20,00 a 20,30	20.85	21.00	N
Come da 20,30 a 20,85 ma color nocciola e mediamente compatti.	21.00	22.00	N
Argille limose grigio chiaro leggermente variegate in marrone. Compatte.	22.00	23.00	N
Impasto di sabbia, ghiaia,ghiaietto e limo argilloso (diam max 3 cm) .	23.00	23.50	N
Sabbie e ghiaie (diam max 4 cm) con abbondante coda limo argillosa. Color nocciola.	23.50	24.00	N
Sabbie e ghiaie con modeste coda limo argillosa color nocciola.	24.00	24.30	N
Impasto di sabbia, ghiaia,ghiaietto e limo argilloso con netta prevalenza della frazione sabbiosa-ghiaiosa.Color nocciola.	24.30	25.00	N

Limi argillosi inglobanti diffusa ghiaia e ghiaietto.Color nocciola . Compatti.	25.00	25.15	N
Sabbia e ghiaia con modesta coda limo argillosa (diam max 2-3 cm) .Color nocciola.	25.15	25.90	N
Come da 25,00 a 25,15.	25.90	26.10	N
Limi argillosi con concrezioni calcaree autigene millimetriche e livelletti cementati a CaCo3 . Compatti.	26.10	27.00	N
Limi argillosi con concrezioni calcaree millimetriche autigene e modestissima coda sabbiosa (a sabbia fine).Variegati grigio azzurro,nocciola.Compatti.	27.00	28.50	N
Limi argillosi con concrezioni calcaree (dia max 1 cm) autigene. Variegati nocciola e grigio.Compatti.	28.50	31.60	N
Limi argillosi e/o argille limose color nocciola prevalente e variegati in grigio . Da molli a poco compatti.	31.60	33.00	N
Come da 28,5 a 31,60 ma con un maggior tenore in concrezioni calcaree autigene millimetriche.Variegati grigio e nocciola.Compatti.	33.00	38.00	N
Argille limose con ghiaia e ghiaietto arrotondati (diam max 1 cm) poco diffusi grigio azzurri.Compatte.Da 40,00 a45,00 da debolmente variegati a variegati grigio azzurro, millimetrici, calcarei autigeni.	38.00	50.00	N

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
rip	rip	rip	0.50
Dr	a	lp	11.00
Da	a	lp	23.00
Da	a	ls	26.00
FPT	a	lp	38.00
FPT	c	ap	50.00

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 837

Data inizio:
Data fine:
Cantiere:
Località: Firenze
Quota p.c. (m): 39.78
Tipologia: Sondaggio

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Limi sabbiosi marroni .	0.00	0.80	N
Limi a modesto legante argilloso,color marrone con sparsi noduli carbonatici autigeni .Rari i noduli millimetrici di Mn.	0.80	2.00	N
Limi argillosi variegati grigio, ocra con sostanza carboniosa e diffusi noduli di Mn e concrezioni calcaree millimetriche.Da mediamente compatti a compatti.	2.00	3.30	N
Impasto di sabbie,ghiaie ,ghiaietto elimi argillosi con clasti subarrotondati (diam max = 6 cm) prevalentemente calcarei.	3.30	4.00	N
Limi argillosi variegati,grigio-ocra-nocciola,con diffusi noduletti millimetrici di Mn e Fe e diffuse concrezioni calcaree (diam max =1cm) .Mediamente compatte.	4.00	6.10	N
Impato minuto di limmi argillosi,nocciola,grigio e ghiaietto essenzialmente calcareo marnoso e noduletti concrezionari. Poco compatti.	6.10	7.00	N
Limi argillosi variegati grigio nocciola ocra con diffusi noduletti millimetrici di Mn e concrezioni (millimetriche) di CaCo3 . Compatti.	7.00	8.60	N
Limi argillosi, nocciola e ocra inglobanti abbondanti ghiaie e ghiaietto (diam max3cm) con clasti arrotondati soprattutto arenacei. Mediamenti compatti.	8.60	10.00	N
Ghiaie e sabbie con coda limo sabbiosa in tenore variabile da 10,0 a 10,3 abbondante , fino a 11.0 modesto.	10.00	11.00	N

Alternanze decimetriche di limi argillosi e limi sabbiosi,variegati grigio ocra inglobanti abbondante ghiaie e ghiaietto essezialmete costitutito da clasti arenacei arrotondati (diam max= 2-3 cm) .Compatti.	11.00	12.40	N
N.B. da 12 a 13 carotati solo 40 cm per effetto tappo.	12.40	13.00	N
Sabbie,ghiaie e ghiaietto con abbondante matrice limo argillosa (diam max =2-3 cm) clasti arrotondati. Poco addensati.	13.00	14.80	N
Come da 11,00 a 12,40 ma con considerevole aumento della frazione sabbiosa e con diffuse concrezioni calcaree millimetriche.	14.80	16.00	N
Come da 12,4 a 14,80.	16.00	17.10	N
Limi argillosi variegati,nocciola,grigio con sostanza carboniosa e diffusi noduletti millimetrici concrezionari di CaCo3. Da mediamente compatti a compatti.	17.10	21.00	N
Limi sabbiosi a modesto legante argilloso inglobanti diffusi clasti arrotondati (diam max 3 cm) . Colore ocra-nocciola chiaro.	21.00	22.00	N
Sabbia e ghiaia con legante argilloso.Mediamente addensato.	22.00	23.40	N
Limi argillosi variegati ocra-grigio-nocciola con diffuse concrezioni calcaree millimetriche e livelletti cementati a CaCo3 con intercalazioni decimetriche di ghiaietto e sabbia con clati arrotondati.Compatti.	23.40	25.00	N
Argille limose color oliva,grigio inglobanti raro ghiaietto arrotondato, rari noduletti millimetrici di ossidi di Fe e Mn e abbondanti clasti arrotondati di varia natura,ma essenzialmete di marne calcaree (diam max =3 cm) e calcare marnoso.	25.00	27.30	N
C.S. Da 25,0 netto passaggio di colori da nocciola chiaro a grigio-oliva scuro .Da molli a poco compatte.	25.00	27.30	N
Limi argillosi variegati grigio-oliva scuro con diffusi noduli calcarei millimetrici ed intercalazioni centimetriche di un minuto ghiaietto costituito da clasti arrotondati sempre immersi in pasta di limo argillosa.Da mediamnte compatti a compatti.	27.30	30.00	N

Argille limose grigio-verdi inglobanti diffuso ghiaietto arrotondato (diam max=1cm).Da molli a poco compatte.	30.00	32.00	N
Limi argillosi variegati grigio-azzurro,caki,con diffuse concrezioni calcaree millimetriche erari clasti arrotondati dalledimensioni di un ghiaietto fine (diam max=3-4 cm).	32.00	35.00	N
Limo argilloso grigio inglobante diffusi clasti arrotondati (diam max 3 cm).Mediamentre compatti.	35.00	35.40	N
Limo sabbioso a modresto legante limo-argilloso, oliva,ghiaia e ghiaietto con clasti arrotondati (diam max = 3 cm,qualcuno anche fino a 6 cm). Prevale il litotipo calcareo-marnoso.Compatti.	35.40	38.20	N
Limi argillosi, inglobanti diffusi noduli concrezionari di CaCo3 millimetrici,variegati oliva-grigio.Compatti.	38.20	40.00	N
Impasto di sabbia, ghiaia e ghiaietto e limo argilloso color oliva.Clasti arrotondati. Da mediamente addensate ad addensate.	40.00	42.60	N
Limi argillosi variegati grigio-azzurro ed oliva inglobanti diffusi noduletti millimetrici a CaCo3 e inglobanti rari clasti arrotondati (diam ma = 3 cm).Da 42,6 a 44,0 compatti; da 44,0 a 45,0 mediamente compatti.	42.60	45.00	N
Argille limose da poco a mediamente comapatte,grigio azzurre.	45.00	47.30	N
Limi argillosi variegati grigio-azzurro,nocciola,ocra con diffusi noduli millimetrici di CaCo3 e rare intercalazioni centimetriche di clasti arrotondati (dimensini di un ghiaietto) inglobati nei limi compatti.	47.30	48.00	N
Limi argillosi variegati grigio-azzurro,nocciola,ocra con diffusi noduli millimetrici di CaCo3 .Compatti.	48.00	50.00	N

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
Dr	a	lp	8.60

Dr	a	ls	11.00
Da	a	ls	13.00
Da	b	ss	17.00
Da	a	lp	23.40
FPT	a	lp	35.50
FPT	a	ls	38.20
FPT	a	lp	40.00
FPT	a	ls	42.60
FPT	c	ap	50.00

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 1707

Data inizio: 2003-01-15
Data fine:
Cantiere: Consorzio castello - Olmatell
Località: Olmatello
Quota p.c. (m): 0.00
Tipologia: Carotaggio continuo a rotazio

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Terreno vegetale - limo con argilla sabbioso, marrone	0.00	0.20	
Limo con argilla, sabbioso e deb. sabbioso, nocciola, consistente	0.20	1.00	
Argilla con limo deb. sabbiosa, a tratti deb. ghiaiosa, colore nocciola, da consistente a molto consistente, con rari clasti alterati	1.00	3.20	
Limo con argilla e ghiaia deb. sabbioso, nocciola con striature grigie con clasti alterati.	3.20	3.70	
Limo con argilla deb. sabbioso e deb. ghiaioso colore nocciola e grigio, consistente con rari clasti	3.70	4.00	
Limo con argilla e ghiaia , deb. sabbioso, nocciola con clasti alterati	4.00	4.50	
Argilla con limo deb. sabbiosa e ghiaiosa con clasti, molto consistente	4.50	5.00	

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
rip	rip	rip	0.20
Dr	a	LP	1.00
Dr	a	LS	5.00

COMUNE DI FIRENZE

Direzione Ambiente

via Benedetto Fortini, 37 - 50125 - e.mail: direz.ambiente@comune.fi.it

Scheda del Sondaggio n. 1709

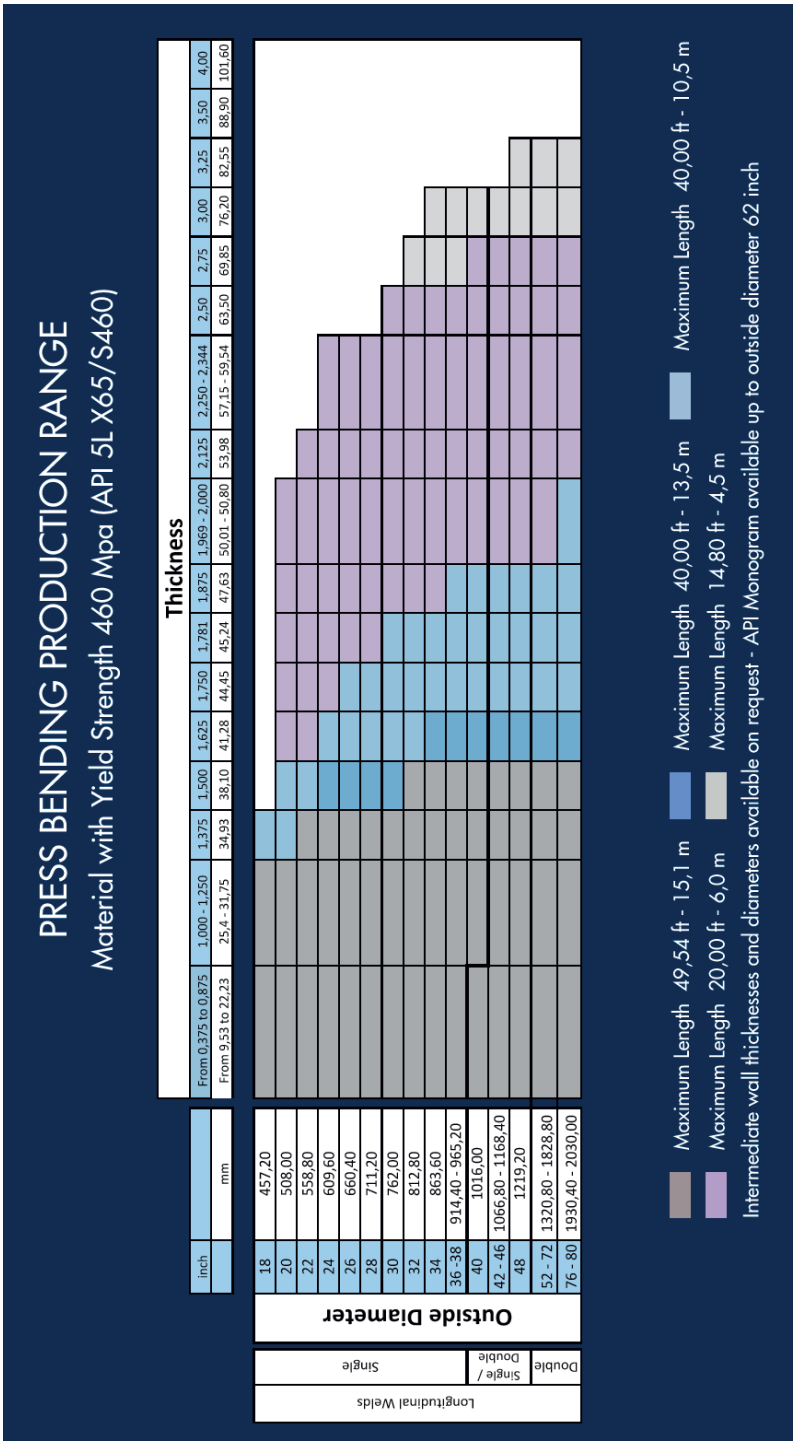
Data inizio: 2003-01-22
Data fine:
Cantiere: Consorzio Castello - Olmatel
Località: Olmatello
Quota p.c. (m): 0.00
Tipologia: Carotaggio continuo a rotazio

Stratigrafia			
Descrizione stratigrafia	**Profondità iniziale (m)**	**Profondità finale (m)**	**Campione**
Terreno vegetale - limo argilloso sabbioso deb. ghiaioso, nocciola	0.00	0.50	
Limo con argilla deolm. sabbioso, nocciola mediamente consistente	0.50	1.10	
Argilla con limo deb. sabbiosa e deb. ghiaiosa, fine, colore nocciola, da consistente a molto consistente	1.10	2.80	
Argilla con limo, deb. sabbiosa e ghiaiosa , colore nocciola da consistente a molto consistente	2.80	4.30	
Argilla con limo, ghiaiosa colore nocciola con striature grigie con clasti alterati	4.30	5.00	

Sintemi			
Sintema	**Subsintema**	**Cod. USCS**	**Profondità (m)**
rip	rip	rip	0.50
Dr	a	LP	2.80
Dr	a	LS	5.00

Appendice B: Gamma di tubi in acciaio

La tabella seguente fornisce la sezione in acciaio a sezione cava circolare (CHS) disponibile da Cimolai S.p.A a Porcia, Italia. Maggiori informazioni qui: https://www.cimolai.com/pipes/



Appendice C: Calcoli del piano terra

Column presizing

For all columns use C 32 /40 fcd= 21 N/mm2

The reinforced concrete column is presized as per the condition of limiting axial force to the following values:

v= 0.40 for central columns
v= 0.35 for edge corners

$$\frac{N_{GS}}{b\,h\,f_d} \leq \nu$$

Additional loads from intermediate roof supported on edge and corner columns

Steel work	=	0.75	kN/m2
Decking & finishes	=	1.25	kN/m2
Ceiling and services	=	1.5	kN/m2
Variable load	=	0.75	kN/m2

The load from intermediate roof spans 25m in one direction & 12.5m in the other

Axial Load for Typical corner column

First floor

DL.f	=	655	kN
LL.f	=	210	kN

Intermediate roof

DL.R =	25m/2 x 12.5m/2 x DL.r=	273	kN
LL.R =	25m/2 x 12.5m/2 x LL.r=	58.6	kN

Total Axial Load

N_{GS}= 1656.281 kN

Minimum column dimensions

$$h = b = \sqrt{\frac{N_{GS}}{\nu\, f_{cd}}} = \quad 440.6 \quad \text{mm}$$

Equivalent area for circular column

Φ = (4xbxh/Π)^0.5 = 497.2 mm say 550 mm

Axial Load for Typical edge column

First floor
DL.f = 1325 kN
LL.f = 405 kN

Intermediate roof
DL.R = 25m/2 x 12.5m/2 x DL.r= 273 kN
LL.R = 25m/2 x 12.5m x LL.r= 117 kN

Total Axial Load
N_{GS}= 2941.172 kN

Minimum column dimensions

$$h = b = \sqrt{\frac{N_{GS}}{\nu f_{cd}}} = \quad 627.6 \quad \text{mm}$$

Equivalent area for circular column

Φ = (4xbxh/Π)^0.5 = 708.4 mm say 750 mm

Axial Load for Typical Central column
DL.f = 2450 kN
LL.f = 785 kN

Intermediate roof
DL.R = 25m x 12.5mx DL.r= 1094 kN
LL.R = 25m x 12.5m x LL.r= 234 kN

Total Axial Load
N_{GS}= 6313.125 kN

Minimum column dimensions

$$h = b = \sqrt{\frac{N_{GS}}{\nu f_{cd}}} = \quad 860.1 \quad \text{mm}$$

Equivalent area for circular column

Φ = (4xbxh/Π)^0.5 = 970.8 mm say 950 mm

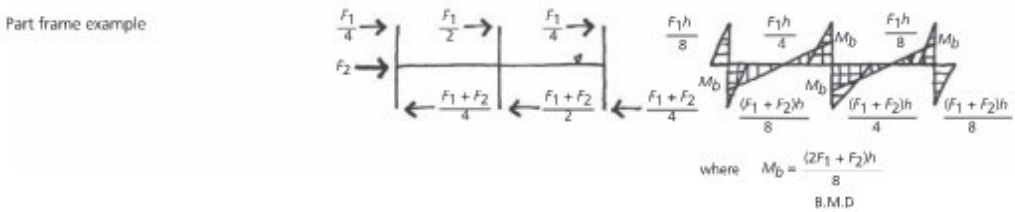
Geometry:
Column height H= 8 m for columns supporting 1st floor
H= 4 m for columns supporting 1st & mezzanine floc

Column design

Typical corner column

Total Axial Load
N_{GS}= 1656.3 kN

For the purpose of this design we will assume a lateral force of 0.2ag N_{GS} where 0.2 represents the seismic force
F= N_{GS} x 0.2ag= 166 kN



M= F x H/8 = 166 kNm

See tedds file "typical corner column"

Typical edge column

Total Axial Load
N_{GS}= 2941.2 kN

For the purpose of this design we will assume a lateral force of 0.2ag $N_{GS/q}$ where 0.2 represents the seismic force and q is the ductility factor of 2
F= N_{GS} x 0.2ag/q= 294 kN

M= F x H/8+ = 294 kNm

See tedds file "typical edge column"

Typical central column

Total Axial Load
N_{GS}= 6313 kN

For the purpose of this design we will assume a lateral force of 0.2ag $N_{GS/q}$ where 0.2 represents the seismic force and q is the ductility factor of 2

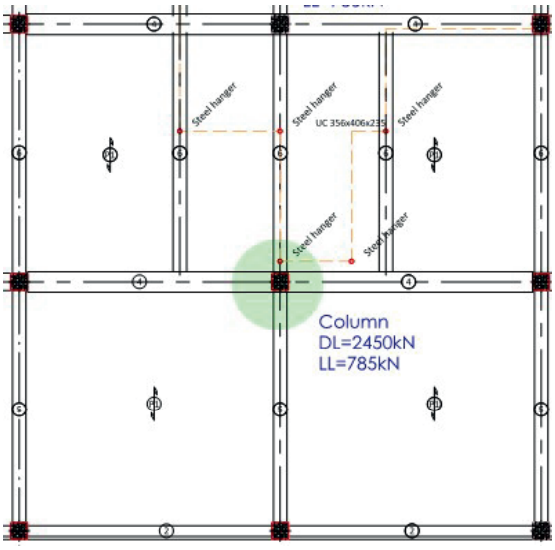
M= F x H/4 = 1263 kNm

Shear force. To avoid column failure under shear force, in design we will assume the equivalent shear force generated by moment

V= 2xM/H = 316 kN

See tedds file "typical edge column"

Central column with hanger loads



Additional loads from mezzanine

Steel work	=	0.75 kN/m2
Decking & finishes	=	7.75 kN/m2
Ceiling and services	=	1.5 kN/m2
Variable load	=	5 kN/m2

First floor

DL.f = 2450 kN

LL.f = 785 kN

Mezzanine floor

Tributary area for column A= 25 m2

DL.m = 250 kN

LL.m = 125 kN

Intermediate roof

DL.R = 25m x 12.5mx DL.r= 1094 kN

LL.R = 25m x 12.5m x LL.r= 234 kN

Total Axial Load

N_{GS}= 6838 kN

For the purpose of this design we will assume a lateral force of 0.2ag $N_{GS/q}$ where 0.2 represents the seismic force and q is the ductility factor of 2

F= N_{GS} x 0.2ag/q= 684 kN

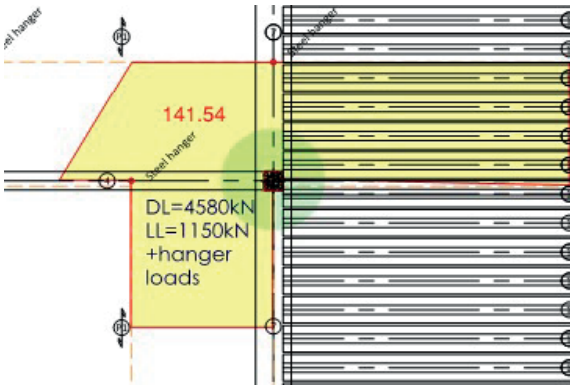
M= F x H/4 = 1368 kNm

See tedds file "Central column with hanger loads"

Central column with hanger loads supporting 25m span mezzanine

Additional loads from mezzanine

Steel work	=	0.75 kN/m2
Decking & finishes	=	7.75 kN/m2
Ceiling and services	=	1.5 kN/m2
Variable load	=	5 kN/m2



First floor

DL.f = 4580 kN

LL.f = 1150 kN

Mezzanine floor

Tributary area for column A= 142 m2

DL.m = 1415 kN

LL.m = 708 kN

Total Axial Load

N_{GS}= 10880 kN

For the purpose of this design we will assume a lateral force of 0.2ag $N_{GS/q}$ where 0.2 represents the seismic force and q is the ductility factor of 2

F= N_{GS} x 0.2ag/q= 1088 kN

M= F x H/4 = 1088 kNm

See tedds file "Central column with hanger loads supporting 25m span mezzanine"

Column at drop off area

Total Axial Load

N_{GS}= 24203 kN

For the purpose of this design we will assume a lateral force of 0.2ag $N_{GS/q}$ where 0.2 represents the seismic force and q is the ductility factor of 2

F= N_{GS} x 0.2ag/q= 2420 kN

M= F x H/8 = 2420 kNm

See tedds file "Column at drop off area "

Ground floor slab

The GF slab will be designed as a suspended slab.

		kN/m2
Finishes	(40mm stone + 20mm bedding)	1.2
Screed	75 mm	1.5
RC slab	350 mm	8.8

Permanent load DL.s= 11.5

Variable load LL.s= 5

Presizing d_{sl} = L/35 = 357.1 m say 350thk

See tedds file "GF slab "

Tie beams between columns at GF

Beam span L= 12.5 m Beams are continuous over supports

Load over beam

DL= DL.s /L = 143.1 kN/m

LL= LL.s /L = 62.5 kN/m

Use RC beam 1200dp x 650w

See tedds file "GF RC beam "

Foundation loads

Foundation under typical corner column

	column load		*GF slab & beam load*			
DL =	928.4	+	2026.4	=	2954.8	kN
LL =	268.6	+	781.2	=	1049.8	kN

See "Corner Column Pilecap Design"

Foundation under typical edge column

	column load		*GF slab & beam load*			
DL =	1598.4	+	3039.6	=	4638.0	kN
LL =	268.6	+	1171.8	=	1440.4	kN

See "Edger Column Pilecap Design"

Foundation under central column

	column load		*GF slab & beam load*			
DL =	3543.8	+	4052.8	=	7596.6	kN
LL =	1019.4	+	1562.4	=	2581.8	kN

See "Central Column Pilecap Design"

Foundation under Central column with hanger loads

	column load		*GF slab & beam load*			
DL =	3793.8	+	4052.8	=	7846.6	kN
LL =	1144.4	+	1562.4	=	2706.8	kN

See" Pilecap under Central column with hanger loads"

Foundation under Central column with hanger loads 25m span area

	column load		*GF slab & beam load*			
DL =	5995.0	+	4052.8	=	10047.8	kN
LL =	1857.5	+	1562.4	=	3419.9	kN

See "Pilecap under Central column with hanger loads (25m span area)"

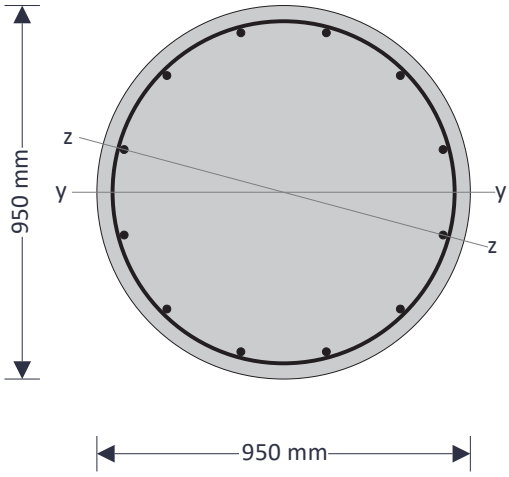
RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1096.78	186.29	0.17	PASS



12 no. 20 mm diameter longitudinal bars
10 mm diameter links
Max link spacing 400 mm generally, 240 mm for 950 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall diameter — h = **950** mm

Column clear height — l_y = **7000** mm

Stability — **Braced**

Concrete details

Concrete strength class — **C32/40**

Partial safety factor for concrete (2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) — α_{cc} = **0.85**

Maximum aggregate size — d_g = **20** mm

Reinforcement details

Nominal cover to links — c_{nom} = **35** mm

Longitudinal bar diameter — ϕ = **20** mm

Link diameter — ϕ_v = **10** mm

Total number of longitudinal bars — N = **12**

Area of longitudinal reinforcement — $A_s = N \times \pi \times \phi^2 / 4$ = **3770** mm^2

Characteristic yield strength — f_{yk} = **500** N/mm^2

Partial safety factor for reinft (2.4.2.4(1)) — γ_S = **1.15**

Modulus of elasticity of reinft (3.2.7(4)) — E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period — R = **120** min

Exposure to fire — **Exposed on more than one side**

Ratio of fire design axial load to design resistance — μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load — N_{Ed} = **1656.3** kN

Moment at top — M_{topy} = **166.0** kNm

Moment at bottom — M_{btmy} = **-166.0** kNm

Column effective length factors

Effective length factor — f_y = **0.70**

Calculated column properties

Concrete properties

Area of concrete — $A_c = \pi \times h^2 / 4$ = **708822** mm^2

Characteristic compression cylinder strength — f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) — λ_{sb} = 0.8

Stress factor (3.1.7(3)) — η = 0.9

Strain limits

Compression strain limit (Table 3.1) — ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) — ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) — $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) — $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **10** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) — a_{fi} = **51** mm

Allowance for deviations from min cover (4.4.1.3) — Δc_{dev} = **10** mm

Min allowable nominal cover — $c_{nom_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **31.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depth and inertia of bars for bending about y axis

For the purposes of determining the bending capacity and interaction diagrams in this Tedds calculation, bending about the y axis is taken to be when there are two furthest equidistant bars on each side of the column centreline. Bending about the z axis is taken to be when there is one furthest bar on each side of the column centreline.

Area per bar — $A_{bar} = \pi \times \phi^2 / 4$ = **314** mm^2

Radial dist from column centre to longitudinal bar — $r_l = h / 2 - c_{nom} - \phi_v - \phi / 2$ = **420.0** mm

Subtended angle between adjacent bars — α = (360 deg) / N = **30.0** deg

Layer 1 — $d_{y1} = h / 2 + r_l \times \cos(\alpha / 2)$ = **880.7** mm

2nd moment of area of reinft about y axis — $I_{y1} = 2 \times A_{bar} \times (d_{y1} - h / 2)^2$ = **10341** cm^4

Library item - Depth layer 1 y circ case 2

Layer 2	$d_{y2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha + \alpha / 2] =$ **772.0** mm
2nd moment of area of reinft about y axis	$I_{y2} = 2 \times A_{bar} \times (d_{y2} - h / 2)^2 =$ **5542** cm^4
Layer 3	$d_{y3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha + \alpha / 2] =$ **583.7** mm
2nd moment of area of reinft about y axis	$I_{y3} = 2 \times A_{bar} \times (d_{y3} - h / 2)^2 =$ **742** cm^4
Layer 4	$d_{y4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha + \alpha / 2] =$ **366.3** mm
2nd moment of area of reinft about y axis	$I_{y4} = 2 \times A_{bar} \times (d_{y4} - h / 2)^2 =$ **742** cm^4
Layer 5	$d_{y5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha + \alpha / 2] =$ **178.0** mm
2nd moment of area of reinft about y axis	$I_{y5} = 2 \times A_{bar} \times (d_{y5} - h / 2)^2 =$ **5542** cm^4
Layer 6	$d_{y6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha + \alpha / 2] =$ **69.3** mm
2nd moment of area of reinft about y axis	$I_{y6} = 2 \times A_{bar} \times (d_{y6} - h / 2)^2 =$ **10341** cm^4
Total 2nd moment of area of reinft about y axis	$I_{sy} =$ **33251** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **297** mm
Effective depth about y axis (5.8.8.3(2))	$d_y = h / 2 + i_{sy} =$ **772** mm

Effective depth of bars for bending about z axis

Layer 1 (tension face)	$d_{z1} = h / 2 + r_l =$ **895.0** mm
2nd moment of area of reinft about z axis	$I_{z1} = A_{bar} \times (d_{z1} - h / 2)^2 =$ **5542** cm^4
Layer 2	$d_{z2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha] =$ **838.7** mm
2nd moment of area of reinft about z axis	$I_{z2} = 2 \times A_{bar} \times (d_{z2} - h / 2)^2 =$ **8313** cm^4
Layer 3	$d_{z3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha] =$ **685.0** mm
2nd moment of area of reinft about z axis	$I_{z3} = 2 \times A_{bar} \times (d_{z3} - h / 2)^2 =$ **2771** cm^4
Layer 4	$d_{z4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha] =$ **475.0** mm
2nd moment of area of reinft about z axis	$I_{z4} = 2 \times A_{bar} \times (d_{z4} - h / 2)^2 =$ **0** cm^4
Layer 5	$d_{z5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha] =$ **265.0** mm
2nd moment of area of reinft about z axis	$I_{z5} = 2 \times A_{bar} \times (d_{z5} - h / 2)^2 =$ **2771** cm^4
Layer 6	$d_{z6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha] =$ **111.3** mm
2nd moment of area of reinft about z axis	$I_{z6} = 2 \times A_{bar} \times (d_{z6} - h / 2)^2 =$ **8313** cm^4
Layer 7	$d_{z7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha] =$ **55.0** mm
2nd moment of area of reinft about z axis	$I_{z7} = 1 \times A_{bar} \times (d_{z7} - h / 2)^2 =$ **5542** cm^4
Total 2nd moment of area of reinft about z axis	$I_{sz} =$ **33251** cm^4
Radius of gyration of reinft about z axis	$i_{sz} = \sqrt{(I_{sz} / A_s)} =$ **297** mm
Effective depth about z axis (5.8.8.3(2))	$d_z = b / 2 + i_{sz} =$ **772** mm

Column effective length

Column effective length	$l_{0y} = f_y \times l_y =$ **4900** mm

Column slenderness

Radius of gyration	$i_y = h / 4 =$ **23.8** cm
Slenderness ratio (5.8.3.2(1))	$\lambda_y = l_{0y} / i_y =$ **20.6**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections	$e_{iy} = l_{0y} /400 =$ **12.2** mm
Minimum end moment	$M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **186.3** kNm
Maximum end moment	$M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **186.3** kNm

Slenderness limit (cl. 5.8.3.1)

Factor A	$A =$ **0.7**
Mechanical reinforcement ratio	$\omega = A_s \times f_{yd} / (A_c \times f_{cd}) =$ **0.128**

Factor B	$B = \sqrt{(1 + 2 \times \omega)}$ = **1.120**
Moment ratio	$r_{my} = -1.0 \times M_{01y} / M_{02y}$ = **-1.000**
Factor C	$C_y = 1.7 - r_{my}$ = **2.700**
Relative normal force	$n = N_{Ed} / (A_c \times f_{cd})$ = **0.129**
Slenderness limit	$\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)}$ = **118.0**

$\lambda_y < \lambda_{limy}$ - Second order effects may be ignored

Design bending moments (cl. 6.1(4))

Design moment	$M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20\text{ mm}))$ = **186.3** kNm

Resultant design bending moment for a circular column

Resultant design moment	$M_{Ed} = \sqrt{(M_{Edy}^2 + M_{Edz}^2)}$ = **186.3** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis	y = **288.0** mm
Depth of stress block	$d_{sby} = \min(\lambda_{sb} \times y, h)$ = **230.4** mm
Area of concrete in compression	A_{sby} = **132789** mm^2
Concrete compression force (3.1.7(3))	$F_{yc} = \eta \times f_{cd} \times A_{sby}$ = **2167.1** kN
Centroid of concrete compression from column cl	y_{sby} = **339.0** mm
Moment of resistance	$M_{Rdyc} = F_{yc} \times y_{sby}$ = **734.6** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y)$ = **-0.00720**
Stress in layer 1	$\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1})$ = **-434.8** N/mm^2
Force in layer 1	$F_{y1} = 2 \times A_{bar} \times \sigma_{y1}$ = **-273.2** kN
Moment of resistance of layer 1	$M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1})$ = **110.8** kNm
Strain in layer 2	$\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y)$ = **-0.00588**
Stress in layer 2	$\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2})$ = **-434.8** N/mm^2
Force in layer 2	$F_{y2} = 2 \times A_{bar} \times \sigma_{y2}$ = **-273.2** kN
Moment of resistance of layer 2	$M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2})$ = **81.1** kNm
Strain in layer 3	$\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y)$ = **-0.00359**
Stress in layer 3	$\sigma_{y3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y3})$ = **-434.8** N/mm^2
Force in layer 3	$F_{y3} = 2 \times A_{bar} \times \sigma_{y3}$ = **-273.2** kN
Moment of resistance of layer 3	$M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3})$ = **29.7** kNm
Strain in layer 4	$\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y)$ = **-0.00095**
Stress in layer 4	$\sigma_{y4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y4})$ = **-190.2** N/mm^2
Force in layer 4	$F_{y4} = 2 \times A_{bar} \times \sigma_{y4}$ = **-119.5** kN
Moment of resistance of layer 4	$M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **-13.0** kNm
Strain in layer 5	$\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **0.00134**
Stress in layer 5	$\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd}$ = **251.1** N/mm^2
Force in layer 5	$F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **157.7** kN
Moment of resistance of layer 5	$M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **46.8** kNm
Strain in layer 6	$\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00266**
Stress in layer 6	$\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 6	$F_{y6} = 2 \times A_{bar} \times \sigma_{y6}$ = **262.9** kN
Moment of resistance of layer 6	$M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **106.7** kNm

Resultant concrete/steel force F_y = **1648.8** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis M_{Rdy} = **1096.8** kNm

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis z = **289.0** mm

Depth of stress block $d_{sbz} = \min(\lambda_{sb} \times z , h)$ = **231.2** mm

Area of concrete in compression A_{sbz} = **133395** mm^2

Concrete compression force (3.1.7(3)) $F_{zc} = \eta \times f_{cd} \times A_{sbz}$ = **2177.0** kN

Centroid of concrete compression from column cl y_{sbz} = **338.5** mm

Moment of resistance $M_{Rdzc} = F_{zc} \times y_{sbz}$ = **737.0** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00734**

Stress in layer 1 $\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-434.8** N/mm^2

Force in layer 1 $F_{z1} = 1 \times A_{bar} \times \sigma_{z1}$ = **-136.6** kN

Moment of resistance of layer 1 $M_{Rdz1} = F_{z1} \times (h / 2 – d_{z1})$ = **57.4** kNm

Strain in layer 2 $\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **-0.00666**

Stress in layer 2 $\sigma_{z2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z2})$ = **-434.8** N/mm^2

Force in layer 2 $F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ = **-273.2** kN

Moment of resistance of layer 2 $M_{Rdz2} = F_{z2} \times (h / 2 – d_{z2})$ = **99.4** kNm

Strain in layer 3 $\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z)$ = **-0.00480**

Stress in layer 3 $\sigma_{z3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z3})$ = **-434.8** N/mm^2

Force in layer 3 $F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ = **-273.2** kN

Moment of resistance of layer 3 $M_{Rdz3} = F_{z3} \times (h / 2 – d_{z3})$ = **57.4** kNm

Strain in layer 4 $\varepsilon_{z4} = \varepsilon_{cu3} \times (1 - d_{z4} / z)$ = **-0.00225**

Stress in layer 4 $\sigma_{z4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z4})$ = **-434.8** N/mm^2

Force in layer 4 $F_{z4} = 2 \times A_{bar} \times \sigma_{z4}$ = **-273.2** kN

Moment of resistance of layer 4 $M_{Rdz4} = F_{z4} \times (h / 2 – d_{z4})$ = **0.0** kNm

Strain in layer 5 $\varepsilon_{z5} = \varepsilon_{cu3} \times (1 - d_{z5} / z)$ = **0.00029**

Stress in layer 5 $\sigma_{z5} = \min(f_{yd}, E_s \times \varepsilon_{z5})$ = **58.1** N/mm^2

Force in layer 5 $F_{z5} = 2 \times A_{bar} \times \sigma_{z5}$ = **36.5** kN

Moment of resistance of layer 5 $M_{Rdz5} = F_{z5} \times (h / 2 – d_{z5})$ = **7.7** kNm

Strain in layer 6 $\varepsilon_{z6} = \varepsilon_{cu3} \times (1 - d_{z6} / z)$ = **0.00215**

Stress in layer 6 $\sigma_{z6} = \min(f_{yd}, E_s \times \varepsilon_{z6}) - \eta \times f_{cd}$ = **414.1** N/mm^2

Force in layer 6 $F_{z6} = 2 \times A_{bar} \times \sigma_{z6}$ = **260.2** kN

Moment of resistance of layer 6 $M_{Rdz6} = F_{z6} \times (h / 2 – d_{z6})$ = **94.6** kNm

Strain in layer 7 $\varepsilon_{z7} = \varepsilon_{cu3} \times (1 - d_{z7} / z)$ = **0.00283**

Stress in layer 7 $\sigma_{z7} = \min(f_{yd}, E_s \times \varepsilon_{z7}) - \eta \times f_{cd}$ = **418.5** N/mm^2

Force in layer 7 $F_{z7} = 1 \times A_{bar} \times \sigma_{z7}$ = **131.5** kN

Moment of resistance of layer 7 $M_{Rdz7} = F_{z7} \times (h / 2 – d_{z7})$ = **55.2** kNm

Resultant concrete/steel force F_z = **1649.0** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis M_{Rdz} = **1108.6** kNm

Minimum moment capacity with axial load N_{Ed}

Minimum moment capacity $M_{Rd} = \min(M_{Rdz}, M_{Rdy})$ = **1096.8** kNm

PASS - The moment capacity exceeds the resultant design bending moment

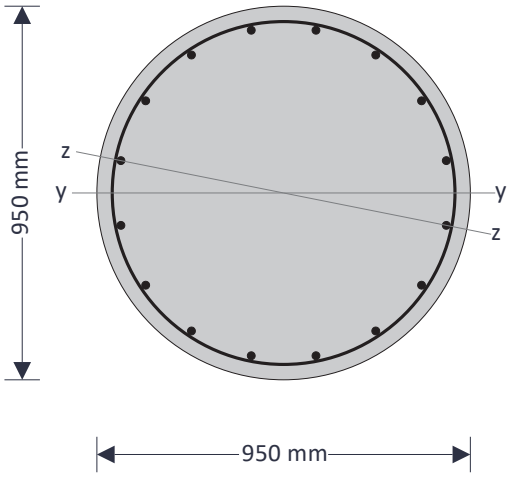
RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1486.32	335.17	0.23	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for 950 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall diameter — h = **950** mm

Column clear height — l_y = **8000** mm

Stability — **Braced**

Concrete details

Concrete strength class — **C32/40**

Partial safety factor for concrete (2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) — α_{cc} = **0.85**

Maximum aggregate size — d_g = **20** mm

Reinforcement details

Nominal cover to links — c_{nom} = **35** mm

Longitudinal bar diameter — ϕ = **20** mm

Link diameter — ϕ_v = **8** mm

Total number of longitudinal bars — N = **16**

Area of longitudinal reinforcement — $A_s = N \times \pi \times \phi^2 / 4$ = **5027** mm^2

Characteristic yield strength — f_{yk} = **500** N/mm^2

Partial safety factor for reinft (2.4.2.4(1)) — γ_S = **1.15**

Modulus of elasticity of reinft (3.2.7(4)) — E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period — R = **120** min

Exposure to fire — **Exposed on more than one side**

Ratio of fire design axial load to design resistance — μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load — N_{Ed} = **2941.0** kN

Moment at top — M_{topy} = **294.0** kNm

Moment at bottom — M_{btmy} = **-294.0** kNm

Column effective length factors

Effective length factor — f_y = **0.70**

Calculated column properties

Concrete properties

Area of concrete — $A_c = \pi \times h^2 / 4$ = **708822** mm^2

Characteristic compression cylinder strength — f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) — λ_{sb} = 0.8

Stress factor (3.1.7(3)) — η = 0.9

Strain limits

Compression strain limit (Table 3.1) — ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) — ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) — $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) — $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **12** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) — a_{fi} = **51** mm

Allowance for deviations from min cover (4.4.1.3) — Δc_{dev} = **10** mm

Min allowable nominal cover — $c_{nom_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **33.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depth and inertia of bars for bending about y axis

For the purposes of determining the bending capacity and interaction diagrams in this Tedds calculation, bending about the y axis is taken to be when there are two furthest equidistant bars on each side of the column centreline. Bending about the z axis is taken to be when there is one furthest bar on each side of the column centreline.

Area per bar — $A_{bar} = \pi \times \phi^2 / 4$ = **314** mm^2

Radial dist from column centre to longitudinal bar — $r_l = h / 2 - c_{nom} - \phi_v - \phi / 2$ = **422.0** mm

Subtended angle between adjacent bars — α = (360 deg) / N = **22.5** deg

Layer 1 — $d_{y1} = h / 2 + r_l \times \cos(\alpha / 2)$ = **888.9** mm

2nd moment of area of reinft about y axis — $I_{y1} = 2 \times A_{bar} \times (d_{y1} - h / 2)^2$ = **10763** cm^4

Library item - Depth layer 1 y circ case 2

Layer 2	$d_{y2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha + \alpha / 2] =$ **825.9** mm
2nd moment of area of reinft about y axis	$I_{y2} = 2 \times A_{bar} \times (d_{y2} - h / 2)^2 =$ **7736** cm^4
Layer 3	$d_{y3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha + \alpha / 2] =$ **709.5** mm
2nd moment of area of reinft about y axis	$I_{y3} = 2 \times A_{bar} \times (d_{y3} - h / 2)^2 =$ **3454** cm^4
Layer 4	$d_{y4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha + \alpha / 2] =$ **557.3** mm
2nd moment of area of reinft about y axis	$I_{y4} = 2 \times A_{bar} \times (d_{y4} - h / 2)^2 =$ **426** cm^4
Layer 5	$d_{y5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha + \alpha / 2] =$ **392.7** mm
2nd moment of area of reinft about y axis	$I_{y5} = 2 \times A_{bar} \times (d_{y5} - h / 2)^2 =$ **426** cm^4
Layer 6	$d_{y6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha + \alpha / 2] =$ **240.5** mm
2nd moment of area of reinft about y axis	$I_{y6} = 2 \times A_{bar} \times (d_{y6} - h / 2)^2 =$ **3454** cm^4
Layer 7	$d_{y7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha + \alpha / 2] =$ **124.1** mm
2nd moment of area of reinft about y axis	$I_{y7} = 2 \times A_{bar} \times (d_{y7} - h / 2)^2 =$ **7736** cm^4
Layer 8	$d_{y8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha + \alpha / 2] =$ **61.1** mm
2nd moment of area of reinft about y axis	$I_{y8} = 2 \times A_{bar} \times (d_{y8} - h / 2)^2 =$ **10763** cm^4
Total 2nd moment of area of reinft about y axis	$I_{sy} =$ **44757** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **298** mm
Effective depth about y axis (5.8.8.3(2))	$d_y = h / 2 + i_{sy} =$ **773** mm

Effective depth of bars for bending about z axis

Layer 1 (tension face)	$d_{z1} = h / 2 + r_l =$ **897.0** mm
2nd moment of area of reinft about z axis	$I_{z1} = A_{bar} \times (d_{z1} - h / 2)^2 =$ **5595** cm^4
Layer 2	$d_{z2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha] =$ **864.9** mm
2nd moment of area of reinft about z axis	$I_{z2} = 2 \times A_{bar} \times (d_{z2} - h / 2)^2 =$ **9551** cm^4
Layer 3	$d_{z3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha] =$ **773.4** mm
2nd moment of area of reinft about z axis	$I_{z3} = 2 \times A_{bar} \times (d_{z3} - h / 2)^2 =$ **5595** cm^4
Layer 4	$d_{z4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha] =$ **636.5** mm
2nd moment of area of reinft about z axis	$I_{z4} = 2 \times A_{bar} \times (d_{z4} - h / 2)^2 =$ **1639** cm^4
Layer 5	$d_{z5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha] =$ **475.0** mm
2nd moment of area of reinft about z axis	$I_{z5} = 2 \times A_{bar} \times (d_{z5} - h / 2)^2 =$ **0** cm^4
Layer 6	$d_{z6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha] =$ **313.5** mm
2nd moment of area of reinft about z axis	$I_{z6} = 2 \times A_{bar} \times (d_{z6} - h / 2)^2 =$ **1639** cm^4
Layer 7	$d_{z7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha] =$ **176.6** mm
2nd moment of area of reinft about z axis	$I_{z7} = 2 \times A_{bar} \times (d_{z7} - h / 2)^2 =$ **5595** cm^4
Layer 8	$d_{z8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha] =$ **85.1** mm
2nd moment of area of reinft about z axis	$I_{z8} = 2 \times A_{bar} \times (d_{z8} - h / 2)^2 =$ **9551** cm^4
Layer 9	$d_{z9} = h / 2 + r_l \times \cos[(9 - 1) \times \alpha] =$ **53.0** mm
2nd moment of area of reinft about z axis	$I_{z9} = 1 \times A_{bar} \times (d_{z9} - h / 2)^2 =$ **5595** cm^4
Total 2nd moment of area of reinft about z axis	$I_{sz} =$ **44757** cm^4
Radius of gyration of reinft about z axis	$i_{sz} = \sqrt{(I_{sz} / A_s)} =$ **298** mm
Effective depth about z axis (5.8.8.3(2))	$d_z = b / 2 + i_{sz} =$ **773** mm

Column effective length

Column effective length	$l_{0y} = f_y \times l_y =$ **5600** mm

Column slenderness

Radius of gyration	$i_y = h / 4 =$ **23.8** cm
Slenderness ratio (5.8.3.2(1))	$\lambda_y = l_{0y} / i_y =$ **23.6**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections $e_{iy} = l_{0y} /400 =$ **14.0** mm

Minimum end moment $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **335.2** kNm

Maximum end moment $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **335.2** kNm

Slenderness limit (cl. 5.8.3.1)

Factor A $A =$ **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) =$ **0.170**

Factor B $B = \sqrt{(1 + 2 \times \omega)} =$ **1.158**

Moment ratio $r_{my} = -1.0 \times M_{01y} / M_{02y} =$ **-1.000**

Factor C $C_y = 1.7 - r_{my} =$ **2.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) =$ **0.229**

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} =$ **91.5**

$\lambda_y < \lambda_{limy}$ - Second order effects may be ignored

Design bending moments (cl. 6.1(4))

Design moment $M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20 \text{ mm})) =$ **335.2** kNm

Resultant design bending moment for a circular column

Resultant design moment $M_{Ed} = \sqrt{(M_{Edy}^2 + M_{Edz}^2)} =$ **335.2** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $y =$ **386.9** mm

Depth of stress block $d_{sby} = \min(\lambda_{sb} \times y , h) =$ **309.5** mm

Area of concrete in compression $A_{sby} =$ **200421** mm^2

Concrete compression force (3.1.7(3)) $F_{yc} = \eta \times f_{cd} \times A_{sby} =$ **3270.9** kN

Centroid of concrete compression from column cl $y_{sby} =$ **293.6** mm

Moment of resistance $M_{Rdyc} = F_{yc} \times y_{sby} =$ **960.3** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) =$ **-0.00454**

Stress in layer 1 $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) =$ **-434.8** N/mm^2

Force in layer 1 $F_{y1} = 2 \times A_{bar} \times \sigma_{y1} =$ **-273.2** kN

Moment of resistance of layer 1 $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) =$ **113.1** kNm

Strain in layer 2 $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) =$ **-0.00397**

Stress in layer 2 $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2}) =$ **-434.8** N/mm^2

Force in layer 2 $F_{y2} = 2 \times A_{bar} \times \sigma_{y2} =$ **-273.2** kN

Moment of resistance of layer 2 $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) =$ **95.9** kNm

Strain in layer 3 $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y) =$ **-0.00292**

Stress in layer 3 $\sigma_{y3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y3}) =$ **-434.8** N/mm^2

Force in layer 3 $F_{y3} = 2 \times A_{bar} \times \sigma_{y3} =$ **-273.2** kN

Moment of resistance of layer 3 $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3}) =$ **64.0** kNm

Strain in layer 4 $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y) =$ **-0.00154**

Stress in layer 4 $\sigma_{y4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y4}) =$ **-308.4** N/mm^2

Force in layer 4 $F_{y4} = 2 \times A_{bar} \times \sigma_{y4} =$ **-193.8** kN

Moment of resistance of layer 4	$M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **16.0** kNm
Strain in layer 5	$\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **-0.00005**
Stress in layer 5	$\sigma_{y5} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y5})$ = **-10.5** N/mm^2
Force in layer 5	$F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **-6.6** kN
Moment of resistance of layer 5	$M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **-0.5** kNm
Strain in layer 6	$\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00132**
Stress in layer 6	$\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **248.4** N/mm^2
Force in layer 6	$F_{y6} = 2 \times A_{bar} \times \sigma_{y6}$ = **156.1** kN
Moment of resistance of layer 6	$M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **36.6** kNm
Strain in layer 7	$\varepsilon_{y7} = \varepsilon_{cu3} \times (1 - d_{y7} / y)$ = **0.00238**
Stress in layer 7	$\sigma_{y7} = \min(f_{yd}, E_s \times \varepsilon_{y7}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 7	$F_{y7} = 2 \times A_{bar} \times \sigma_{y7}$ = **262.9** kN
Moment of resistance of layer 7	$M_{Rdy7} = F_{y7} \times (h / 2 - d_{y7})$ = **92.3** kNm
Strain in layer 8	$\varepsilon_{y8} = \varepsilon_{cu3} \times (1 - d_{y8} / y)$ = **0.00295**
Stress in layer 8	$\sigma_{y8} = \min(f_{yd}, E_s \times \varepsilon_{y8}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 8	$F_{y8} = 2 \times A_{bar} \times \sigma_{y8}$ = **262.9** kN
Moment of resistance of layer 8	$M_{Rdy8} = F_{y8} \times (h / 2 - d_{y8})$ = **108.8** kNm
Resultant concrete/steel force	F_y = **2932.9** kN
	PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis	M_{Rdy} = **1486.3** kNm

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis	z = **387.3** mm
Depth of stress block	$d_{sbz} = \min(\lambda_{sb} \times z , h)$ = **309.8** mm
Area of concrete in compression	A_{sbz} = **200727** mm^2
Concrete compression force (3.1.7(3))	$F_{zc} = \eta \times f_{cd} \times A_{sbz}$ = **3275.9** kN
Centroid of concrete compression from column cl	y_{sbz} = **293.4** mm
Moment of resistance	$M_{Rdzc} = F_{zc} \times y_{sbz}$ = **961.1** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00461**
Stress in layer 1	$\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-434.8** N/mm^2
Force in layer 1	$F_{z1} = 1 \times A_{bar} \times \sigma_{z1}$ = **-136.6** kN
Moment of resistance of layer 1	$M_{Rdz1} = F_{z1} \times (h / 2 – d_{z1})$ = **57.6** kNm
Strain in layer 2	$\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **-0.00432**
Stress in layer 2	$\sigma_{z2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z2})$ = **-434.8** N/mm^2
Force in layer 2	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ = **-273.2** kN
Moment of resistance of layer 2	$M_{Rdz2} = F_{z2} \times (h / 2 – d_{z2})$ = **106.5** kNm
Strain in layer 3	$\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z)$ = **-0.00349**
Stress in layer 3	$\sigma_{z3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z3})$ = **-434.8** N/mm^2
Force in layer 3	$F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ = **-273.2** kN
Moment of resistance of layer 3	$M_{Rdz3} = F_{z3} \times (h / 2 – d_{z3})$ = **81.5** kNm

Strain in layer 4	$\varepsilon_{z4} = \varepsilon_{cu3} \times (1 - d_{z4} / z) =$ **-0.00225**
Stress in layer 4	$\sigma_{z4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z4}) =$ **-434.8** N/mm^2
Force in layer 4	$F_{z4} = 2 \times A_{bar} \times \sigma_{z4} =$ **-273.2** kN
Moment of resistance of layer 4	$M_{Rdz4} = F_{z4} \times (h / 2 – d_{z4}) =$ **44.1** kNm
Strain in layer 5	$\varepsilon_{z5} = \varepsilon_{cu3} \times (1 - d_{z5} / z) =$ **-0.00079**
Stress in layer 5	$\sigma_{z5} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z5}) =$ **-158.5** N/mm^2
Force in layer 5	$F_{z5} = 2 \times A_{bar} \times \sigma_{z5} =$ **-99.6** kN
Moment of resistance of layer 5	$M_{Rdz5} = F_{z5} \times (h / 2 – d_{z5}) =$ **0.0** kNm
Strain in layer 6	$\varepsilon_{z6} = \varepsilon_{cu3} \times (1 - d_{z6} / z) =$ **0.00067**
Stress in layer 6	$\sigma_{z6} = \min(f_{yd}, E_s \times \varepsilon_{z6}) =$ **133.4** N/mm^2
Force in layer 6	$F_{z6} = 2 \times A_{bar} \times \sigma_{z6} =$ **83.8** kN
Moment of resistance of layer 6	$M_{Rdz6} = F_{z6} \times (h / 2 – d_{z6}) =$ **13.5** kNm
Strain in layer 7	$\varepsilon_{z7} = \varepsilon_{cu3} \times (1 - d_{z7} / z) =$ **0.00190**
Stress in layer 7	$\sigma_{z7} = \min(f_{yd}, E_s \times \varepsilon_{z7}) - \eta \times f_{cd} =$ **364.5** N/mm^2
Force in layer 7	$F_{z7} = 2 \times A_{bar} \times \sigma_{z7} =$ **229.0** kN
Moment of resistance of layer 7	$M_{Rdz7} = F_{z7} \times (h / 2 – d_{z7}) =$ **68.3** kNm
Strain in layer 8	$\varepsilon_{z8} = \varepsilon_{cu3} \times (1 - d_{z8} / z) =$ **0.00273**
Stress in layer 8	$\sigma_{z8} = \min(f_{yd}, E_s \times \varepsilon_{z8}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 8	$F_{z8} = 2 \times A_{bar} \times \sigma_{z8} =$ **262.9** kN
Moment of resistance of layer 8	$M_{Rdz8} = F_{z8} \times (h / 2 – d_{z8}) =$ **102.5** kNm
Strain in layer 9	$\varepsilon_{z9} = \varepsilon_{cu3} \times (1 - d_{z9} / z) =$ **0.00302**
Stress in layer 9	$\sigma_{z9} = \min(f_{yd}, E_s \times \varepsilon_{z9}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 9	$F_{z9} = 1 \times A_{bar} \times \sigma_{z9} =$ **131.5** kN
Moment of resistance of layer 9	$M_{Rdz9} = F_{z9} \times (h / 2 – d_{z9}) =$ **55.5** kNm
Resultant concrete/steel force	$F_z =$ **2927.3** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis $M_{Rdz} =$ **1490.7** kNm

Minimum moment capacity with axial load N_{Ed}

Minimum moment capacity $M_{Rd} = \min(M_{Rdz}, M_{Rdy}) =$ **1486.3** kNm

PASS - The moment capacity exceeds the resultant design bending moment

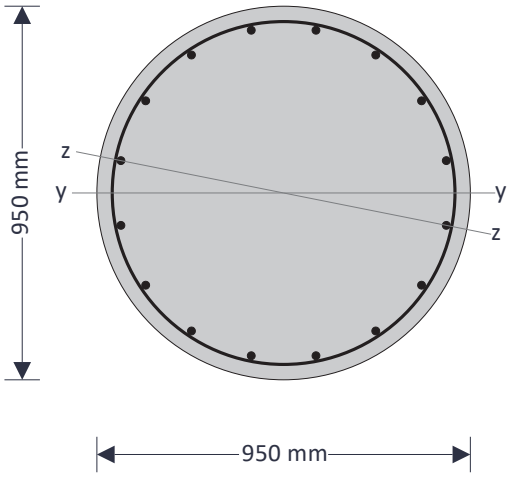
RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1604.34	1340.33	0.84	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for 950 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall diameter — h = **950** mm

Column clear height — l_y = **7000** mm

Stability — **Braced**

Concrete details

Concrete strength class — **C32/40**

Partial safety factor for concrete (2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) — α_{cc} = **0.85**

Maximum aggregate size — d_g = **20** mm

Reinforcement details

Nominal cover to links — c_{nom} = **35** mm

Longitudinal bar diameter — ϕ = **20** mm

Link diameter — ϕ_v = **8** mm

Total number of longitudinal bars — N = **16**

Area of longitudinal reinforcement — $A_s = N \times \pi \times \phi^2 / 4$ = **5027** mm^2

Characteristic yield strength — f_{yk} = **500** N/mm^2

Partial safety factor for reinft (2.4.2.4(1)) — γ_S = **1.15**

Modulus of elasticity of reinft (3.2.7(4)) — E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period — R = **120** min

Exposure to fire — **Exposed on more than one side**

Ratio of fire design axial load to design resistance — μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load — N_{Ed} = **6313.0** kN

Moment at top — M_{topy} = **1263.0** kNm

Moment at bottom — M_{btmy} = **-1263.0** kNm

Column effective length factors

Effective length factor — f_y = **0.70**

Calculated column properties

Concrete properties

Area of concrete — $A_c = \pi \times h^2 / 4$ = **708822** mm^2

Characteristic compression cylinder strength — f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) — λ_{sb} = 0.8

Stress factor (3.1.7(3)) — η = 0.9

Strain limits

Compression strain limit (Table 3.1) — ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) — ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) — $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) — $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **12** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) — a_{fi} = **51** mm

Allowance for deviations from min cover (4.4.1.3) — Δc_{dev} = **10** mm

Min allowable nominal cover — $c_{nom_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **33.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depth and inertia of bars for bending about y axis

For the purposes of determining the bending capacity and interaction diagrams in this Tedds calculation, bending about the y axis is taken to be when there are two furthest equidistant bars on each side of the column centreline. Bending about the z axis is taken to be when there is one furthest bar on each side of the column centreline.

Area per bar — $A_{bar} = \pi \times \phi^2 / 4$ = **314** mm^2

Radial dist from column centre to longitudinal bar — $r_l = h / 2 - c_{nom} - \phi_v - \phi / 2$ = **422.0** mm

Subtended angle between adjacent bars — α = (360 deg) / N = **22.5** deg

Layer 1 — $d_{y1} = h / 2 + r_l \times \cos(\alpha / 2)$ = **888.9** mm

2nd moment of area of reinft about y axis — $I_{y1} = 2 \times A_{bar} \times (d_{y1} - h / 2)^2$ = **10763** cm^4

Library item - Depth layer 1 y circ case 2

Layer 2	$d_{y2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha + \alpha / 2] =$ **825.9** mm
2nd moment of area of reinft about y axis	$I_{y2} = 2 \times A_{bar} \times (d_{y2} - h / 2)^2 =$ **7736** cm^4
Layer 3	$d_{y3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha + \alpha / 2] =$ **709.5** mm
2nd moment of area of reinft about y axis	$I_{y3} = 2 \times A_{bar} \times (d_{y3} - h / 2)^2 =$ **3454** cm^4
Layer 4	$d_{y4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha + \alpha / 2] =$ **557.3** mm
2nd moment of area of reinft about y axis	$I_{y4} = 2 \times A_{bar} \times (d_{y4} - h / 2)^2 =$ **426** cm^4
Layer 5	$d_{y5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha + \alpha / 2] =$ **392.7** mm
2nd moment of area of reinft about y axis	$I_{y5} = 2 \times A_{bar} \times (d_{y5} - h / 2)^2 =$ **426** cm^4
Layer 6	$d_{y6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha + \alpha / 2] =$ **240.5** mm
2nd moment of area of reinft about y axis	$I_{y6} = 2 \times A_{bar} \times (d_{y6} - h / 2)^2 =$ **3454** cm^4
Layer 7	$d_{y7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha + \alpha / 2] =$ **124.1** mm
2nd moment of area of reinft about y axis	$I_{y7} = 2 \times A_{bar} \times (d_{y7} - h / 2)^2 =$ **7736** cm^4
Layer 8	$d_{y8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha + \alpha / 2] =$ **61.1** mm
2nd moment of area of reinft about y axis	$I_{y8} = 2 \times A_{bar} \times (d_{y8} - h / 2)^2 =$ **10763** cm^4
Total 2nd moment of area of reinft about y axis	$I_{sy} =$ **44757** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **298** mm
Effective depth about y axis (5.8.8.3(2))	$d_y = h / 2 + i_{sy} =$ **773** mm

Effective depth of bars for bending about z axis

Layer 1 (tension face)	$d_{z1} = h / 2 + r_l =$ **897.0** mm
2nd moment of area of reinft about z axis	$I_{z1} = A_{bar} \times (d_{z1} - h / 2)^2 =$ **5595** cm^4
Layer 2	$d_{z2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha] =$ **864.9** mm
2nd moment of area of reinft about z axis	$I_{z2} = 2 \times A_{bar} \times (d_{z2} - h / 2)^2 =$ **9551** cm^4
Layer 3	$d_{z3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha] =$ **773.4** mm
2nd moment of area of reinft about z axis	$I_{z3} = 2 \times A_{bar} \times (d_{z3} - h / 2)^2 =$ **5595** cm^4
Layer 4	$d_{z4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha] =$ **636.5** mm
2nd moment of area of reinft about z axis	$I_{z4} = 2 \times A_{bar} \times (d_{z4} - h / 2)^2 =$ **1639** cm^4
Layer 5	$d_{z5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha] =$ **475.0** mm
2nd moment of area of reinft about z axis	$I_{z5} = 2 \times A_{bar} \times (d_{z5} - h / 2)^2 =$ **0** cm^4
Layer 6	$d_{z6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha] =$ **313.5** mm
2nd moment of area of reinft about z axis	$I_{z6} = 2 \times A_{bar} \times (d_{z6} - h / 2)^2 =$ **1639** cm^4
Layer 7	$d_{z7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha] =$ **176.6** mm
2nd moment of area of reinft about z axis	$I_{z7} = 2 \times A_{bar} \times (d_{z7} - h / 2)^2 =$ **5595** cm^4
Layer 8	$d_{z8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha] =$ **85.1** mm
2nd moment of area of reinft about z axis	$I_{z8} = 2 \times A_{bar} \times (d_{z8} - h / 2)^2 =$ **9551** cm^4
Layer 9	$d_{z9} = h / 2 + r_l \times \cos[(9 - 1) \times \alpha] =$ **53.0** mm
2nd moment of area of reinft about z axis	$I_{z9} = 1 \times A_{bar} \times (d_{z9} - h / 2)^2 =$ **5595** cm^4
Total 2nd moment of area of reinft about z axis	$I_{sz} =$ **44757** cm^4
Radius of gyration of reinft about z axis	$i_{sz} = \sqrt{(I_{sz} / A_s)} =$ **298** mm
Effective depth about z axis (5.8.8.3(2))	$d_z = b / 2 + i_{sz} =$ **773** mm

Column effective length

Column effective length	$l_{0y} = f_y \times l_y =$ **4900** mm

Column slenderness

Radius of gyration	$i_y = h / 4 =$ **23.8** cm
Slenderness ratio (5.8.3.2(1))	$\lambda_y = l_{0y} / i_y =$ **20.6**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections $e_{iy} = l_{0y} /400 =$ **12.2** mm

Minimum end moment $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **1340.3** kNm

Maximum end moment $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **1340.3** kNm

Slenderness limit (cl. 5.8.3.1)

Factor A $A =$ **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) =$ **0.170**

Factor B $B = \sqrt{(1 + 2 \times \omega)} =$ **1.158**

Moment ratio $r_{my} = -1.0 \times M_{01y} / M_{02y} =$ **-1.000**

Factor C $C_y = 1.7 - r_{my} =$ **2.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) =$ **0.491**

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} =$ **62.4**

$\lambda_y < \lambda_{limy}$ - Second order effects may be ignored

Design bending moments (cl. 6.1(4))

Design moment $M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20 \text{ mm})) =$ **1340.3** kNm

Resultant design bending moment for a circular column

Resultant design moment $M_{Ed} = \sqrt{(M_{Edy}^2 + M_{Edz}^2)} =$ **1340.3** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $y =$ **595.6** mm

Depth of stress block $d_{sby} = \min(\lambda_{sb} \times y , h) =$ **476.5** mm

Area of concrete in compression $A_{sby} =$ **355823** mm^2

Concrete compression force (3.1.7(3)) $F_{yc} = \eta \times f_{cd} \times A_{sby} =$ **5807.0** kN

Centroid of concrete compression from column cl $y_{sby} =$ **200.8** mm

Moment of resistance $M_{Rdyc} = F_{yc} \times y_{sby} =$ **1166.0** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) =$ **-0.00172**

Stress in layer 1 $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) =$ **-344.7** N/mm^2

Force in layer 1 $F_{y1} = 2 \times A_{bar} \times \sigma_{y1} =$ **-216.6** kN

Moment of resistance of layer 1 $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) =$ **89.6** kNm

Strain in layer 2 $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) =$ **-0.00135**

Stress in layer 2 $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2}) =$ **-270.6** N/mm^2

Force in layer 2 $F_{y2} = 2 \times A_{bar} \times \sigma_{y2} =$ **-170.0** kN

Moment of resistance of layer 2 $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) =$ **59.7** kNm

Strain in layer 3 $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y) =$ **-0.00067**

Stress in layer 3 $\sigma_{y3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y3}) =$ **-133.8** N/mm^2

Force in layer 3 $F_{y3} = 2 \times A_{bar} \times \sigma_{y3} =$ **-84.1** kN

Moment of resistance of layer 3 $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3}) =$ **19.7** kNm

Strain in layer 4 $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y) =$ **0.00022**

Stress in layer 4 $\sigma_{y4} = \min(f_{yd}, E_s \times \varepsilon_{y4}) =$ **45.0** N/mm^2

Force in layer 4 $F_{y4} = 2 \times A_{bar} \times \sigma_{y4} =$ **28.3** kN

Moment of resistance of layer 4	$M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **-2.3** kNm
Strain in layer 5	$\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **0.00119**
Stress in layer 5	$\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd}$ = **222.2** N/mm^2
Force in layer 5	$F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **139.6** kN
Moment of resistance of layer 5	$M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **11.5** kNm
Strain in layer 6	$\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00209**
Stress in layer 6	$\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **401.0** N/mm^2
Force in layer 6	$F_{y6} = 2 \times A_{bar} \times \sigma_{y6}$ = **251.9** kN
Moment of resistance of layer 6	$M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **59.1** kNm
Strain in layer 7	$\varepsilon_{y7} = \varepsilon_{cu3} \times (1 - d_{y7} / y)$ = **0.00277**
Stress in layer 7	$\sigma_{y7} = \min(f_{yd}, E_s \times \varepsilon_{y7}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 7	$F_{y7} = 2 \times A_{bar} \times \sigma_{y7}$ = **262.9** kN
Moment of resistance of layer 7	$M_{Rdy7} = F_{y7} \times (h / 2 - d_{y7})$ = **92.3** kNm
Strain in layer 8	$\varepsilon_{y8} = \varepsilon_{cu3} \times (1 - d_{y8} / y)$ = **0.00314**
Stress in layer 8	$\sigma_{y8} = \min(f_{yd}, E_s \times \varepsilon_{y8}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 8	$F_{y8} = 2 \times A_{bar} \times \sigma_{y8}$ = **262.9** kN
Moment of resistance of layer 8	$M_{Rdy8} = F_{y8} \times (h / 2 - d_{y8})$ = **108.8** kNm
Resultant concrete/steel force	F_y = **6282.0** kN
	PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis	M_{Rdy} = **1604.3** kNm

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis	z = **597.0** mm
Depth of stress block	$d_{sbz} = \min(\lambda_{sb} \times z , h)$ = **477.6** mm
Area of concrete in compression	A_{sbz} = **356910** mm^2
Concrete compression force (3.1.7(3))	$F_{zc} = \eta \times f_{cd} \times A_{sbz}$ = **5824.8** kN
Centroid of concrete compression from column cl	y_{sbz} = **200.2** mm
Moment of resistance	$M_{Rdzc} = F_{zc} \times y_{sbz}$ = **1166.0** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00176**
Stress in layer 1	$\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-351.7** N/mm^2
Force in layer 1	$F_{z1} = 1 \times A_{bar} \times \sigma_{z1}$ = **-110.5** kN
Moment of resistance of layer 1	$M_{Rdz1} = F_{z1} \times (h / 2 – d_{z1})$ = **46.6** kNm
Strain in layer 2	$\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **-0.00157**
Stress in layer 2	$\sigma_{z2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z2})$ = **-314.0** N/mm^2
Force in layer 2	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ = **-197.3** kN
Moment of resistance of layer 2	$M_{Rdz2} = F_{z2} \times (h / 2 – d_{z2})$ = **76.9** kNm
Strain in layer 3	$\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z)$ = **-0.00103**
Stress in layer 3	$\sigma_{z3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z3})$ = **-206.8** N/mm^2
Force in layer 3	$F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ = **-129.9** kN
Moment of resistance of layer 3	$M_{Rdz3} = F_{z3} \times (h / 2 – d_{z3})$ = **38.8** kNm

Strain in layer 4	$\varepsilon_{z4} = \varepsilon_{cu3} \times (1 - d_{z4} / z) =$ **-0.00023**
Stress in layer 4	$\sigma_{z4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z4}) =$ **-46.3** N/mm^2
Force in layer 4	$F_{z4} = 2 \times A_{bar} \times \sigma_{z4} =$ **-29.1** kN
Moment of resistance of layer 4	$M_{Rdz4} = F_{z4} \times (h / 2 – d_{z4}) =$ **4.7** kNm
Strain in layer 5	$\varepsilon_{z5} = \varepsilon_{cu3} \times (1 - d_{z5} / z) =$ **0.00072**
Stress in layer 5	$\sigma_{z5} = \min(f_{yd}, E_s \times \varepsilon_{z5}) - \eta \times f_{cd} =$ **126.8** N/mm^2
Force in layer 5	$F_{z5} = 2 \times A_{bar} \times \sigma_{z5} =$ **79.6** kN
Moment of resistance of layer 5	$M_{Rdz5} = F_{z5} \times (h / 2 – d_{z5}) =$ **0.0** kNm
Strain in layer 6	$\varepsilon_{z6} = \varepsilon_{cu3} \times (1 - d_{z6} / z) =$ **0.00166**
Stress in layer 6	$\sigma_{z6} = \min(f_{yd}, E_s \times \varepsilon_{z6}) - \eta \times f_{cd} =$ **316.1** N/mm^2
Force in layer 6	$F_{z6} = 2 \times A_{bar} \times \sigma_{z6} =$ **198.6** kN
Moment of resistance of layer 6	$M_{Rdz6} = F_{z6} \times (h / 2 – d_{z6}) =$ **32.1** kNm
Strain in layer 7	$\varepsilon_{z7} = \varepsilon_{cu3} \times (1 - d_{z7} / z) =$ **0.00246**
Stress in layer 7	$\sigma_{z7} = \min(f_{yd}, E_s \times \varepsilon_{z7}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 7	$F_{z7} = 2 \times A_{bar} \times \sigma_{z7} =$ **262.9** kN
Moment of resistance of layer 7	$M_{Rdz7} = F_{z7} \times (h / 2 – d_{z7}) =$ **78.5** kNm
Strain in layer 8	$\varepsilon_{z8} = \varepsilon_{cu3} \times (1 - d_{z8} / z) =$ **0.00300**
Stress in layer 8	$\sigma_{z8} = \min(f_{yd}, E_s \times \varepsilon_{z8}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 8	$F_{z8} = 2 \times A_{bar} \times \sigma_{z8} =$ **262.9** kN
Moment of resistance of layer 8	$M_{Rdz8} = F_{z8} \times (h / 2 – d_{z8}) =$ **102.5** kNm
Strain in layer 9	$\varepsilon_{z9} = \varepsilon_{cu3} \times (1 - d_{z9} / z) =$ **0.00319**
Stress in layer 9	$\sigma_{z9} = \min(f_{yd}, E_s \times \varepsilon_{z9}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 9	$F_{z9} = 1 \times A_{bar} \times \sigma_{z9} =$ **131.5** kN
Moment of resistance of layer 9	$M_{Rdz9} = F_{z9} \times (h / 2 – d_{z9}) =$ **55.5** kNm
Resultant concrete/steel force	$F_z =$ **6293.6** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis $M_{Rdz} =$ **1601.5** kNm

Minimum moment capacity with axial load N_{Ed}

Minimum moment capacity $M_{Rd} = \min(M_{Rdz}, M_{Rdy}) =$ **1601.5** kNm

PASS - The moment capacity exceeds the resultant design bending moment

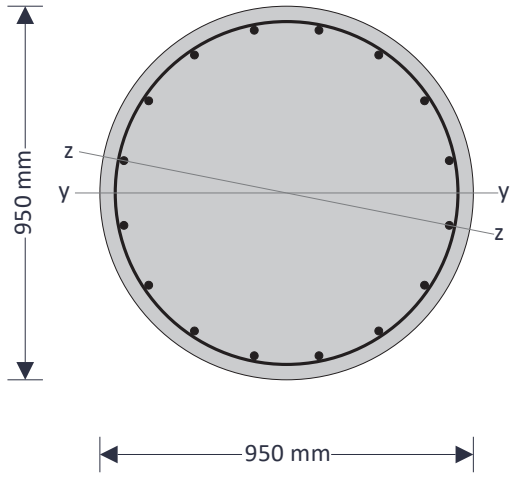
RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	1569.39	1451.77	0.93	PASS



16 no. 20 mm diameter longitudinal bars
8 mm diameter links
Max link spacing 400 mm generally, 240 mm for 950 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall diameter — h = **950** mm

Column clear height — l_y = **7000** mm

Stability — **Braced**

Concrete details

Concrete strength class — **C32/40**

Partial safety factor for concrete (2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) — α_{cc} = **0.85**

Maximum aggregate size — d_g = **20** mm

Reinforcement details

Nominal cover to links — c_{nom} = **35** mm

Longitudinal bar diameter — ϕ = **20** mm

Link diameter — ϕ_v = **8** mm

Total number of longitudinal bars — N = **16**

Area of longitudinal reinforcement — $A_s = N \times \pi \times \phi^2 / 4$ = **5027** mm^2

Characteristic yield strength — f_{yk} = **500** N/mm^2

Partial safety factor for reinft (2.4.2.4(1)) — γ_S = **1.15**

Modulus of elasticity of reinft (3.2.7(4)) — E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period — R = **120** min

Exposure to fire — **Exposed on more than one side**

Ratio of fire design axial load to design resistance — μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load — N_{Ed} = **6838.0** kN

Moment at top — M_{topy} = **1368.0** kNm

Moment at bottom — M_{btmy} = **-1368.0** kNm

Column effective length factors

Effective length factor — f_y = **0.70**

Calculated column properties

Concrete properties

Area of concrete — $A_c = \pi \times h^2 / 4$ = **708822** mm^2

Characteristic compression cylinder strength — f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) — λ_{sb} = 0.8

Stress factor (3.1.7(3)) — η = 0.9

Strain limits

Compression strain limit (Table 3.1) — ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) — ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) — $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) — $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **12** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) — a_{fi} = **51** mm

Allowance for deviations from min cover (4.4.1.3) — Δc_{dev} = **10** mm

Min allowable nominal cover — $c_{nom_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **33.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depth and inertia of bars for bending about y axis

For the purposes of determining the bending capacity and interaction diagrams in this Tedds calculation, bending about the y axis is taken to be when there are two furthest equidistant bars on each side of the column centreline. Bending about the z axis is taken to be when there is one furthest bar on each side of the column centreline.

Area per bar — $A_{bar} = \pi \times \phi^2 / 4$ = **314** mm^2

Radial dist from column centre to longitudinal bar — $r_l = h / 2 - c_{nom} - \phi_v - \phi / 2$ = **422.0** mm

Subtended angle between adjacent bars — α = (360 deg) / N = **22.5** deg

Layer 1 — $d_{y1} = h / 2 + r_l \times \cos(\alpha / 2)$ = **888.9** mm

2nd moment of area of reinft about y axis — $I_{y1} = 2 \times A_{bar} \times (d_{y1} - h / 2)^2$ = **10763** cm^4

Library item - Depth layer 1 y circ case 2

Layer 2	$d_{y2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha + \alpha / 2] =$ **825.9** mm
2nd moment of area of reinft about y axis	$I_{y2} = 2 \times A_{bar} \times (d_{y2} - h / 2)^2 =$ **7736** cm^4
Layer 3	$d_{y3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha + \alpha / 2] =$ **709.5** mm
2nd moment of area of reinft about y axis	$I_{y3} = 2 \times A_{bar} \times (d_{y3} - h / 2)^2 =$ **3454** cm^4
Layer 4	$d_{y4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha + \alpha / 2] =$ **557.3** mm
2nd moment of area of reinft about y axis	$I_{y4} = 2 \times A_{bar} \times (d_{y4} - h / 2)^2 =$ **426** cm^4
Layer 5	$d_{y5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha + \alpha / 2] =$ **392.7** mm
2nd moment of area of reinft about y axis	$I_{y5} = 2 \times A_{bar} \times (d_{y5} - h / 2)^2 =$ **426** cm^4
Layer 6	$d_{y6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha + \alpha / 2] =$ **240.5** mm
2nd moment of area of reinft about y axis	$I_{y6} = 2 \times A_{bar} \times (d_{y6} - h / 2)^2 =$ **3454** cm^4
Layer 7	$d_{y7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha + \alpha / 2] =$ **124.1** mm
2nd moment of area of reinft about y axis	$I_{y7} = 2 \times A_{bar} \times (d_{y7} - h / 2)^2 =$ **7736** cm^4
Layer 8	$d_{y8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha + \alpha / 2] =$ **61.1** mm
2nd moment of area of reinft about y axis	$I_{y8} = 2 \times A_{bar} \times (d_{y8} - h / 2)^2 =$ **10763** cm^4
Total 2nd moment of area of reinft about y axis	$I_{sy} =$ **44757** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **298** mm
Effective depth about y axis (5.8.8.3(2))	$d_y = h / 2 + i_{sy} =$ **773** mm

Effective depth of bars for bending about z axis

Layer 1 (tension face)	$d_{z1} = h / 2 + r_l =$ **897.0** mm
2nd moment of area of reinft about z axis	$I_{z1} = A_{bar} \times (d_{z1} - h / 2)^2 =$ **5595** cm^4
Layer 2	$d_{z2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha] =$ **864.9** mm
2nd moment of area of reinft about z axis	$I_{z2} = 2 \times A_{bar} \times (d_{z2} - h / 2)^2 =$ **9551** cm^4
Layer 3	$d_{z3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha] =$ **773.4** mm
2nd moment of area of reinft about z axis	$I_{z3} = 2 \times A_{bar} \times (d_{z3} - h / 2)^2 =$ **5595** cm^4
Layer 4	$d_{z4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha] =$ **636.5** mm
2nd moment of area of reinft about z axis	$I_{z4} = 2 \times A_{bar} \times (d_{z4} - h / 2)^2 =$ **1639** cm^4
Layer 5	$d_{z5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha] =$ **475.0** mm
2nd moment of area of reinft about z axis	$I_{z5} = 2 \times A_{bar} \times (d_{z5} - h / 2)^2 =$ **0** cm^4
Layer 6	$d_{z6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha] =$ **313.5** mm
2nd moment of area of reinft about z axis	$I_{z6} = 2 \times A_{bar} \times (d_{z6} - h / 2)^2 =$ **1639** cm^4
Layer 7	$d_{z7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha] =$ **176.6** mm
2nd moment of area of reinft about z axis	$I_{z7} = 2 \times A_{bar} \times (d_{z7} - h / 2)^2 =$ **5595** cm^4
Layer 8	$d_{z8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha] =$ **85.1** mm
2nd moment of area of reinft about z axis	$I_{z8} = 2 \times A_{bar} \times (d_{z8} - h / 2)^2 =$ **9551** cm^4
Layer 9	$d_{z9} = h / 2 + r_l \times \cos[(9 - 1) \times \alpha] =$ **53.0** mm
2nd moment of area of reinft about z axis	$I_{z9} = 1 \times A_{bar} \times (d_{z9} - h / 2)^2 =$ **5595** cm^4
Total 2nd moment of area of reinft about z axis	$I_{sz} =$ **44757** cm^4
Radius of gyration of reinft about z axis	$i_{sz} = \sqrt{(I_{sz} / A_s)} =$ **298** mm
Effective depth about z axis (5.8.8.3(2))	$d_z = b / 2 + i_{sz} =$ **773** mm

Column effective length

Column effective length	$l_{0y} = f_y \times l_y =$ **4900** mm

Column slenderness

Radius of gyration	$i_y = h / 4 =$ **23.8** cm
Slenderness ratio (5.8.3.2(1))	$\lambda_y = l_{0y} / i_y =$ **20.6**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections $e_{iy} = l_{0y} /400 =$ **12.2** mm

Minimum end moment $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **1451.8** kNm

Maximum end moment $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **1451.8** kNm

Slenderness limit (cl. 5.8.3.1)

Factor A $A =$ **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) =$ **0.170**

Factor B $B = \sqrt{(1 + 2 \times \omega)} =$ **1.158**

Moment ratio $r_{my} = -1.0 \times M_{01y} / M_{02y} =$ **-1.000**

Factor C $C_y = 1.7 - r_{my} =$ **2.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) =$ **0.532**

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} =$ **60.0**

$\lambda_y < \lambda_{limy}$ - Second order effects may be ignored

Design bending moments (cl. 6.1(4))

Design moment $M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20 \text{ mm})) =$ **1451.8** kNm

Resultant design bending moment for a circular column

Resultant design moment $M_{Ed} = \sqrt{(M_{Edy}^2 + M_{Edz}^2)} =$ **1451.8** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $y =$ **627.5** mm

Depth of stress block $d_{sby} = \min(\lambda_{sb} \times y , h) =$ **502.0** mm

Area of concrete in compression $A_{sby} =$ **380047** mm^2

Concrete compression force (3.1.7(3)) $F_{yc} = \eta \times f_{cd} \times A_{sby} =$ **6202.4** kN

Centroid of concrete compression from column cl $y_{sby} =$ **187.1** mm

Moment of resistance $M_{Rdyc} = F_{yc} \times y_{sby} =$ **1160.4** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) =$ **-0.00146**

Stress in layer 1 $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) =$ **-291.6** N/mm^2

Force in layer 1 $F_{y1} = 2 \times A_{bar} \times \sigma_{y1} =$ **-183.2** kN

Moment of resistance of layer 1 $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) =$ **75.8** kNm

Strain in layer 2 $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) =$ **-0.00111**

Stress in layer 2 $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2}) =$ **-221.3** N/mm^2

Force in layer 2 $F_{y2} = 2 \times A_{bar} \times \sigma_{y2} =$ **-139.0** kN

Moment of resistance of layer 2 $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) =$ **48.8** kNm

Strain in layer 3 $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y) =$ **-0.00046**

Stress in layer 3 $\sigma_{y3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y3}) =$ **-91.4** N/mm^2

Force in layer 3 $F_{y3} = 2 \times A_{bar} \times \sigma_{y3} =$ **-57.4** kN

Moment of resistance of layer 3 $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3}) =$ **13.5** kNm

Strain in layer 4 $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y) =$ **0.00039**

Stress in layer 4 $\sigma_{y4} = \min(f_{yd}, E_s \times \varepsilon_{y4}) =$ **78.3** N/mm^2

Force in layer 4 $F_{y4} = 2 \times A_{bar} \times \sigma_{y4} =$ **49.2** kN

Moment of resistance of layer 4	$M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **-4.0** kNm
Strain in layer 5	$\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **0.00131**
Stress in layer 5	$\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd}$ = **245.6** N/mm^2
Force in layer 5	$F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **154.3** kN
Moment of resistance of layer 5	$M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **12.7** kNm
Strain in layer 6	$\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00216**
Stress in layer 6	$\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **415.3** N/mm^2
Force in layer 6	$F_{y6} = 2 \times A_{bar} \times \sigma_{y6}$ = **261.0** kN
Moment of resistance of layer 6	$M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **61.2** kNm
Strain in layer 7	$\varepsilon_{y7} = \varepsilon_{cu3} \times (1 - d_{y7} / y)$ = **0.00281**
Stress in layer 7	$\sigma_{y7} = \min(f_{yd}, E_s \times \varepsilon_{y7}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 7	$F_{y7} = 2 \times A_{bar} \times \sigma_{y7}$ = **262.9** kN
Moment of resistance of layer 7	$M_{Rdy7} = F_{y7} \times (h / 2 - d_{y7})$ = **92.3** kNm
Strain in layer 8	$\varepsilon_{y8} = \varepsilon_{cu3} \times (1 - d_{y8} / y)$ = **0.00316**
Stress in layer 8	$\sigma_{y8} = \min(f_{yd}, E_s \times \varepsilon_{y8}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 8	$F_{y8} = 2 \times A_{bar} \times \sigma_{y8}$ = **262.9** kN
Moment of resistance of layer 8	$M_{Rdy8} = F_{y8} \times (h / 2 - d_{y8})$ = **108.8** kNm
Resultant concrete/steel force	F_y = **6813.0** kN
	PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis	M_{Rdy} = **1569.4** kNm

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis	z = **628.5** mm
Depth of stress block	$d_{sbz} = \min(\lambda_{sb} \times z , h)$ = **502.8** mm
Area of concrete in compression	A_{sbz} = **380806** mm^2
Concrete compression force (3.1.7(3))	$F_{zc} = \eta \times f_{cd} \times A_{sbz}$ = **6214.8** kN
Centroid of concrete compression from column cl	y_{sbz} = **186.7** mm
Moment of resistance	$M_{Rdzc} = F_{zc} \times y_{sbz}$ = **1160.0** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00150**
Stress in layer 1	$\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-299.0** N/mm^2
Force in layer 1	$F_{z1} = 1 \times A_{bar} \times \sigma_{z1}$ = **-93.9** kN
Moment of resistance of layer 1	$M_{Rdz1} = F_{z1} \times (h / 2 – d_{z1})$ = **39.6** kNm
Strain in layer 2	$\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **-0.00132**
Stress in layer 2	$\sigma_{z2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z2})$ = **-263.3** N/mm^2
Force in layer 2	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ = **-165.4** kN
Moment of resistance of layer 2	$M_{Rdz2} = F_{z2} \times (h / 2 – d_{z2})$ = **64.5** kNm
Strain in layer 3	$\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z)$ = **-0.00081**
Stress in layer 3	$\sigma_{z3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z3})$ = **-161.4** N/mm^2
Force in layer 3	$F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ = **-101.4** kN
Moment of resistance of layer 3	$M_{Rdz3} = F_{z3} \times (h / 2 – d_{z3})$ = **30.3** kNm

Strain in layer 4	$\varepsilon_{z4} = \varepsilon_{cu3} \times (1 - d_{z4} / z) =$ **-0.00004**
Stress in layer 4	$\sigma_{z4} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z4}) =$ **-8.9** N/mm^2
Force in layer 4	$F_{z4} = 2 \times A_{bar} \times \sigma_{z4} =$ **-5.6** kN
Moment of resistance of layer 4	$M_{Rdz4} = F_{z4} \times (h / 2 – d_{z4}) =$ **0.9** kNm
Strain in layer 5	$\varepsilon_{z5} = \varepsilon_{cu3} \times (1 - d_{z5} / z) =$ **0.00085**
Stress in layer 5	$\sigma_{z5} = \min(f_{yd}, E_s \times \varepsilon_{z5}) - \eta \times f_{cd} =$ **154.6** N/mm^2
Force in layer 5	$F_{z5} = 2 \times A_{bar} \times \sigma_{z5} =$ **97.2** kN
Moment of resistance of layer 5	$M_{Rdz5} = F_{z5} \times (h / 2 – d_{z5}) =$ **0.0** kNm
Strain in layer 6	$\varepsilon_{z6} = \varepsilon_{cu3} \times (1 - d_{z6} / z) =$ **0.00175**
Stress in layer 6	$\sigma_{z6} = \min(f_{yd}, E_s \times \varepsilon_{z6}) - \eta \times f_{cd} =$ **334.5** N/mm^2
Force in layer 6	$F_{z6} = 2 \times A_{bar} \times \sigma_{z6} =$ **210.2** kN
Moment of resistance of layer 6	$M_{Rdz6} = F_{z6} \times (h / 2 – d_{z6}) =$ **33.9** kNm
Strain in layer 7	$\varepsilon_{z7} = \varepsilon_{cu3} \times (1 - d_{z7} / z) =$ **0.00252**
Stress in layer 7	$\sigma_{z7} = \min(f_{yd}, E_s \times \varepsilon_{z7}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 7	$F_{z7} = 2 \times A_{bar} \times \sigma_{z7} =$ **262.9** kN
Moment of resistance of layer 7	$M_{Rdz7} = F_{z7} \times (h / 2 – d_{z7}) =$ **78.5** kNm
Strain in layer 8	$\varepsilon_{z8} = \varepsilon_{cu3} \times (1 - d_{z8} / z) =$ **0.00303**
Stress in layer 8	$\sigma_{z8} = \min(f_{yd}, E_s \times \varepsilon_{z8}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 8	$F_{z8} = 2 \times A_{bar} \times \sigma_{z8} =$ **262.9** kN
Moment of resistance of layer 8	$M_{Rdz8} = F_{z8} \times (h / 2 – d_{z8}) =$ **102.5** kNm
Strain in layer 9	$\varepsilon_{z9} = \varepsilon_{cu3} \times (1 - d_{z9} / z) =$ **0.00320**
Stress in layer 9	$\sigma_{z9} = \min(f_{yd}, E_s \times \varepsilon_{z9}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 9	$F_{z9} = 1 \times A_{bar} \times \sigma_{z9} =$ **131.5** kN
Moment of resistance of layer 9	$M_{Rdz9} = F_{z9} \times (h / 2 – d_{z9}) =$ **55.5** kNm
Resultant concrete/steel force	$F_z =$ **6813.1** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis $M_{Rdz} =$ **1565.7** kNm

Minimum moment capacity with axial load N_{Ed}

Minimum moment capacity $M_{Rd} = \min(M_{Rdz}, M_{Rdy}) =$ **1565.7** kNm

PASS - The moment capacity exceeds the resultant design bending moment

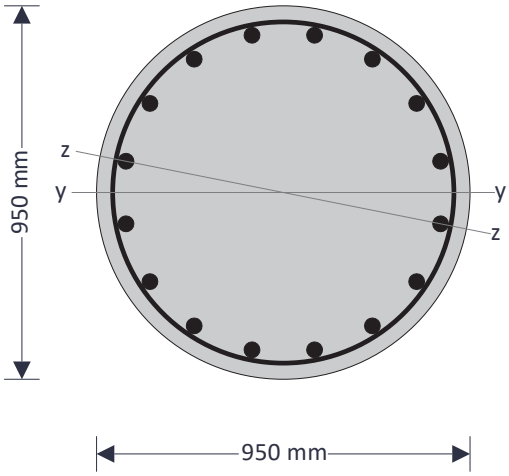
RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	2383.97	2309.28	0.97	PASS



16 no. 40 mm diameter longitudinal bars
12 mm diameter links
Max link spacing 400 mm generally, 240 mm for 950 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall diameter — h = **950** mm

Column clear height — l_y = **7000** mm

Stability — **Braced**

Concrete details

Concrete strength class — **C32/40**

Partial safety factor for concrete (2.4.2.4(1)) — γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) — α_{cc} = **0.85**

Maximum aggregate size — d_g = **20** mm

Reinforcement details

Nominal cover to links — c_{nom} = **35** mm

Longitudinal bar diameter — ϕ = **40** mm

Link diameter — ϕ_v = **12** mm

Total number of longitudinal bars — N = **16**

Area of longitudinal reinforcement — $A_s = N \times \pi \times \phi^2 / 4$ = **20106** mm^2

Characteristic yield strength — f_{yk} = **500** N/mm^2

Partial safety factor for reinft (2.4.2.4(1)) — γ_S = **1.15**

Modulus of elasticity of reinft (3.2.7(4)) — E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period — R = **120** min

Exposure to fire — **Exposed on more than one side**

Ratio of fire design axial load to design resistance — μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load — N_{Ed} = **10880.0** kN

Moment at top — M_{topy} = **2176.0** kNm

Moment at bottom — M_{btmy} = **-2176.0** kNm

Column effective length factors

Effective length factor — f_y = **0.70**

Calculated column properties

Concrete properties

Area of concrete — $A_c = \pi \times h^2 / 4$ = **708822** mm^2

Characteristic compression cylinder strength — f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) — $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) — E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) — λ_{sb} = 0.8

Stress factor (3.1.7(3)) — η = 0.9

Strain limits

Compression strain limit (Table 3.1) — ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) — ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) — $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) — $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **28** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) — a_{fi} = **51** mm

Allowance for deviations from min cover (4.4.1.3) — Δc_{dev} = **10** mm

Min allowable nominal cover — $c_{nom_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **38.0** mm

FAIL - the nominal cover is less than the minimum required

Effective depth and inertia of bars for bending about y axis

For the purposes of determining the bending capacity and interaction diagrams in this Tedds calculation, bending about the y axis is taken to be when there are two furthest equidistant bars on each side of the column centreline. Bending about the z axis is taken to be when there is one furthest bar on each side of the column centreline.

Area per bar — $A_{bar} = \pi \times \phi^2 / 4$ = **1257** mm^2

Radial dist from column centre to longitudinal bar — $r_l = h / 2 - c_{nom} - \phi_v - \phi / 2$ = **408.0** mm

Subtended angle between adjacent bars — α = (360 deg) / N = **22.5** deg

Layer 1 — $d_{y1} = h / 2 + r_l \times \cos(\alpha / 2)$ = **875.2** mm

2nd moment of area of reinft about y axis — $I_{y1} = 2 \times A_{bar} \times (d_{y1} - h / 2)^2$ = **40245** cm^4

Library item - Depth layer 1 y circ case 2

Layer 2	$d_{y2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha + \alpha / 2] =$ **814.2** mm
2nd moment of area of reinft about y axis	$I_{y2} = 2 \times A_{bar} \times (d_{y2} - h / 2)^2 =$ **28924** cm^4
Layer 3	$d_{y3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha + \alpha / 2] =$ **701.7** mm
2nd moment of area of reinft about y axis	$I_{y3} = 2 \times A_{bar} \times (d_{y3} - h / 2)^2 =$ **12913** cm^4
Layer 4	$d_{y4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha + \alpha / 2] =$ **554.6** mm
2nd moment of area of reinft about y axis	$I_{y4} = 2 \times A_{bar} \times (d_{y4} - h / 2)^2 =$ **1592** cm^4
Layer 5	$d_{y5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha + \alpha / 2] =$ **395.4** mm
2nd moment of area of reinft about y axis	$I_{y5} = 2 \times A_{bar} \times (d_{y5} - h / 2)^2 =$ **1592** cm^4
Layer 6	$d_{y6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha + \alpha / 2] =$ **248.3** mm
2nd moment of area of reinft about y axis	$I_{y6} = 2 \times A_{bar} \times (d_{y6} - h / 2)^2 =$ **12913** cm^4
Layer 7	$d_{y7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha + \alpha / 2] =$ **135.8** mm
2nd moment of area of reinft about y axis	$I_{y7} = 2 \times A_{bar} \times (d_{y7} - h / 2)^2 =$ **28924** cm^4
Layer 8	$d_{y8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha + \alpha / 2] =$ **74.8** mm
2nd moment of area of reinft about y axis	$I_{y8} = 2 \times A_{bar} \times (d_{y8} - h / 2)^2 =$ **40245** cm^4
Total 2nd moment of area of reinft about y axis	$I_{sy} =$ **167348** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **288** mm
Effective depth about y axis (5.8.8.3(2))	$d_y = h / 2 + i_{sy} =$ **763** mm

Effective depth of bars for bending about z axis

Layer 1 (tension face)	$d_{z1} = h / 2 + r_l =$ **883.0** mm
2nd moment of area of reinft about z axis	$I_{z1} = A_{bar} \times (d_{z1} - h / 2)^2 =$ **20918** cm^4
Layer 2	$d_{z2} = h / 2 + r_l \times \cos[(2 - 1) \times \alpha] =$ **851.9** mm
2nd moment of area of reinft about z axis	$I_{z2} = 2 \times A_{bar} \times (d_{z2} - h / 2)^2 =$ **35710** cm^4
Layer 3	$d_{z3} = h / 2 + r_l \times \cos[(3 - 1) \times \alpha] =$ **763.5** mm
2nd moment of area of reinft about z axis	$I_{z3} = 2 \times A_{bar} \times (d_{z3} - h / 2)^2 =$ **20918** cm^4
Layer 4	$d_{z4} = h / 2 + r_l \times \cos[(4 - 1) \times \alpha] =$ **631.1** mm
2nd moment of area of reinft about z axis	$I_{z4} = 2 \times A_{bar} \times (d_{z4} - h / 2)^2 =$ **6127** cm^4
Layer 5	$d_{z5} = h / 2 + r_l \times \cos[(5 - 1) \times \alpha] =$ **475.0** mm
2nd moment of area of reinft about z axis	$I_{z5} = 2 \times A_{bar} \times (d_{z5} - h / 2)^2 =$ **0** cm^4
Layer 6	$d_{z6} = h / 2 + r_l \times \cos[(6 - 1) \times \alpha] =$ **318.9** mm
2nd moment of area of reinft about z axis	$I_{z6} = 2 \times A_{bar} \times (d_{z6} - h / 2)^2 =$ **6127** cm^4
Layer 7	$d_{z7} = h / 2 + r_l \times \cos[(7 - 1) \times \alpha] =$ **186.5** mm
2nd moment of area of reinft about z axis	$I_{z7} = 2 \times A_{bar} \times (d_{z7} - h / 2)^2 =$ **20918** cm^4
Layer 8	$d_{z8} = h / 2 + r_l \times \cos[(8 - 1) \times \alpha] =$ **98.1** mm
2nd moment of area of reinft about z axis	$I_{z8} = 2 \times A_{bar} \times (d_{z8} - h / 2)^2 =$ **35710** cm^4
Layer 9	$d_{z9} = h / 2 + r_l \times \cos[(9 - 1) \times \alpha] =$ **67.0** mm
2nd moment of area of reinft about z axis	$I_{z9} = 1 \times A_{bar} \times (d_{z9} - h / 2)^2 =$ **20918** cm^4
Total 2nd moment of area of reinft about z axis	$I_{sz} =$ **167348** cm^4
Radius of gyration of reinft about z axis	$i_{sz} = \sqrt{(I_{sz} / A_s)} =$ **288** mm
Effective depth about z axis (5.8.8.3(2))	$d_z = b / 2 + i_{sz} =$ **763** mm

Column effective length

Column effective length	$l_{0y} = f_y \times l_y =$ **4900** mm

Column slenderness

Radius of gyration	$i_y = h / 4 =$ **23.8** cm
Slenderness ratio (5.8.3.2(1))	$\lambda_y = l_{0y} / i_y =$ **20.6**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections $e_{iy} = l_{0y} /400 =$ **12.2** mm

Minimum end moment $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **2309.3** kNm

Maximum end moment $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} =$ **2309.3** kNm

Slenderness limit (cl. 5.8.3.1)

Factor A $A =$ **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) =$ **0.680**

Factor B $B = \sqrt{(1 + 2 \times \omega)} =$ **1.536**

Moment ratio $r_{my} = -1.0 \times M_{01y} / M_{02y} =$ **-1.000**

Factor C $C_y = 1.7 - r_{my} =$ **2.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) =$ **0.846**

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} =$ **63.1**

$\lambda_y < \lambda_{limy}$ - Second order effects may be ignored

Design bending moments (cl. 6.1(4))

Design moment $M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20 \text{ mm})) =$ **2309.3** kNm

Resultant design bending moment for a circular column

Resultant design moment $M_{Ed} = \sqrt{(M_{Edy}^2 + M_{Edz}^2)} =$ **2309.3** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $y =$ **712.5** mm

Depth of stress block $d_{sby} = \min(\lambda_{sb} \times y , h) =$ **570.0** mm

Area of concrete in compression $A_{sby} =$ **444056** mm^2

Concrete compression force (3.1.7(3)) $F_{yc} = \eta \times f_{cd} \times A_{sby} =$ **7247.0** kN

Centroid of concrete compression from column cl $y_{sby} =$ **151.3** mm

Moment of resistance $M_{Rdyc} = F_{yc} \times y_{sby} =$ **1096.8** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) =$ **-0.00080**

Stress in layer 1 $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) =$ **-159.8** N/mm^2

Force in layer 1 $F_{y1} = 2 \times A_{bar} \times \sigma_{y1} =$ **-401.6** kN

Moment of resistance of layer 1 $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) =$ **160.7** kNm

Strain in layer 2 $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) =$ **-0.00050**

Stress in layer 2 $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2}) =$ **-100.0** N/mm^2

Force in layer 2 $F_{y2} = 2 \times A_{bar} \times \sigma_{y2} =$ **-251.2** kN

Moment of resistance of layer 2 $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) =$ **85.2** kNm

Strain in layer 3 $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y) =$ **0.00005**

Stress in layer 3 $\sigma_{y3} = \min(f_{yd}, E_s \times \varepsilon_{y3}) =$ **10.6** N/mm^2

Force in layer 3 $F_{y3} = 2 \times A_{bar} \times \sigma_{y3} =$ **26.7** kN

Moment of resistance of layer 3 $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3}) =$ **-6.1** kNm

Strain in layer 4 $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y) =$ **0.00078**

Stress in layer 4 $\sigma_{y4} = \min(f_{yd}, E_s \times \varepsilon_{y4}) - \eta \times f_{cd} =$ **138.8** N/mm^2

Force in layer 4 $F_{y4} = 2 \times A_{bar} \times \sigma_{y4} =$ **348.9** kN

Moment of resistance of layer 4	$M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **-27.8** kNm
Strain in layer 5	$\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **0.00156**
Stress in layer 5	$\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd}$ = **295.2** N/mm^2
Force in layer 5	$F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **742.0** kN
Moment of resistance of layer 5	$M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **59.1** kNm
Strain in layer 6	$\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00228**
Stress in layer 6	$\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 6	$F_{y6} = 2 \times A_{bar} \times \sigma_{y6}$ = **1051.7** kN
Moment of resistance of layer 6	$M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **238.4** kNm
Strain in layer 7	$\varepsilon_{y7} = \varepsilon_{cu3} \times (1 - d_{y7} / y)$ = **0.00283**
Stress in layer 7	$\sigma_{y7} = \min(f_{yd}, E_s \times \varepsilon_{y7}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 7	$F_{y7} = 2 \times A_{bar} \times \sigma_{y7}$ = **1051.7** kN
Moment of resistance of layer 7	$M_{Rdy7} = F_{y7} \times (h / 2 - d_{y7})$ = **356.8** kNm
Strain in layer 8	$\varepsilon_{y8} = \varepsilon_{cu3} \times (1 - d_{y8} / y)$ = **0.00313**
Stress in layer 8	$\sigma_{y8} = \min(f_{yd}, E_s \times \varepsilon_{y8}) - \eta \times f_{cd}$ = **418.5** N/mm^2
Force in layer 8	$F_{y8} = 2 \times A_{bar} \times \sigma_{y8}$ = **1051.7** kN
Moment of resistance of layer 8	$M_{Rdy8} = F_{y8} \times (h / 2 - d_{y8})$ = **420.9** kNm
Resultant concrete/steel force	F_y = **10866.8** kN
	PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis	M_{Rdy} = **2384.0** kNm

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis	z = **712.5** mm
Depth of stress block	$d_{sbz} = \min(\lambda_{sb} \times z , h)$ = **570.0** mm
Area of concrete in compression	A_{sbz} = **444056** mm^2
Concrete compression force (3.1.7(3))	$F_{zc} = \eta \times f_{cd} \times A_{sbz}$ = **7247.0** kN
Centroid of concrete compression from column cl	y_{sbz} = **151.3** mm
Moment of resistance	$M_{Rdzc} = F_{zc} \times y_{sbz}$ = **1096.8** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00084**
Stress in layer 1	$\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-167.5** N/mm^2
Force in layer 1	$F_{z1} = 1 \times A_{bar} \times \sigma_{z1}$ = **-210.5** kN
Moment of resistance of layer 1	$M_{Rdz1} = F_{z1} \times (h / 2 – d_{z1})$ = **85.9** kNm
Strain in layer 2	$\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **-0.00068**
Stress in layer 2	$\sigma_{z2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z2})$ = **-137.0** N/mm^2
Force in layer 2	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2}$ = **-344.3** kN
Moment of resistance of layer 2	$M_{Rdz2} = F_{z2} \times (h / 2 – d_{z2})$ = **129.8** kNm
Strain in layer 3	$\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z)$ = **-0.00025**
Stress in layer 3	$\sigma_{z3} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z3})$ = **-50.1** N/mm^2
Force in layer 3	$F_{z3} = 2 \times A_{bar} \times \sigma_{z3}$ = **-125.9** kN
Moment of resistance of layer 3	$M_{Rdz3} = F_{z3} \times (h / 2 – d_{z3})$ = **36.3** kNm

Strain in layer 4	$\varepsilon_{z4} = \varepsilon_{cu3} \times (1 - d_{z4} / z) =$ **0.00040**
Stress in layer 4	$\sigma_{z4} = \min(f_{yd}, E_s \times \varepsilon_{z4}) =$ **79.9** N/mm^2
Force in layer 4	$F_{z4} = 2 \times A_{bar} \times \sigma_{z4} =$ **200.9** kN
Moment of resistance of layer 4	$M_{Rdz4} = F_{z4} \times (h / 2 – d_{z4}) =$ **-31.4** kNm
Strain in layer 5	$\varepsilon_{z5} = \varepsilon_{cu3} \times (1 - d_{z5} / z) =$ **0.00117**
Stress in layer 5	$\sigma_{z5} = \min(f_{yd}, E_s \times \varepsilon_{z5}) - \eta \times f_{cd} =$ **217.0** N/mm^2
Force in layer 5	$F_{z5} = 2 \times A_{bar} \times \sigma_{z5} =$ **545.4** kN
Moment of resistance of layer 5	$M_{Rdz5} = F_{z5} \times (h / 2 – d_{z5}) =$ **0.0** kNm
Strain in layer 6	$\varepsilon_{z6} = \varepsilon_{cu3} \times (1 - d_{z6} / z) =$ **0.00193**
Stress in layer 6	$\sigma_{z6} = \min(f_{yd}, E_s \times \varepsilon_{z6}) - \eta \times f_{cd} =$ **370.4** N/mm^2
Force in layer 6	$F_{z6} = 2 \times A_{bar} \times \sigma_{z6} =$ **930.9** kN
Moment of resistance of layer 6	$M_{Rdz6} = F_{z6} \times (h / 2 – d_{z6}) =$ **145.4** kNm
Strain in layer 7	$\varepsilon_{z7} = \varepsilon_{cu3} \times (1 - d_{z7} / z) =$ **0.00258**
Stress in layer 7	$\sigma_{z7} = \min(f_{yd}, E_s \times \varepsilon_{z7}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 7	$F_{z7} = 2 \times A_{bar} \times \sigma_{z7} =$ **1051.7** kN
Moment of resistance of layer 7	$M_{Rdz7} = F_{z7} \times (h / 2 – d_{z7}) =$ **303.4** kNm
Strain in layer 8	$\varepsilon_{z8} = \varepsilon_{cu3} \times (1 - d_{z8} / z) =$ **0.00302**
Stress in layer 8	$\sigma_{z8} = \min(f_{yd}, E_s \times \varepsilon_{z8}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 8	$F_{z8} = 2 \times A_{bar} \times \sigma_{z8} =$ **1051.7** kN
Moment of resistance of layer 8	$M_{Rdz8} = F_{z8} \times (h / 2 – d_{z8}) =$ **396.4** kNm
Strain in layer 9	$\varepsilon_{z9} = \varepsilon_{cu3} \times (1 - d_{z9} / z) =$ **0.00317**
Stress in layer 9	$\sigma_{z9} = \min(f_{yd}, E_s \times \varepsilon_{z9}) - \eta \times f_{cd} =$ **418.5** N/mm^2
Force in layer 9	$F_{z9} = 1 \times A_{bar} \times \sigma_{z9} =$ **525.9** kN
Moment of resistance of layer 9	$M_{Rdz9} = F_{z9} \times (h / 2 – d_{z9}) =$ **214.5** kNm
Resultant concrete/steel force	$F_z =$ **10872.8** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis $M_{Rdz} =$ **2377.2** kNm

Minimum moment capacity with axial load N_{Ed}

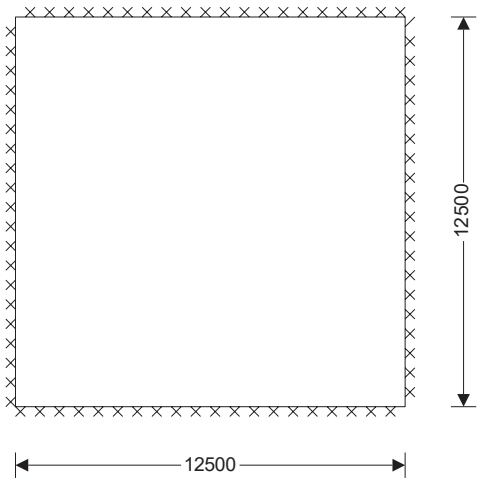
Minimum moment capacity $M_{Rd} = \min(M_{Rdz}, M_{Rdy}) =$ **2377.2** kNm

PASS - The moment capacity exceeds the resultant design bending moment

RC SLAB DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.18



Slab definition

Slab reference name	**GF slab**
Type of slab	**Two way spanning with restrained edges**
Overall slab depth	h = **350** mm
Shorter effective span of panel	l_x = **12500** mm
Longer effective span of panel	l_y = **12500** mm
Support conditions	**Four edges continuous (interior panel)**
Top outer layer of reinforcement	**Short span direction**
Bottom outer layer of reinforcement	**Short span direction**

Loading

Characteristic permanent action	G_k = **13.0** kN/m^2
Characteristic variable action	Q_k = **5.0** kN/m^2
Partial factor for permanent action	γ_G = **1.35**
Partial factor for variable action	γ_Q = **1.50**
Quasi-permanent value of variable action	ψ_2 = **0.30**
Design ultimate load	$q = \gamma_G \times G_k + \gamma_Q \times Q_k$ = **25.1** kN/m^2
Quasi-permanent load	$q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k$ = **14.5** kN/m^2

Concrete properties

Concrete strength class	C32/40
Characteristic cylinder strength	f_{ck} = **32** N/mm^2
Partial factor (Table 2.1N)	γ_C = **1.50**
Compressive strength factor (cl. 3.1.6)	α_{cc} = **0.85**
Design compressive strength (cl. 3.1.6)	f_{cd} = **18.1** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **3.0** N/mm^2
Maximum aggregate size	d_g = **20** mm

Reinforcement properties

Characteristic yield strength	f_{yk} = **500** N/mm^2

Partial factor (Table 2.1N) γ_S = **1.15**

Design yield strength (fig. 3.8) $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Concrete cover to reinforcement

Nominal cover to outer top reinforcement c_{nom_t} = **35** mm

Nominal cover to outer bottom reinforcement c_{nom_b} = **35** mm

Fire resistance period to top of slab R_{top} = **120** min

Fire resistance period to bottom of slab R_{btm} = **30** min

Axia distance to top reinft (Table 5.8) a_{fi_t} = **20** mm

Axia distance to bottom reinft (Table 5.8) a_{fi_b} = **10** mm

Min. top cover requirement with regard to bond c_{min,b_t} = **16** mm

Min. btm cover requirement with regard to bond c_{min,b_b} = **12** mm

Reinforcement fabrication **Subject to QA system**

Cover allowance for deviation Δc_{dev} = **5** mm

Min. required nominal cover to top reinft $c_{nom_t_min}$ = **21.0** mm

Min. required nominal cover to bottom reinft $c_{nom_b_min}$ = **17.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient β_{sx_p} = **0.0240**

Design bending moment $M_{x_p} = \beta_{sx_p} \times q \times l_x^2$ = **93.9** kNm/m

Reinforcement provided 12 mm dia. bars at 100 mm centres

Area provided A_{sx_p} = 1131 mm^2/m

Effective depth to tension reinforcement $d_{x_p} = h - c_{nom_b} - \phi_{x_p} / 2$ = **309.0** mm

K factor $K = M_{x_p} / (b \times d_{x_p}^2 \times f_{ck})$ = **0.031**

Redistribution ratio δ = 1.0

K' factor $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21$ = **0.208**

K < K' - Compression reinforcement is not required

Lever arm $z = \min(0.95 \times d_{x_p}, d_{x_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **293.6** mm

Area of reinforcement required for bending $A_{sx_p_m} = M_{x_p} / (f_{yd} \times z)$ = **736** mm^2/m

Minimum area of reinforcement required $A_{sx_p_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x_p}, 0.0013 \times b \times d_{x_p})$ = **486** mm^2/m

Area of reinforcement required $A_{sx_p_req} = \max(A_{sx_p_m}, A_{sx_p_min})$ = **736** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sx_p} = (f_{yk} / \gamma_S) \times \min((A_{sx_p_m}/A_{sx_p}), 1.0) \times q_{SLS} / q$ = **163.8** N/mm^2

Maximum allowable spacing (Table 7.3N) $s_{max_x_p}$ = **295** mm

Actual bar spacing s_{x_p} = **100** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient β_{sy_p} = **0.0240**

Design bending moment $M_{y_p} = \beta_{sy_p} \times q \times l_x^2$ = **93.9** kNm/m

Reinforcement provided 12 mm dia. bars at 100 mm centres

Area provided A_{sy_p} = 1131 mm^2/m

Effective depth to tension reinforcement $d_{y_p} = h - c_{nom_b} - \phi_{x_p} - \phi_{y_p} / 2$ = **297.0** mm

K factor $K = M_{y_p} / (b \times d_{y_p}^2 \times f_{ck})$ = **0.033**

Redistribution ratio δ = 1.0

K' factor — $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm — $z = \min(0.95 \times d_{y_p}, d_{y_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **282.2** mm

Area of reinforcement required for bending — $A_{sy_p_m} = M_{y_p} / (f_{yd} \times z) =$ **766** mm²/m

Minimum area of reinforcement required — $A_{sy_p_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y_p}, 0.0013 \times b \times d_{y_p}) =$ **467** mm²/m

Area of reinforcement required — $A_{sy_p_req} = \max(A_{sy_p_m}, A_{sy_p_min}) =$ **766** mm²/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress — $\sigma_{sy_p} = (f_{yk} / \gamma_S) \times \min((A_{sy_p_m}/A_{sy_p}), 1.0) \times q_{SLS} / q =$ **170.4** N/mm²

Maximum allowable spacing (Table 7.3N) — $s_{max_y_p} =$ **287** mm

Actual bar spacing — $s_{y_p} =$ **100** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at continuous support in short span direction (cl.6.1)

Bending moment coefficient — $\beta_{sx_n} =$ **0.0310**

Design bending moment — $M_{x_n} = \beta_{sx_n} \times q \times l_x^2 =$ **121.3** kNm/m

Reinforcement provided — 16 mm dia. bars at 100 mm centres

Area provided — $A_{sx_n} = 2011$ mm²/m

Effective depth to tension reinforcement — $d_{x_n} = h - c_{nom_t} - \phi_{x_n} / 2 =$ **307.0** mm

K factor — $K = M_{x_n} / (b \times d_{x_n}^2 \times f_{ck}) =$ **0.040**

Redistribution ratio — $\delta = 1.0$

K' factor — $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm — $z = \min(0.95 \times d_{x_n}, d_{x_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **291.7** mm

Area of reinforcement required for bending — $A_{sx_n_m} = M_{x_n} / (f_{yd} \times z) =$ **957** mm²/m

Minimum area of reinforcement required — $A_{sx_n_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x_n}, 0.0013 \times b \times d_{x_n}) =$ **483** mm²/m

Area of reinforcement required — $A_{sx_n_req} = \max(A_{sx_n_m}, A_{sx_n_min}) =$ **957** mm²/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress — $\sigma_{sx_n} = (f_{yk} / \gamma_S) \times \min((A_{sx_n_m}/A_{sx_n}), 1.0) \times q_{SLS} / q =$ **119.8** N/mm²

Maximum allowable spacing (Table 7.3N) — $s_{max_x_n} =$ **300** mm

Actual bar spacing — $s_{x_n} =$ **100** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at continuous support in long span direction (cl.6.1)

Bending moment coefficient — $\beta_{sy_n} =$ **0.0320**

Design bending moment — $M_{y_n} = \beta_{sy_n} \times q \times l_x^2 =$ **125.3** kNm/m

Reinforcement provided — 16 mm dia. bars at 100 mm centres

Area provided — $A_{sy_n} = 2011$ mm²/m

Effective depth to tension reinforcement — $d_{y_n} = h - c_{nom_t} - \phi_{x_n} - \phi_{y_n} / 2 =$ **291.0** mm

K factor — $K = M_{y_n} / (b \times d_{y_n}^2 \times f_{ck}) =$ **0.046**

Redistribution ratio — $\delta = 1.0$

K' factor — $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm — $z = \min(0.95 \times d_{y_n}, d_{y_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **276.5** mm

Area of reinforcement required for bending — $A_{sy_n_m} = M_{y_n} / (f_{yd} \times z) =$ **1042** mm²/m

Minimum area of reinforcement required — $A_{sy_n_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y_n}, 0.0013 \times b \times d_{y_n}) =$ **458** mm²/m

Area of reinforcement required $A_{sy_n_req} = \max(A_{sy_n_m}, A_{sy_n_min}) = \mathbf{1042}$ mm²/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy_n} = (f_{yk} / \gamma_S) \times \min((A_{sy_n_m}/A_{sy_n}), 1.0) \times q_{SLS} / q = \mathbf{130.4}$ N/mm²

Maximum allowable spacing (Table 7.3N) $s_{max_y_n} = \mathbf{300}$ mm

Actual bar spacing $s_{y_n} = \mathbf{100}$ mm

PASS - The reinforcement spacing is acceptable

Shear capacity check at short span continuous support

Shear force $V_{x_n} = q \times l_x / 2 = \mathbf{156.6}$ kN/m

Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200\text{ mm} / d_{x_n})^{0.5}) = \mathbf{1.807}$

Reinforcement ratio $\rho_l = \min(0.02, A_{sx_n} / (b \times d_{x_n})) = \mathbf{0.0065}$

Minimum shear resistance (Exp. 6.3N) $V_{Rd,c_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{x_n}$

$V_{Rd,c_min} = \mathbf{147.7}$ kN/m

Shear resistance constant (cl. 6.2.2) $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C = \mathbf{0.12}$ N/mm²

Shear resistance (Exp. 6.2a)

$$V_{Rd,c_x_n} = \max(V_{Rd,c_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\text{ N/mm}^2))^{0.333} \times b \times d_{x_n}) = \mathbf{183.4}\text{ kN/m}$$

PASS - Shear capacity is adequate

Shear capacity check at long span continuous support

Shear force $V_{y_n} = q \times l_x / 2 = \mathbf{156.6}$ kN/m

Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200\text{ mm} / d_{y_n})^{0.5}) = \mathbf{1.829}$

Reinforcement ratio $\rho_l = \min(0.02, A_{sy_n} / (b \times d_{y_n})) = \mathbf{0.0069}$

Minimum shear resistance (Exp. 6.3N) $V_{Rd,c_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{y_n}$

$V_{Rd,c_min} = \mathbf{142.5}$ kN/m

Shear resistance constant (cl. 6.2.2) $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C = \mathbf{0.12}$ N/mm²

Shear resistance (Exp. 6.2a)

$$V_{Rd,c_y_n} = \max(V_{Rd,c_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\text{ N/mm}^2))^{0.333} \times b \times d_{y_n}) = \mathbf{179.1}\text{ kN/m}$$

PASS - Shear capacity is adequate

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio $\rho_0 = (f_{ck} / 1\text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0057}$

Required tension reinforcement ratio $\rho = \max(0.0035, A_{sx_p_req} / (b \times d_{x_p})) = \mathbf{0.0035}$

Required compression reinforcement ratio $\rho' = A_{scx_p_req} / (b \times d_{x_p}) = \mathbf{0.0000}$

Stuctural system factor (Table 7.4N) $K_\delta = \mathbf{1.5}$

Basic limit span-to-depth ratio (Exp. 7.16)

$$\text{ratio}_{lim_x_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}] = \mathbf{50.21}$$

Mod span-to-depth ratio limit

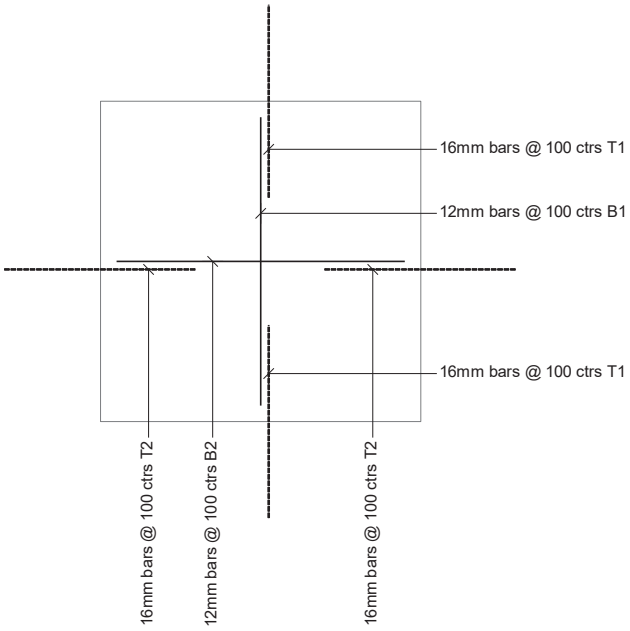
$$\text{ratio}_{lim_x} = \min(40 \times K_\delta, \min(1.5, (500\text{ N/mm}^2 / f_{yk}) \times (A_{sx_p} / A_{sx_p_m})) \times \text{ratio}_{lim_x_bas}) = \mathbf{60.00}$$

Actual span-to-eff. depth ratio $\text{ratio}_{act_x} = l_x / d_{x_p} = \mathbf{40.45}$

PASS - Actual span-to-effective depth ratio is acceptable

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.


16mm bars @ 100 ctrs T1
12mm bars @ 100 ctrs B1
16mm bars @ 100 ctrs T1
16mm bars @ 100 ctrs T2
12mm bars @ 100 ctrs B2
16mm bars @ 100 ctrs T2

RC MEMBER ANALYSIS & DESIGN (EN1992-1-1:2004)

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.2.02

Design summary

Span 1 moment design reinforcement (including crack requirements)

	Unit	Provided	Required	Utilisation	Result
Zone 1 top (-4070.5 kNm)	mm^2	10757	10196	0.948	PASS
Zone 1 bot (508.8 kNm)	mm^2	1885	1167	0.619	PASS (crack width)
Zone 2 top (0.0 kNm)	mm^2	1206	0	0.000	PASS
Zone 2 bot (2035.3 kNm)	mm^2	5102	4642	0.910	PASS
Zone 3 top (-4070.5 kNm)	mm^2	10757	10196	0.948	PASS
Zone 3 bot (508.8 kNm)	mm^2	3493	1136	0.325	PASS (crack width)
Span to depth ratio		11.43	20.17	0.566	PASS

Span 1 shear design reinforcement

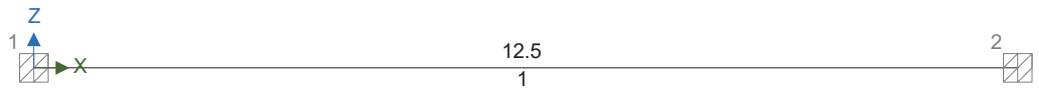
	Unit	Provided	Required	Utilisation	Result
Zone 1 (1608.0 kN)	mm^2/m	2011	1611	0.801	PASS
Zone 2 (976.9 kN)	mm^2/m	1005	891	0.886	PASS
Zone 3 (1609.1 kN)	mm^2/m	2011	1612	0.802	PASS

ANALYSIS

Tedds calculation version 1.0.27

Geometry

Geometry (m) - Concrete (C32 2500 Quartzite) - R 650x1200

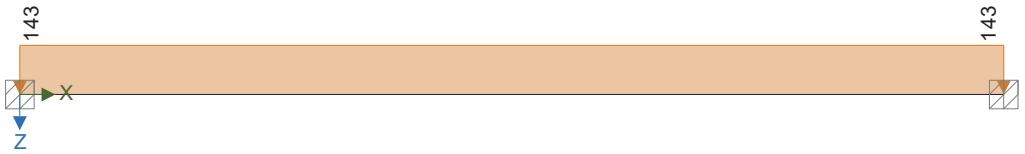


Span	Length (m)	Section	Start Support	End Support
1	12.5	R 650x1200	Fixed	Fixed
R 650x1200: Area 7800 cm^2, Inertia Major 9360000 cm^4, Inertia Minor 2746250 cm^4, Shear area parallel to Minor 6500 cm^2, Shear area parallel to Major = 6500 cm^2				
Concrete (C32 2500 Quartzite): Density 2500 kg/m^3, Youngs 33.3457645 kN/mm^2, Shear 13.8940685 kN/mm^2, Thermal 0.00001 $°C^{-1}$				

Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00
1.0G + 1.0ψ_2Q (Quasi)	1.00	1.00	0.30

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	143 kN/m
Beam	Imposed	UDL	GlobalZ	62.5 kN/m

Results

Total deflection

1.35G + 1.5Q + 1.5RQ (Strength) - Total deflection

Member results

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Member	Position (m)	Deflection (mm)		Axial deflection (mm)	
Beam	0	0		0	
	6.25	7 (max)		0	
	12.5	0		0	

1.0G + 1.0Q + 1.0RQ (Service) - Total deflection

Member results

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Position (m)	Deflection (mm)		Axial deflection (mm)	
Beam	0	0		0	
	6.25	5.1 (max)		0	
	12.5	0		0	

1.0G + 1.0ψ2Q (Quasi) - Total deflection

Member results

Load combination: 1.0G + 1.0ψ2Q (Quasi)

Member	Position (m)	Deflection (mm)		Axial deflection (mm)	
Beam	0	0		0	
	6.25	4.1 (max)		0	
	12.5	0		0	

Node deflections

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0	
2	0	0	0	

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0	
2	0	0	0	

Load combination: 1.0G + 1.0ψ2Q (Quasi)

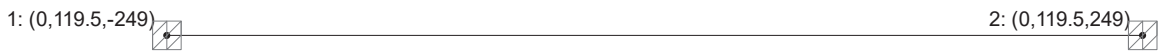
Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0	
2	0	0	0	

Total base reactions

Load case/combination	Force FX (kN)	Force FZ (kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	3907.7
1.0G + 1.0Q + 1.0RQ (Service)	0	2807.8
1.0G + 1.0ψ2Q (Quasi)	0	2260.9

Reactions

Self Weight - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load case: Self Weight

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	119.5	-249
2	0	119.5	249

Permanent - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

Load case: Permanent

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	893.7	-1862
2	0	893.8	1862

Imposed - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

Load case: Imposed

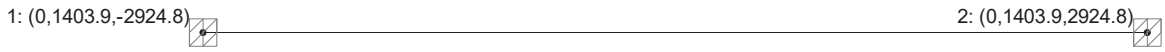
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	390.6	-813.8
2	0	390.6	813.8

1.35G + 1.5Q + 1.5RQ (Strength) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	1953.9	-4070.5
2	0	1953.9	4070.5

1.0G + 1.0Q + 1.0RQ (Service) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	1403.9	-2924.8
2	0	1403.9	2924.8

1.0G + 1.0ψ2Q (Quasi) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

Load combination: 1.0G + 1.0ψ2Q (Quasi)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	0	1130.5	-2355.1
2	0	1130.5	2355.1

Element end forces

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	12.5	1	0	-1953.9	4070.5
		2	0	-1953.9	-4070.5

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

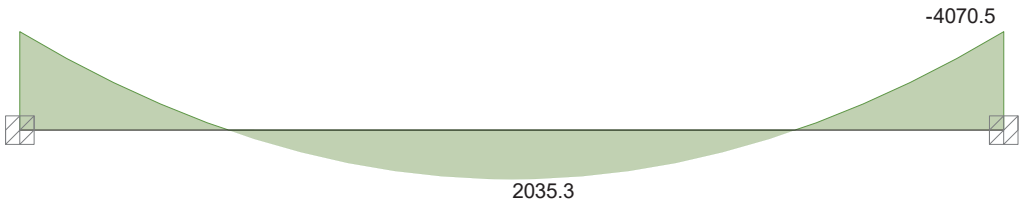
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	12.5	1	0	-1403.9	2924.8
		2	0	-1403.9	-2924.8

Load combination: 1.0G + 1.0ψ2Q (Quasi)

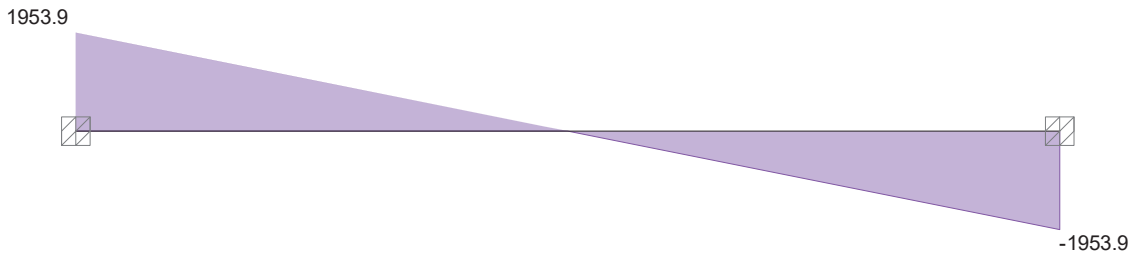
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	12.5	1	0	-1130.5	2355.1
		2	0	-1130.5	-2355.1

Forces

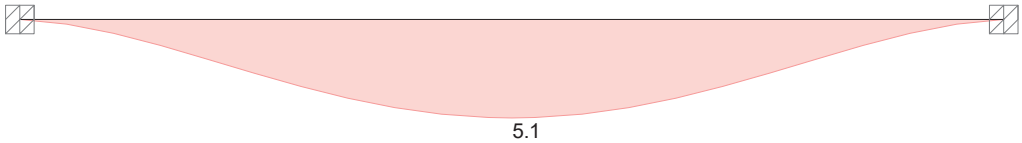
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Member results

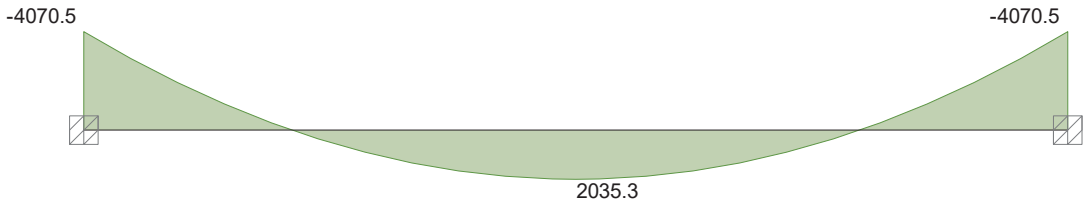
Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Beam	0	1953.9		-4070.5 (min)	
	6.25	0		2035.3 (max)	
	12.5	-1953.9 (max abs)		-4070.5 (min)	

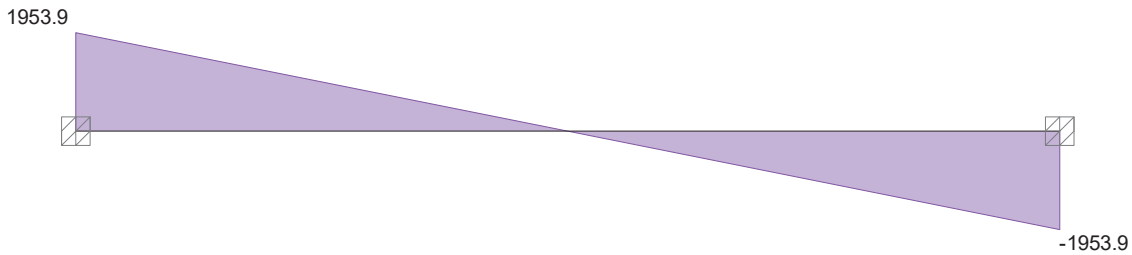
Envelope - Service combinations

Member	Position (m)	Deflection (mm)	
Beam	6.25	5.1 (max)	
	12.5	0	

1.35G + 1.5Q + 1.5RQ (Strength) - Moment (kNm)



1.35G + 1.5Q + 1.5RQ (Strength) - Shear (kN)



Member results

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Beam	0	1953.9		-4070.5 (min)	
	6.25	0		2035.3 (max)	
	12.5	-1953.9 (max abs)		-4070.5 (min)	

Member results

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Member	Position (m)	Deflection (mm)	
Beam	6.25	5.1 (max)	
	12.5	0	

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class	C32/40
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **32** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck}$ + 8 N/mm^2 = **40** N/mm^2
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm^2 × (f_{ck}/ 1 N/mm^2)$^{2/3}$ = **3.0** N/mm^2
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm^2×[f_{cm}/10 N/mm^2]$^{0.3}$ × AAF = **33346** N/mm^2
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Coefficient k_1	k_1 = **0.40**
Coefficient k_2	k_2 = 1.0 × (0.6 + 0.0014 / ε_{cu2}) = **1.00**
Coefficient k_3	k_3 = **0.40**
Coefficient k_4	k_4 = 1.0 × (0.6 + 0.0014 / ε_{cu2}) = **1.00**
Partial factor for concrete -Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **21.3** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Monolithic simple support moment factor	β_1 = **0.25**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **40** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **40** mm
Nominal cover to side reinforcement	c_{nom_s} = **40** mm

Fire resistance

Standard fire resistance period	R = **120** min
Number of sides exposed to fire	3
Minimum width of beam - EN1992-1-2 Table 5.5	b_{min} = **200** mm

Beam - Span 1

Rectangular section details

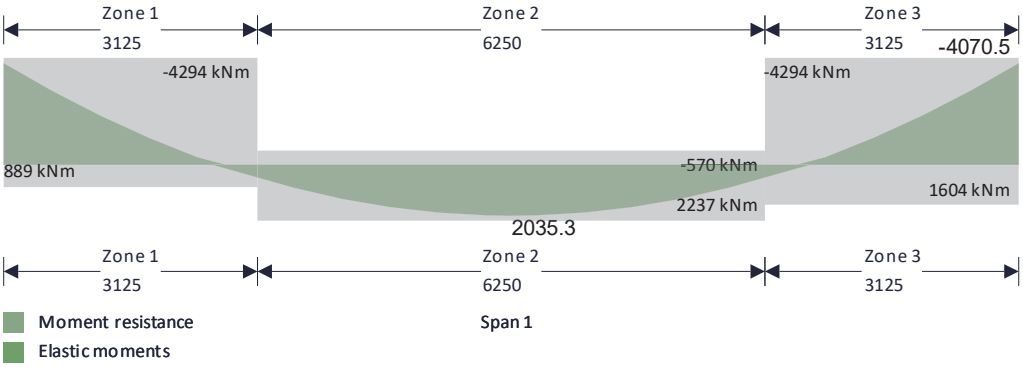
Section width	b = **650** mm
Section depth	h = **1200** mm

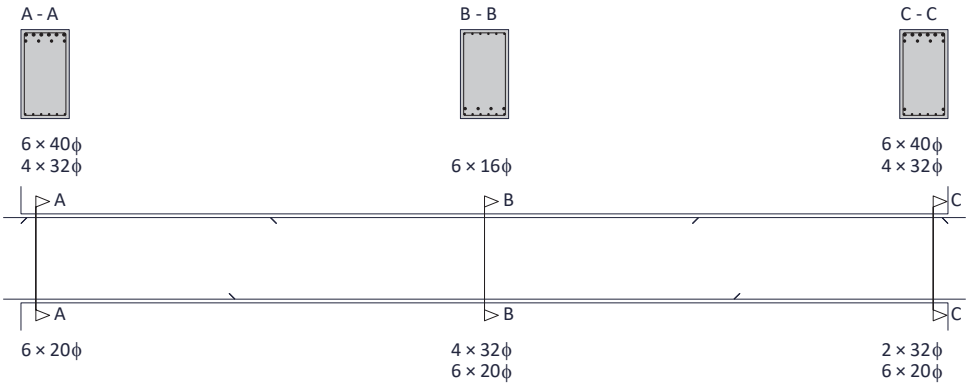
PASS - Minimum dimensions for fire resistance met

WARNING

Section is greater than or equal to 1000mm and requires additional skin reinforcement to control cracking on the sides of the beam in accordance with EN1992-1-1 clause 7.3.3(3). This is beyond the scope of this calculation.

Moment design





Zone 1 (0 mm - 3125 mm) Positive moment - section 6.1

Design bending moment	$M = abs(M_{m1_s1_z1_max_red})$ = **508.8** kNm
Effective depth of tension reinforcement	d = **1142** mm
Redistribution ratio	$\delta = min(M_{pos_red_z1} / M_{pos_z1}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.019**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**
	K' > K - No compression reinforcement is required
Lever arm	$z = min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **1085** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **143** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **1079** mm^2
Tension reinforcement provided	6 × 20ϕ
Area of tension reinforcement provided	$A_{s,prov}$ = **1885** mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1167** mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h$ = **31200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf – 3.2.7(4)	E_s = **200000** N/mm^2

Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = \mathbf{3.0}$ N/mm^2
Stress distribution coefficient	$k_c = \mathbf{0.4}$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) = \mathbf{0.76}$
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / (N_{m1_s1_z1_b_L1} - 1) + \phi_{m1_s1_z1_b_L1} = \mathbf{106.8}$ mm
Maximum stress permitted - Table 7.3N	$\sigma_s = \mathbf{315}$ N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = \mathbf{6.00}$
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{594}$ mm
Area of concrete in the tensile zone	$A_{ct} = b \times y = \mathbf{385796}$ mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{1120}$ mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z1_pos_quasi})) = \mathbf{294.4}$kNm
Permanent load ratio	$R_{PL} = M_{QP} / M = \mathbf{0.58}$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{144}$ N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max} = \mathbf{300}$ mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 1 (0 mm - 3125 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1_s1_max_red}), \text{abs}(M_{m1_s1_z1_min_red})) = \mathbf{4070.5}$ kNm
Effective depth of tension reinforcement	$d = \mathbf{1106}$ mm
Redistribution ratio	$\delta = \min(M_{neg_red_z1} / M_{neg_z1}, 1) = \mathbf{1.000}$
	$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.160}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{918}$ mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = \mathbf{470}$ mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{10196}$ mm^2
Tension reinforcement provided	Layer 1 - 6 × 40ϕ, Spacing - 50mm, Layer 2 - 4 × 32ϕ
Area of tension reinforcement provided	$A_{s,prov} = \mathbf{10757}$ mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{1131}$ mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = \mathbf{31200}$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	$w_k = \mathbf{0.3}$ mm
Design value modulus of elasticity reinf – 3.2.7(4)	$E_s = \mathbf{200000}$ N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = \mathbf{3.0}$ N/mm^2
Stress distribution coefficient	$k_c = \mathbf{0.4}$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) = \mathbf{0.76}$
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1})) / (N_{m1_s1_z1_t_L1} - 1) + \phi_{m1_s1_z1_t_L1} = \mathbf{102.8}$ mm
Maximum stress permitted - Table 7.3N	$\sigma_s = \mathbf{318}$ N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm} =$ **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ **567** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y =$ **368781** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **1060** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment — $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_pos_quasi}), \text{abs}(M_{m1_s1_z1_neg_quasi})) =$ **2355.1**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M =$ **0.58**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **238** N/mm^2

Maximum bar spacing - Tables 7.3N — $s_{bar,max} =$ **202** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing — $s_{top} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1})) / (N_{m1_s1_z1_t_L1} - 1) =$ **62.8** mm

Minimum allowable top bar spacing — $s_{top,min} = \max(\phi_{m1_s1_z1_t_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) =$ **40.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing — $s_{bot} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / (N_{m1_s1_z1_b_L1} - 1) =$ **86.8** mm

Minimum allowable bottom bar spacing — $s_{bot,min} = \max(\phi_{m1_s1_z1_b_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) =$ **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (3125 mm - 9375 mm) Positive moment - section 6.1

Design bending moment — $M = \text{abs}(M_{m1_s1_z2_max_red}) =$ **2035.3** kNm

Effective depth of tension reinforcement — $d =$ **1094** mm

Redistribution ratio — $\delta = \min(M_{pos_red_z2} / M_{pos_z2}, 1) =$ **1.000**

$K = M / (b \times d^2 \times f_{ck}) =$ **0.082**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) =$ **1008** mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda =$ **214** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z) =$ **4642** mm^2

Tension reinforcement provided — Layer 1 - 6 × 20ϕ, Spacing - 50mm, Layer 2 - 4 × 32ϕ

Area of tension reinforcement provided — $A_{s,prov} =$ **5102** mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **1118** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h =$ **31200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width — $w_k =$ **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4) — $E_s =$ **200000** N/mm^2

Mean value of concrete tensile strength — $f_{ct,eff} = f_{ctm} =$ **3.0** N/mm^2

Stress distribution coefficient — $k_c =$ **0.4**

Non-uniform self-equilibrating stress coefficient — $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) =$ **0.76**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1) + \phi_{m1_s1_z2_b_L1}$ = **106.8** mm

Maximum stress permitted - Table 7.3N — σ_s = **315** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **584** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **379834** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1103** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment — $M_{QP} = abs(M_{m1_s1_z2_pos_quasi})$ = **1177.6**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M$ = **0.58**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **229** N/mm^2

Maximum bar spacing - Tables 7.3N — $s_{bar,max}$ = **213.9** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_{m0} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000$ = **0.00566**

Required tension reinforcement ratio — $\rho_m = A_{s,req} / (b \times d)$ = **0.00653**

Required compression reinforcement ratio — $\rho'_m = A_{s2,req} / (b \times d)$ = **0.00000**

Structural system factor - Table 7.4N — K_b = **1.0**

Basic allowable span to depth ratio — $span_to_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / (\rho_m - \rho'_m) + (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho'_m / \rho_{m0})^{0.5} / 12]$ = **18.354**

Reinforcement factor - exp.7.17 — $K_s = min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5)$ = **1.099**

Flange width factor — F1 = 1 = **1.000**

Long span supporting brittle partition factor — F2 = 1 = **1.000**

Allowable span to depth ratio — $span_to_depth_{allow} = min(span_to_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b)$ = **20.173**

Actual span to depth ratio — $span_to_depth_{actual} = L_{m1_s1} / d$ = **11.425**

PASS - Actual span to depth ratio is within the allowable limit

Minimum bar spacing (Section 8.2)

Top bar spacing — $s_{top} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_t_L1} \times N_{m1_s1_z2_t_L1})) / (N_{m1_s1_z2_t_L1} - 1)$ = **91.6** mm

Minimum allowable top bar spacing — $s_{top,min} = max(\phi_{m1_s1_z2_t_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing — $s_{bot} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1)$ = **86.8** mm

Minimum allowable bottom bar spacing — $s_{bot,min} = max(\phi_{m1_s1_z2_b_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (9375 mm - 12500 mm) Positive moment - section 6.1

Design bending moment — $M = abs(M_{m1_s1_z3_max_red})$ = **508.8** kNm

Effective depth of tension reinforcement — d = **1112** mm

Redistribution ratio — $\delta = min(M_{pos_red_z3} / M_{pos_z3}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.020**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **1056** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **139** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **1108** mm^2
Tension reinforcement provided	Layer 1 - 6 × 20ϕ, Spacing - 40mm, Layer 2 - 2 × 32ϕ
Area of tension reinforcement provided	$A_{s,prov}$ = **3493** mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1136** mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h$ = **31200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf – 3.2.7(4)	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1)$ = **0.76**
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1) + \phi_{m1_s1_z3_b_L1}$ = **106.8** mm
Maximum stress permitted - Table 7.3N	σ_s = **315** N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.00**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **589** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **382719** mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1111** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z3_pos_quasi}))$ = **294.4**kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.58**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **80** N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 3 (9375 mm - 12500 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1_s1_max_red}), \text{abs}(M_{m1_s1_z3_min_red}))$ = **4070.5** kNm
Effective depth of tension reinforcement	d = **1106** mm
Redistribution ratio	$\delta = \min(M_{neg_red_z3} / M_{neg_z3}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.160**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **918** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **470** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **10196** mm^2
Tension reinforcement provided	Layer 1 - 6 × 40ϕ, Spacing - 50mm, Layer 2 - 4 × 32ϕ

Area of tension reinforcement provided — $A_{s,prov}$ = **10757** mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1131** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h$ = **31200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width — w_k = **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4) — E_s = **200000** N/mm^2

Mean value of concrete tensile strength — $f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2

Stress distribution coefficient — k_c = **0.4**

Non-uniform self-equilibrating stress coefficient — $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1)$ = **0.76**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1})) / (N_{m1_s1_z3_t_L1} - 1) + \phi_{m1_s1_z3_t_L1}$ = **102.8** mm

Maximum stress permitted - Table 7.3N — σ_s = **318** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **567** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **368781** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1060** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment — $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_pos_quasi}), \text{abs}(M_{m1_s1_z3_neg_quasi}))$ = **2355.1**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M$ = **0.58**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **238** N/mm^2

Maximum bar spacing - Tables 7.3N — $s_{bar,max}$ = **202** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing — $s_{top} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1})) / (N_{m1_s1_z3_t_L1} - 1)$ = **62.8** mm

Minimum allowable top bar spacing — $s_{top,min} = \max(\phi_{m1_s1_z3_t_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **40.0** mm

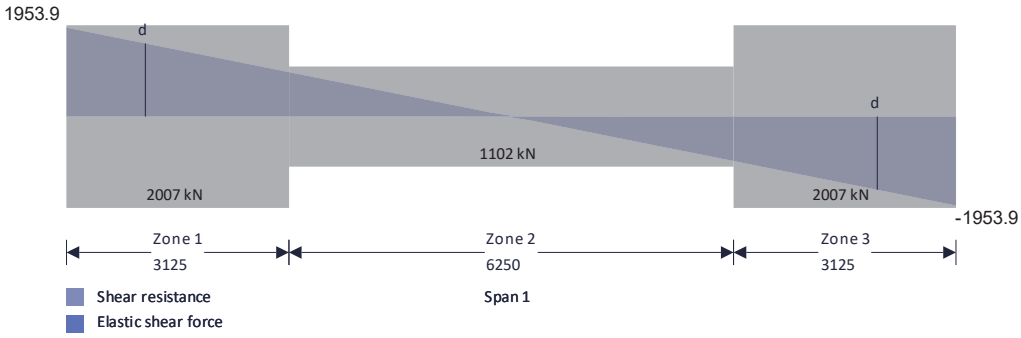
PASS - Actual bar spacing exceeds minimum allowable

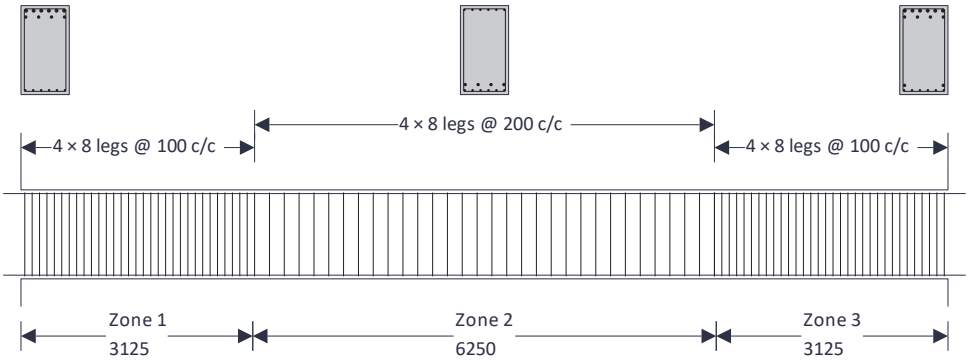
Bottom bar spacing — $s_{bot} = (b - (2 \times (c_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1)$ = **86.8** mm

Minimum allowable bottom bar spacing — $s_{bot,min} = \max(\phi_{m1_s1_z3_b_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Angle of comp. shear strut for maximum shear	θ_{max} = 45 deg
Strength reduction factor - cl.6.2.3(3)	$v_1 = 0.6 \times (1 - f_{ck} / 250\ N/mm^2)$ = **0.523**
Compression chord coefficient - cl.6.2.3(3)	α_{cw} = **1.00**
Minimum area of shear reinforcement - exp.9.5N	$A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **588** mm^2/m

Zone 1 (0 mm - 3125 mm) shear - section 6.2

Design shear force at support	$V_{Ed,max} = max(abs(V_{z1_max}), abs(V_{z1_red_max}))$ = **1954** kN
Min lever arm in shear zone	z = **918** mm
Maximum design shear resistance - exp.6.9	$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (cot(\theta_{max}) + tan(\theta_{max}))$ = **3331** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force at 1106mm from support	V_{Ed} = **1608** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **2.694** N/mm^2
Angle of concrete compression strut - cl.6.2.3	$\theta = min(max(0.5 \times Asin(min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ deg), 45deg)$ = **21.8** deg
Area of shear reinforcement required - exp.6.8	$A_{sv,des} = v_{Ed} \times b / (f_{yd} \times cot(\theta))$ = **1611** mm^2/m
Area of shear reinforcement required	$A_{sv,req} = max(A_{sv,min}, A_{sv,des})$ = **1611** mm^2/m
Shear reinforcement provided	4 × 8 legs @ 100 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **2011** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N	$s_{vl,max} = 0.75 \times d$ = **830** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 2 (3125 mm - 9375 mm) shear - section 6.2

Design shear force at support	$V_{Ed,max} = max(abs(V_{z2_max}), abs(V_{z2_red_max}))$ = **977** kN
Min lever arm in shear zone	z = **1008** mm
Maximum design shear resistance - exp.6.9	$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (cot(\theta_{max}) + tan(\theta_{max}))$ = **3658** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force within zone	V_{Ed} = **977** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **1.490** N/mm^2
Angle of concrete compression strut - cl.6.2.3	$\theta = min(max(0.5 \times Asin(min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ deg), 45deg)$ = **21.8** deg
Area of shear reinforcement required - exp.6.8	$A_{sv,des} = v_{Ed} \times b / (f_{yd} \times cot(\theta))$ = **891** mm^2/m
Area of shear reinforcement required	$A_{sv,req} = max(A_{sv,min}, A_{sv,des})$ = **891** mm^2/m
Shear reinforcement provided	4 × 8 legs @ 200 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N	$s_{vl,max} = 0.75 \times d$ = **821** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 3 (9375 mm - 12500 mm) shear - section 6.2

Design shear force at support	$V_{Ed,max}$ = max(abs(V_{z3_max}), abs($V_{z3_red_max}$)) = **1954** kN
Min lever arm in shear zone	z = **918** mm
Maximum design shear resistance - exp.6.9	$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **3331** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force at 1106mm from support	V_{Ed} = **1608** kN
Design shear stress	$v_{Ed} = V_{Ed} / (b \times z)$ = **2.694** N/mm^2
Angle of concrete compression strut - cl.6.2.3	θ = min(max(0.5 × Asin(min(2 × v_{Ed} / ($\alpha_{cw} \times f_{cwd} \times v_1$),1)), 21.8 deg), 45deg) = **21.8** deg
Area of shear reinforcement required - exp.6.8	$A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **1611** mm^2/m
Area of shear reinforcement required	$A_{sv,req}$ = max($A_{sv,min}$, $A_{sv,des}$) = **1611** mm^2/m
Shear reinforcement provided	4 × 8 legs @ 100 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **2011** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N	$s_{vl,max}$ = 0.75 × d = **830** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Project	Florence Airport	The Concrete Centre	**The Concrete Centre**		
Client			Made by: MC	Date: 30-Nov-21	Page: -
Location	Corner Column Pilecap Design		Checked: -	Revision: -	Job No: -
	PILECAP DESIGN to EN 1992-1: 2004 (without UK NA)	*Double Pilecap*			
	Originated from TCC82.xls v 3.0 on CD	© 2003-2006 TCC			

Usage: Traffic<=30kN

MATERIALS

fck	32	N/mm²	d_g	20	mm	γ_c	1.5	concrete
fyk	500	N/mm²	&S cover	50	mm	γ_s	1.15	steel
Steel class	A		Btm cover	75	mm	onc density	25	kN/m³
Pile capacity	3815.1	kN	based on 6N/mm2 max stress in a pile					

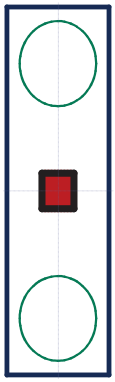
DIMENSIONS mm

COLUMN 550mm dia

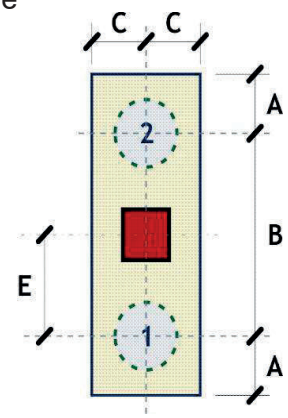
PILECAP

A = 600
B = 2700
C = 600
E = 1350
depth H = 1350

Pile Ø = 900
Tolerance = 75



PLOT *(to scale)*



KEY

COLUMN ACTIONS kN, kNm characteristic

	DEAD	IMPOSED	WIND
Axial (kN)	2955	1050	
ψ_0 =		0.7	0.5

STATUS VALID DESIGN

REINFORCEMENT

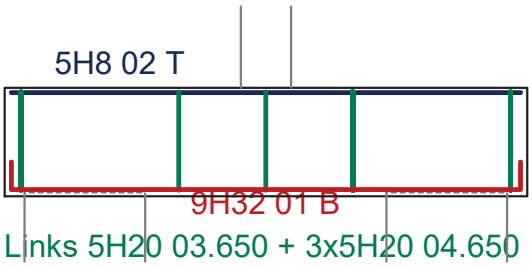
BOTTOM M = 3,482.5 kNm
d = 1,239.0 mm, As = 6,842 mm²
9H32 B = 7,238 mm²

TOP M = 0.0 kNm
d = 1,276.0 mm, As = 0 mm²
5H8 T = 251 mm²

LINKS
VEd = 2874 kN, vEd = 1.933 N/mm²
vRd 2d/av = 1.076 N/mm²
Asw/S = 2.3711 mm
5 Legs H20 @ 650 LINKS = 2.4166 mm

PILE REACTIONS kN

	PILE 1	PILE 2
Gk + Qk	2081.5	2081.5
Gk + Qk +Wk	2081.5	2081.5



ELEVATION

Project	Florence Airport	The Concrete Centre		
Client		Made by MC	Date 30-Nov-21	Page -
Location	Corner Column Pilecap Design - Detailed Calculations	Checked -	Revision -	Job No -

PILECAP DESIGN to EN 1992-1: 2004 (without L **Double Pilecap**

Originated from TCC82.xls v 3.0 on CD © 2003-2006 TCC

Cap load (kN) = 158.0 iles @ (m) 2.700

or (kN/m) = 40.50

OVERTURNING MOMENTS - kNm characteristic

DEAD	IMPOSED	WIND
0.0	0.0	0.0

PILE REACTIONS kN

	PILE 1	PILE 2
Gk + Qk	2081.5	2081.5
Gk + Qk +Wk	2081.5	2081.5
1.35Gk + 1.5Qk	2888.7	2888.7
1.0Gk + 1.5Wk	1556.5	1556.5
1.35Gk + 1.5Qk + 0.75Wk	2888.7	2888.7

BENDING MOMENTS - kNm

	1.35Gk + 1.5Qk	1.0Gk + 1.5Wk	1.35Gk + 1.5Qk + 0.75Wk	
M ↓ of col	3482.5	1857.8	3482.5	arm (m) 1.2375
M ↑ of col	3482.5	1857.8	3482.5	arm (m) 1.2375

(including tolerance)

BOTTOM STEEL

Bottom M =	3482.5	Ø32
d =	1239.0	
K =	0.0591	
z =	1170.6	
As =	6842	
Provide	6842	
No =	9	
As prov =	7238	
σs =	293	
Max S =	183.7	
Min S =	64.0	
S =	128.5	**9H32 B**

TOP STEEL

Top M =	0.0	Ø8
d =	1276.0	
K =	0.0000	
z =	1212.2	
As =	0	
Provide	0	
No =	5	
As prov =	251	
σs =	0	
Max S =	300.0	
Min S =	33.0	
S =	263.0	**5H8 T**

K' = 0.2067

min As = 0.157%

SHEAR Crit section is 270.0 from pile centres 0.487%

PILE 1

	1.35Gk + 1.5Qk	1.0Gk + 1.5Wk	1.35Gk + 1.5Qk + 0.75Wk
V_{Ed}	2874.0	1545.5	2874.0

a_v = 967.5 v_{Ed} = 1.9330 cotθ = 2.50 v_{Rd} 2d/a_v = 1.0760

PILE 2

V_{Ed}	2874.0	1545.5	2874.0

a_v = 967.5 v_{Ed} = 1.9330 cotθ = 2.50 v_{Rd} 2d/a_v = 1.0760

$v_{Rd,max}$ = 3.849

Asw/S required= 2.3711 mm Ø20 links

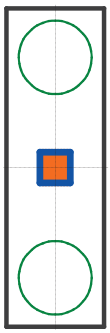
No of legs = 5 Link spacing = 650 **5 Legs H20 @ 650 LINKS**

Project	Florence Airport	The Concrete Centre	Made by	Date	
Client			MC	30-Nov-21	
Location	Edge Column Pilecap Design	Double Pilecap	Checked	Revision	Job No
			-	-	

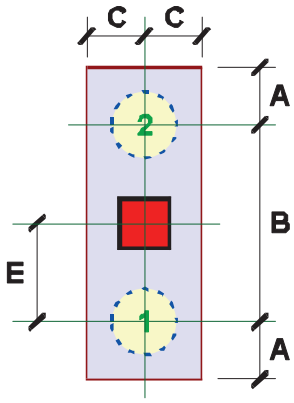
MATERIALS									
	fcu	35	N/mm²	h agg	20	mm	γc	1.5	concrete
	fy	500	N/mm²	&S cover	50	mm	γs	1.15	steel
	Pile capacity	3815.1	kN	tm cover	75	mm	steel class	A	
				Conc density	23.6	kN/m³			

DIMENSIONS mm

COLUMN	PILECAP
	A = 600
	B = 2700
	C = 600
Pile Ø = 900	E = 1350
Tolerance = 75	depth H = 1350



PLOT *(to scale)*



COLUMN ACTIONS kN, kNm characteristic

	DEAD	IMPOSED	
Axial (kN)	4638	1440	0
M (kNm)	0	0	0
H (kN)	0	0	0

STATUS VALID DESIGN

REINFORCEMENT

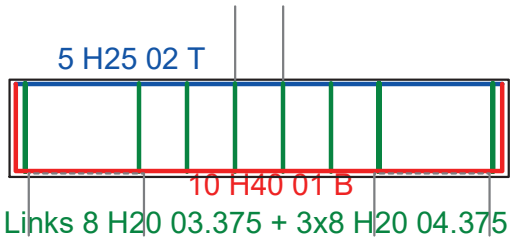
BOTTOM M = 5,482.1 kNm
d = 1,235.0 mm, As = 11,425 mm²
10 H40 B = 12,566 mm²

TOP M = 0.0 kNm
d = 1,267.5 mm, As min = 2,106 mm²
5 H25 T = 2,454 mm²

LINKS
V = 4488.5 kN, v = 3.029 N/mm²
vc 2d/av = 1.53 N/mm²
(v - vc)b = 1,797.9 N/mm
5 Legs H20 @ 375 LINKS = 1,821.2 N/mm

PILE REACTIONS kN

	PILE 1	PILE 2
Gk + Qk	3113.6	3113.6
Gk + Qk +Wk	3113.6	3113.6



ELEVATION

Project	Florence Airport	The Concrete Centre		
Client	0	Made by: MC	Date: 30-Nov-21	Page: 1
Location	Edge Column Pilecap Design - Deta 0	Checked: -	Revision: -	Job No: 0

The Concrete Centre™ **Double Pilecap**

Cap load (kN) = 149.1 iles @ (m) 2.700
or (kN/m) = 38.23

OVERTURNING MOMENTS - kNm characteristic

DEAD	IMPOSED	WIND
0.0	0.0	0.0

PILE REACTIONS kN

	PILE 1	PILE 2
Gk + Qk	3113.6	3113.6
Gk + Qk +Wk	3113.6	3113.6
1.4Gk + 1.6Qk	4503.0	4503.0
Gk + 1.4Wk	2393.6	2393.6
1.2(Gk+Qk+Wk)	3736.3	3736.3

BENDING MOMENTS - kNm

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)	
M ↓ of col	5482.1	2897.5	4546.2	arm (m) 1.2375
M ↑ of col	5482.1	2897.5	4546.2	arm (m) 1.2375

(including tolerance)

BOTTOM STEEL

Bottom M =	5482.1	Ø40
d =	1235.0	
K =	0.0856	
z =	1103.6	
As =	11425	
Provide	11425	
No =	10	
As prov =	12566	
fs =	303	
Max clear S =	155.1	
Min clear S =	40.0	
Clear S =	73.3	10 H40 B

TOP STEEL

Top M =	0.0	Ø25
d =	1267.5	
K =	0.0000	
z =	1204.1	
As =	0	
Provide	2106	
No =	5	
As prov =	2454	
fs =	0	
Max clear S =	950.6	
Min clear S =	25.0	
Clear S =	233.8	5 H25 T

K' = 0.1558
min As = 0.13%

SHEAR Crit section is 270.0 from pile centres 0.848 %

PILE 1

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)
V	4488.5	2383.2	3723.9

Av = 1080.0 v = 3.0287 vc 2d/av = 1.5305

PILE 2

V	4488.5	2383.2	3723.9

Av = 1080.0 v = 3.0287 vc 2d/av = 1.5305

Max (v - vc)b = 1797.9 Ø20 links
No of legs = 5 Link spacing = 375 5 Legs H20 @ 375 LINKS

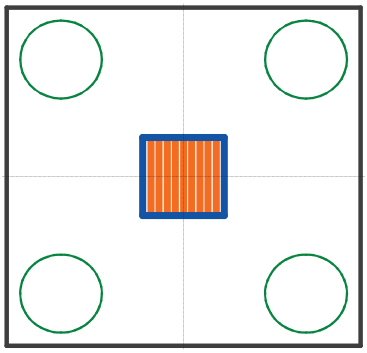
Project	Florence Airport	The Concrete Centre		
Client	0	Made by: MC	Date: 30-Nov-21	Page:
Location	Pilecap under central column	4 Pile Cap	Checked: -	Revision: - / Job No:

Originated from RCC82.xls v 3.2 on CD

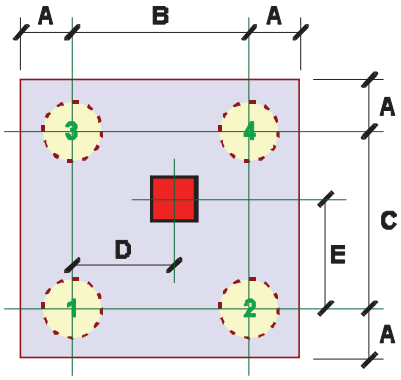
© 2006 TCC

DIMENSIONS mm

COLUMN	PILECAP
	A = 600
	B = 2700
	C = 2700
	D = 1350
Pile Ø = 900	E = 1350
Tolerance = 75	depth H = 1350



PLOT *(to scale)*



KEY

COLUMN ACTIONS kN, kNm characteristic

	DEAD	IMPOSED	
Axial (kN)	7597.0	2582.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN

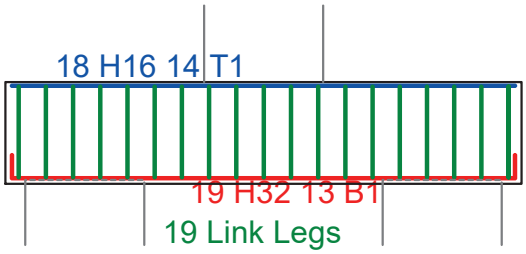
PILE REACTIONS kN

	PILE 1	PILE 2	PILE 3	PILE 4
Gk + Qk	2665.9	2665.9	2665.9	2665.9
Gk + Qk +Wk	2669.3	2669.3	2669.3	2669.3

REINFORCEMENT

EW(1/3-2/4) M = 7,529.6 kNm, b = 3,900 mm
d = 1,259.0 mm, As = 14,479 mm²
19 H32 B1 = 15,281 mm²

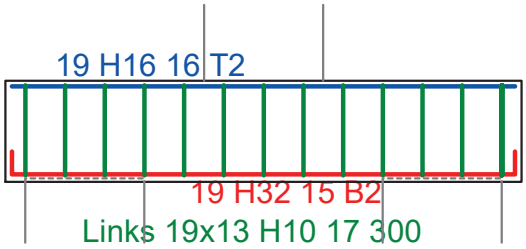
V = 7,722.7 kN, bv = 3,900 mm
v = 1.573 N/mm², (v-vc)b = 1,777 N/mm



↑ ELEVATION

NS(1/2-3/4) M = 7,529.6 kNm, b = 3,900 mm
d = 1,227.0 mm, As = 14,857 mm²
19 H32 B2 = 15,281 mm²

V = 7,722.7 kN, bv = 3,900 mm
v = 1.614 N/mm², (v-vc)b = 2,012 N/mm
19 Legs H10 @ 300 LINKS = 2,163 N/mm



← ELEVATION

Project	Florence Airport	The Concrete Centre	Made by	Date	Page
Client	0	4 Pile Cap	MC	30-Nov-21	1
Location	Pilecap under central column		Checked	Revision	Job No
	0		-	-	0

Originated from RCC82.xls v 3.2 on CD

© 2006 TCC

Cap load = 484.6 kN

OVERTURNING MOMENTS - kNm character

	DEAD	IMPOSED	WIND
Mx	0.0	0.0	0.0
My	0.0	0.0	0.0

PILE REACTIONS kN

	PILE 1	PILE 2	PILE 3	PILE 4
Gk + Qk	2665.9	2665.9	2665.9	2665.9
Gk + Qk +Wk	2669.3	2669.3	2669.3	2669.3
.4Gk + 1.6Qk	3861.4	3861.4	3861.4	3861.4
Gk + 1.4Wk	2025.2	2025.2	2025.2	2025.2
(Gk+Qk+Wk)	3203.2	3203.2	3203.2	3203.2

BENDING MOMENTS - kNm

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)
My v of col	7529.6	3949.1	6246.2
My ^ of col	7529.6	3949.1	6246.2
My < of col	7529.6	3949.1	6246.2
My > of col	7529.6	3949.1	6246.2

(including tolerance)

E-W STEEL	BOTTOM	TOP	N-S STEE	BOTTOM	TOP
	Ø32	Ø16		Ø32	Ø16
M =	7529.6	0.0	M =	7529.6	0.0
b =	3900	3900	b =	3900	3900
d =	1259.0	1292.0	d =	1227.0	1276.0
K =	0.0348	0.0000	K =	0.0366	0.0000
z =	1196.1	1227.4	z =	1165.7	1212.2
As =	14479	0	As =	14857	0
Provide	14479	0	Provide	14857	0
No =	19	18	No =	19	19
As prov =	15281	3619	As prov =	15281	3820
fs =	316	18 H16 T1	fs =	324	19 H16 T2
Max clear S =	268.0		Max clear S =	268.0	
Min clear S =	32.0		Min clear S =	32.0	
Clear S =	176.2		Clear S =	176.2	
	19 H32 B1			19 H32 B2	

K' = 0.1558

min As = 0.13%

BEAM SHEAR Critical section is 270 mm from pile centres

EW (NS plane) Ø10 links

V = 7722.7 v = 1.5728 As% = 0.311

av = 1080.0 vc = 1.1171 (v - vc)b = 1777.5

No of legs = 18 Spacing = 300 18 Legs H10 @ 300 LINKS

NS (EW plane)

V = 7722.7 v = 1.6138 As% = 0.319

av = 1080.0 vc = 1.0980 (v - vc)b = 2011.6

No of legs = 19 Spacing = 300 19 Legs H10 @ 300 LINKS

PUNCHING Column Face No other critical section

V = 14767.0 μ = 8640 d ave = 1243

v = 3.300 v = 0.000 av = 1080.0

v max = 4.733 ok vc 2d/av = 1.108 ok

Project	Florence Airport	The Concrete Centre		
Client		Made by: MC	Date: 30-Nov-21	Page:
Location	Pilecap under Central column with hanger loads	4 Pile Cap	Checked: -	Revision: - / Job No:

Originated from RCC82.xls v 3.2 on CD

© 2006 TCC

DIMENSIONS mm

COLUMN

Pile Ø = 900

Tolerance = 75

PILECAP

A = 600

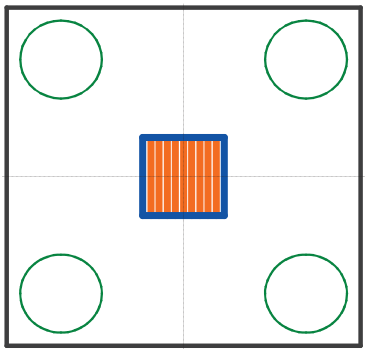
B = 2700

C = 2700

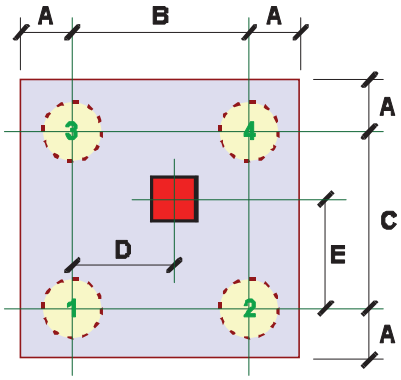
D = 1350

E = 1350

depth H = 1350



PLOT *(to scale)*



KEY

COLUMN ACTIONS kN, kNm characteristic

	DEAD	*IMPOSED*	*WIND*
Axial (kN)	7847.0	2707.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN

PILE REACTIONS kN

	PILE 1	*PILE 2*	*PILE 3*	*PILE 4*
Gk + Qk	2759.6	2759.6	2759.6	2759.6
Gk + Qk +Wk	2763.0	2763.0	2763.0	2763.0

REINFORCEMENT

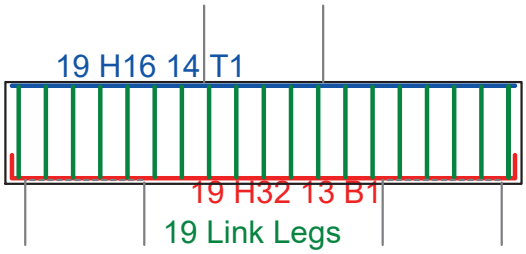
EW(1/3-2/4) M = 7,797.8 kNm, b = 3,900 mm

d = 1,259.0 mm, As = 14,995 mm²

19 H32 B1 = 15,281 mm²

V = 7,997.7 kN, bv = 3,900 mm

v = 1.629 N/mm², (v-vc)b = 1,996 N/mm



↑ ELEVATION

NS(1/2-3/4) M = 7,797.8 kNm, b = 3,900 mm

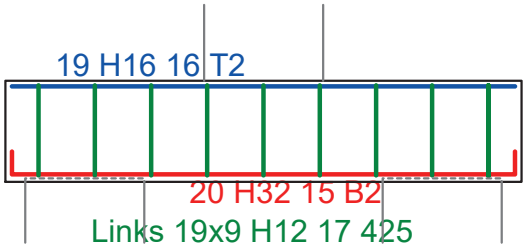
d = 1,227.0 mm, As = 15,386 mm²

20 H32 B2 = 16,085 mm²

V = 7,997.7 kN, bv = 3,900 mm

v = 1.671 N/mm², (v-vc)b = 2,162 N/mm

19 Legs H12 @ 425 LINKS = 2,198 N/mm



← ELEVATION

Project	Florence Airport	The Concrete Centre	Made by	Date	Page
Client	0	4 Pile Cap	MC	30-Nov-21	1
Location	Pilecap under Central column with h		Checked	Revision	Job No
	0		-	-	0

Originated from RCC82.xls v 3.2 on CD

© 2006 TCC

Cap load = 484.6 kN

OVERTURNING MOMENTS - kNm character

	DEAD	IMPOSED	WIND
Mx	0.0	0.0	0.0
My	0.0	0.0	0.0

PILE REACTIONS kN

	PILE 1	PILE 2	PILE 3	PILE 4
Gk + Qk	2759.6	2759.6	2759.6	2759.6
Gk + Qk +Wk	2763.0	2763.0	2763.0	2763.0
.4Gk + 1.6Qk	3998.9	3998.9	3998.9	3998.9
Gk + 1.4Wk	2087.7	2087.7	2087.7	2087.7
(Gk+Qk+Wk)	3315.7	3315.7	3315.7	3315.7

BENDING MOMENTS - kNm

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)
My v of col	7797.8	4070.9	6465.5
My ^ of col	7797.8	4070.9	6465.5
My < of col	7797.8	4070.9	6465.5
My > of col	7797.8	4070.9	6465.5

(including tolerance)

E-W STEEL	BOTTOM	TOP	N-S STEE	BOTTOM	TOP
	Ø32	Ø16		Ø32	Ø16
M =	7797.8	0.0	M =	7797.8	0.0
b =	3900	3900	b =	3900	3900
d =	1259.0	1292.0	d =	1227.0	1276.0
K =	0.0360	0.0000	K =	0.0379	0.0000
z =	1196.1	1227.4	z =	1165.7	1212.2
As =	14995	0	As =	15386	0
Provide	14995	0	Provide	15386	0
No =	19	19	No =	20	19
As prov =	15281	3820	As prov =	16085	3820
fs =	327	19 H16 T1	fs =	319	19 H16 T2
Max clear S =	268.0		Max clear S =	268.0	
Min clear S =	32.0		Min clear S =	32.0	
Clear S =	176.0		Clear S =	165.1	
	19 H32 B1			20 H32 B2	

K' = 0.1558

min As = 0.13%

BEAM SHEAR Critical section is 270 mm from pile centres

EW (NS plane) Ø12 links

V = 7997.7 v = 1.6288 As% = 0.311

av = 1080.0 vc = 1.1171 (v - vc)b = 1995.9

No of legs = 19 Spacing = 425 19 Legs H12 @ 425 LINKS

NS (EW plane)

V = 7997.7 v = 1.6713 As% = 0.336

av = 1080.0 vc = 1.1170 (v - vc)b = 2161.9

No of legs = 19 Spacing = 425 19 Legs H12 @ 425 LINKS

PUNCHING Column Face No other critical section

V = 15317.0 μ = 8640 d ave = 1243

v = 3.423 v = 0.000 av = 1080.0

v max = 4.733 ok vc 2d/av = 1.117 ok

Project	Florence Airport	The Concrete Centre		
Client		Made by: MC	Date: 30-Nov-21	Page:
Location	Pilecap under Central column with hanger loads (25m span area)	4 Pile Cap	Checked: -	Revision: - / Job No:

Originated from RCC82.xls v 3.2 on CD © 2006 TCC

DIMENSIONS mm

COLUMN

Pile Ø = 900

Tolerance = 75

PILECAP

A = 600

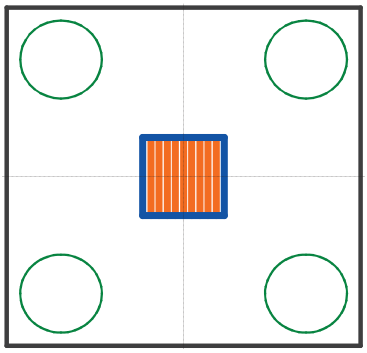
B = 2700

C = 2700

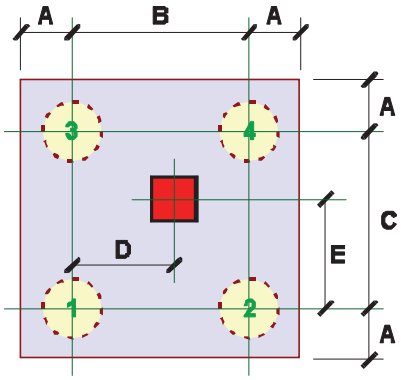
D = 1350

E = 1350

depth H = 1350



PLOT *(to scale)*



KEY

COLUMN ACTIONS kN, kNm characteristic

	DEAD	*IMPOSED*	*WIND*
Axial (kN)	10048.0	3420.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN

PILE REACTIONS kN

	PILE 1	*PILE 2*	*PILE 3*	*PILE 4*
Gk + Qk	3488.1	3488.1	3488.1	3488.1
Gk + Qk +Wk	3488.1	3488.1	3488.1	3488.1

REINFORCEMENT

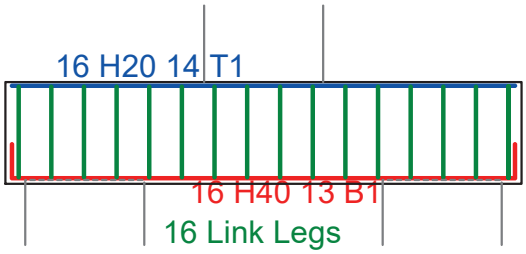
EW(1/3-2/4) M = 9,856.1 kNm, b = 3,900 mm

d = 1,255.0 mm, As = 19,091 mm²

16 H40 B1 = 20,106 mm²

V = 10,108.8 kN, bv = 3,900 mm

v = 2.065 N/mm², (v-vc)b = 3,291 N/mm



↑ ELEVATION

NS(1/2-3/4) M = 9,856.1 kNm, b = 3,900 mm

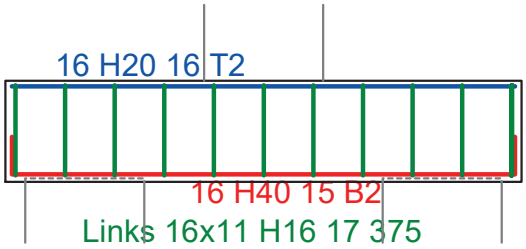
d = 1,215.0 mm, As = 19,800 mm²

16 H40 B2 = 20,106 mm²

V = 10,108.8 kN, bv = 3,900 mm

v = 2.133 N/mm², (v-vc)b = 3,658 N/mm

16 Legs H16 @ 375 LINKS = 3,730 N/mm



← ELEVATION

Project	Florence Airport	The Concrete Centre	Made by	Date	Page
Client	0	4 Pile Cap	MC	30-Nov-21	1
Location	Pilecap under Central column with h		Checked	Revision	Job No
	0		-	-	0

Originated from RCC82.xls v 3.2 on CD

© 2006 TCC

Cap load = 484.6 kN

OVERTURNING MOMENTS - kNm character

	DEAD	IMPOSED	WIND
Mx	0.0	0.0	0.0
My	0.0	0.0	0.0

PILE REACTIONS kN

	PILE 1	PILE 2	PILE 3	PILE 4
Gk + Qk	3488.1	3488.1	3488.1	3488.1
Gk + Qk +Wk	3488.1	3488.1	3488.1	3488.1
.4Gk + 1.6Qk	5054.4	5054.4	5054.4	5054.4
Gk + 1.4Wk	2633.1	2633.1	2633.1	2633.1
(Gk+Qk+Wk)	4185.8	4185.8	4185.8	4185.8

BENDING MOMENTS - kNm

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)
My v of col	9856.1	5134.6	8162.3
My ^ of col	9856.1	5134.6	8162.3
My < of col	9856.1	5134.6	8162.3
My > of col	9856.1	5134.6	8162.3

(including tolerance)

E-W STEEL	BOTTOM	TOP	N-S STEE	BOTTOM	TOP
	Ø40	Ø20		Ø40	Ø20
M =	9856.1	0.0	M =	9856.1	0.0
b =	3900	3900	b =	3900	3900
d =	1255.0	1290.0	d =	1215.0	1270.0
K =	0.0458	0.0000	K =	0.0489	0.0000
z =	1187.4	1225.5	z =	1144.9	1206.5
As =	19091	0	As =	19800	0
Provide	19091	0	Provide	19800	0
No =	16	16	No =	16	16
As prov =	20106	5027	As prov =	20106	5027
fs =	316	16 H20 T1	fs =	328	16 H20 T2
Max clear S =	260.0		Max clear S =	260.0	
Min clear S =	40.0		Min clear S =	40.0	
Clear S =	208.5		Clear S =	208.5	
	16 H40 B1			16 H40 B2	

K' = 0.1558

min As = 0.13%

BEAM SHEAR Critical section is 270 mm from pile centres

EW (NS plane) Ø16 links

V = 10108.8 v = 2.0653 As% = 0.411

av = 1080.0 vc = 1.2215 (v - vc)b = 3291.1

No of legs = 16 Spacing = 375 16 Legs H16 @ 375 LINKS

NS (EW plane)

V = 10108.8 v = 2.1333 As% = 0.424

av = 1080.0 vc = 1.1954 (v - vc)b = 3658.0

No of legs = 16 Spacing = 375 16 Legs H16 @ 375 LINKS

PUNCHING Column Face No other critical section

V = 19539.2 μ = 8640 d ave = 1235

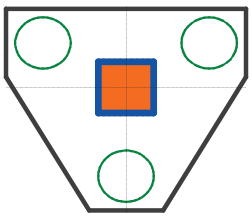
v = 4.395 v = 0.000 av = 1080.0

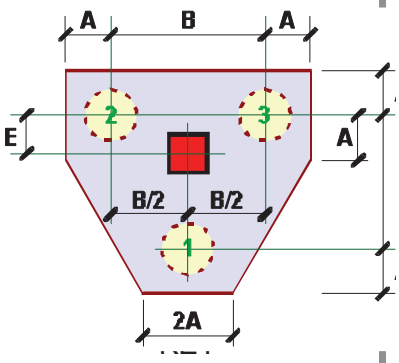
v max = 4.733 ok vc 2d/av = 1.209 ok

Project	Florence Airport	The Concrete Centre	Made by	Date	Page
Client			MC	30-Nov-21	
Location	Pilecap under central column supporting only 1st floor	Triple Pilecap	Checked	Revision	Job No
	Originated from RCC82.xls v 3.2 on CD	© 2006 TCC	-	-	

DIMENSIONS mm

COLUMN	PILECAP
	A = 600
	B = 2700
	C = 2339
Pile Ø = 900	E = 779.67
in spacing = 2700	depth H = 1350
Tolerance = 75	





PLOT *(to scale)*

COLUMN ACTIONS kN, kNm characteristic

	DEAD	IMPOSED	WIND
Axial (kN)	6502.8	2347.4	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

STATUS VALID DESIGN

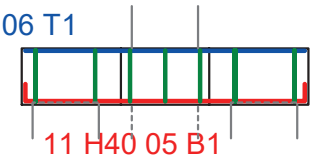
PILE REACTIONS kN characteristic

	PILE 1	PILE 2	PILE 3
Gk + Qk	3063.1	3063.1	3063.1
Gk + Qk +Wk	3063.1	3063.1	3063.1

REINFORCEMENT

EW (2-3) M = 6,333.9 kNm, b = 1,800 mm
d = 1,239.0 mm, As = 12,767 mm²
11 H40 B = 13,823 mm²

6 H12 06 T1



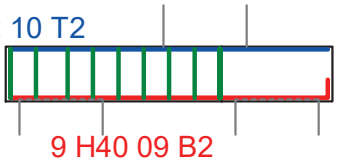
11 H40 05 B1

V = 4,444.8 kN, bv = 1,800 mm
v = 1.993 N/mm², (v-vc)b = 1,098 N/mm
6 Legs H16 @ 475 LINKS = 1,104 N/mm

Links 7 H16 07 475 + 4x7 H16 08 475
↑ ELEVATION

NS (1-2/3) M = 5,264.2 kNm, b = 3,173 mm
d = 1,199.0 mm, As = 10,630 mm²
9 H40 B = 11,310 mm²

5 H12 10 T2



9 H40 09 B2

V = 4,444.8 kN, bv = 2,204 mm
v = 1.682 N/mm², (v-vc)b = 1,523 N/mm
5 Legs H16 @ 275 LINKS = 1,589 N/mm

Links 9 H16 11 275 + 3x9 H16 12 275
← ELEVATION

Project	Florence Airport	The Concrete Centre		
Client		Made by	Date	Page
Location	Pilecap under central column suppo	MC	30-Nov-21	15
	0	Checked	Revision	Job No
	Originated from RCC82.xls v 3.2 on CD © 2006 TCC	-	-	P2000

Triple Pilecap

Cap load = 339.1 kN Group centre @ 1559 m from pile 1

PILE REACTIONS kN

	PILE 1	PILE 2	PILE 3
Gk + Qk	3063.1	3063.1	3063.1
Gk + Qk +Wk	3063.1	3063.1	3063.1
.4Gk + 1.6Qk	4444.8	4444.8	4444.8
Gk + 1.4Wk	2280.6	2280.6	2280.6
(Gk+Qk+Wk)	3675.7	3675.7	3675.7

OVERTURNING MOMENTS - kNm characte

	DEAD	IMPOSED	WIND
Mx	0.0	0.0	0.0
My	0.0	0.0	0.0

BENDING MOMENTS - kNm

	1.4Gk + 1.6Qk	Gk + 1.4Wk	1.2(Gk+Qk+Wk)
My v of col	5264.2	2701.0	4353.3
My ^ of col	3597.4	1845.8	2974.9
Mx	6333.9	3249.9	5237.9

(including tolerance)

E-W STEEL

M =	6333.9	Ø40
b =	1800	
d =	1239.0	
K =	0.0655	
z =	1141.1	
As =	12767	
Provide	12767	
No =	11	
As prov =	13823	
fs =	308	
Max clear S =	246.3	
Min clear S =	40.0	
Clear S =	62.8	11 H40 B

N-S STEEL

M =	5264.2	Ø40
b =	3173	
d =	1199.0	
K =	0.0330	
z =	1139.1	
As =	10630	
Provide	10630	
No =	9	
As prov =	11310	
fs =	313	
Max clear S =	260.0	
Min clear S =	40.0	
Clear S =	88.5	9 H40 B

K' = 0.1558

min As = 0.13%

SHEAR Crit section is 270.0 from pile centres

PILES 2 & 3 Ø16 links

V = 4444.8 b = 1800 As% = 0.620 .

av = 1080.0 v = 1.9930 vc = 1.3831

(v - vc)b = 1097.8 ɔ of legs = 6 Spacing = 475 6 Legs H16 @ 475 LINKS

PILE 1 Ø16 links

V = 4444.8 b = 2204 As% = 0.428 .

av = 1289.3 v = 1.6818 vc = 0.9909

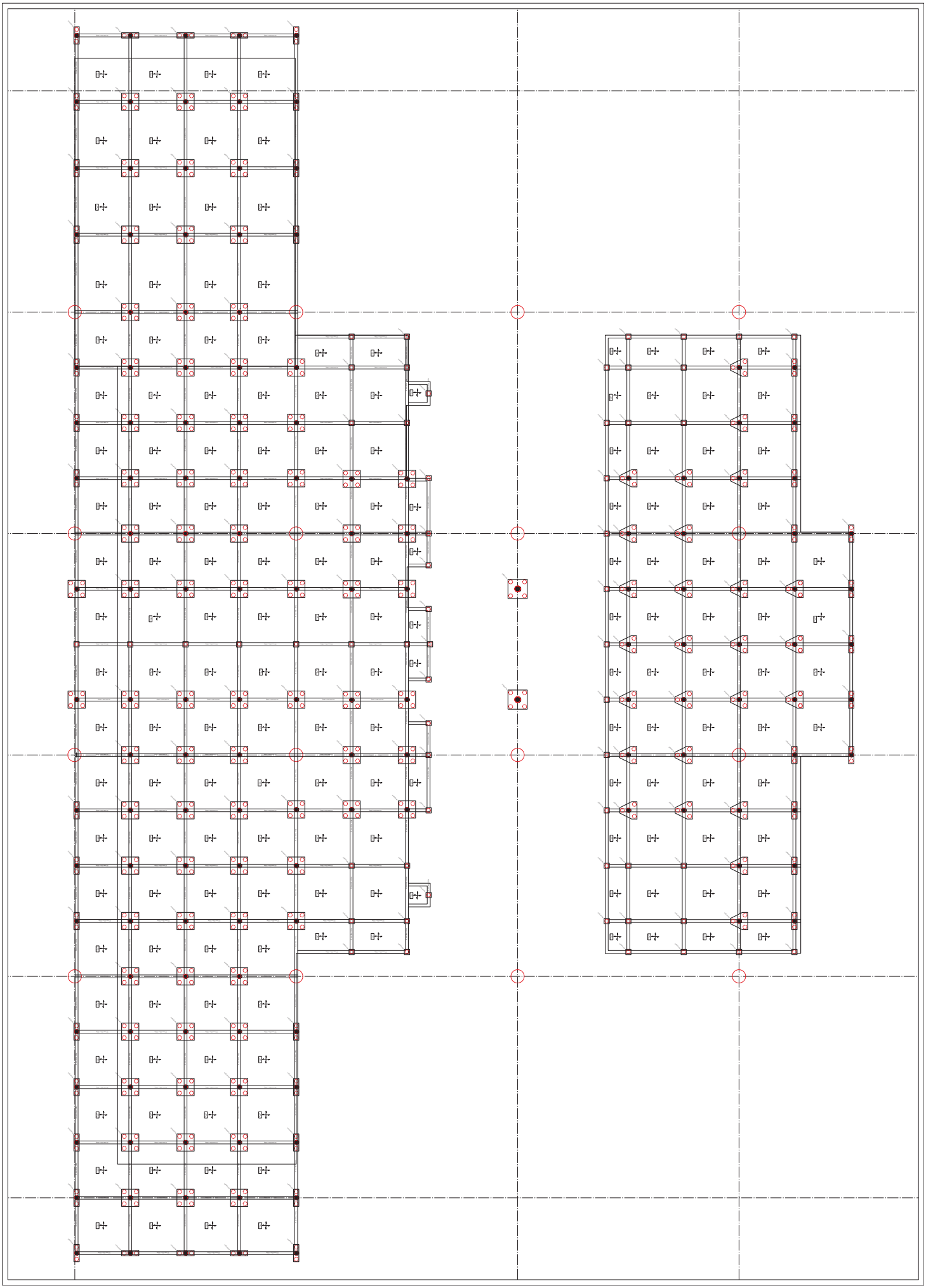
(v - vc)b = 1522.9 ɔ of legs = 5 Spacing = 275 5 Legs H16 @ 275 LINKS

PUNCHING Column Face At Fig 3.23 critical section

V = 12859.8 μ = 9718 d ave = 1219

v = 2.930 v = 1.086 av = 989.8

v max = 4.733 ok vc 2d/av = 1.343 ok



Appendice D: Calcoli del primo piano

Surface loading

				kN/m2
Finishes	(40mm stone + 20mm bedding)			1.2
Screed	75 mm	Screed density assumed 20kN/m3		1.5
Insitu RC topping	50 mm	Concrete density assumed 25kN/m3		1.25
Hollowcore planks	400mm deep			4.68
Services				0.5
Ceiling				1
			DL=	10.2
Variable load			LL=	5

Linear loading

			kN/m2	kN/m
Perimetral beams facade	8 m height	FL=	1.50	12.00
Beam self weight	automatcally applied in software			

Safety factors

γ_{DL}	1.35	
γ_{LL}	1.50	
Tributary width	12.5	m

Type (3) **Typical beam load** SLS

DL [kN/m]	129.9
LL [kN/m]	62.5

. .

Prestressed Y beam

Beams at 1200mm c/c

L adj = L(1.2/S)^0.5

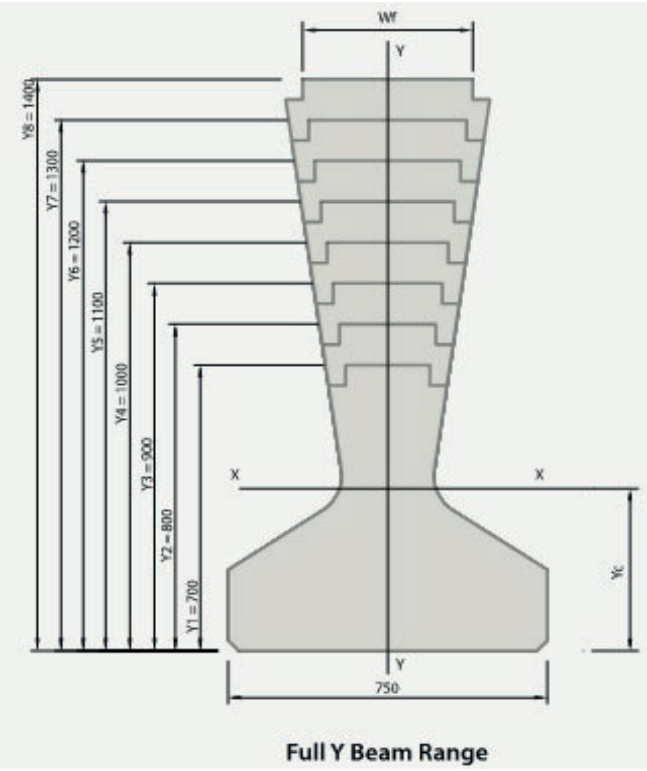
Beam c/c: say S= 1.2 m
Beam span 24 m

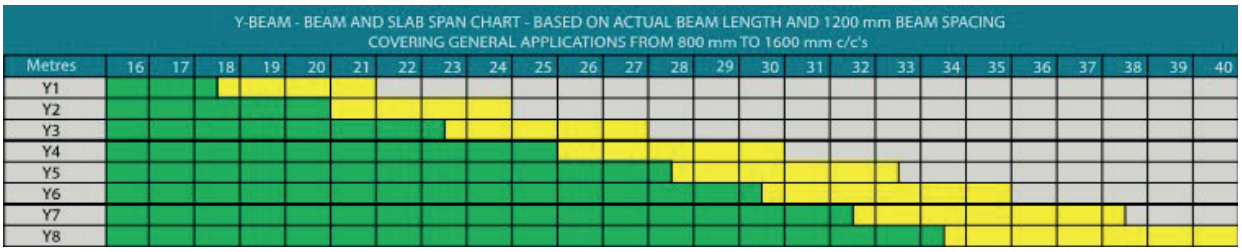
From table below use beam type:

Y5 H= 1100 mm

Beam max span: L 27 m

L adj = L(1.2/S)^0.5 26.45 m
Max c/c for beam 1.250 m





Y 5 beam self weight

Area = 450230 mm2 = 0.45 m2
Self weight = 11.26 kN/m

Floor DL = 12.24 kN/m
Floor LL = 6 kN/m

Beam reaction [SLS]

Permanent = 147 kN
Variable = 72 kN

HOLLOWCORE LOAD SPAN TABLES

150	150mm Deep Slab with 10 No. 9.3mm Diameter Strands												
	Clear Span (m)	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0
Maximum Load Capacity (kN/m^2)														
Without Structural Screed		71.0	47.5	33.0	23.8	17.9	13.8	10.8	8.6	6.9	5.6	4.5	3.6	2.9
With 75mm Structural Screed		125.0	91.6	62.5	44.0	30.0	24.0	18.7	13.6	10.3	7.6	5.5	3.8	2.4

Note: Self-weight of unit = 2.4kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)

200	200mm Deep Slab with 6 No. 9.3mm and 2 No. 12.5mm Diameter Strands												
	Clear Span (m)	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0
Maximum Load Capacity (kN/m^2)														
Without Structural Screed			35.4	26.5	20.4	16.5	13.2	10.8	8.8	7.3	6.0	5.0	4.2	3.4
With 75mm Structural Screed			50.0	37.5	29.0	22.9	18.4	14.9	12.1	9.7	7.4	5.7	4.2	3.0

Note: Self-weight of unit = 2.4kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)

250	250mm Deep Slab with 8 No. 12.5mm Diameter Strands												
	Clear Span (m)	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0
Maximum Load Capacity (kN/m^2)														
Without Structural Screed		26.6	25.0	20.8	16.6	14.1	11.6	10.0	8.3	7.5	6.3	5.4	4.8	
With 75mm Structural Screed		25.8	23.3	20.8	19.1	17.0	15.4	13.3	11.1	9.1	7.5	6.0	4.5	3.6

Note: Self-weight of unit = 3.6kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)

300	300mm Deep Slab with 8 No. 12.5mm Diameter Strands												
	Clear Span (m)	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0	11.5	12.0
Maximum Load Capacity (kN/m^2)														
Without Structural Screed		26.6	22.5	19.0	15.8	13.8	11.6	10.0	8.8	7.5	6.6	5.8	5.0	4.1
With 75mm Structural Screed							14.1	12.9	10.4	8.5	7.5	5.8	4.6	3.7

Note: Self-weight of unit = 4.1kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)

400	400mm Deep Slab with 12 No. 12.5mm Diameter Strands on Bottom, 2 No. 9.3mm Diameter Strands on Top												
	Clear Span (m)	10.0	10.5	11.0	1.5	12.0	12.5	13.0	13.5	14.0	14.5	15.0	15.5	16.0
Maximum Load Capacity (kN/m^2)														
Without Structural Screed		16.0	14.4	12.8	11.4	10.2	9.0	8.0	7.2	6.5	5.7	5.2	4.6	4.0
With 75mm Structural Screed		16.0	15.0	13.5	12.3	11.3	10.2	9.2	8.1	7.0	6.2	5.3	4.5	3.7

Note: Self-weight of unit = 4.68kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)

500	500mm Slab with 14 No. 12.5mm Diameter Strands on Bottom, 4 No. 9.3mm Diameter Strands on Top												
	Clear Span (m)	8.0	8.5	9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.5	13	13.5	14
Maximum Load Capacity (kN/m^2)														
Without Structural Screed		30.0	27.0	24.5	22.0	20.0	18.5	17.0	15.5	14.0	13.0	12.0	11.0	10.0
With 75mm Structural Screed		32.5	30.0	28.0	26.0	25.0	23.0	21.5	19.5	18.0	16.5	15.0	14	12.0
500	500mm Slab with 14 No. 12.5mm Diameter Strands on Bottom, 4 No. 9.3mm Diameter Strands on Top												
	Clear Span	14.5	15.0	15.5	16.0	16.5	17.0	17.5	18.0				
Maximum Load Capacity (kN/m^2)													
Without Structural Screed		9.0	8.0	7.0	6.5	5.5	5.0	4.5	4.0				
With 75mm Structural Screed		11.0	9.5	8.5	7.0	6.0	5.5	4.0	2.5				

Note: Self-weight of unit = 5.38kN/m^2 (already deducted from table)
Structural screed = 1.8kN/m^2 (already deducted from table)
Based on 150mm Bearing

From table above, the precast 450 would be able to support the total load of 11kN

Eckersley O'Callaghan

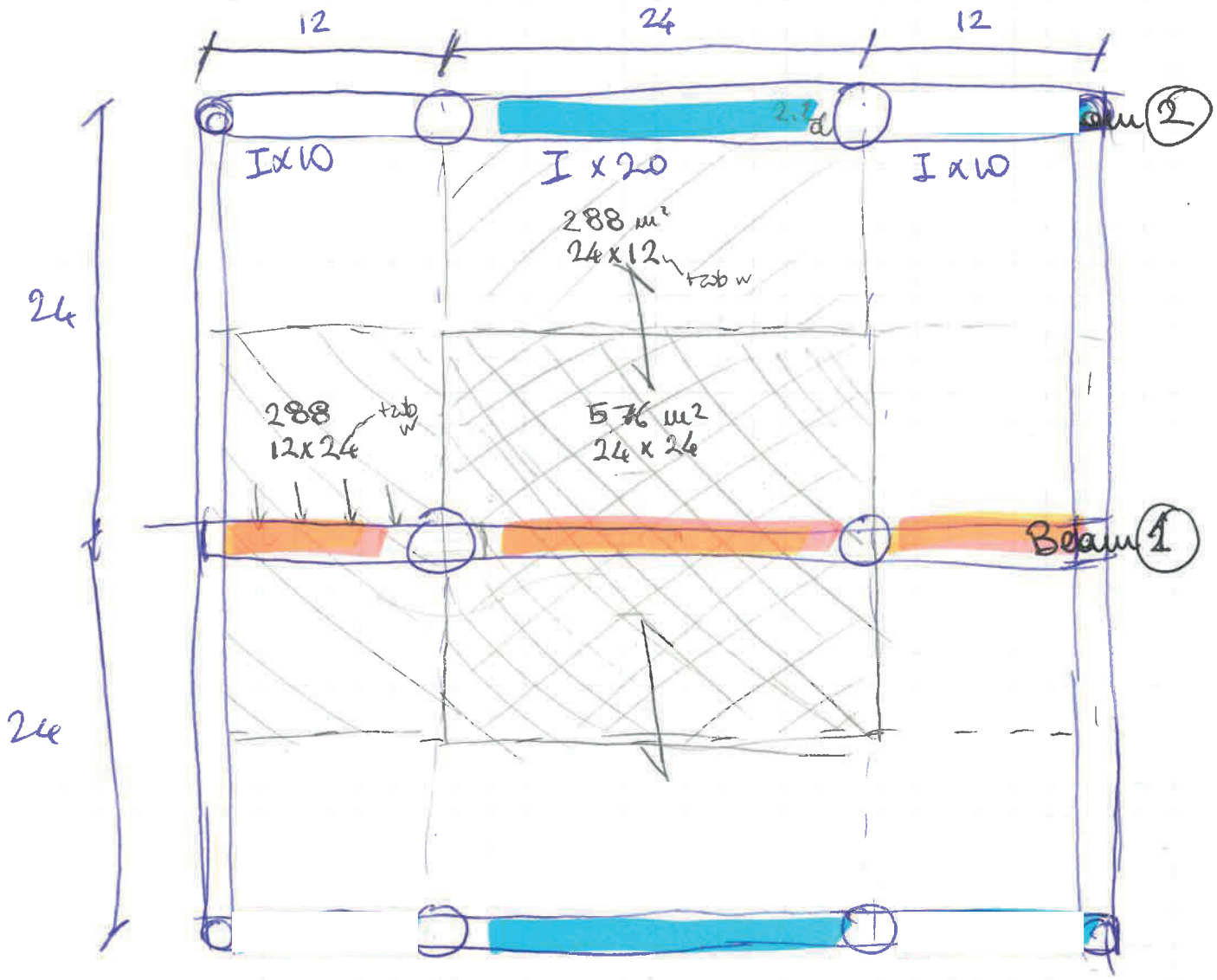
Project Name: Baugher

Section

Project No. | Page | Date

eocengineers.com | Revision | By | Checked

1st Floor



SDL = 4.2 KPa

LL = 5 KPa

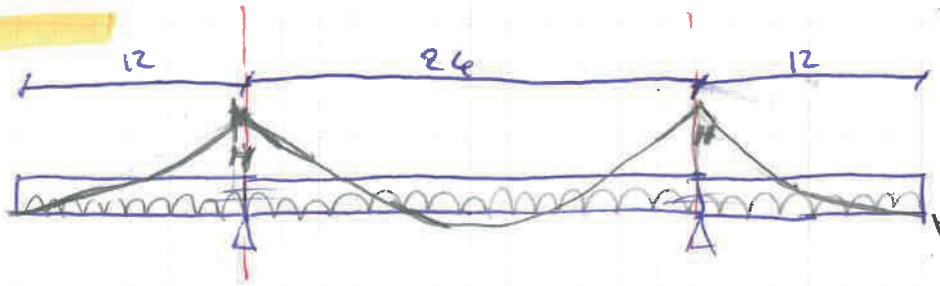
Dead: Beams : 11.3 KN/m spaced @ 1.2 m. = 9.5 KPa

+ 200 mm cover = 9.5 + 5 = 14.5 KPa

tot dead = DL + SDL = 18.7 KPa × 1.35 = 25.3 KPa

LL = 5 × 1.5 = 7.5 KPa

1



3m d

24m tab width

cantilevering

Continuous across supports

Post tensioned

Slab contributes to support

SW: assume 1.5x 3d beam

SW = 3x1.5x 25 = 113 KN/m

ULS = 152

ULS loads

DL = 304 + 152 (476) tot = 636 KN/m

LL = 180 KN/m

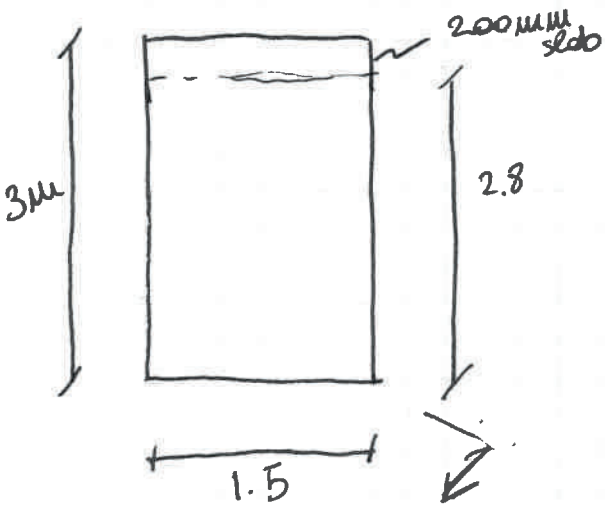
Normal RC doesn't work → PT

"Deconstruct Beam"

Beam continuous & hyperstatic

for now examine cantilever & simply supported

Assumed beam size:

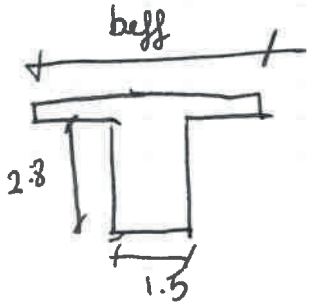


SW = 25 x 2.8 x 1.5 = ~~105~~ 152 KN/m

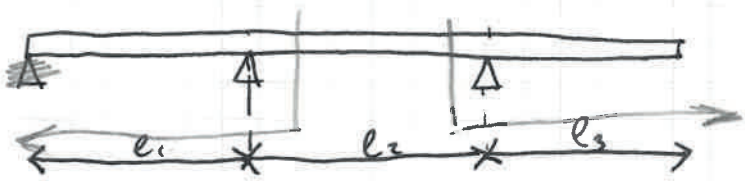
DL = 18.7 x 24 = 449 KN/m

total = ~~554~~ 601 KN/m.

Slab contributes to load-bearing

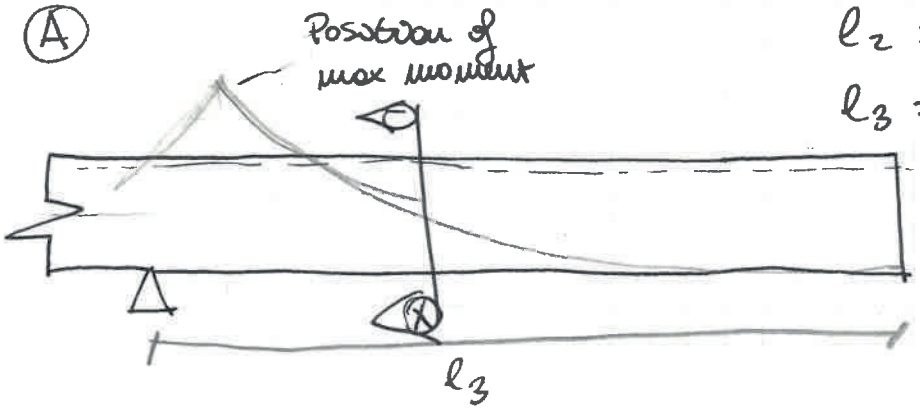


(B) $l_0 = 0.7 l_2$ (A) Cantilever



From EC2

(A)

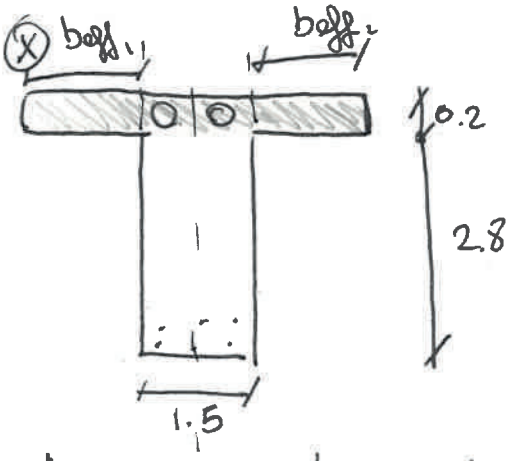


$l_2 = 24\,m$

$l_3 = 12\,m$

UDL = 554 kN/m

EC2 (5.7)



$b_1 = b_2 = 24/2$

Max moment = hogging

→ get PT in slab

$b_{eff,1} = 0.2 \times 24 + 0.1 l_0 = b_{eff,2}$

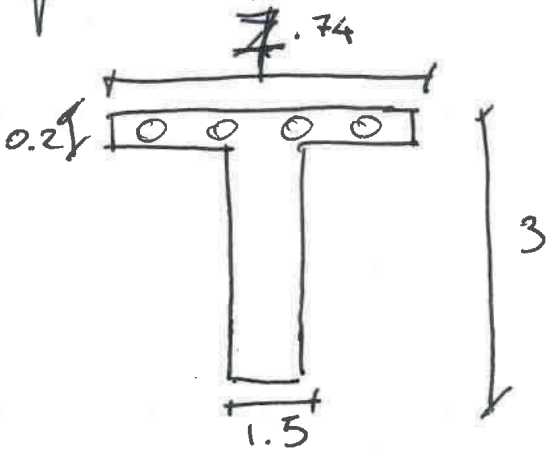
$= 3.96 > 0.2 l_0$

$l_0 = 0.15 \times 24 + 12 = 15.6\,m$

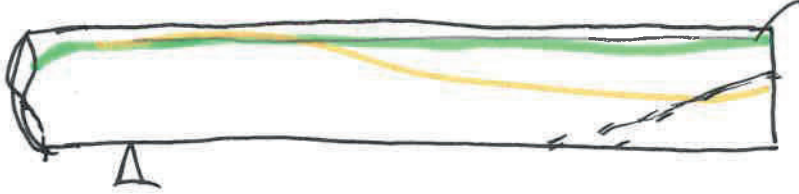
$\therefore c_{eff,i} = 0.2 l_0 = 3.12\,m$

$B_{eff} = 1.5 + 2 \times 3.12 = 7.74\,m$

Cant beam ⇒



PT design



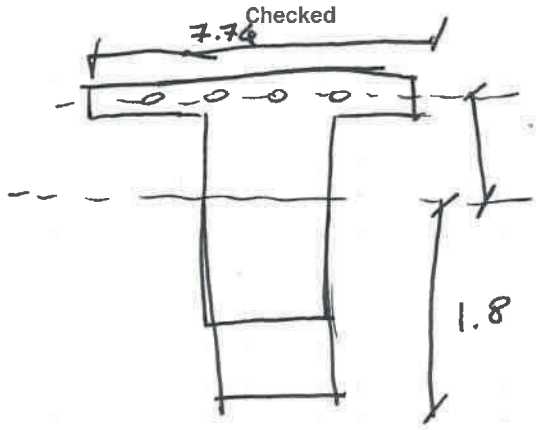
Tendon position should start @ ℄ but beam should be tapered

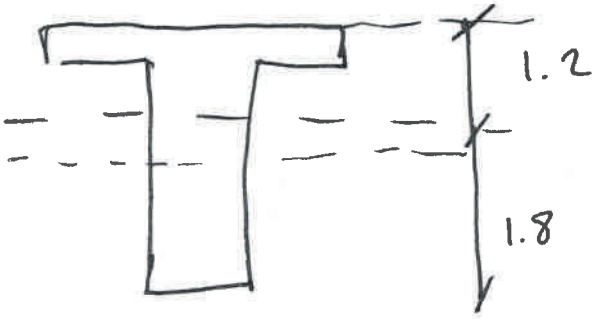
PT to sit on slab

Find tendon to fit on 200 mm

$w_p = 0.8 \times 554 = 443.2$ kN/m

Neutral axis





From top of slab:

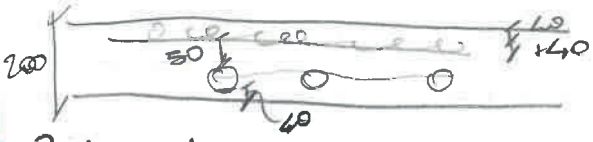
40 mm → cover

20+20 → 2 way reinf. ~~2 way slab~~

= 80

- spacing - 40 other side

= 120 - spacing = 170



ϕ30 PT bars ideally.

$f = 1.2 - 0.17 - 0.03F$

$= 1$

$P_o = \frac{0.8 M_{od}}{0.85 \times f}$

~~$P_o = \frac{w_p L^2}{8f} = 1383$ kN~~

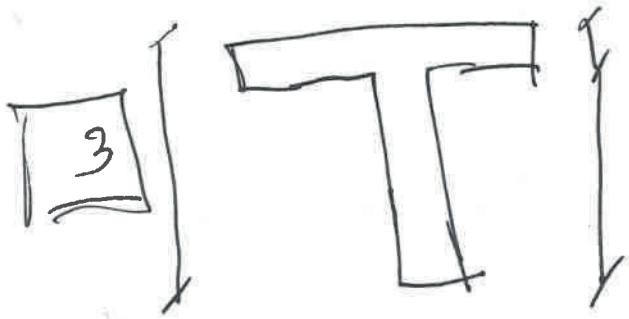
? $L_1 = L_0$ or $L = 12$

Needs 18 strands

Space needed for 18 strands ϕ 60 mm

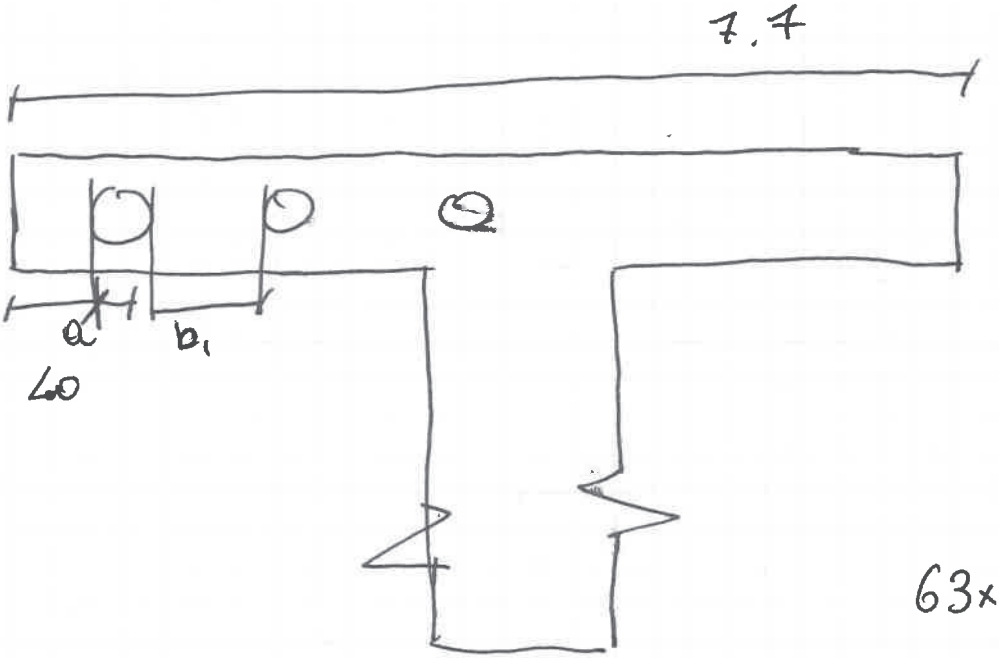
↓

Needs deeper slab ∴ change overall dimensions



250 - 300

2.75 - 2.7



63 x 8 = 504

N tendons needed = 16065/2204 = 7.3 ≈ 10

Spacing needed = $\Sigma d\phi + \Sigma$ spacing + 2 x cover

Should be ok

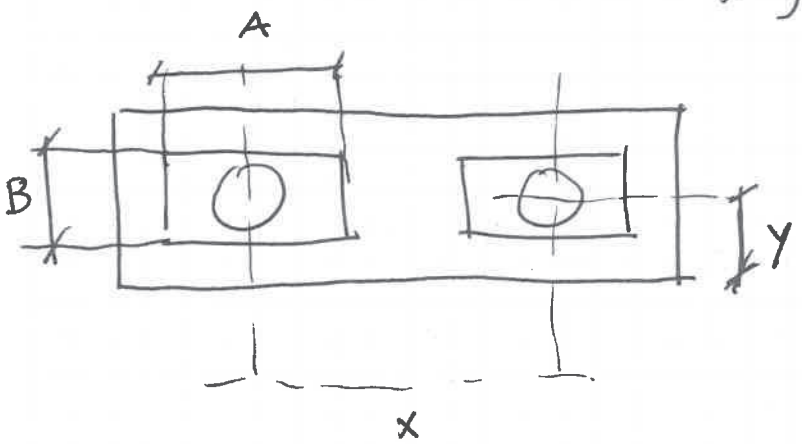
$b_1 = 0.7 \times 63 = 44.1$ tot space = 1.1 m ok.

Dimension of duct.

SW = 25 x (0.25 x 7.7 + 2.75 x 1.5) = 152 kN/m

~~Check ULS~~

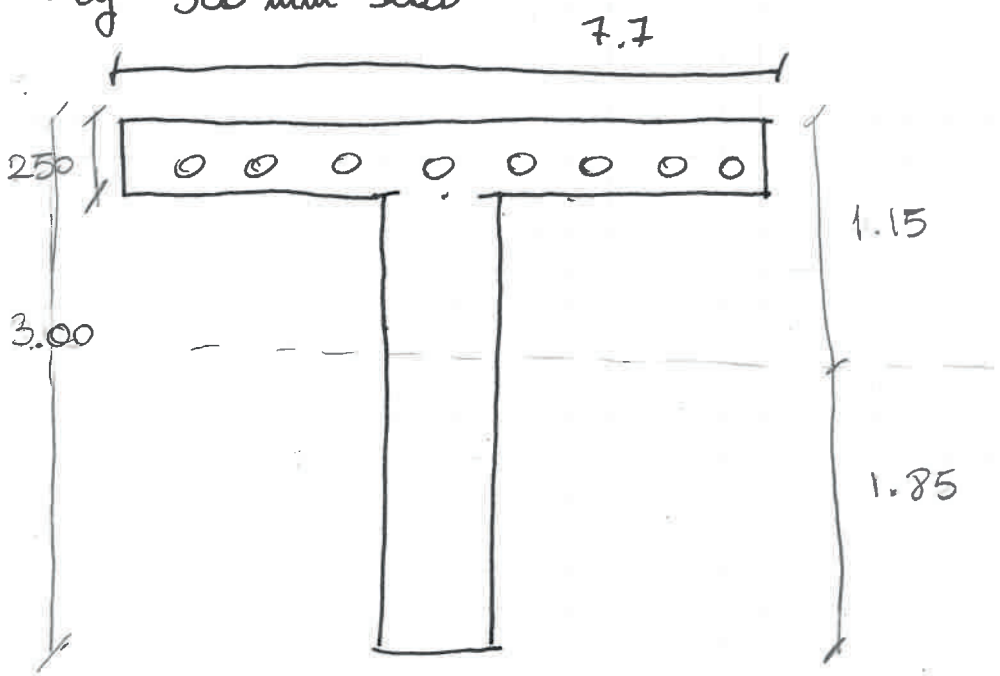
~~$w = 554.8 - 443.2 = 110.8 + \} \times 1.35 =$~~



$x \geq A + 30$ mm

$y > B + 30$ mm

Try 300 mm slab



$u_p = 443.2$ KN/m

$L = l_0$ or L?

assume $L = \cancel{12}$ m 15.6

40 mm top cover

+ 40 mm reinforcement (passive)

+ 50 mm between passive rebar & PT

+ 40 mm bottom cover

= 170 mm → Should fit a $\phi 80$ tendon in 250 mm slab

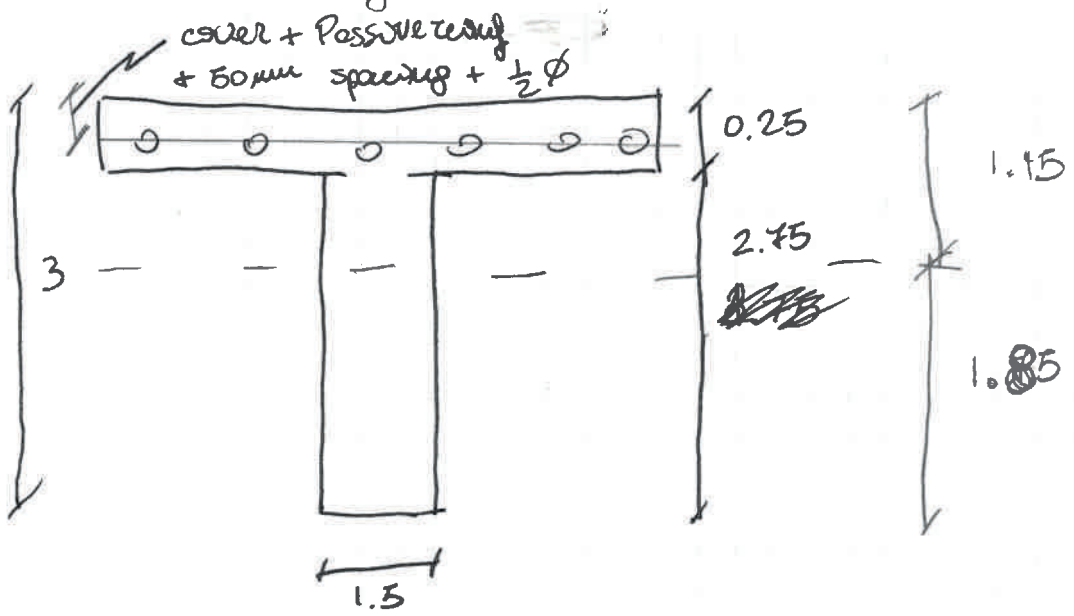
Try tendon diam = $\phi 64$

$f = 1.15 - (40 + 40 + 50 + 32) = 0.988$

~~$P_\alpha = 13656$ (18505) $L = 15.6$ $P_o = 16065$ KN~~

~~$P_\alpha = 8075$ (8680) $L = 12$ $P_o = 9500$ KN (9989 KN)~~

Revised loading



Dead load: Beams = 9.5 KPa

+ 250 mm cover = 6.25 ∴ 15.75 KPa

× 24 = 378 KN/m

SDL ~~7~~ 4.2 × 24 = 100.8 = 479 KN/m

SW = 2.75 × 1.5 × 25 = 104 KN/m

Total DL = 583 KN/m

u_p = 0.8 TotDL = 466 KN/m

$M_{Edp} = \frac{466 \times 12^2}{2} = 33552$

g = 1.15 − (60 + 40 + 50 + 32) = 0.988

$P_o = \frac{33552}{0.85 \times 0.988} = 39952$

L_o = ~~15.6 m~~ Use 12 m.

$P_\infty = \frac{466 \times 15.6^2}{8 \times 0.988} = 14368$ KN

P_o = ~~16880~~ KN ~~aim to have 7 tends.~~

= ~~2412~~ KN } ~~Need 10 strands~~

~~13 strands pr EN 10138~~

~~$P_\infty = \frac{3627}{} \times 0.85$~~

~~= 3083~~

13 ~~of pr EN 10138~~

~~Guaranteed breaking load~~

~~15.7 → 279 → 9 strands~~

Assume ~~5B15~~ + ~~4B15~~ → ~~33 9~~ = A

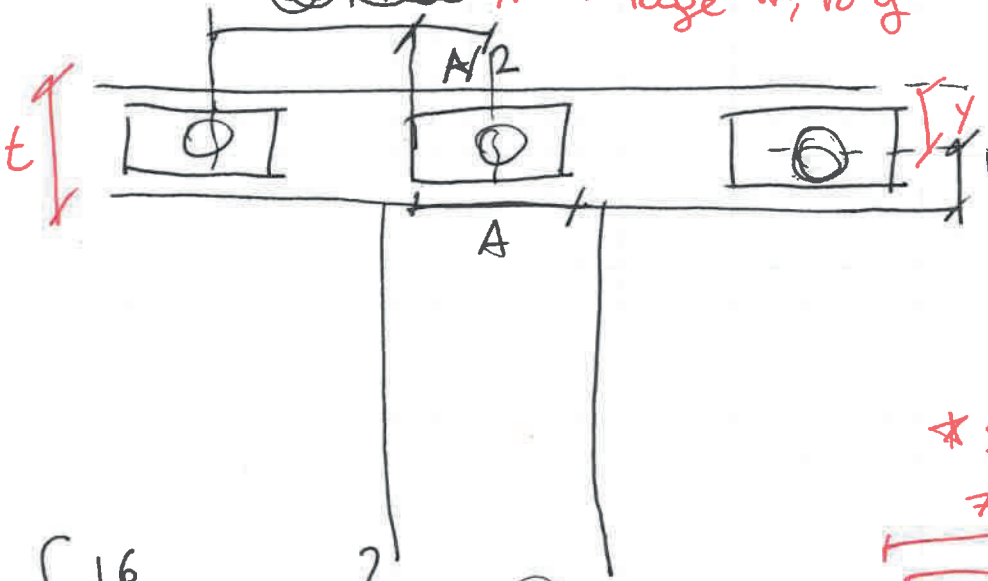
369 ~~130~~ = ~~B~~

Bonded
↳ Grout after PT

~~(33 9+030)~~ X → Page 17, 18 of

A = 250 X_1 = 0.5 × (B+30) + 40

B = 200 = 155 *



~~130+30~~

~~= 160~~

y' = ~~110~~ ~~= 220~~ ~~× 2 = 220~~

* slab depth = 600 mm

7.76 m

{16 15PT | 15} (5) 15

~~P_o = 9989 kN~~ 39952

Assume 4 S15 Strand 15 = 15.7 mm

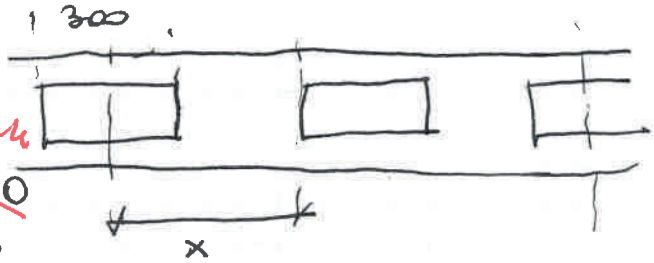
* pr EN10138

* $FP_{0,1}$ - Yield load = 246 kN

Page 20 Freyssinet catalogue

Max spacing: ~~475~~ mm A+30 = 250 +3 = 280

∴ (7700 − ~~475~~ 280) / ~~475~~ 280 = ~~15~~ 27 (can fit ~~15~~ 27 anchors)



If ~~4~~ 13 S15 → ~~4~~ 13 × 246 = ~~984~~ 3198 kN

× ~~15~~ 13 = ~~14760~~ 41574

>> P_o

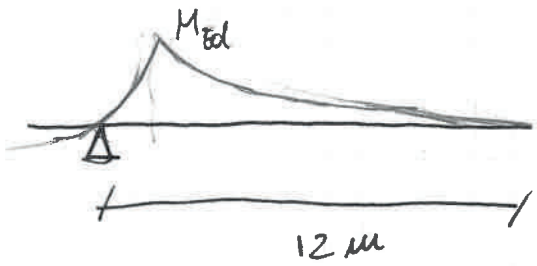
~~If~~ Could use even 11 tendons

PT OK ✓

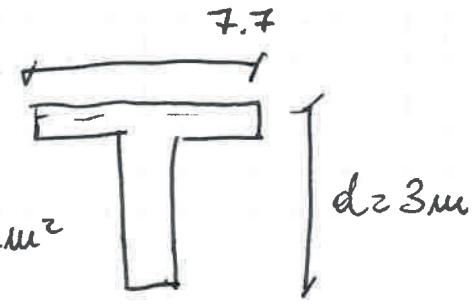
Check ULS

$w = w - w_p = 583 - 466 = 117$ kN/m (SLS)

ULS : DL = 158 kN/m
LL = 7.5 × 24 × 1.5 = 270 kN/m } tot = 428

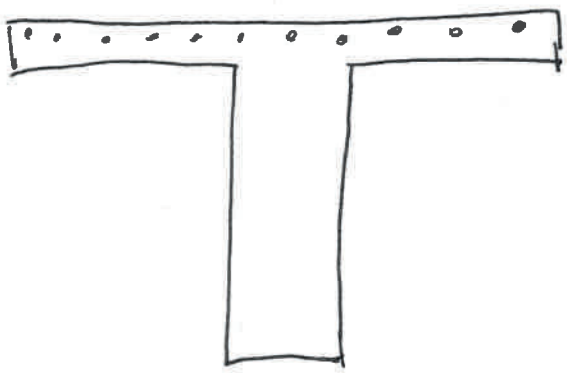


$M_{Ed} = \frac{1}{2} w L^2 = 30\,816$ kNm



$A_s = \frac{30\,816 \times 10^3}{0.9 \times 3000 \times 235} = 26\,238$ mm²

Assume $\phi = 20$ (→ 314 mm²) ⇒ Need 84 bars?



84/7 = 12 → Need 12 bars per meter → 12.5

80 mm spacing between bars

3927 × 7 = 27489 mm²
> 26 238

ULS OK ✓

Deflection: DL (excluding w_p) = 117 kN/m LL = 180 kN/m

$\delta = \frac{wL^4}{8EI} = 35.6$ mm L/500 = 24 mm

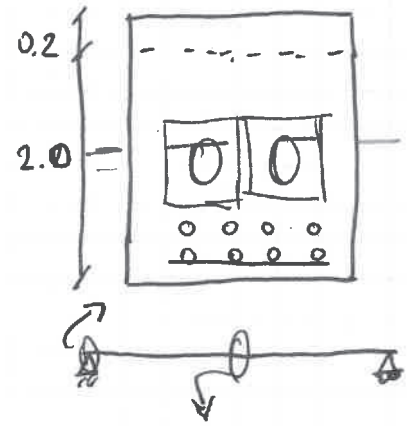
LL only: 22 mm L/360 = 33.3 mm

Beam 24 m span, 12 m trib area LL = 5 KPa

DL = 18.7 KPa

Passive reinforcement assumed

2 layers of ø20 + longitudinal



$DL = 18.7 \times 12 + 2 \times 1.8 \times 25 = 315$ KN/m

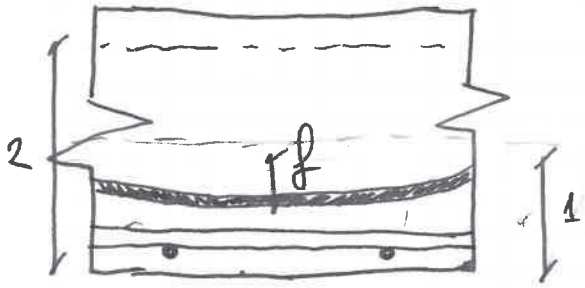
Cover = 40 mm

$f = 1.1000 - (40 + 16 + 2 \times 20 + 60 + 50 + 160/2)$

$= 11000 - 286 \sim 8\frac{1}{4}$ mm

$u_p = DL \times 0.8 = 252$ KN/m

$P_\infty = \frac{u_p L^2}{8 \times 0.814} = 22290$ KN



$P_o = P_\infty / 0.85 = 26224$ KN

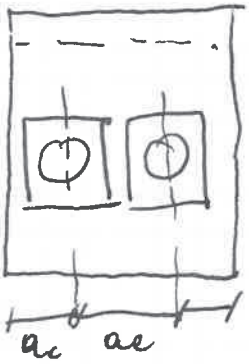
∴ Max per tendon 13 112 KN

PT works, Strand size 15.7 mm

Nom area 150 mm² → 5806, 48 Strands

Page 26 table : Tendon Properties

Spacing



Page 24

∴ 5506 → $a_c = 445$ mm

$a_e = 905$ mm

tot width $= 2a_c + a_e = 1795$ mm

Technical Data of anchorages BBR VT CONA CMI SP

ULS Design

Reduced DL

$DL_{New} = 315 - 252 = 63$ KN/m

ULS = 85.1 KN/m

LL = 90 KN/m ULS

tot = 175.1 KN/m

Assumption for simply supported

∴ conservative

$M_{Ed} = \frac{wL^2}{8} = 12\,608$ KNm

$A_s = \frac{12\,608 \times 10^6}{0.9 \times 2000 \times 435} = 16\,103$ mm² not considering slab

14 639 mm² considering slab action

Need 47 ϕ20

23 bars per row

spacing : 50 mm

23 bars ϕ20 + 22 50 mm spacing = 1560 mm

+ 80 mm cover

+ 16 shear link

= 1600 mm

Beam OK ✓

Eckersley O'Callaghan

eocengineers.com

Project Name

Section

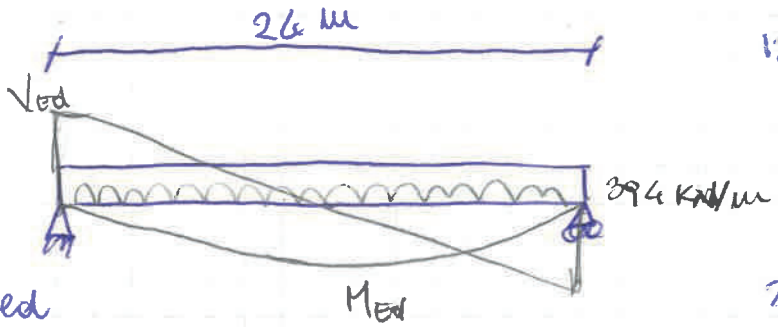
Project No. | Page | Date

Revision | By | Checked

SS

2

2.2 m d
12 m tub width



Simply supported

Post tensioned

? depth
2.2 x 1.5
SW = 83 KN/m

ULS loads

DL = 25.3 x 12 = 303.6 KN/m → + SW = 627 KN/m

LL = 7.5 x 12 = 90 KN/m

tot = 394 KN/m
+ SW = 517

$M_{Ed} = \frac{wL^2}{8} = 683 \times 26^2 = 34776$ KNm

$A_s = \frac{34776 \times 10^3}{0.9 \times 2.2 \times 10^3 \times 435} = 40\,378$ mm²

$V_{Ed} = \frac{wL}{2} = 5796$ KN

[Unfactored DL = 18.7 x 12 + 83 = 308 KN/m] 225

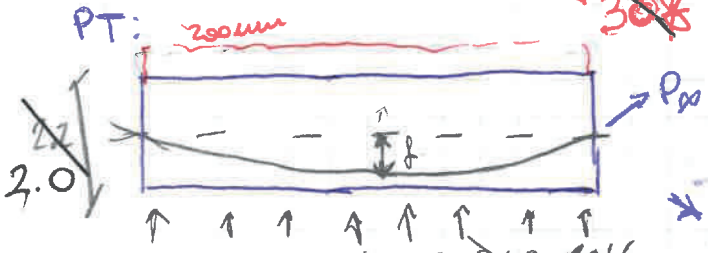
Dead load ULS only

SS Values DL = 393 KN/m → 308

PT

Consider P as 80% of DL
UNFACTORED

PT:



$up \sim \frac{8 P_\infty f}{L^2} \rightarrow P_\infty = \frac{up L^2}{8f}$

$P_0 = P_\infty / 0.85$

[up = 0.8 x 308 = 266 KN/m] assuming f = 250 mm

P_∞ = ~~26683~~ 20823 KN

P_0 = ~~31392~~ 24506 KN

19680

23153

assume breakage

For loading purposes:

1.2 wide x 2.2 deep

1.5 x 2.2 x 25 = 83 KN/m

x 1.35 = 124 KN/m

ULS self weight for 1.5 x 2.2 beam

$f = \frac{2.2}{2} - 40 - 50 - \frac{185}{2} - 50$

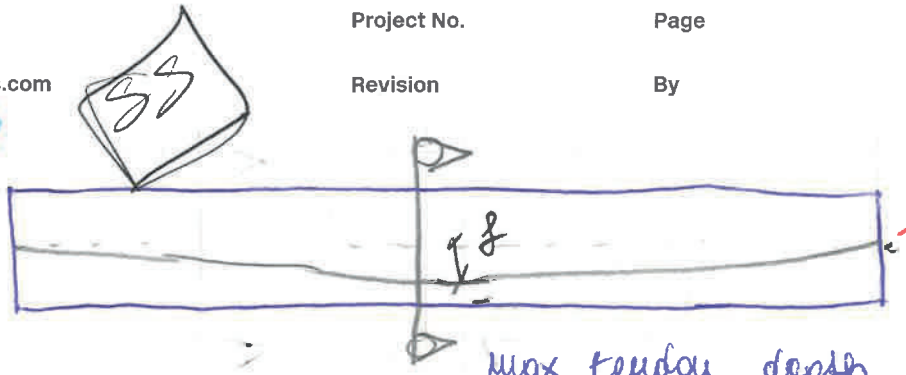
cover; 2x16 or 20 reinf.; max diameter; space between tendons & reinf

→ f = 0.85 m (approx)

Dod ~~not~~ consider slab depth → conservative

Yes consider slab depth → f = 0.9 !

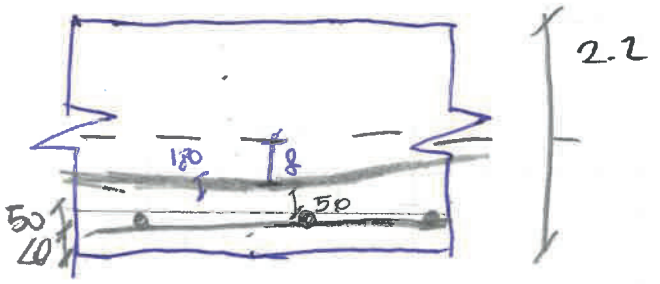
Concrete grade C60/50



P/tendon = 12~~25~~ KN

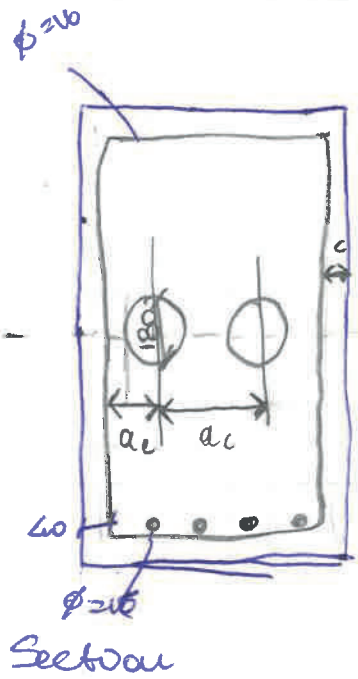
max tendon depth

or 11577 KN



Tendon chosen 5506 or 4206 *

d_{max} = 165 mm



Max beam width = 2 m

If that choice → tot width required → w = 1795 mm

↓

∴ Must consider 200 mm slab

P_o = 11577 KN Per tendon → 13 MN

Table: Multi strand Post tensioning Page 24 BBR VT CONA CMSP 5506

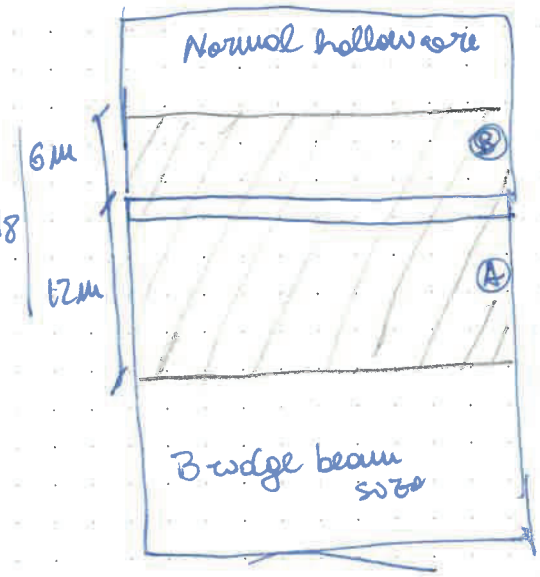
Max centre spacing

a_e = 665 mm a_c = 905 mm

tot w = 1800 mm

! Primary (Blue) Beam

Span: 12 m



1.2 x 15

SW = 45 KN/m

Ⓐ tributary = 12 m

Load: DL = 25.3 KPa + SW =
LL = 7.5 KPa

Ⓑ 400 + 75 mm Plank {8 KPa}
+ SDL (4.2 KPa)

LL = 90 KN/m
SDL = 76 KN/m

$DL_A = 14.5 \times 12 = 174$ KN/m

$DL_B = (4.32 + 0.075 \times 25) \times 6 = 6.2 \times 6 = 38$ KN/m

tot DL = 212 + SW = 257 KN/m + SDL = 333 KN
x 1.35
= 450

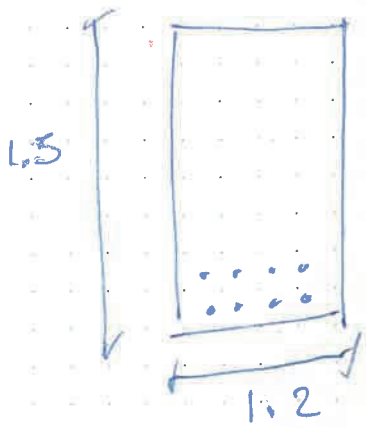
LL = 90 KN/m = 135

tot w = 585 KN/m

$M = \frac{wL^2}{8} = \cancel{10330}$ 10530 KN/m

$A_s = \frac{M_{ed}}{0.9d\ 435} = \cancel{20016}$ 17931 mm²

36 ~~20~~ ø25



2 Layers ~~of~~ 20 bars ø25

Spacing: 50 mm 3 Layers 15 ø25

Check Standard Primaries GL5 - GLB

Tributary width: 12 m

DL: 400 mm hollowcore + 50 mm topping
4.68 + 1.25 = 6 KPa → 71.2

SDL: 4.2 KPa → 50.4 tot DL = 122 KN/m

LL: 5 KPa → 60

Assume simply supported

122 × 1.35
60 × 1.5

$\frac{wL^2}{8}$

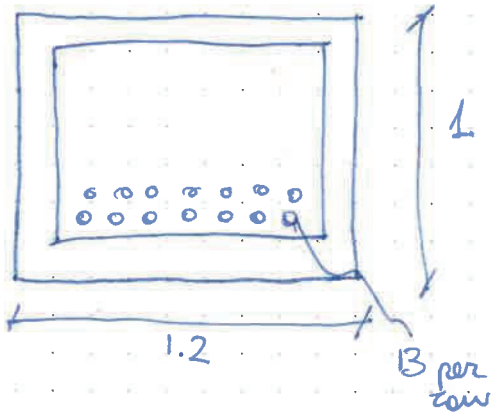
$M_{Ed} = \frac{254.7 \times 12^2}{8} = 4585$ KNm

$A_s = \frac{4585 \times 10^6}{0.9 \times 1000 \times 435} = 11711$ mm²

2 layers of 13 ϕ25s

Spacing ≈ 60 mm

tot space required
= 1125. OK✓



With Space Capacity.

Appendice E: Calcoli del piano intermedio

Mezzanine floor loads

				kN/m2
Finishes	(40mm stone + 20mm bedding)			1.2
Screed	75 mm	Screed density assumed 20kN/m3		1.5
Insitu RC topping	75 mm	Concrete density assumed 25kN/m3		1.875
150thk composite slab		Concrete volume = 0.14m3/m		3.5
Services				0.5
Ceiling				1
			DL=	9.6
Variable load			LL=	5.0

Beam Load

Tryburary width		3.125 m
Total DL	=	29.9
Total LL	=	15.6

SCI	Tata Steel		v9.0.34.24751
Job Reference:	Florence airport	Date:	27/11/2021
Deck Reference:	CF80/1.2_350	Time:	9:38:36
Company Name:		Job No:	
Client Name:		Calcs By:	
Checked By:		File Name:	Mezzanine floor

Full Output

Note: Section Designed to Eurocodes, United Kingdom National Annex

Construction Stage:	PASS	**Max Unity Factor:**	0.43
Normal Stage:	PASS	**Max Unity Factor:**	0.79
Fire Condition:	PASS	**Max Unity Factor:**	0.90
Serviceability:	SATISFACTORY	**Max Unity Factor:**	0.57

***** Section Adequate *****

Floor Plan Data (unpropped composite construction with ComFlor 80/1.2/S350 decking)

Beam centres	3.20 m	Profile span type	Single
Beam or wall width	150 mm	Propping	None
		Concrete span type	End

Profile Data (ComFlor 80/1.2/S350 decking.)

Depth	80 mm	Pitch of deck ribs	300 mm
Trough width	120 mm	Crest width	150.0 mm
Nominal sheet thickness	1.20 mm	Design sheet thickness	1.16 mm
Deck weight	0.15 kN/m²	Yield strength	350 N/mm²

Concrete Slab (Normal Weight Concrete ; Grade C30/37; Mesh : A393)

Overall slab depth	150 mm		
Concrete characteristic strength	30 N/mm²	Concrete wet density	2550 kg/m³
Modular ratio	10	Concrete dry density	2450 kg/m³
Bar reinforcement	None		
Mesh reinforcement :			
Mesh	A393	Yield strength	500 N/mm²
Cover to Mesh	30 mm	Mesh Layers	Single
Account for End Anchorage	No	Shear connectors per rib	N/A
Diameter of Shear Connectors	N/A		
Screed depth	75 mm	Screed density	2000 kg/m³

Section Properties

*** Note - 1: All values of inertia are expressed in steel units
*** Note - 2: Average inertia is used for deflection calculations for the composite stage
*** Note - 3: Cracked dynamic inertia is used for natural frequency calculations

Deck Profile

Sagging Inertia, Iy	213.000 cm4/m	Area of profile (Net), Ap	1864 mm²/m
Hogging Inertia, Iy	203.000 cm4/m	Effective area of profile	1678.00 mm²/m

Composite

Inertia, Iy - Uncracked	2296 cm4/m	Inertia, Iy - Cracked	1186 cm4/m
Average inertia	1741 cm4/m	Cracked inertia (dynamic)	1335 cm4/m
Shear bond coefficients - Tau	0.21		
Concrete volume	0.104 m³/m/m		

Loads Acting on Slab (Actions)

*** Note: Slab subjected to uniformly distributed loads (UDL) ONLY

Imposed (occupancy)	5.00 kN/m²	Partitions	1.00 kN/m²
Ceilings and services	1.50 kN/m²	Finishes	3.00 kN/m²
Self weight of concrete slab (wet)	2.61 kN/m²	Self weight of decking	0.15 kN/m²
Self weight of concrete slab (dry)	2.51 kN/m²	Self weight of screeds	1.47 kN/m²
Construction load	1.50 kN/m²	(applied over middle 3m length, 0.75kN/m² elsewhere)	

Line Loads Perpendicular to Deck Span (Actions)
None

Line Loads Parallel to Deck Span (Actions)
None

Fire Data

Design method	Mesh + Deck Method	Fire resistance period	60 mins
Non-permanent imposed loads	N/A	It is assumed the fire protection will be provided by other means to achieve the required protection.	

Partial Safety Factors

Actions		Materials	
Permanent, gamma G	1.35	Structural steel - elastic, gamma M0	1.00
Permanent - accidental, gamma GA	N/A	Structural steel - buckling, gamma M1	1.00
Variable, gamma Q	1.50	Concrete, gamma C	1.50
Combination factor - Fire, psi 1	0.70	Reinforcement, gamma S	1.15
Combination factor, psi 0	0.70	Combination factor, psi 2	0.60

Construction Stage

Loadings	@ SLS (kN/m²)	@ ULS (kN/m²)
Self weight of decking	0.15	0.20
Self weight of concrete slab (wet)	2.61	3.91
Reinforcement	0.07	0.09
Total weight of slab	2.83	4.21
Construction live load	0.75	1.13
Construction live load patch	0.75	1.13

Effective Span of Deck

Effective span Le, is the smaller of

1) c/c of supports	= 3.20 m
2) clear span + deck depth	= 3.05 + 80.0 / 1000
	= 3.13 m
Therefore Le	= 3.13 m

Shear Resistance Check (BS EN 1993-1-3 Clause 6.1.5 and 6.1.7.3)

Applied shear	10.11 kN/m			
Web shear resistance, Pv	106.69 kN/m	Unity Factor	0.09	PASS
Applied reaction	10.11 kN/m			
Web crushing resistance, Pw	40.90 kN/m	Unity Factor	0.25	PASS

Bending Resistance Check (BS EN 1993-1-3 Clause 6.1.4.1)

Sagging

Max applied moment	7.91 kNm/m			
Moment resistance	18.49 kNm/m	Unity Factor	0.43	PASS

Combined Effects

Bending and Web Crushing (BS EN 1993-1-3 Clause 6.1.11)

Design unity factor is the worst case of

1. Maximum hogging:

(0.00 / 40.90 + 0.00 / 14.59) / 1.25 = 0.00

2. Maximum reaction:

(0.00 / 40.90 + 0.00 / 14.59) / 1.25 = 0.00

Design unity factor 0.00 PASS

Bending and Shear (BS EN 1993-1-3 Clause 6.1.10)

*** Note: Low shear - This check is not required

Support Interaction Check at Serviceability Limit State (BS EN 1993-1-3 Clause 7.2)

Deflection

Allowable deflection is the lesser of

1) Effective span / deflection limit without ponding	17.39 mm
2) Deflection limit without ponding, absolute maximum value	20.00 mm
3) Slab depth / 10	15.00 mm

Max self weight deflection = 7.90mm <= 15.00mm SATISFACTORY

Normal Stage

Span

The effective composite span is 3.15 m

Loadings	@ SLS (kN/m²)	@ ULS (kN/m²)
Dead (Profile, concrete, reinforcement)	2.73	3.68
Imposed	6.00	9.00
Superimp (Ceiling, services, screed, finishes)	5.97	8.06
Total	14.70	20.74

All line and point described above in 'Loading Details' are applied at the Normal stage

Shear Resistance Check

Vertical Shear (BS EN 1992-1-1 Clause 6.2.2)

Maximum applied shear 31.33 kN/m

Shear resistance of end diaphragm (ComFlor 225 only) 0.00 kN/m ***test value

Vertical shear resistance is the greater of:

1. (0.54 * 400.00 * 104.99) / 1000 + 0.00
2. (0.12 * 2.00 * (100 * 0.02 * 30.00)^1/3) * 400.00 * 104.99) / 1000 + 0.00

= 39.46 kN/m

Unity Factor = 31.33/39.46 = 0.79 < 1 PASS

Punching Shear (BS EN 1994-1-1 Clause 9.7.6)

N/A - no concentrated loads have been applied

Bending Resistance Check (BS EN 1994-1-1 Clause 9.7.2)

Applied bending moment	22.74 kNm/m
Depth of concrete stress block	13.45 mm
Lever arm	94.88 mm
Compression in concrete	228.58 kN/m
Moment Resistance	35.80 kNm/m

Unity Factor = 22.74/35.80 = 0.64 < 1 PASS

Fire Resistance

Effective span in fire	3.05 m
Fire total UDL	13.06 kN/m²
Fire free moment	15.18 kNm/m
Moment resistance	12.69 kNm/m
Total moment resistance	16.92 kNm/m
Unity Factor	0.90

PASS

Deflection

Properties

Modular ratio	10.00
Uncracked section inertia	22963160.00 mm4
Cracked section inertia	11858250.00 mm4

Deflection Checks

Imposed load deflection	2.10 mm	
Allowable deflection (20 mm max)	20.00 mm	SATISFACTORY
Total deflection	4.19 mm	
Allowable deflection	12.62 mm	PASS

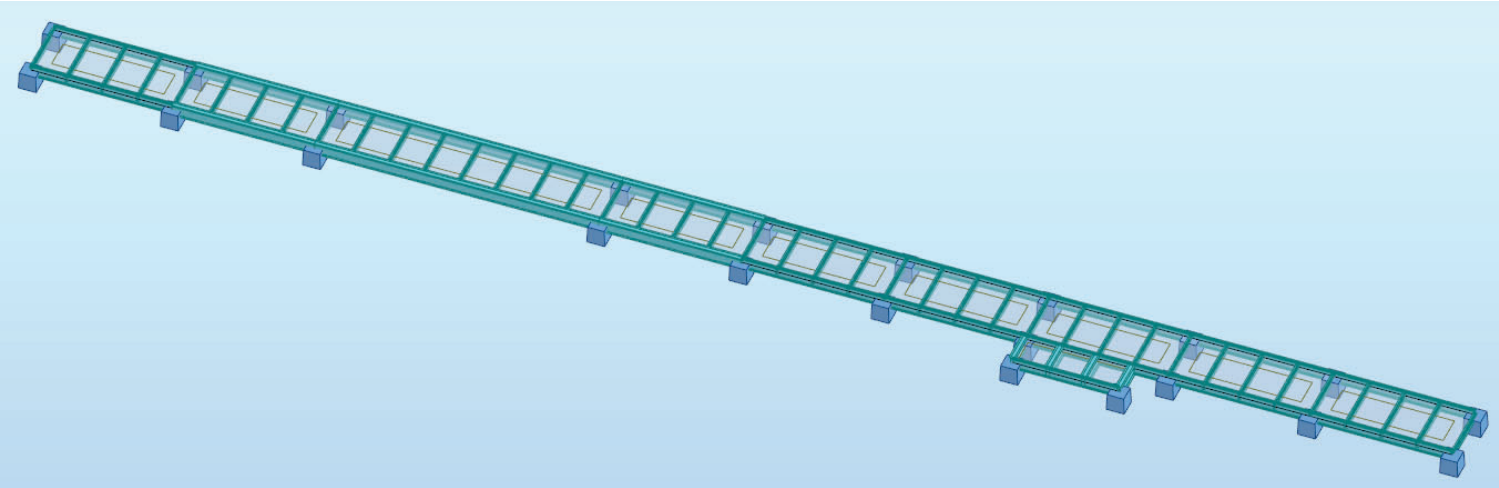
Dynamic Sensitivity

Dynamic inertia (cracked section)	1335.28 cm4
Maximum deflection	4.24 mm
Frequency	8.74 Hz
Unity Factor = 5.00/8.74 = 0.57 < 1	PASS

MEZZANINE FLOOR

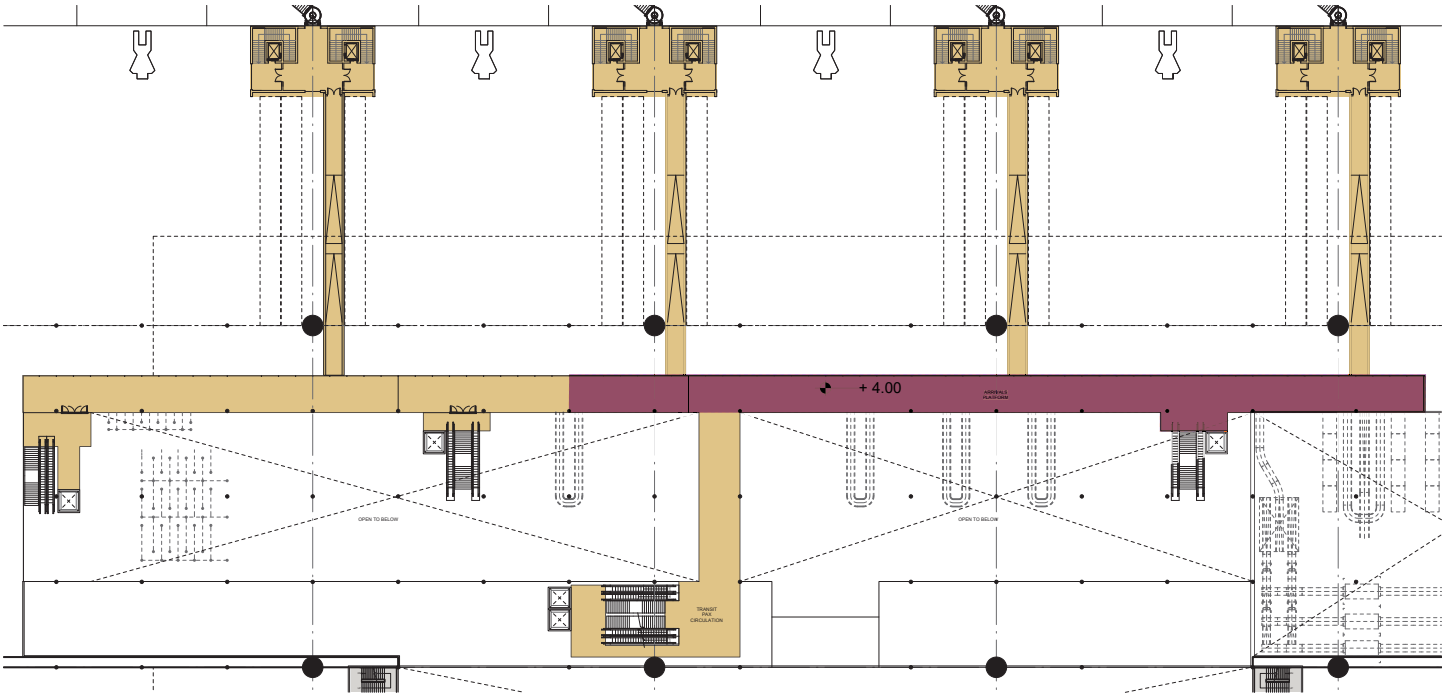
Part of the floor has been modeled in the FE model to understand the behavior of the composite floor.
For the purpose of vibration checks, the supports have been considered fixed, as under the excitation force, the rigidity of the connections can be considered as fixed connections.

The image below shows the part of the floor modeled and includes the 25m long floor without any intermediary supports.

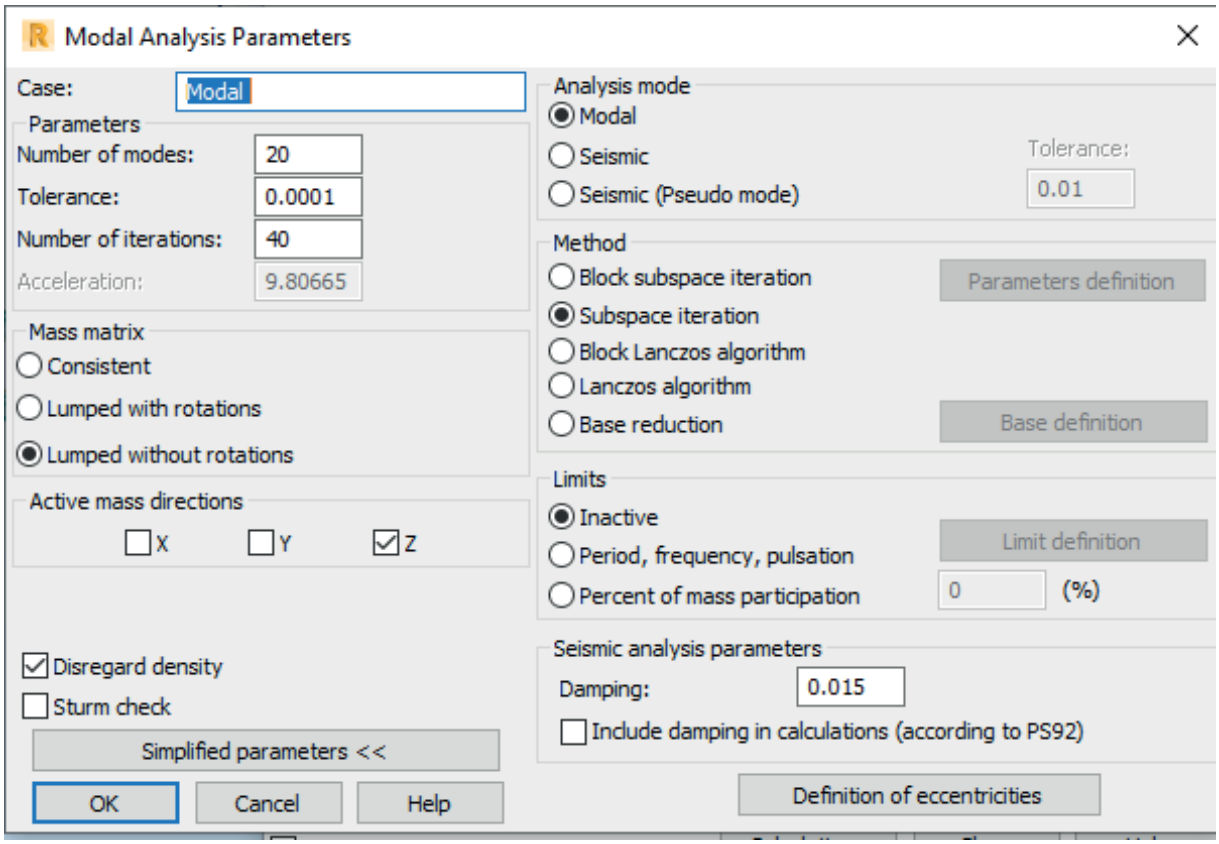


Part of the floor has been modeled in the FE model to understand the behavior of the composite floor.
For the purpose of vibration checks, the supports have been considered fixed, as under the excitation force, the rigidity of the connections can be considered as fixed connections.

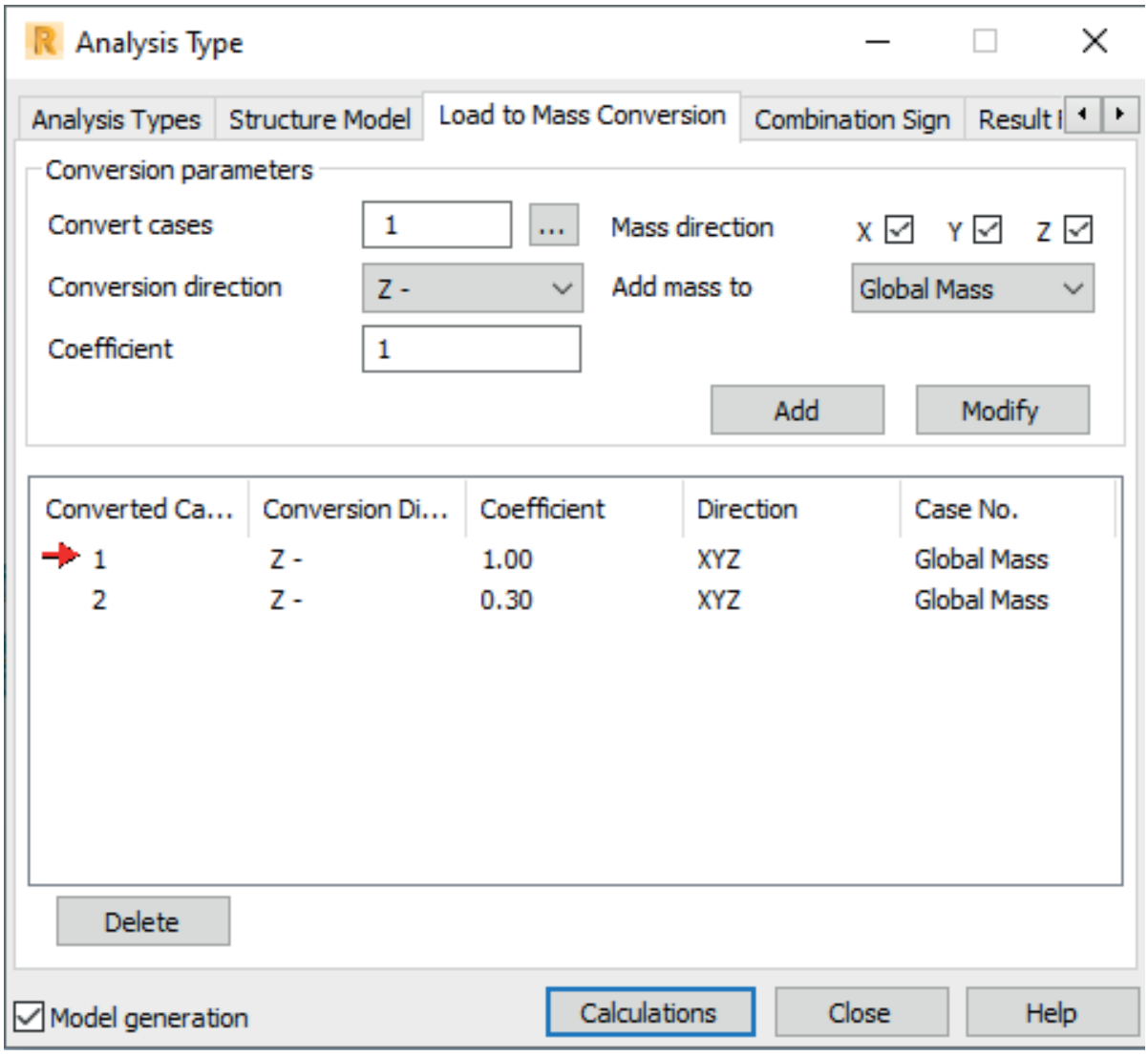
The image below shows the part of the floor modeled and includes the 25m long floor without any intermediary supports.



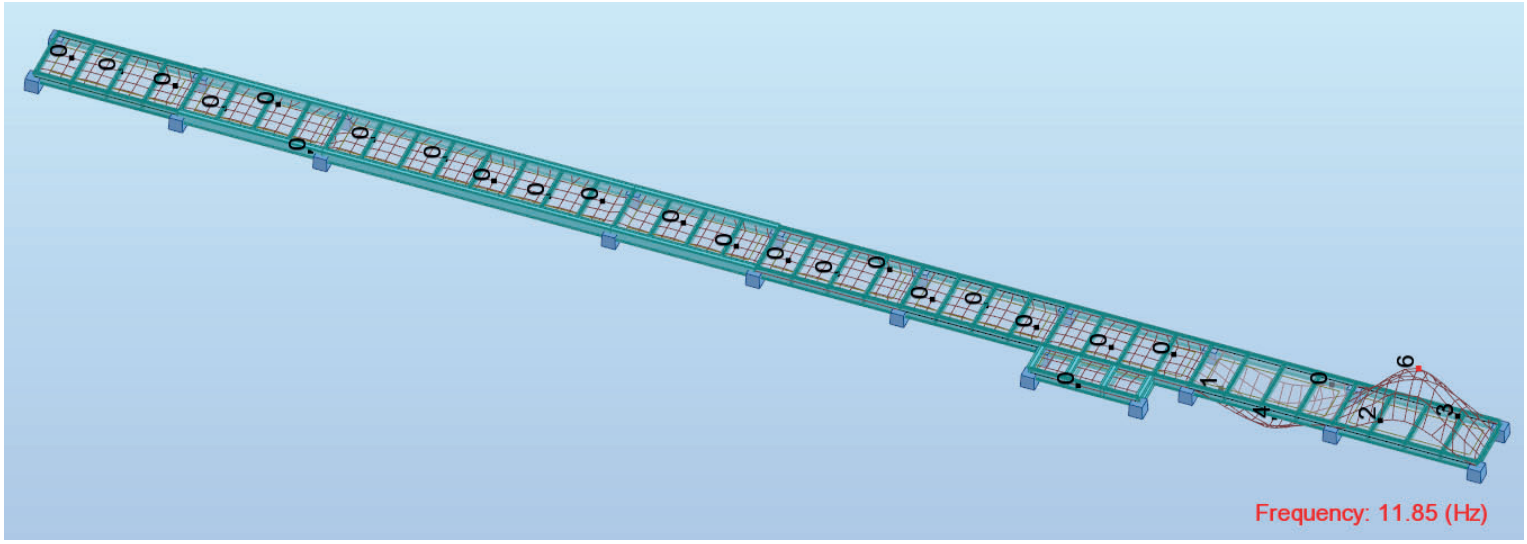
Definition of modal cases



Mass conversion

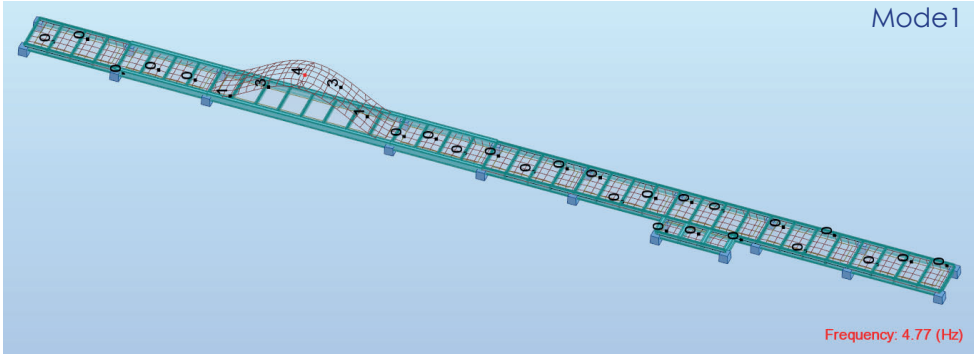


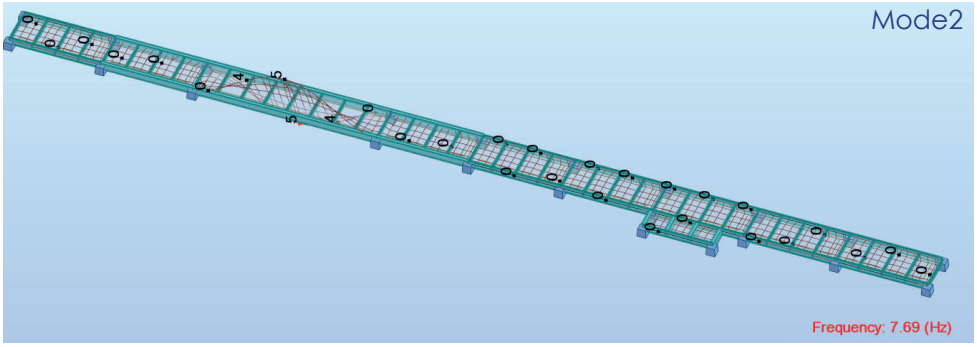
Mode 4 is the first mode that activates the typical bay of 12.5m span with a natural frequency of 11.85 Hz

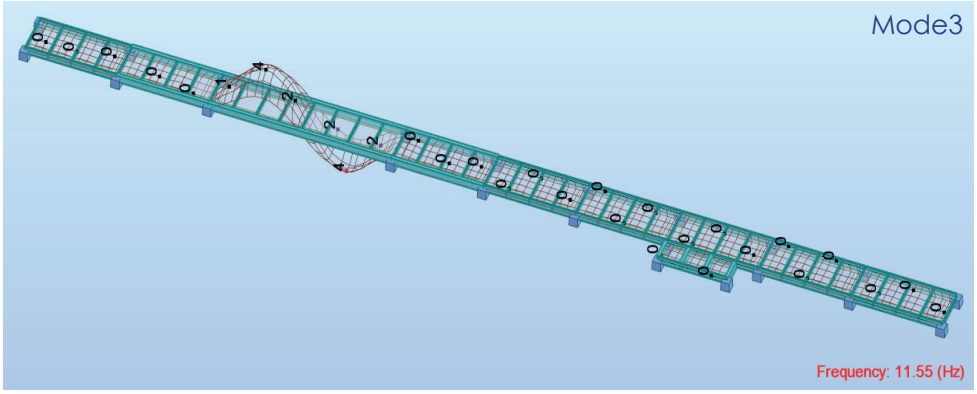


Mode 1 has the lowest frequency value, all the other modes having a value above 7hz, which is considered to be acceptable.

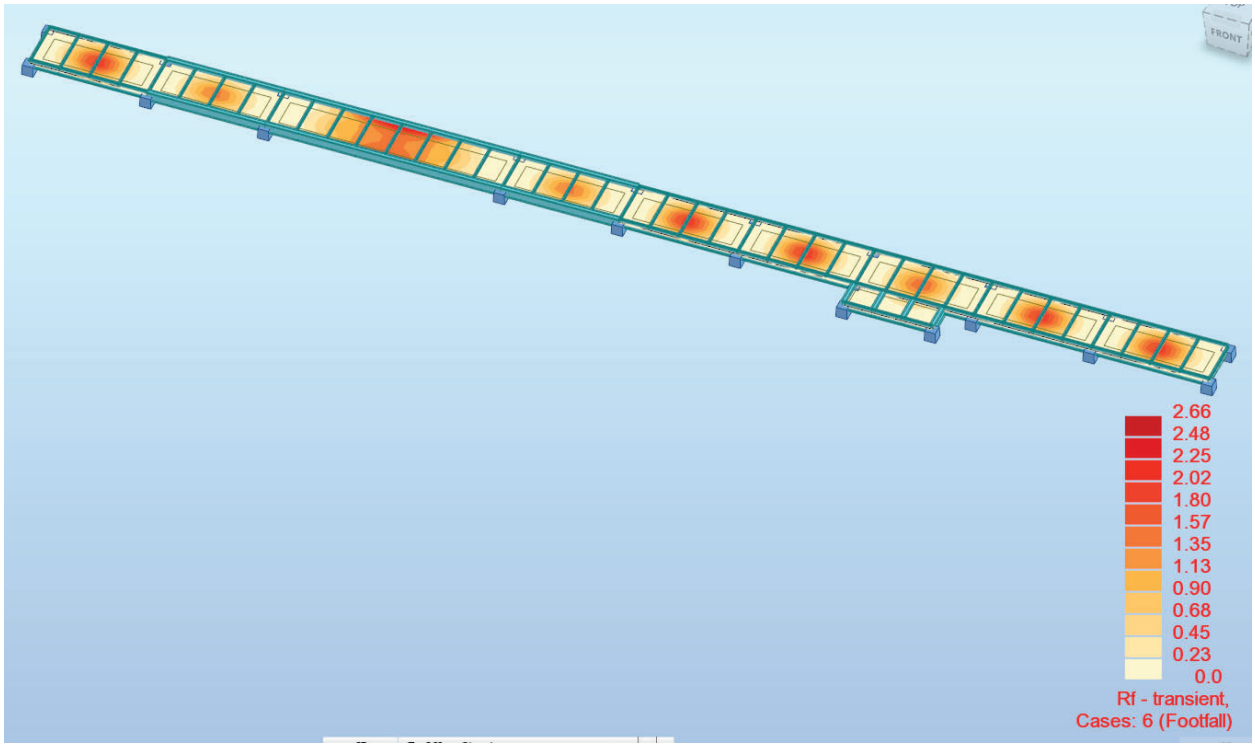
Also, considering the walking frequency of 1.2Hz the 3rd harmonic will be under the 1st mode frequency of the floor.

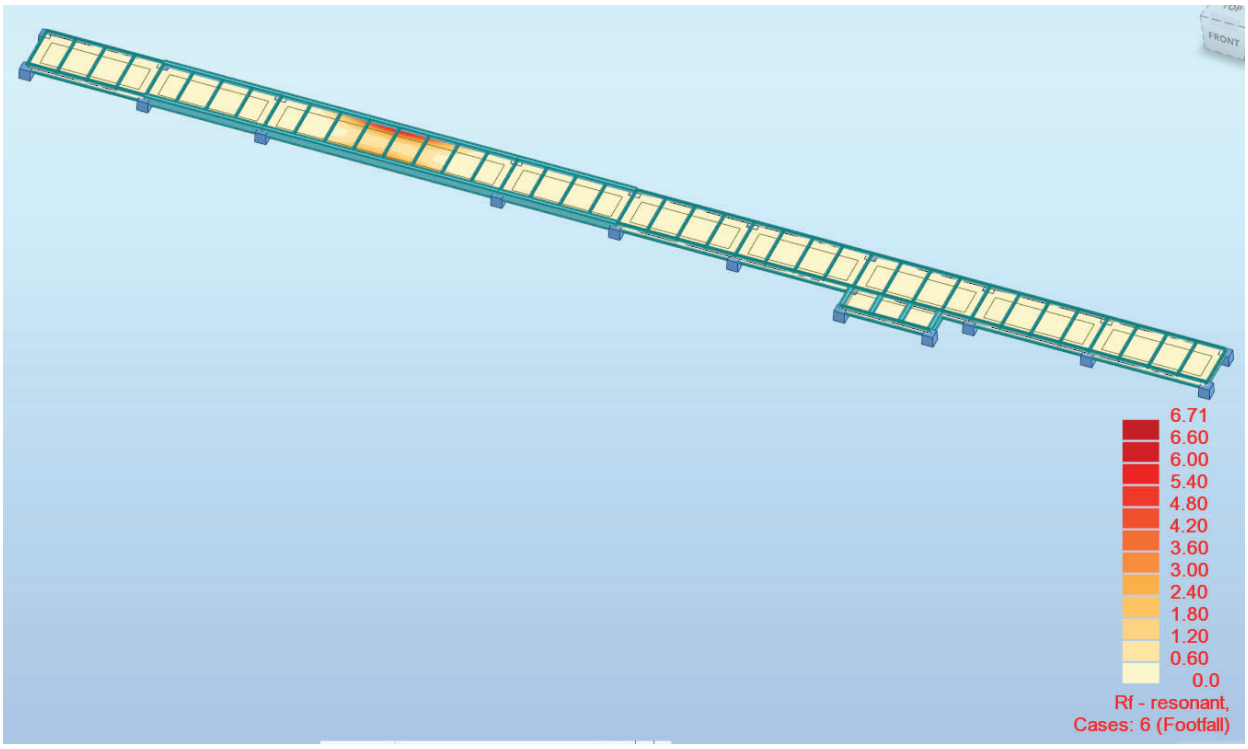






A footfall check is showing a response factor of 6.71 in the resonant case and 2.66 for the transient case.

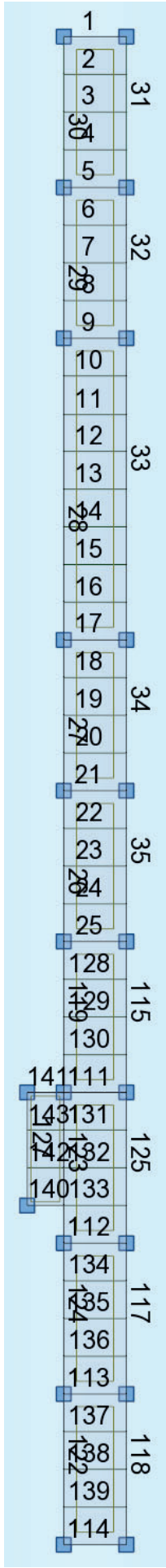




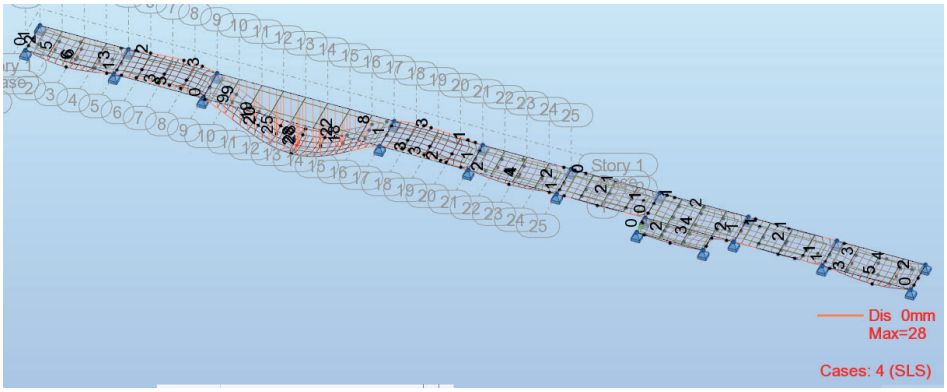
For the purpose of concept design, it can be assumed the structure is safe.

Check of the steel elements

Elements numbers



Maximum deflection



STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 1 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 114.89 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.03 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.05 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.06 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 2 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 203.19 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.05 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.10 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 3 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.57 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.01 kN
	Vz,T,Rd = 2378.27 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 4 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.79 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.06 kN
	Vz,T,Rd = 2378.25 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 5 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 211.44 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.03 kN
	Vz,T,Rd = 2378.27 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 6 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.58 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.03 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 7 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.59 kN*m
My,pl,Rd = 2184.20 kN*m

My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.02 kN
Vz,T,Rd = 2378.27 kN
Tt,Ed = 0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 8 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4

tf=40 mm Wply=6331000 mm3 Wplz=1939000 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = 208.38 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.04 kN
Vz,T,Rd = 2378.27 kN
Tt,Ed = -0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 9 B **POINT:** 1 **COORDINATE:** x = 0.50 L = 2.58 m

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 213.78 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -1.65 kN
	Vz,T,Rd = 2378.27 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 10 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:

Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:

S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 209.61 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.07 kN
	Vz,T,Rd = 2378.22 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 11 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.03 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.02 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 12 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 205.31 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.01 kN
	Vz,T,Rd = 2378.27 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 13 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.37 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.02 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 14 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.69 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.00 kN
	Vz,T,Rd = 2378.28 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 15 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.05 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Mb,Rd = 1954.83 kN*m

Vz,Ed = -0.03 kN
Vz,T,Rd = 2378.28 kN
Tt,Ed = -0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 16 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 208.09 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.03 kN
	Vz,T,Rd = 2378.27 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 17 B **POINT:** 3 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 213.43 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.97 kN
	Vz,T,Rd = 2378.25 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 18 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 208.27 kN*m
My,pl,Rd = 2184.20 kN*m

My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.06 kN
Vz,T,Rd = 2378.26 kN
Tt,Ed = -0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 19 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4

tf=40 mm Wply=6331000 mm3 Wplz=1939000 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.49 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.15 kN
Vz,T,Rd = 2378.28 kN
Tt,Ed = -0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 20 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.57 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.23 kN
	Vz,T,Rd = 2378.26 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 21 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:

Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:

S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 210.57 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.21 kN
	Vz,T,Rd = 2378.23 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 22 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.59 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.03 kN
	Vz,T,Rd = 2378.20 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis:

About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 23 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 205.66 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.01 kN
	Vz,T,Rd = 2378.22 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 24 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.56 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.05 kN
	Vz,T,Rd = 2378.25 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 25 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 214.39 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.02 kN
	Vz,T,Rd = 2378.04 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.04 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 26 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -720.38 kN*m
My,pl,Rd = 5716.65 kN*m
My,c,Rd = 5716.65 kN*m
Mb,Rd = 1760.09 kN*m
Vz,Ed = 300.86 kN
Vz,T,Rd = 4676.44 kN
Tt,Ed = 0.04 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.13 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.41 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 27 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 12.50 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2569.12 kN*m	
My,pl,Rd = 10006.26 kN*m	
My,c,Rd = 10006.26 kN*m	Vz,Ed = -471.78 kN
	Vz,T,Rd = 7023.85 kN
Mb,Rd = 9018.71 kN*m	Tt,Ed = -0.93 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 81096.74 kN*m	Curve,LT - d	XLT = 0.89
Lcr,low=12.50 m	Lam_LT = 0.35	fi,LT = 0.62	XLT,mod = 0.90

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.26 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.07 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.28 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 28 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2569.12 kN*m
My,pl,Rd = 10006.26 kN*m
My,c,Rd = 10006.26 kN*m
Mb,Rd = 8075.51 kN*m

Vz,Ed = 674.65 kN
Vz,T,Rd = 7023.65 kN
Tt,Ed = 1.72 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 42127.67 kN*m	Curve,LT - d	XLT = 0.79
Lcr,low=25.00 m	Lam_LT = 0.49	fi,LT = 0.73	XLT,mod = 0.81

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.26 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.10 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.32 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 125 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 27 mm < uz max = L/200.00 = 125 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 29 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2508.81 kN*m
My,pl,Rd = 10006.26 kN*m

My,c,Rd = 10006.26 kN*m

Mb,Rd = 9018.71 kN*m

Vz,Ed = 447.85 kN
Vz,T,Rd = 7023.76 kN
Tt,Ed = 1.27 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 81096.74 kN*m	Curve,LT - d	XLT = 0.89
Lcr,low=12.50 m	Lam_LT = 0.35	fi,LT = 0.62	XLT,mod = 0.90

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.25 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.28 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 30 B **POINT:** 1 **COORDINATE:** x = 0.50 L = 6.25 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4

tf=40 mm Wply=16570000 mm3 Wplz=1940000 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = 799.00 kN*m
My,pl,Rd = 5716.65 kN*m
My,c,Rd = 5716.65 kN*m

Mb,Rd = 1760.09 kN*m

Vz,Ed = -32.27 kN
Vz,T,Rd = 4676.52 kN
Tt,Ed = 0.03 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,upp=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.14 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.01 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.45 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 62 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 5 mm < uz max = L/200.00 = 62 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 31 B **POINT:** 3 **COORDINATE:** x = 0.50 L = 6.25 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 798.91 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 32.24 kN
	Vz,T,Rd = 4676.55 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.03 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,upp=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.14 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.01 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.45 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 5 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 32 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 12.50 m

---

LOADS:

Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2507.89 kN*m
My,pl,Rd = 10006.26 kN*m
My,c,Rd = 10006.26 kN*m
Mb,Rd = 9018.71 kN*m

Vz,Ed = -447.57 kN
Vz,T,Rd = 7023.75 kN
Tt,Ed = -1.30 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 81096.74 kN*m	Curve,LT - d	XLT = 0.89
Lcr,low=12.50 m	Lam_LT = 0.35	fi,LT = 0.62	XLT,mod = 0.90

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.25 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.28 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 33 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 25.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2573.17 kN*m	
My,pl,Rd = 10006.26 kN*m	
My,c,Rd = 10006.26 kN*m	Vz,Ed = -675.05 kN
	Vz,T,Rd = 7023.78 kN
Mb,Rd = 8075.51 kN*m	Tt,Ed = -1.22 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 42127.67 kN*m	Curve,LT - d	XLT = 0.79
Lcr,low=25.00 m	Lam_LT = 0.49	fi,LT = 0.73	XLT,mod = 0.81

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.26 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.10 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.32 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 125 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 27 mm < uz max = L/200.00 = 125 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 34 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -2573.17 kN*m	
My,pl,Rd = 10006.26 kN*m	
My,c,Rd = 10006.26 kN*m	Vz,Ed = 473.51 kN
	Vz,T,Rd = 7023.65 kN
Mb,Rd = 9018.71 kN*m	Tt,Ed = 1.72 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 81096.74 kN*m	Curve,LT - d	XLT = 0.89
Lcr,low=12.50 m	Lam_LT = 0.35	fi,LT = 0.62	XLT,mod = 0.90

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.26 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.07 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.29 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 35 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 12.50 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -783.81 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = -307.59 kN
	Vz,T,Rd = 4676.26 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.05 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.14 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.07 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.45 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 111 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -212.77 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 213.61 kN
	Vz,T,Rd = 2376.77 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.23 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,low=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.09 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 112 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 214.43 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.10 kN
	Vz,T,Rd = 2377.59 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.11 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 113 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 214.95 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -0.03 kN
	Vz,T,Rd = 2378.10 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.03 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 114 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 114.65 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Vz,Ed = -0.02 kN
Vz,T,Rd = 2378.17 kN
Mb,Rd = 1954.83 kN*m
Tt,Ed = -0.02 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.05 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.06 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 115 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -783.77 kN*m
My,pl,Rd = 5716.65 kN*m

My,c,Rd = 5716.65 kN*m

Mb,Rd = 1760.09 kN*m

Vz,Ed = 281.03 kN
Vz,T,Rd = 4676.10 kN
Tt,Ed = 0.07 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.14 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.45 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 117 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 12.50 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4

tf=40 mm Wply=16570000 mm3 Wplz=1940000 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = -877.21 kN*m
My,pl,Rd = 5716.65 kN*m
My,c,Rd = 5716.65 kN*m
Mb,Rd = 1760.09 kN*m

Vz,Ed = -290.51 kN
Vz,T,Rd = 4675.55 kN
Tt,Ed = -0.12 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.15 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.50 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 118 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -877.11 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 342.94 kN
	Vz,T,Rd = 4676.85 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.00 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.15 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.07 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.50 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 4 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 119 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 12.50 m

---

LOADS:

Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:

S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -908.48 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = -285.11 kN
	Vz,T,Rd = 4668.45 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.73 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.16 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.52 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 122 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -832.45 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 339.24 kN
	Vz,T,Rd = 4676.23 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.06 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.15 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.07 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.47 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 4 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 123 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -898.18 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 433.67 kN
	Vz,T,Rd = 4674.91 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = 0.17 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.16 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.09 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.51 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 3 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 124 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -833.69 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 273.07 kN
	Vz,T,Rd = 4676.15 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = 0.06 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.15 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.47 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 0 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 125 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -675.55 kN*m	
My,pl,Rd = 5716.65 kN*m	
My,c,Rd = 5716.65 kN*m	Vz,Ed = 267.75 kN
	Vz,T,Rd = 4674.20 kN
Mb,Rd = 1760.09 kN*m	Tt,Ed = -0.23 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1869.68 kN*m	Curve,LT - c	XLT = 0.31
Lcr,low=12.50 m	Lam_LT = 1.75	fi,LT = 1.98	XLT,mod = 0.31

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.12 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.06 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.38 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 2 mm < uz max = L/200.00 = 63 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 127 B **POINT:** 1 **COORDINATE:** x = 0.67 L = 6.25 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 1000

h=1008 mm	gM0=1.00	gM1=1.00	
b=302 mm	Ay=26172 mm2	Az=23480 mm2	Ax=44400 mm2
tw=21 mm	Iy=7223000000 mm4	Iz=184600000 mm4	Ix=17130000 mm4
tf=40 mm	Wply=16570000 mm3	Wplz=1940000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 407.09 kN*m
My,pl,Rd = 5716.65 kN*m
My,c,Rd = 5716.65 kN*m
Mb,Rd = 2285.42 kN*m

Vz,Ed = -115.93 kN
Vz,T,Rd = 4676.25 kN
Tt,Ed = 0.06 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 2627.58 kN*m	Curve,LT - c	XLT = 0.40
Lcr,upp=9.38 m	Lam_LT = 1.48	fi,LT = 1.58	XLT,mod = 0.40

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.07 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.18 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 47 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 2 mm < uz max = L/200.00 = 47 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 128 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.31 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.04 kN
Vz,T,Rd = 2377.96 kN
Tt,Ed = -0.05 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 129 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 207.36 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Vz,Ed = -0.03 kN
Vz,T,Rd = 2378.15 kN
Mb,Rd = 1954.83 kN*m
Tt,Ed = -0.02 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 130 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 200.79 kN*m
My,pl,Rd = 2184.20 kN*m

My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.44 kN
Vz,T,Rd = 2378.25 kN
Tt,Ed = 0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.10 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 131 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4

tf=40 mm Wply=6331000 mm3 Wplz=1939000 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = 170.67 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m

Mb,Rd = 1954.83 kN*m

Vz,Ed = 12.98 kN
Vz,T,Rd = 2377.21 kN
Tt,Ed = 0.16 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.08 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.01 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.09 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 132 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 224.27 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -7.94 kN
	Vz,T,Rd = 2378.22 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 2 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 133 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:

Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:

S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 213.29 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = -2.31 kN
	Vz,T,Rd = 2377.49 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.12 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.10 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 134 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.55 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.05 kN
	Vz,T,Rd = 2378.09 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = -0.03 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 135 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 205.74 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.03 kN
	Vz,T,Rd = 2378.15 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.02 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 136 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.65 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.06 kN
	Vz,T,Rd = 2378.01 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.04 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 137 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.38 kN*m	
My,pl,Rd = 2184.20 kN*m	
My,c,Rd = 2184.20 kN*m	Vz,Ed = 0.02 kN
	Vz,T,Rd = 2378.21 kN
Mb,Rd = 1954.83 kN*m	Tt,Ed = 0.01 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 138 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 206.67 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.01 kN
Vz,T,Rd = 2378.26 kN
Tt,Ed = -0.00 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.11 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 139 B **POINT:** 2 **COORDINATE:** x = 0.50 L = 2.58 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: HEM 450

h=478 mm	gM0=1.00	gM1=1.00	
b=307 mm	Ay=26276 mm2	Az=11940 mm2	Ax=33500 mm2
tw=21 mm	Iy=1315000000 mm4	Iz=193400000 mm4	Ix=15340000 mm4
tf=40 mm	Wply=6331000 mm3	Wplz=1939000 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 203.22 kN*m
My,pl,Rd = 2184.20 kN*m
My,c,Rd = 2184.20 kN*m
Mb,Rd = 1954.83 kN*m

Vz,Ed = 0.08 kN
Vz,T,Rd = 2378.22 kN
Tt,Ed = -0.01 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 4455.17 kN*m	Curve,LT - b	XLT = 0.87
Lcr,upp=5.15 m	Lam_LT = 0.70	fi,LT = 0.73	XLT,mod = 0.89

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.09 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.00 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.10 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 1 mm < uz max = L/200.00 = 26 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 140 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 3.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = 0.07 kN*m
My,pl,Rd = 10006.26 kN*m
My,c,Rd = 10006.26 kN*m
Mb,Rd = 10006.26 kN*m

Vz,Ed = -61.82 kN
Vz,T,Rd = 7018.53 kN
Tt,Ed = 21.50 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 273148.39 kN*m	Curve,LT - d	XLT = 1.00
Lcr,upp=3.00 m	Lam_LT = 0.19	fi,LT = 0.52	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.00 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.01 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.00 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 0 mm < uz max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 141 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -214.64 kN*m
My,pl,Rd = 10006.26 kN*m

My,c,Rd = 10006.26 kN*m

Mb,Rd = 10006.26 kN*m

Vz,Ed = 129.89 kN
Vz,T,Rd = 7021.71 kN
Tt,Ed = 9.19 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 273148.39 kN*m	Curve,LT - d	XLT = 1.00
Lcr,low=3.00 m	Lam_LT = 0.19	fi,LT = 0.52	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.02 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.02 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 0 mm < uz max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 142 B **POINT:** 3 **COORDINATE:** x = 1.00 L = 3.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4

tf=36 mm Wply=29003648 mm3 Wplz=11769600 mm3

INTERNAL FORCES AND CAPACITIES:

My,Ed = -0.21 kN*m
My,pl,Rd = 10006.26 kN*m
My,c,Rd = 10006.26 kN*m

Mb,Rd = 10006.26 kN*m

Vz,Ed = -115.54 kN
Vz,T,Rd = 7018.05 kN
Tt,Ed = -23.38 kN*m
Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 273148.39 kN*m	Curve,LT - d	XLT = 1.00
Lcr,low=3.00 m	Lam_LT = 0.19	fi,LT = 0.52	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:
My,Ed/My,c,Rd = 0.00 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)
Global stability check of member:
My,Ed/Mb,Rd = 0.00 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):
uy = 0 mm < uy max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 0 mm < uz max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

STEEL DESIGN

CODE: *BS-EN 1993-1:2005/NA:2008/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 143 B **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 m

---

LOADS:
Governing Load Case: 3 ULS 1*1.35+2*1.50

MATERIAL:
S355 (S355) fy = 345.00 MPa

SECTION PARAMETERS: BOX2_1

h=1000 mm	gM0=1.00	gM1=1.00	
b=600 mm	Ay=43200 mm2	Az=35264 mm2	Ax=78464 mm2
tw=19 mm	Iy=12571795115 mm4	Iz=2090500859 mm4	Ix=2831772189 mm4
tf=36 mm	Wply=29003648 mm3	Wplz=11769600 mm3	

INTERNAL FORCES AND CAPACITIES:

My,Ed = -66.47 kN*m	
My,pl,Rd = 10006.26 kN*m	
My,c,Rd = 10006.26 kN*m	Vz,Ed = 123.56 kN
	Vz,T,Rd = 7010.46 kN
Mb,Rd = 10006.26 kN*m	Tt,Ed = -52.73 kN*m
	Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 273148.39 kN*m	Curve,LT - d	XLT = 1.00
Lcr,low=3.00 m	Lam_LT = 0.19	fi,LT = 0.52	XLT,mod = 1.00

BUCKLING PARAMETERS:

About y axis: About z axis:

VERIFICATION FORMULAS:

Section strength check:

My,Ed/My,c,Rd = 0.01 < 1.00 (6.2.5.(1))
Vz,Ed/Vz,T,Rd = 0.02 < 1.00 (6.2.6-7)
Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.01 < 1.00 (6.2.6)
Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

My,Ed/Mb,Rd = 0.01 < 1.00 (6.3.2.1.(1))

LIMIT DISPLACEMENTS

Deflections (LOCAL SYSTEM):

uy = 0 mm < uy max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70
uz = 0 mm < uz max = L/200.00 = 15 mm Verified
Governing Load Case: 4 SLS 1*1.00+2*0.70

Displacements (GLOBAL SYSTEM): *Not analyzed*

Section OK !!!

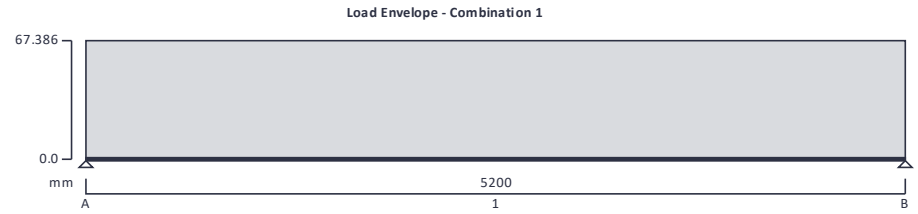
Tekla Tedds	Project: Florence airport				Job no.	
	Calcs for: Typ Secondary beam				Start page no./Revision: 1	
	Calcs by: M	Calcs date: 27/11/2021	Checked by	Checked date	Approved by	Approved date

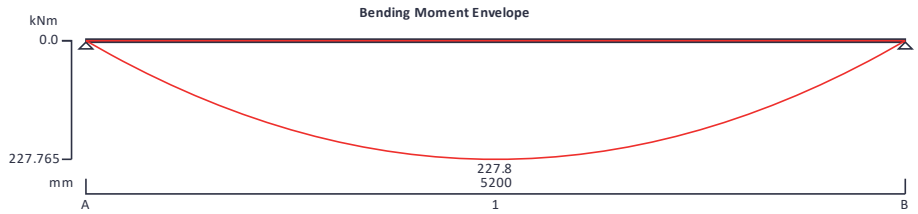
A series of hand checks have been done for the worst cases for each beam

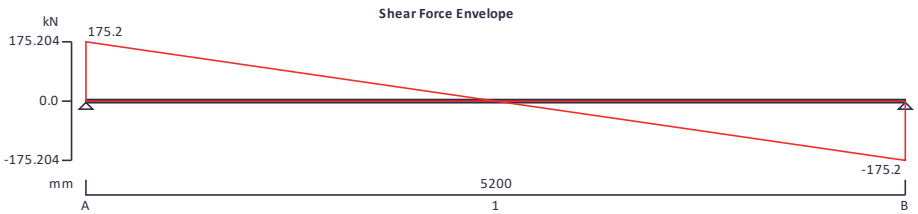
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	SW - Permanent self weight of beam × 1
	DL - Permanent full UDL 30 kN/m
	LL - Variable full UDL 15.6 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

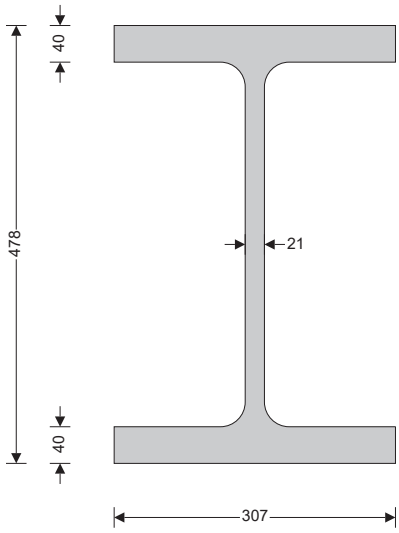
Maximum moment	M_{max} = **227.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **175.2** kN	V_{min} = **-175.2** kN
Deflection	δ_{max} = **1.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **175.2** kN	R_{A_min} = **175.2** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **84.7** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **40.6** kN	
Maximum reaction at support B	R_{B_max} = **175.2** kN	R_{B_min} = **175.2** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **84.7** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **40.6** kN	

Section details

Section type	**HE 450 M (Arcelor)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **40.0** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.83}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d = \mathbf{344}$ mm

$c / t_w = 19.8 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{116}$ mm

$c / t_f = 3.5 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{398}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) = \mathbf{175.2}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{11984}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{2387}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(abs(M_{s1_max}), abs(M_{s1_min})) = \mathbf{227.8}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{2184.2}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{1}$

$C_1 = 1 / k_c^2 = \mathbf{1}$

Destabilised load condition factor — $D = \mathbf{2}$

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.924}$

Poissons ratio — $\nu = \mathbf{0.3}$

Shear modulus — $G = E / [2 \times (1 + \nu)] = \mathbf{80769}$ N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{5200}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **5821.7** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{1.225}$

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections — $\beta = \mathbf{0.75}$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.203}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.565}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{1.000}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.565}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{1233.8}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **14.4** mm

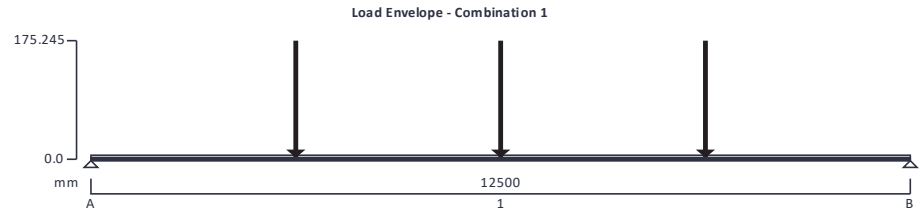
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **1.661** mm

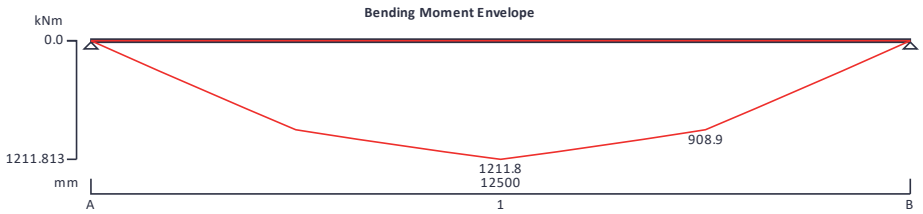
PASS - Maximum deflection does not exceed deflection limit

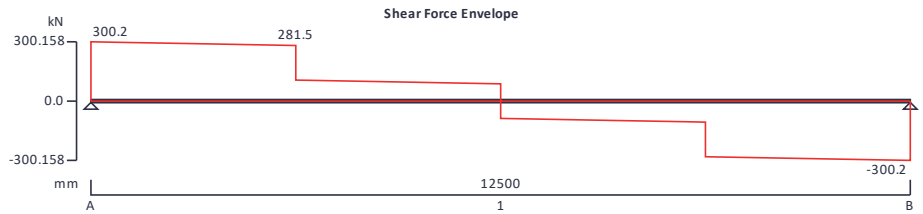
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	SW - Permanent self weight of beam × 1
	DL - Permanent point load 84.7 kN at 3125 mm
	LL - Variable point load 40.6 kN at 3125 mm
	DL - Permanent point load 84.7 kN at 6250 mm
	LL - Variable point load 40.6 kN at 6250 mm
	DL - Permanent point load 84.7 kN at 9375 mm
	LL - Variable point load 40.6 kN at 9375 mm
	DL - Permanent full UDL 1 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35

		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

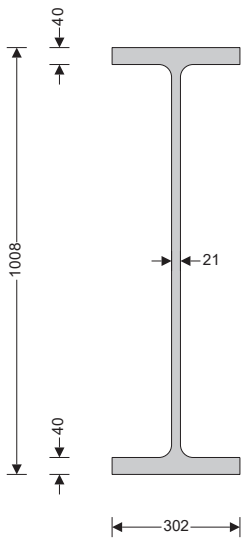
Maximum moment	M_{max} = **1211.8** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1_seg1_max}$ = **908.9** kNm	$M_{s1_seg1_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1_seg2_max}$ = **1211.8** kNm	$M_{s1_seg2_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1_seg3_max}$ = **1211.8** kNm	$M_{s1_seg3_min}$ = **0** kNm
Maximum moment span 1 segment 4	$M_{s1_seg4_max}$ = **908.9** kNm	$M_{s1_seg4_min}$ = **0** kNm
Maximum shear	V_{max} = **300.2** kN	V_{min} = **-300.2** kN
Maximum shear span 1 segment 1	$V_{s1_seg1_max}$ = **300.2** kN	$V_{s1_seg1_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1_seg2_max}$ = **106.3** kN	$V_{s1_seg2_min}$ = **-87.6** kN
Maximum shear span 1 segment 3	$V_{s1_seg3_max}$ = **0** kN	$V_{s1_seg3_min}$ = **-281.5** kN
Maximum shear span 1 segment 4	$V_{s1_seg4_max}$ = **0** kN	$V_{s1_seg4_min}$ = **-300.2** kN
Deflection segment 5	δ_{max} = **8.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **300.2** kN	R_{A_min} = **300.2** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **154.7** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **60.9** kN	
Maximum reaction at support B	R_{B_max} = **300.2** kN	R_{B_min} = **300.2** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **154.7** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **60.9** kN	

Section details

Section type	**HE 1000 M (Arcelor)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **40.0** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports plus quarter points

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.83**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **868** mm

$c / t_w = 50.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **110.5** mm

$c / t_f = 3.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **928** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **300.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **23501** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **4681** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 2 major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min}))$ = **1211.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **5715.9** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **1**

$C_1 = 1 / k_c^2$ = **1**

Destabilised load condition factor D = **2**

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.987**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length $L = 1.2 \times L_{s1_seg2}$ = **3750** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **14716.4** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.246**

Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5	d
Imperfection factor - Table 6.3	α_{LT} = **0.76**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.404**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.434**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.434**
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **2482.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

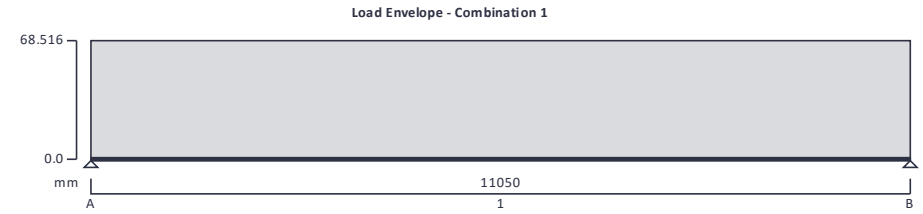
Limiting deflection	$\delta_{lim} = L_{s1} / 360$ = **34.7** mm
Maximum deflection span 1	$\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **8.909** mm

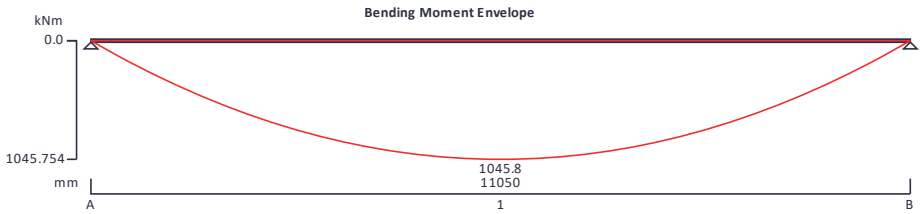
PASS - Maximum deflection does not exceed deflection limit

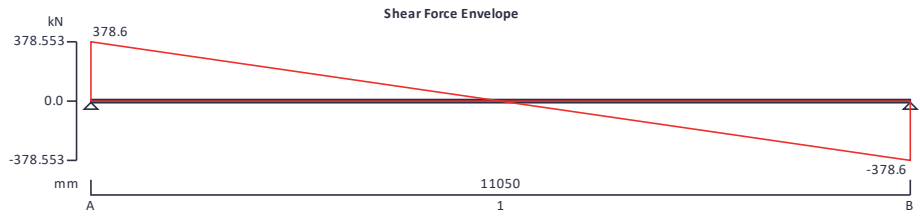
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	SW - Permanent self weight of beam × 1
	DL - Permanent full UDL 30 kN/m
	LL - Variable full UDL 15.6 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

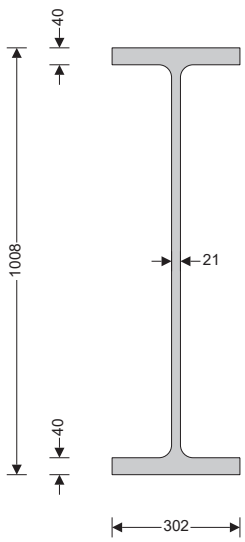
Maximum moment	M_{max} = **1045.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **378.6** kN	V_{min} = **-378.6** kN
Deflection	δ_{max} = **6.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **378.6** kN	R_{A_min} = **378.6** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **184.6** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **86.2** kN	
Maximum reaction at support B	R_{B_max} = **378.6** kN	R_{B_min} = **378.6** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **184.6** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **86.2** kN	

Section details

Section type	**HE 1000 M (Arcelor)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **40.0** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.83}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d = \mathbf{868}$ mm

$c / t_w = 50.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{110.5}$ mm

$c / t_f = 3.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{928}$ mm

Shear area factor $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) = \mathbf{378.6}$ kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{23501}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{4681}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(abs(M_{s1_max}), abs(M_{s1_min})) = \mathbf{1045.8}$ kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{5715.9}$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

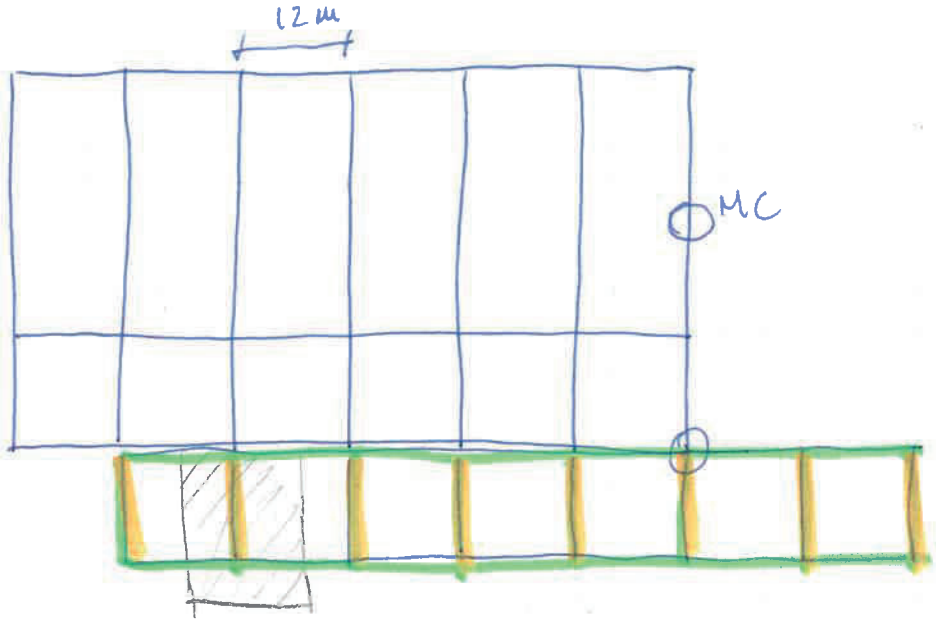
Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{30.7}$ mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{6.274}$ mm

PASS - Maximum deflection does not exceed deflection limit

Mezzanine HVAC Structure

Steel Structure beams @ 12 m centres



Primary

secondary steelwork includes secondary beams

LOADS: From last issued report:

tot DL (DL+SDL) = 3.5 KPa

add 1 KPa (more cladding) = 4.5 KPa

HVAC unit: 0.2 KPa ≈ 0.5 KPa

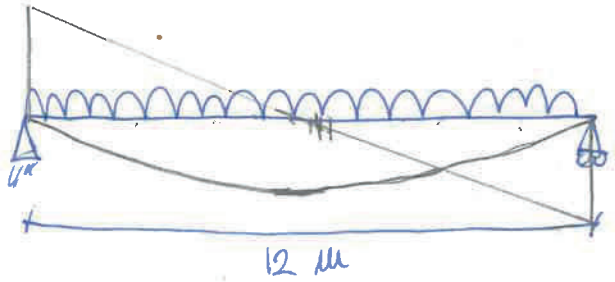
→ tot DL = 5 KPa

LL = 0.6 KPa

Trib. width = 12 m

SLS: 5 x 12 + 0.6 x 12 = 60 + 7.2 = 67.2 KN/m

ULS: 81 + 11 = 92 KN/m



92 KN/m

$V_{Ed} = \frac{wL}{2} = 552\,KN$

$M_{Ed} = \frac{wL^2}{8} = 1656\,KN/m$

Assume Beam of family ~~533~~ 762 x 262 x 197
Leading Bending
Shear capacity: 2350
Moment Capacity: 2470 KN/m

Assume secondary beams & deck restrain every 4m → 1960

Check With Tedds

$E = 205000$

Deflection check: $\delta = \frac{5}{384} \frac{wL^4}{EI} = 37\,mm$

$I = 24$

$I = 36 \times 10^7\,mm^2$

26
32*

* Beam: 838 x 292 x 194

$I = 279000\,cm^4$

$V_{Rd} = 2610\,KN$

$M_{Rd} = 2150\,KN/m$ if restrained every 4m

$\delta_{max} = 32\,mm$

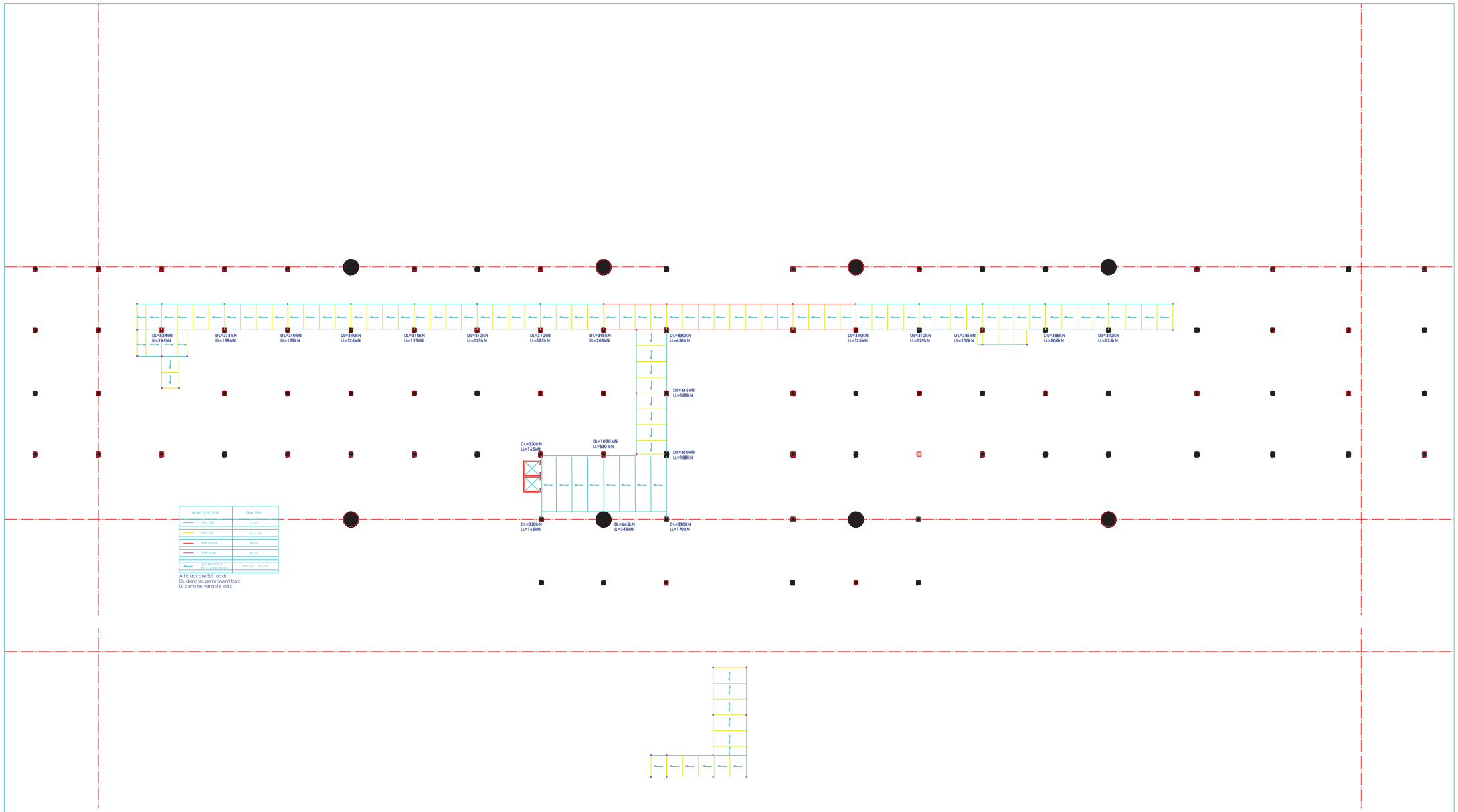
$L/360 = 33.3\,mm$ ok

Find European equivalent

~~IPE 750 x 220~~

HE 800A $\delta = 29.2\,mm$

HE 800 A 224

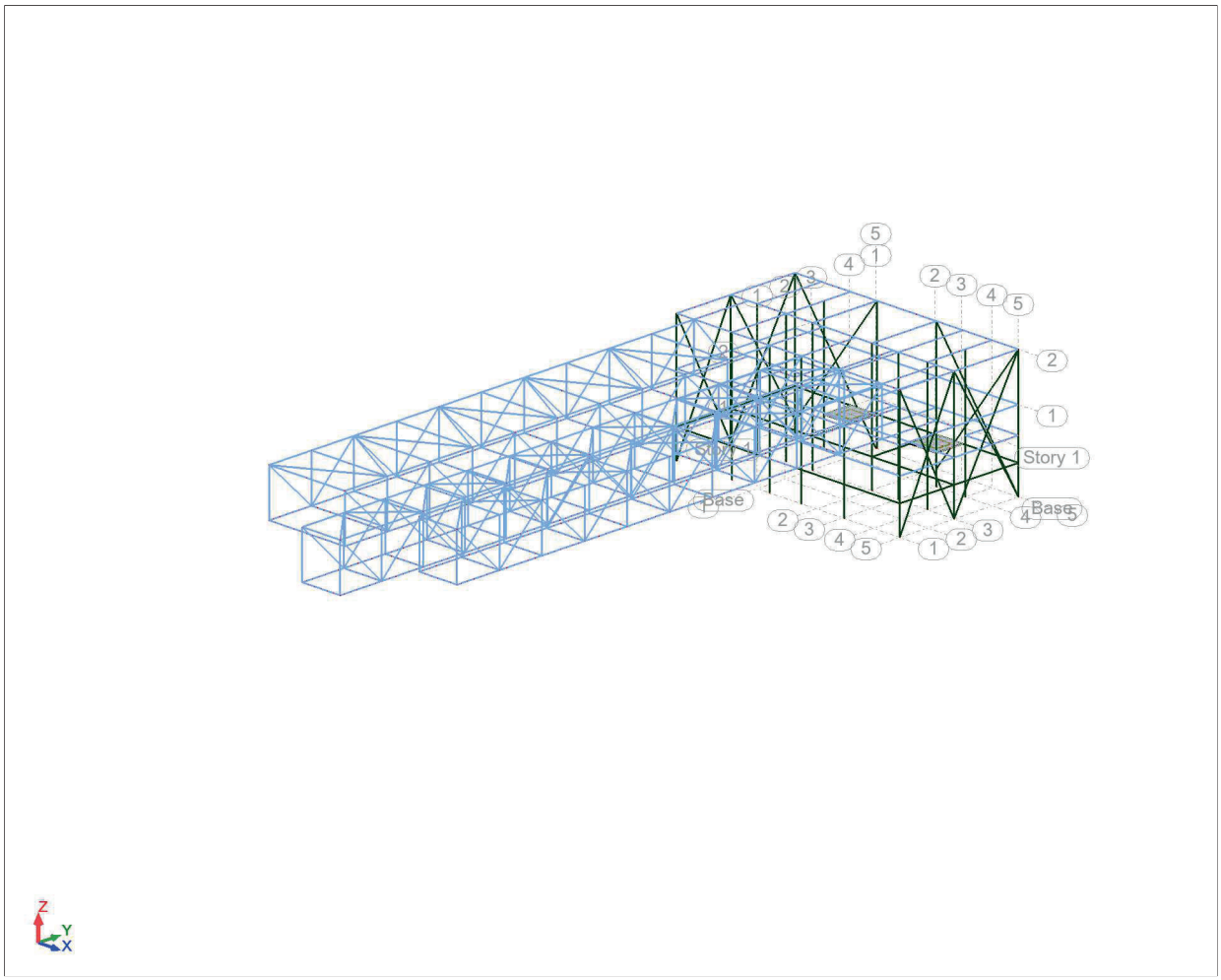


BEAM SCHEDULE	Quantities
HEM 1100	512 m
HEM 450	1115 m
2xHEM 1000	88 m
Steel Hanger	80 m
Steel Hanger 80/12 & 100 diameter	177.65 m2 39.6 m3

All loads are SLS loads
DL denotes permanent load
LL denotes variable load

Appendice F: Calcoli ponte d'imbarco

Structure View



Data - Bars

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
1	1	2	IPE 450	*S355*	4.70	0.0	Beam
2	2	3	IPE 450	*S355*	6.50	0.0	Beam
3	3	4	IPE 450	*S355*	5.10	0.0	Beam
4	4	5	IPE 450	*S355*	4.30	0.0	Beam
5	5	6	HEA 340	*S355*	8.80	0.0	Column
6	4	7	HEA 260	*S355*	8.80	0.0	Column
7	3	8	HEA 260	*S355*	8.80	0.0	Column
8	2	9	HEA 260	*S355*	8.80	0.0	Column
9	10	11	HEA 260	*S355*	8.80	0.0	Column
10	12	13	HEA 260	*S355*	8.80	0.0	Column
11	14	15	HEA 260	*S355*	8.80	0.0	Column
12	16	17	HEA 260	*S355*	8.80	0.0	Column
13	18	19	HEA 260	*S355*	8.80	0.0	Column
14	20	21	HEA 340	*S355*	8.80	0.0	Column
15	2	18	IPE 450	*S355*	5.10	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
16	4891	31	IPE 450	*S355*	4.40	0.0	Beam
17	5	20	HEA 340	*S355*	7.80	0.0	Beam
18	18	16	IPE 450	*S355*	2.10	0.0	Beam
19	2139	26	IPE 450	*S355*	4.30	0.0	Beam
20	16	10	IPE 450	*S355*	2.30	0.0	Beam
21	10	4	IPE 450	*S355*	2.10	0.0	Beam
22	31	4892	IPE 450	*S355*	3.40	0.0	Beam
23	25	36	HEA 340	*S355*	4.40	0.0	Beam
24	36	26	HEA 340	*S355*	3.40	0.0	Beam
25	16	14	IPE 450	*S355*	3.00	0.0	Beam
26	12	10	IPE 450	*S355*	3.00	0.0	Beam
27	6	4	SHS 120x5	*S355*	9.79	0.0	Simple bar
28	4895	33	IPE 450	*S355*	4.40	0.0	Beam
29	25	27	IPE 450	*S355*	4.30	0.0	Beam
30	27	29	IPE 450	*S355*	5.10	0.0	Beam
31	29	30	IPE 450	*S355*	6.50	0.0	Beam
32	30	28	IPE 450	*S355*	5.10	0.0	Beam
33	37	35	IPE 450	*S355*	2.30	0.0	Beam
34	577	39	IPE 450	*S355*	4.30	0.0	Beam
35	2140	20	IPE 450	*S355*	4.30	0.0	Beam
36	35	28	IPE 450	*S355*	2.10	0.0	Beam
37	33	4896	IPE 450	*S355*	3.40	0.0	Beam
38	576	38	IPE 450	*S355*	4.30	0.0	Beam
39	7	5	SHS 120x5	*S355*	9.79	0.0	Simple bar
40	41	30	IPE 450	*S355*	4.70	0.0	Beam
41	42	28	IPE 450	*S355*	4.70	0.0	Beam
42	43	26	IPE 450	*S355*	2.50	0.0	Beam
43	44	18	IPE 450	*S355*	4.70	0.0	Beam
44	45	20	IPE 450	*S355*	2.50	0.0	Beam
45	35	36	IPE 450	*S355*	4.30	0.0	Beam
46	16	40	IPE 450	*S355*	4.30	0.0	Beam
47	7	3	SHS 120x5	*S355*	10.17	0.0	Simple bar
48	8	4	SHS 120x5	*S355*	10.17	0.0	Simple bar
49	8	2	SHS 120x5	*S355*	10.94	0.0	Simple bar
50	9	3	SHS 120x5	*S355*	10.94	0.0	Simple bar
51	22	584	SHS 120x5	*S355*	10.88	0.0	Simple bar
52	589	1	SHS 120x5	*S355*	10.94	0.0	Simple bar
53	589	585	SHS 120x5	*S355*	10.17	0.0	Simple bar
54	588	584	SHS 120x5	*S355*	10.17	0.0	Simple bar
55	588	586	SHS 120x5	*S355*	9.79	0.0	Simple bar
56	587	585	SHS 120x5	*S355*	9.79	0.0	Simple bar
57	615	32	HEA 340	*S355*	8.80	-180.0	Column
58	72	73	SHS 200x1	*S355*	3.00	0.0	Beam
59	74	75	SHS 200x1	*S355*	3.00	0.0	Beam
60	76	77	SHS 200x1	*S355*	3.00	0.0	Beam
61	78	79	SHS 200x1	*S355*	3.00	0.0	Beam
62	80	81	SHS 200x1	*S355*	3.00	0.0	Beam
63	82	83	SHS 200x1	*S355*	3.00	0.0	Beam
64	84	85	SHS 200x1	*S355*	3.00	0.0	Beam
65	86	87	SHS 200x1	*S355*	3.00	0.0	Beam
66	88	89	SHS 200x1	*S355*	3.00	0.0	Beam
70	96	97	SHS 200x1	*S355*	3.00	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
71	38	96	RHS 400x	*S355*	33.70	0.0	Beam
72	36	97	RHS 400x	*S355*	33.70	0.0	Beam
73	72	100	SHS 200x1	*S355*	3.30	0.0	Beam
74	73	101	SHS 200x1	*S355*	3.30	0.0	Beam
75	74	102	SHS 200x1	*S355*	3.30	0.0	Beam
76	75	103	SHS 200x1	*S355*	3.30	0.0	Beam
77	76	104	SHS 200x1	*S355*	3.30	0.0	Beam
78	77	105	SHS 200x1	*S355*	3.30	0.0	Beam
79	78	106	SHS 200x1	*S355*	3.30	0.0	Beam
80	79	107	SHS 200x1	*S355*	3.30	0.0	Beam
81	80	108	SHS 200x1	*S355*	3.30	0.0	Beam
82	81	109	SHS 200x1	*S355*	3.30	0.0	Beam
83	82	110	SHS 200x1	*S355*	3.30	0.0	Beam
84	83	111	SHS 200x1	*S355*	3.30	0.0	Beam
85	84	112	SHS 200x1	*S355*	3.30	0.0	Beam
86	85	113	SHS 200x1	*S355*	3.30	0.0	Beam
87	86	114	SHS 200x1	*S355*	3.30	0.0	Beam
88	87	115	SHS 200x1	*S355*	3.30	0.0	Beam
89	88	116	SHS 200x1	*S355*	3.30	0.0	Beam
90	89	117	SHS 200x1	*S355*	3.30	0.0	Beam
101	100	101	SHS 200x1	*S355*	3.00	0.0	Beam
102	102	103	SHS 200x1	*S355*	3.00	0.0	Beam
103	104	105	SHS 200x1	*S355*	3.00	0.0	Beam
104	106	107	SHS 200x1	*S355*	3.00	0.0	Beam
105	108	109	SHS 200x1	*S355*	3.00	0.0	Beam
106	110	111	SHS 200x1	*S355*	3.00	0.0	Beam
107	112	113	SHS 200x1	*S355*	3.00	0.0	Beam
108	114	115	SHS 200x1	*S355*	3.00	0.0	Beam
109	116	117	SHS 200x1	*S355*	3.00	0.0	Beam
113	136	135	SHS 200x1	*S355*	3.00	0.0	Beam
114	39	136	RHS 400x	*S355*	33.70	0.0	Beam
115	40	135	RHS 400x	*S355*	33.70	0.0	Beam
119	136	88	SHS 200x1	*S355*	4.72	0.0	Beam
120	88	114	SHS 200x1	*S355*	4.72	0.0	Beam
121	114	84	SHS 200x1	*S355*	4.72	0.0	Beam
122	84	110	SHS 200x1	*S355*	4.72	0.0	Beam
123	110	80	SHS 200x1	*S355*	4.72	0.0	Beam
124	80	106	SHS 200x1	*S355*	4.72	0.0	Beam
125	106	76	SHS 200x1	*S355*	4.72	0.0	Beam
126	76	102	SHS 200x1	*S355*	4.72	0.0	Beam
127	102	72	SHS 200x1	*S355*	4.72	0.0	Beam
128	72	39	SHS 200x1	*S355*	4.72	0.0	Beam
129	39	38	SHS 200x1	*S355*	3.30	0.0	Beam
130	36	40	SHS 200x1	*S355*	3.30	0.0	Beam
131	40	73	SHS 200x1	*S355*	4.72	0.0	Beam
132	73	103	SHS 200x1	*S355*	4.72	0.0	Beam
133	103	77	SHS 200x1	*S355*	4.72	0.0	Beam
134	77	107	SHS 200x1	*S355*	4.72	0.0	Beam
135	107	81	SHS 200x1	*S355*	4.72	0.0	Beam
136	81	111	SHS 200x1	*S355*	4.72	0.0	Beam
137	111	85	SHS 200x1	*S355*	4.72	0.0	Beam
138	85	115	SHS 200x1	*S355*	4.72	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
139	115	89	SHS 200x1	*S355*	4.72	0.0	Beam
140	89	135	SHS 200x1	*S355*	4.72	0.0	Beam
147	136	117	SHS 200x1	*S355*	4.51	0.0	Beam
148	117	114	SHS 200x1	*S355*	4.51	0.0	Beam
149	114	113	SHS 200x1	*S355*	4.51	0.0	Beam
150	113	110	SHS 200x1	*S355*	4.51	0.0	Beam
151	110	109	SHS 200x1	*S355*	4.51	0.0	Beam
152	109	106	SHS 200x1	*S355*	4.51	0.0	Beam
153	106	105	SHS 200x1	*S355*	4.51	0.0	Beam
154	105	102	SHS 200x1	*S355*	4.51	0.0	Beam
155	102	101	SHS 200x1	*S355*	4.51	0.0	Beam
156	101	39	SHS 200x1	*S355*	4.51	0.0	Beam
158	136	96	RHS 400x	*S355*	3.30	0.0	Column
159	97	135	RHS 400x	*S355*	3.30	0.0	Column
160	14	126	IPE 450	*S355*	2.10	0.0	Column
161	12	127	IPE 450	*S355*	2.10	0.0	Column
166	36	134	HEA 340	*S355*	5.50	0.0	Column
168	1	584	IPE 450	*S355*	6.50	0.0	Beam
169	584	585	IPE 450	*S355*	5.10	0.0	Beam
170	585	586	IPE 450	*S355*	4.30	0.0	Beam
171	586	587	HEA 340	*S355*	8.80	180.0	Column
172	585	588	HEA 260	*S355*	8.80	180.0	Column
173	584	589	HEA 260	*S355*	8.80	180.0	Column
174	1	590	HEA 260	*S355*	8.80	180.0	Column
175	591	592	HEA 260	*S355*	8.80	180.0	Column
176	593	594	HEA 260	*S355*	8.80	180.0	Column
177	595	596	HEA 260	*S355*	8.80	180.0	Column
178	597	598	HEA 260	*S355*	8.80	180.0	Column
179	44	599	HEA 260	*S355*	8.80	180.0	Column
181	1	44	IPE 450	*S355*	5.10	0.0	Beam
183	586	45	HEA 340	*S355*	7.40	0.0	Beam
184	44	597	IPE 450	*S355*	2.10	0.0	Beam
185	604	43	HEA 340	*S355*	7.40	0.0	Beam
186	597	591	IPE 450	*S355*	2.30	0.0	Beam
187	591	585	IPE 450	*S355*	2.10	0.0	Beam
191	597	595	IPE 450	*S355*	3.00	0.0	Beam
192	593	591	IPE 450	*S355*	3.00	0.0	Beam
193	604	602	IPE 450	*S355*	4.30	0.0	Beam
194	602	608	IPE 450	*S355*	5.10	0.0	Beam
195	608	41	IPE 450	*S355*	6.50	0.0	Beam
196	41	42	IPE 450	*S355*	5.10	0.0	Beam
197	621	609	IPE 450	*S355*	2.30	0.0	Beam
200	609	42	IPE 450	*S355*	2.10	0.0	Beam
201	595	616	IPE 450	*S355*	2.10	0.0	Column
202	593	617	IPE 450	*S355*	2.10	0.0	Column
206	619	612	IPE 450	*S355*	4.30	0.0	Beam
207	603	613	IPE 450	*S355*	4.30	0.0	Beam
208	609	614	IPE 450	*S355*	4.30	0.0	Beam
209	597	615	IPE 450	*S355*	4.30	0.0	Beam
366	1635	1636	SHS 200x1	*S355*	3.00	0.0	Beam
367	1637	1638	SHS 200x1	*S355*	3.00	0.0	Beam
368	1639	1640	SHS 200x1	*S355*	3.00	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
369	1641	1642	SHS 200x1	*S355*	3.00	0.0	Beam
370	1643	1644	SHS 200x1	*S355*	3.00	0.0	Beam
371	1645	1646	SHS 200x1	*S355*	3.00	0.0	Beam
372	1647	1648	SHS 200x1	*S355*	3.00	0.0	Beam
373	1649	1650	SHS 200x1	*S355*	3.00	0.0	Beam
374	1651	1652	SHS 200x1	*S355*	3.00	0.0	Beam
375	1653	1654	SHS 200x1	*S355*	3.00	0.0	Beam
376	1655	1653	RHS 400x	*S355*	39.56	0.0	Beam
377	26	1654	RHS 400x	*S355*	39.56	0.0	Beam
378	1635	1657	SHS 200x1	*S355*	3.30	0.0	Beam
379	1636	1658	SHS 200x1	*S355*	3.30	0.0	Beam
380	1637	1659	SHS 200x1	*S355*	3.30	0.0	Beam
381	1638	1660	SHS 200x1	*S355*	3.30	0.0	Beam
382	1639	1661	SHS 200x1	*S355*	3.30	0.0	Beam
383	1640	1662	SHS 200x1	*S355*	3.30	0.0	Beam
384	1641	1663	SHS 200x1	*S355*	3.30	0.0	Beam
385	1642	1664	SHS 200x1	*S355*	3.30	0.0	Beam
386	1643	1665	SHS 200x1	*S355*	3.30	0.0	Beam
387	1644	1666	SHS 200x1	*S355*	3.30	0.0	Beam
388	1645	1667	SHS 200x1	*S355*	3.30	0.0	Beam
389	1646	1668	SHS 200x1	*S355*	3.30	0.0	Beam
390	1647	1669	SHS 200x1	*S355*	3.30	0.0	Beam
391	1648	1670	SHS 200x1	*S355*	3.30	0.0	Beam
392	1649	1671	SHS 200x1	*S355*	3.30	0.0	Beam
393	1650	1672	SHS 200x1	*S355*	3.30	0.0	Beam
394	1651	1673	SHS 200x1	*S355*	3.30	0.0	Beam
395	1652	1674	SHS 200x1	*S355*	3.30	0.0	Beam
396	1675	1653	RHS 400x	*S355*	3.30	0.0	Column
397	1654	1676	RHS 400x	*S355*	3.30	0.0	Column
398	1657	1658	SHS 200x1	*S355*	3.00	0.0	Beam
399	1659	1660	SHS 200x1	*S355*	3.00	0.0	Beam
400	1661	1662	SHS 200x1	*S355*	3.00	0.0	Beam
401	1663	1664	SHS 200x1	*S355*	3.00	0.0	Beam
402	1665	1666	SHS 200x1	*S355*	3.00	0.0	Beam
403	1667	1668	SHS 200x1	*S355*	3.00	0.0	Beam
404	1669	1670	SHS 200x1	*S355*	3.00	0.0	Beam
405	1671	1672	SHS 200x1	*S355*	3.00	0.0	Beam
406	1673	1674	SHS 200x1	*S355*	3.00	0.0	Beam
407	1675	1676	SHS 200x1	*S355*	3.00	0.0	Beam
408	1677	1675	RHS 400x	*S355*	39.56	0.0	Beam
409	20	1676	RHS 400x	*S355*	39.56	0.0	Beam
411	1651	1671	SHS 200x1	*S355*	4.66	0.0	Beam
412	1671	1647	SHS 200x1	*S355*	4.66	0.0	Beam
413	1647	1667	SHS 200x1	*S355*	4.66	0.0	Beam
414	1667	1643	SHS 200x1	*S355*	4.66	0.0	Beam
415	1643	1663	SHS 200x1	*S355*	4.66	0.0	Beam
416	1663	1639	SHS 200x1	*S355*	4.72	0.0	Beam
417	1639	1659	SHS 200x1	*S355*	4.66	0.0	Beam
418	1659	1635	SHS 200x1	*S355*	4.66	0.0	Beam
419	1635	1677	SHS 200x1	*S355*	4.66	0.0	Beam
420	1677	1655	SHS 200x1	*S355*	3.30	0.0	Beam
422	20	1636	SHS 200x1	*S355*	4.66	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
423	1636	1660	SHS 200x1	*S355*	4.66	0.0	Beam
424	1660	1640	SHS 200x1	*S355*	4.66	0.0	Beam
425	1640	1664	SHS 200x1	*S355*	4.72	0.0	Beam
426	1664	1644	SHS 200x1	*S355*	4.66	0.0	Beam
427	1644	1668	SHS 200x1	*S355*	4.66	0.0	Beam
428	1668	1648	SHS 200x1	*S355*	4.66	0.0	Beam
429	1648	1672	SHS 200x1	*S355*	4.66	0.0	Beam
430	1672	1652	SHS 200x1	*S355*	4.66	0.0	Beam
433	1674	1671	SHS 200x1	*S355*	4.45	0.0	Beam
434	1671	1670	SHS 200x1	*S355*	4.45	0.0	Beam
435	1670	1667	SHS 200x1	*S355*	4.45	0.0	Beam
436	1667	1666	SHS 200x1	*S355*	4.45	0.0	Beam
437	1666	1663	SHS 200x1	*S355*	4.45	0.0	Beam
438	1663	1662	SHS 200x1	*S355*	4.51	0.0	Beam
439	1662	1659	SHS 200x1	*S355*	4.45	0.0	Beam
440	1659	1658	SHS 200x1	*S355*	4.45	0.0	Beam
441	1658	1677	SHS 200x1	*S355*	4.45	0.0	Beam
443	45	2142	HEA 340	*S355*	8.80	-180.0	Column
444	2143	2144	SHS 200x1	*S355*	6.00	0.0	Beam
445	2145	2146	SHS 200x1	*S355*	6.00	0.0	Beam
446	2147	2148	SHS 200x1	*S355*	6.00	0.0	Beam
447	2149	2150	SHS 200x1	*S355*	6.00	0.0	Beam
448	2151	2152	SHS 200x1	*S355*	6.00	0.0	Beam
449	2153	2154	SHS 200x1	*S355*	6.00	0.0	Beam
450	2155	2156	SHS 200x1	*S355*	6.00	0.0	Beam
451	2157	2158	SHS 200x1	*S355*	6.00	0.0	Beam
452	2159	2160	SHS 200x1	*S355*	6.00	0.0	Beam
453	2161	2162	SHS 200x1	*S355*	6.00	0.0	Beam
454	613	2161	RHS 400x	*S355*	33.70	0.0	Beam
456	2143	2163	SHS 200x1	*S355*	3.30	180.0	Beam
458	2145	2165	SHS 200x1	*S355*	3.30	180.0	Beam
460	2147	2167	SHS 200x1	*S355*	3.30	180.0	Beam
462	2149	2169	SHS 200x1	*S355*	3.30	180.0	Beam
464	2151	2171	SHS 200x1	*S355*	3.30	180.0	Beam
466	2153	2173	SHS 200x1	*S355*	3.30	180.0	Beam
468	2155	2175	SHS 200x1	*S355*	3.30	180.0	Beam
470	2157	2177	SHS 200x1	*S355*	3.30	180.0	Beam
472	2159	2179	SHS 200x1	*S355*	3.30	180.0	Beam
474	2181	2161	RHS 400x	*S355*	3.30	180.0	Column
476	2163	2164	SHS 200x1	*S355*	6.00	0.0	Beam
477	2165	2166	SHS 200x1	*S355*	6.00	0.0	Beam
478	2167	2168	SHS 200x1	*S355*	6.00	0.0	Beam
479	2169	2170	SHS 200x1	*S355*	6.00	0.0	Beam
480	2171	3156	SHS 200x1	*S355*	6.00	0.0	Beam
481	2173	2174	SHS 200x1	*S355*	6.00	0.0	Beam
482	2175	3155	SHS 200x1	*S355*	6.00	0.0	Beam
483	2177	2178	SHS 200x1	*S355*	6.00	0.0	Beam
484	2179	3154	SHS 200x1	*S355*	6.00	0.0	Beam
485	2181	2182	SHS 200x1	*S355*	6.00	0.0	Beam
486	612	2181	RHS 400x	*S355*	33.70	0.0	Beam
488	2181	2159	SHS 200x1	*S355*	4.72	0.0	Beam
489	2159	2177	SHS 200x1	*S355*	4.72	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
490	2177	2155	SHS 200x1	*S355*	4.72	0.0	Beam
491	2155	2173	SHS 200x1	*S355*	4.72	0.0	Beam
492	2173	2151	SHS 200x1	*S355*	4.72	0.0	Beam
493	2151	2169	SHS 200x1	*S355*	4.72	0.0	Beam
494	2169	2147	SHS 200x1	*S355*	4.72	0.0	Beam
495	2147	2165	SHS 200x1	*S355*	4.72	0.0	Beam
496	2165	2143	SHS 200x1	*S355*	4.72	0.0	Beam
497	2143	612	SHS 200x1	*S355*	4.72	0.0	Beam
498	612	613	SHS 200x1	*S355*	3.30	180.0	Beam
510	2181	3154	SHS 200x1	*S355*	6.88	0.0	Beam
511	3154	2177	SHS 200x1	*S355*	6.88	0.0	Beam
512	2177	3155	SHS 200x1	*S355*	6.88	0.0	Beam
513	3155	2173	SHS 200x1	*S355*	6.88	0.0	Beam
514	2173	3156	SHS 200x1	*S355*	6.88	0.0	Beam
515	3156	2169	SHS 200x1	*S355*	6.88	0.0	Beam
516	2169	2168	SHS 200x1	*S355*	6.88	0.0	Beam
517	2168	2165	SHS 200x1	*S355*	6.88	0.0	Beam
518	2165	2164	SHS 200x1	*S355*	6.88	0.0	Beam
519	2164	612	SHS 200x1	*S355*	6.88	0.0	Beam
533	43	2162	RHS 400x	*S355*	33.70	0.0	Beam
535	2144	2164	SHS 200x1	*S355*	3.30	-180.0	Beam
537	2146	2166	SHS 200x1	*S355*	3.30	-180.0	Beam
539	2148	2168	SHS 200x1	*S355*	3.30	180.0	Beam
541	2150	2170	SHS 200x1	*S355*	3.30	180.0	Beam
543	2152	3156	SHS 200x1	*S355*	3.30	180.0	Beam
545	2154	2174	SHS 200x1	*S355*	3.30	180.0	Beam
547	2156	3155	SHS 200x1	*S355*	3.30	180.0	Beam
549	2158	2178	SHS 200x1	*S355*	3.30	180.0	Beam
551	2160	3154	SHS 200x1	*S355*	3.30	180.0	Beam
553	2162	2182	RHS 400x	*S355*	3.30	180.0	Column
565	45	2182	RHS 400x	*S355*	33.70	0.0	Beam
578	45	2144	SHS 200x1	*S355*	4.72	0.0	Beam
579	2144	2166	SHS 200x1	*S355*	4.72	0.0	Beam
580	2166	2148	SHS 200x1	*S355*	4.72	0.0	Beam
581	2148	2170	SHS 200x1	*S355*	4.72	0.0	Beam
582	2170	2152	SHS 200x1	*S355*	4.72	0.0	Beam
583	2152	2174	SHS 200x1	*S355*	4.72	0.0	Beam
584	2174	2156	SHS 200x1	*S355*	4.72	0.0	Beam
585	2156	2178	SHS 200x1	*S355*	4.72	0.0	Beam
586	2178	2160	SHS 200x1	*S355*	4.72	0.0	Beam
587	2160	2182	SHS 200x1	*S355*	4.72	0.0	Beam
599	611	45	IPE 450	*S355*	4.30	0.0	Beam
601	3149	1677	IPE 450	*S355*	4.30	0.0	Beam
602	3152	1655	IPE 450	*S355*	4.30	0.0	Beam
603	610	43	IPE 450	*S355*	4.30	0.0	Beam
604	3159	3160	SHS 200x1	*S355*	3.00	0.0	Beam
605	3159	3161	SHS 200x1	*S355*	3.30	0.0	Beam
606	3160	3162	SHS 200x1	*S355*	3.30	0.0	Beam
607	3161	3162	SHS 200x1	*S355*	3.00	0.0	Beam
608	3182	3183	SHS 200x1	*S355*	3.00	0.0	Beam
609	3182	3184	SHS 200x1	*S355*	3.30	0.0	Beam
610	3183	3185	SHS 200x1	*S355*	3.30	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
611	3184	3185	SHS 200x1	*S355*	3.00	0.0	Beam
615	3205	3206	IPE 450	*S355*	5.10	0.0	Beam
616	3207	3208	IPE 450	*S355*	5.10	0.0	Beam
617	3209	3210	IPE 450	*S355*	5.10	0.0	Beam
618	3211	3212	IPE 450	*S355*	5.10	0.0	Beam
619	3213	3214	IPE 450	*S355*	5.10	0.0	Beam
620	3215	3216	IPE 450	*S355*	5.10	0.0	Beam
621	3217	3218	IPE 450	*S355*	5.10	0.0	Beam
622	3219	3220	IPE 450	*S355*	5.10	0.0	Beam
637	3239	3240	IPE 450	*S355*	2.10	0.0	Beam
640	3243	3244	IPE 450	*S355*	2.10	0.0	Beam
641	3245	3246	IPE 450	*S355*	2.10	0.0	Beam
644	3249	3250	IPE 450	*S355*	2.10	0.0	Beam
652	3239	3258	IPE 450	*S355*	2.10	0.0	Column
654	3243	3260	IPE 450	*S355*	2.10	0.0	Column
655	3245	3261	IPE 450	*S355*	2.10	0.0	Column
657	3249	3263	IPE 450	*S355*	2.10	0.0	Column
659	3206	3207	IPE 450	*S355*	6.50	0.0	Beam
660	3210	3211	IPE 450	*S355*	6.50	0.0	Beam
661	3214	3215	IPE 450	*S355*	6.50	0.0	Beam
662	3218	3219	IPE 450	*S355*	6.50	0.0	Beam
663	37	27	IPE 450	*S355*	2.10	0.0	Beam
665	621	602	IPE 450	*S355*	2.10	0.0	Beam
666	3266	3267	IPE 450	*S355*	2.30	0.0	Beam
667	3212	3208	IPE 450	*S355*	4.70	0.0	Beam
668	3267	3208	IPE 450	*S355*	2.10	0.0	Beam
669	3268	3212	IPE 450	*S355*	2.10	0.0	Beam
670	3266	3205	IPE 450	*S355*	2.10	0.0	Beam
671	3268	3270	IPE 450	*S355*	2.30	0.0	Beam
672	3270	3209	IPE 450	*S355*	2.10	0.0	Beam
673	3278	3279	IPE 450	*S355*	2.30	0.0	Beam
674	3220	3216	IPE 450	*S355*	4.70	0.0	Beam
675	3279	3216	IPE 450	*S355*	2.10	0.0	Beam
676	3280	3220	IPE 450	*S355*	2.10	0.0	Beam
677	3278	3213	IPE 450	*S355*	2.10	0.0	Beam
678	3280	3282	IPE 450	*S355*	2.30	0.0	Beam
679	3282	3217	IPE 450	*S355*	2.10	0.0	Beam
1034	4893	4892	IPE 450	*S355*	2.50	0.0	Beam
1035	4894	4893	IPE 450	*S355*	7.40	0.0	Beam
1037	4897	4896	IPE 450	*S355*	2.50	0.0	Beam
1038	4898	4897	IPE 450	*S355*	7.40	0.0	Beam
1039	4891	3205	IPE 450	*S355*	4.30	0.0	Beam
1040	4894	3209	IPE 450	*S355*	4.30	0.0	Beam
1041	4895	3213	IPE 450	*S355*	4.30	0.0	Beam
1042	4898	3217	IPE 450	*S355*	4.30	0.0	Beam
1043	3211	3207	IPE 450	*S355*	4.70	0.0	Beam
1044	3219	3215	IPE 450	*S355*	4.70	0.0	Beam
1060	1675	3182	SHS 200x1	*S355*	4.66	0.0	Beam
1061	3182	3161	SHS 200x1	*S355*	4.66	0.0	Beam
1062	3161	1651	SHS 200x1	*S355*	4.66	0.0	Beam
1063	1652	3162	SHS 200x1	*S355*	4.66	0.0	Beam
1064	3162	3183	SHS 200x1	*S355*	4.66	0.0	Beam

Bar	Node 1	Node 2	Section	Material	Length (m)	Gamma (Deg)	Type
1065	3183	1676	SHS 200x1	*S355*	4.66	0.0	Beam
1066	1675	3185	SHS 200x1	*S355*	4.45	0.0	Beam
1067	3185	3161	SHS 200x1	*S355*	4.45	0.0	Beam
1068	3161	1674	SHS 200x1	*S355*	4.45	0.0	Beam

Data - Panels

Panel	Thickness	Material	Meshing type	Reinforcement type
167	Comflor60_150THK_1.0	Conc_Normal_We	Coons	RC floor 1
442	Comflor60_150THK_1.0	Conc_Normal_We	Coons	RC floor 1
521	Comflor60_150THK_1.0	Conc_Normal_We	Coons	RC floor 1
613	TH50_Steel_mesh_dec	STEEL	Coons	RC floor
614	TH50_Steel_mesh_dec	STEEL	Coons	RC floor
680	TH50_Steel_mesh_dec	STEEL	Coons	RC floor
681	TH50_Steel_mesh_dec	STEEL	Coons	RC floor
682	TH50_Steel_mesh_dec	STEEL	Coons	RC floor
683	TH50_Steel_mesh_dec	STEEL	Coons	RC floor

Data - Sections

	Section name	Bar list	AX (mm2)	AY (mm2)	AZ (mm2)	IX (mm4)
	IPE 450	1to4 15 16 18to2	9880	5548	4230	667000
	RHS 400x300x16	71 72 114 115 1	20300	9600	12800	587300000
	SHS 200x10	58to66 73to90 1	7550	4000	4000	70200000
	HEA 260	6to13 172to179	8680	6500	1875	542000
	HEA 340	5 14 17 23 24 57	13300	9900	3135	1310000
	SHS 120x5	27 39 47to56	2290	1200	1200	7750000

	IY (mm4)	IZ (mm4)
	337400000	16760000
	443500000	285350000
	45250000	45250000
	104500000	36680000
	276900000	74360000
	5030000	5030000

Data - Supports

	Support name	List of nodes	List of edges	List of objects
	Pinned	6to9 11to21By2 32 134 5		
	Slider	96 97 1653 1654 2161 21		

	Support conditions
	UX UY UZ

	Support conditions
	UY UZ

Loads - Cases

Case	Label	Case name	Nature	Analysis type
1	DL1	DL1	Structural	Static - Linear
2	LL1	LL1	Category A	Static - Linear
3		ULS	Structural	Linear Combination
4		SLS	Structural	Linear Combination
5	WIND1	WIND1	wind	Static - Linear
6	SN1	SN1	snow	Static - Linear
7	TEMP1	TEMP d0	temperature	Static - Linear
8	TEMP2	TEMP d10	temperature	Static - Linear
9		ULS Wind Leading + Snow + Temp	Structural	Linear Combination
10		ULS Wind Leading + Snow + Temp	Structural	Linear Combination
11		ULS 6.10 G+Q+Snow+Tempd0	Structural	Linear Combination
12		ULS 6.10 G+Q+Snow+Tempd10	Structural	Linear Combination
13	13	Footfall		Footfall
14	SDL1	SDL1	Structural	Static - Linear
15	FDL1	Facade	Structural	Static - Linear
16	MOD16	Modal		Modal
17		SLS DL	Structural	Linear Combination
18		SLS LL	Structural	Linear Combination
19		ULS DL	Structural	Linear Combination
20		ULS LL	Structural	Linear Combination
21		ULS WL	Structural	Linear Combination

Loads - Values

	Case	Load type	List	Load values
	1	uniform load		PZ=-7.60(kN/m)
	1	uniform load		PZ=-16.00(kN/m)
	1	self-weight	1to66 70to90	PZ Negative Factor=1.00
	2	(FE) uniform	680to683	PZ=-5.00(kN/m2)
	2	(FE) uniform	613	PZ=-5.00(kN/m2)
	2	uniform load	101to109 113	PZ=-1.90(kN/m)
	2	uniform load		PZ=-0.60(kN/m)
	2	uniform load		PZ=-1.30(kN/m)
	2	(FE) uniform	167 442 521	PZ=-5.00(kN/m2)
	5	uniform load	72 115 377 4	PX=2.56(kN/m)
	5	uniform load	71 114 376 4	PX=1.26(kN/m)
	5	uniform load	1 2 31 40 70	PY=-1.26(kN/m)
	5	uniform load	3 4 29 30 169	PX=1.26(kN/m)
	6	(FE) uniform	614	PZ=-1.00(kN/m2)
	6	uniform load	113 407 485	PZ=-1.70(kN/m)
	6	uniform load	101to109 398	PZ=-3.37(kN/m)
	6	trapezoidal load (2p)	17 183	PZ2=-1.70(kN/m) PZ1=-1.70(kN/m) X2=4.50(m) X1=1.50(m) glo
	7	thermal load	1to26 28to38	

	Case	Load type	List	Load values
	8	thermal load	1to26 28to38	TX=10.00(°C)
	14	(FE) uniform	613	PZ=-3.70(kN/m2)
	14	(FE) uniform	167 442 521	PZ=-3.70(kN/m2)
	14	uniform load	17 113 183 4	PZ=-1.85(kN/m)
	14	uniform load	101to109 476	PZ=-3.70(kN/m)
	14	(FE) uniform	680to683	PZ=-3.70(kN/m2)
	14	uniform load	407	PZ=-1.65(kN/m)
	14	uniform load	398to406 607	PZ=-3.30(kN/m)
	15	uniform load	28 37 619 62	PZ=-0.90(kN/m)
	15	uniform load	16 22 615 61	PZ=-1.00(kN/m)
	15	uniform load	23 24 29to31	PZ=-1.85(kN/m)
	15	uniform load	1043	PZ=-1.00(kN/m)
	15	uniform load	1044	PZ=-0.90(kN/m)

Manual Combinations

- Cases: 3 4 9to13 17to21 Active modes: CQC

Combinations	Name	Analysis type	Combination	Case nature
3 (C)	ULS	Linear Combinati	ULS	Structural
4 (C)	SLS	Linear Combinati	SLS	Structural
9 (C)	ULS Wind Leadi	Linear Combinati	ULS	Structural
10 (C)	ULS Wind Leadi	Linear Combinati	ULS	Structural
11 (C)	ULS 6.10 G+Q+	Linear Combinati	ULS	Structural
12 (C)	ULS 6.10 G+Q+	Linear Combinati	ULS	Structural
13	Footfall	Footfall		
17 (C)	SLS DL	Linear Combinati	SLS	Structural
18 (C)	SLS LL	Linear Combinati	SLS	Structural
19 (C)	ULS DL	Linear Combinati	ULS	Structural
20 (C)	ULS LL	Linear Combinati	ULS	Structural
21 (C)	ULS WL	Linear Combinati	ULS	Structural

Combinations	Definition
3 (C)	(1+14+15)*1.35+(2+6)*1.50
4 (C)	(1+14+15)*1.00+2*0.70
9 (C)	(1+14)*1.35+6*0.50+7*0.60+5*1.50+2*0.70
10 (C)	(1+15+14)*1.35+6*0.50+8*0.60+5*1.50+2*0.70
11 (C)	(1+14+15)*1.35+2*1.50+(5+6)*0.50+7*0.60
12 (C)	(1+15+14)*1.35+2*1.50+(5+6)*0.50+8*0.60
13	
17 (C)	(1+14+15)*1.00
18 (C)	(2+5+6)*1.00
19 (C)	(1+14+15)*1.35
20 (C)	(2+6)*1.50
21 (C)	5*1.50

Stories - Displacements

Case/Story	UX (mm)	UY (mm)	dr UX (mm)	dr UY (mm)	d UX	d UY
1/ 1	N/A	N/A	N/A	N/A	N/A	N/A
2/ 1	N/A	N/A	N/A	N/A	N/A	N/A
3 (C)/ 1	N/A	N/A	N/A	N/A	N/A	N/A
4 (C)/ 1	N/A	N/A	N/A	N/A	N/A	N/A
5/ 1	N/A	N/A	N/A	N/A	N/A	N/A
6/ 1	N/A	N/A	N/A	N/A	N/A	N/A
7/ 1	N/A	N/A	N/A	N/A	N/A	N/A
8/ 1	N/A	N/A	N/A	N/A	N/A	N/A
9 (C)/ 1	N/A	N/A	N/A	N/A	N/A	N/A
10 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
11 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
12 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
13/ 1	N/A	N/A	N/A	N/A	N/A	N/A
14/ 1	N/A	N/A	N/A	N/A	N/A	N/A
15/ 1	N/A	N/A	N/A	N/A	N/A	N/A
17 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
18 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
19 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
20 (C)/	N/A	N/A	N/A	N/A	N/A	N/A
21 (C)/	N/A	N/A	N/A	N/A	N/A	N/A

Case/Story	Max UX (mm)	Max UY (mm)	Min UX (mm)	Min UY (mm)
1/ 1	N/A	N/A	N/A	N/A
2/ 1	N/A	N/A	N/A	N/A
3 (C)/ 1	N/A	N/A	N/A	N/A
4 (C)/ 1	N/A	N/A	N/A	N/A
5/ 1	N/A	N/A	N/A	N/A
6/ 1	N/A	N/A	N/A	N/A
7/ 1	N/A	N/A	N/A	N/A
8/ 1	N/A	N/A	N/A	N/A
9 (C)/ 1	N/A	N/A	N/A	N/A
10 (C)/	N/A	N/A	N/A	N/A
11 (C)/	N/A	N/A	N/A	N/A
12 (C)/	N/A	N/A	N/A	N/A
13/ 1	N/A	N/A	N/A	N/A
14/ 1	N/A	N/A	N/A	N/A
15/ 1	N/A	N/A	N/A	N/A
17 (C)/	N/A	N/A	N/A	N/A
18 (C)/	N/A	N/A	N/A	N/A
19 (C)/	N/A	N/A	N/A	N/A
20 (C)/	N/A	N/A	N/A	N/A
21 (C)/	N/A	N/A	N/A	N/A

Reactions ULS: global extremes

in the coordinate system: global - Cases: 1to3 5to15 19to21 Active modes: CQC

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	26.44	1410.75	1710.60	0.00	0.00	0.00
Node	9	2161	134	2162	1653	2161
Case	3 (C)	10 (C)	3 (C)	3 (C)	10 (C)	21 (C)
MIN	-151.65	-1547.56	-400.28	-0.00	-0.00	-0.00
Node	9	96	589	1654	2162	2162
Case	21 (C)	21 (C)	9 (C)	3 (C)	21 (C)	21 (C)

Displacements SLS: global extremes

- Cases: 1 2 4to8 13to15 17 18 Active modes: CQC

	UX (mm)	UY (mm)	UZ (mm)	RX (Rad)	RY (Rad)	RZ (Rad)
MAX	43	6	5	0.004	0.003	0.002
Node	1675	2182	1667	2778	11	29
Case	18 (C)	4 (C)	8	4 (C)	18 (C)	18 (C)
MIN	-1	-4	-34	-0.003	-0.002	-0.001
Node	3210	1195	2672	3072	1019	589
Case	8	18 (C)	4 (C)	4 (C)	4 (C)	4 (C)

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
1 B	OK	IPE 450	S355	25.43	114.11	0.34	10 ULS Wind Leading + Snow + T	0.01
2 B	OK	IPE 450	S355	35.17	157.82	0.30	10 ULS Wind Leading + Snow + T	0.00
3 B	OK	IPE 450	S355	27.60	123.83	0.13	12 ULS 6.10 G+Q+Snow+Tempd	0.01
4 B	OK	IPE 450	S355	23.27	104.40	0.10	8 TEMP d10	0.01
5 B	OK	HEA 340	S355	60.99	117.69	0.72	12 ULS 6.10 G+Q+Snow+Tempd	-
6 B	OK	HEA 260	S355	80.20	135.37	0.75	10 ULS Wind Leading + Snow + T	-
7 B	OK	HEA 260	S355	80.20	135.37	0.77	10 ULS Wind Leading + Snow + T	-
8 B	OK	HEA 260	S355	80.20	135.37	0.48	3 ULS	-
9 B	OK	HEA 260	S355	80.20	135.37	0.44	10 ULS Wind Leading + Snow + T	-
10 B	OK	HEA 260	S355	80.20	135.37	0.17	3 ULS	-
11 B	OK	HEA 260	S355	80.20	135.37	0.20	21 ULS WL	-
12 B	OK	HEA 260	S355	80.20	135.37	0.51	9 ULS Wind Leading + Snow + Tempd0	-
13 B	OK	HEA 260	S355	80.20	135.37	0.53	9 ULS Wind Leading + Snow + Tempd0	-
14 B	OK	HEA 340	S355	60.99	117.69	0.94	11 ULS 6.10 G+Q+Snow+Tempd	-
15 B	OK	IPE 450	S355	27.60	123.83	0.11	12 ULS 6.10 G+Q+Snow+Tempd	0.00
16 B	OK	IPE 450	S355	23.81	106.83	0.40	9 ULS Wind Leading + Snow + Tempd0	0.02
17 B	OK	HEA 340	S355	54.06	104.32	0.61	3 ULS	0.00
18 B	OK	IPE 450	S355	11.36	50.99	0.09	10 ULS Wind Leading + Snow + T	0.00
19 B	OK	IPE 450	S355	23.27	104.40	0.44	3 ULS	0.00
20 B	OK	IPE 450	S355	12.45	55.84	0.07	12 ULS 6.10 G+Q+Snow+Tempd	0.00
21 B	OK	IPE 450	S355	11.36	50.99	0.07	8 TEMP d10	0.01
22 B	OK	IPE 450	S355	18.40	82.55	0.26	21 ULS WL	0.01
23 B	OK	HEA 340	S355	30.49	58.84	0.58	9 ULS Wind Leading + Snow + Tempd0	0.01
24 B	OK	HEA 340	S355	23.56	45.47	0.93	10 ULS Wind Leading + Snow + T	0.02
25 B	OK	IPE 450	S355	16.23	72.84	0.08	12 ULS 6.10 G+Q+Snow+Tempd	0.00
26 B	OK	IPE 450	S355	16.23	72.84	0.11	10 ULS Wind Leading + Snow + T	0.00
28 B	OK	IPE 450	S355	23.81	106.83	0.60	10 ULS Wind Leading + Snow + T	0.02
29 B	OK	IPE 450	S355	23.27	104.40	0.42	10 ULS Wind Leading + Snow + T	0.00
30 B	OK	IPE 450	S355	27.60	123.83	0.18	10 ULS Wind Leading + Snow + T	0.08
31 B	OK	IPE 450	S355	35.17	157.82	0.14	10 ULS Wind Leading + Snow + T	0.11
32 B	OK	IPE 450	S355	27.60	123.83	0.08	11 ULS 6.10 G+Q+Snow+Tempd	0.02
33 B	OK	IPE 450	S355	12.45	55.84	0.10	10 ULS Wind Leading + Snow + T	0.01
34 B	OK	IPE 450	S355	23.27	104.40	0.18	12 ULS 6.10 G+Q+Snow+Tempd	0.00
35 B	OK	IPE 450	S355	23.27	104.40	0.34	3 ULS	0.00
36 B	OK	IPE 450	S355	11.36	50.99	0.11	21 ULS WL	0.00
37 B	OK	IPE 450	S355	18.40	82.55	0.37	10 ULS Wind Leading + Snow + T	0.01
38 B	OK	IPE 450	S355	23.27	104.40	0.29	10 ULS Wind Leading + Snow + T	0.00
40 B	OK	IPE 450	S355	25.43	114.11	0.20	10 ULS Wind Leading + Snow + T	0.01
41 B	OK	IPE 450	S355	25.43	114.11	0.24	10 ULS Wind Leading + Snow + T	0.01
42 B	OK	IPE 450	S355	13.53	60.70	0.43	10 ULS Wind Leading + Snow + T	0.00
43 B	OK	IPE 450	S355	25.43	114.11	0.18	10 ULS Wind Leading + Snow + T	0.01
44 B	OK	IPE 450	S355	13.53	60.70	0.22	10 ULS Wind Leading + Snow + T	0.01
45 B	OK	IPE 450	S355	23.27	104.40	0.37	3 ULS	0.00
46 B	OK	IPE 450	S355	23.27	104.40	0.24	3 ULS	0.00
57 B	OK	HEA 340	S355	60.99	117.69	0.73	3 ULS	-
58 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
59 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
60 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
61 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
62 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
63 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
64 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
65 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
66 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
70 B	OK	SHS 200x10	S355	38.75	38.75	0.13	10 ULS Wind Leading + Snow + T	0.01
71 B	OK	RHS 400x300x16	S355	228.00	284.24	0.18	10 ULS Wind Leading + Snow + T	0.02
72 B	OK	RHS 400x300x16	S355	228.00	284.24	0.32	10 ULS Wind Leading + Snow + T	0.02
73 B	OK	SHS 200x10	S355	42.63	42.63	0.10	3 ULS	0.01
74 B	OK	SHS 200x10	S355	42.63	42.63	0.10	3 ULS	0.02
75 B	OK	SHS 200x10	S355	42.63	42.63	0.06	3 ULS	0.01
76 B	OK	SHS 200x10	S355	42.63	42.63	0.05	12 ULS 6.10 G+Q+Snow+Tempd	0.01
77 B	OK	SHS 200x10	S355	42.63	42.63	0.04	11 ULS 6.10 G+Q+Snow+Tempd	0.01
78 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.00
79 B	OK	SHS 200x10	S355	42.63	42.63	0.05	3 ULS	0.00
80 B	OK	SHS 200x10	S355	42.63	42.63	0.04	12 ULS 6.10 G+Q+Snow+Tempd	0.00
81 B	OK	SHS 200x10	S355	42.63	42.63	0.02	3 ULS	0.00
82 B	OK	SHS 200x10	S355	42.63	42.63	0.03	10 ULS Wind Leading + Snow + T	0.00
83 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.00
84 B	OK	SHS 200x10	S355	42.63	42.63	0.04	12 ULS 6.10 G+Q+Snow+Tempd	0.00
85 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.01
86 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.01
87 B	OK	SHS 200x10	S355	42.63	42.63	0.06	3 ULS	0.01
88 B	OK	SHS 200x10	S355	42.63	42.63	0.06	3 ULS	0.01
89 B	OK	SHS 200x10	S355	42.63	42.63	0.09	3 ULS	0.01
90 B	OK	SHS 200x10	S355	42.63	42.63	0.09	3 ULS	0.01
101 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
102 B	OK	SHS 200x10	S355	38.75	38.75	0.06	3 ULS	0.00
103 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
104 B	OK	SHS 200x10	S355	38.75	38.75	0.06	3 ULS	0.00
105 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
106 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
107 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
108 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
109 B	OK	SHS 200x10	S355	38.75	38.75	0.07	10 ULS Wind Leading + Snow + T	0.00
113 B	OK	SHS 200x10	S355	38.75	38.75	0.16	10 ULS Wind Leading + Snow + T	0.00
114 B	OK	RHS 400x300x16	S355	228.00	284.24	0.31	3 ULS	0.01
115 B	OK	RHS 400x300x16	S355	228.00	284.24	0.32	3 ULS	0.01
119 B	OK	SHS 200x10	S355	60.93	60.93	0.29	3 ULS	0.01
120 B	OK	SHS 200x10	S355	60.93	60.93	0.25	3 ULS	0.01
121 B	OK	SHS 200x10	S355	60.93	60.93	0.17	3 ULS	0.00
122 B	OK	SHS 200x10	S355	60.93	60.93	0.11	3 ULS	0.01
123 B	OK	SHS 200x10	S355	60.93	60.93	0.03	3 ULS	0.00
124 B	OK	SHS 200x10	S355	60.93	60.93	0.04	9 ULS Wind Leading + Snow + Tempd0	0.00
125 B	OK	SHS 200x10	S355	60.93	60.93	0.10	12 ULS 6.10 G+Q+Snow+Tempd	0.01

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
126 B	OK	SHS 200x10	S355	60.93	60.93	0.16	3 ULS	0.01
127 B	OK	SHS 200x10	S355	60.93	60.93	0.24	3 ULS	0.01
128 B	OK	SHS 200x10	S355	60.93	60.93	0.28	3 ULS	0.03
129 B	OK	SHS 200x10	S355	42.63	42.63	0.17	12 ULS 6.10 G+Q+Snow+Tempd	0.01
130 B	OK	SHS 200x10	S355	42.63	42.63	0.28	3 ULS	0.01
131 B	OK	SHS 200x10	S355	60.93	60.93	0.24	3 ULS	0.00
132 B	OK	SHS 200x10	S355	60.93	60.93	0.22	3 ULS	0.01
133 B	OK	SHS 200x10	S355	60.93	60.93	0.15	11 ULS 6.10 G+Q+Snow+Tempd	0.01
134 B	OK	SHS 200x10	S355	60.93	60.93	0.10	12 ULS 6.10 G+Q+Snow+Tempd	0.01
135 B	OK	SHS 200x10	S355	60.93	60.93	0.03	9 ULS Wind Leading + Snow + Tempd0	0.01
136 B	OK	SHS 200x10	S355	60.93	60.93	0.03	3 ULS	0.00
137 B	OK	SHS 200x10	S355	60.93	60.93	0.11	3 ULS	0.01
138 B	OK	SHS 200x10	S355	60.93	60.93	0.16	3 ULS	0.00
139 B	OK	SHS 200x10	S355	60.93	60.93	0.24	3 ULS	0.00
140 B	OK	SHS 200x10	S355	60.93	60.93	0.28	3 ULS	0.01
147 B	OK	SHS 200x10	S355	58.28	58.28	0.06	10 ULS Wind Leading + Snow + T	0.00
148 B	OK	SHS 200x10	S355	58.28	58.28	0.01	21 ULS WL	0.00
149 B	OK	SHS 200x10	S355	58.28	58.28	0.02	3 ULS	0.01
150 B	OK	SHS 200x10	S355	58.28	58.28	0.02	3 ULS	0.00
151 B	OK	SHS 200x10	S355	58.28	58.28	0.03	3 ULS	0.00
152 B	OK	SHS 200x10	S355	58.28	58.28	0.05	10 ULS Wind Leading + Snow + T	0.00
153 B	OK	SHS 200x10	S355	58.28	58.28	0.05	21 ULS WL	0.00
154 B	OK	SHS 200x10	S355	58.28	58.28	0.06	10 ULS Wind Leading + Snow + T	0.00
155 B	OK	SHS 200x10	S355	58.28	58.28	0.07	21 ULS WL	0.00
156 B	OK	SHS 200x10	S355	58.28	58.28	0.06	21 ULS WL	0.01
158 B	OK	RHS 400x300x16	S355	22.33	27.83	0.15	3 ULS	-
159 B	OK	RHS 400x300x16	S355	22.33	27.83	0.14	3 ULS	-
160 B	OK	IPE 450	S355	11.36	50.99	0.06	12 ULS 6.10 G+Q+Snow+Tempd	-
161 B	OK	IPE 450	S355	11.36	50.99	0.07	10 ULS Wind Leading + Snow + T	-
166 B	OK	HEA 340	S355	38.12	73.56	0.78	12 ULS 6.10 G+Q+Snow+Tempd	-
168 B	OK	IPE 450	S355	35.17	157.82	0.29	10 ULS Wind Leading + Snow + T	0.00
169 B	OK	IPE 450	S355	27.60	123.83	0.16	12 ULS 6.10 G+Q+Snow+Tempd	0.01
170 B	OK	IPE 450	S355	23.27	104.40	0.14	11 ULS 6.10 G+Q+Snow+Tempd	0.01
172 B	OK	HEA 260	S355	80.20	135.37	0.44	21 ULS WL	-
173 B	OK	HEA 260	S355	80.20	135.37	0.34	3 ULS	-
174 B	OK	HEA 260	S355	80.20	135.37	0.74	10 ULS Wind Leading + Snow + T	-
175 B	OK	HEA 260	S355	80.20	135.37	0.52	9 ULS Wind Leading + Snow + Tempd0	-
176 B	OK	HEA 260	S355	80.20	135.37	0.20	9 ULS Wind Leading + Snow + Tempd0	-
177 B	OK	HEA 260	S355	80.20	135.37	0.21	9 ULS Wind Leading + Snow + Tempd0	-
178 B	OK	HEA 260	S355	80.20	135.37	0.57	9 ULS Wind Leading + Snow + Tempd0	-
179 B	OK	HEA 260	S355	80.20	135.37	0.76	9 ULS Wind Leading + Snow + Tempd0	-
181 B	OK	IPE 450	S355	27.60	123.83	0.13	12 ULS 6.10 G+Q+Snow+Tempd	0.00
183 B	OK	HEA 340	S355	51.29	98.97	0.82	3 ULS	0.00
184 B	OK	IPE 450	S355	11.36	50.99	0.08	10 ULS Wind Leading + Snow + T	0.00
185 B	OK	HEA 340	S355	51.29	98.97	0.92	3 ULS	0.00
186 B	OK	IPE 450	S355	12.45	55.84	0.08	10 ULS Wind Leading + Snow + T	0.00
187 B	OK	IPE 450	S355	11.36	50.99	0.07	8 TEMP d10	0.01

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
191 B	OK	IPE 450	S355	16.23	72.84	0.12	10 ULS Wind Leading + Snow + T	0.00
192 B	OK	IPE 450	S355	16.23	72.84	0.09	10 ULS Wind Leading + Snow + T	0.00
193 B	OK	IPE 450	S355	23.27	104.40	0.41	3 ULS	0.00
194 B	OK	IPE 450	S355	27.60	123.83	0.22	10 ULS Wind Leading + Snow + T	0.02
195 B	OK	IPE 450	S355	35.17	157.82	0.19	10 ULS Wind Leading + Snow + T	0.07
196 B	OK	IPE 450	S355	27.60	123.83	0.08	3 ULS	0.02
197 B	OK	IPE 450	S355	12.45	55.84	0.08	10 ULS Wind Leading + Snow + T	0.00
200 B	OK	IPE 450	S355	11.36	50.99	0.07	8 TEMP d10	0.00
201 B	OK	IPE 450	S355	11.36	50.99	0.09	10 ULS Wind Leading + Snow + T	-
202 B	OK	IPE 450	S355	11.36	50.99	0.08	10 ULS Wind Leading + Snow + T	-
206 B	OK	IPE 450	S355	23.27	104.40	0.26	3 ULS	0.00
207 B	OK	IPE 450	S355	23.27	104.40	0.42	3 ULS	0.00
208 B	OK	IPE 450	S355	23.27	104.40	0.18	12 ULS 6.10 G+Q+Snow+Tempd	0.00
209 B	OK	IPE 450	S355	23.27	104.40	0.15	12 ULS 6.10 G+Q+Snow+Tempd	0.00
366 B	OK	SHS 200x10	S355	38.75	38.75	0.07	11 ULS 6.10 G+Q+Snow+Tempd	0.01
367 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
368 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
369 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
370 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
371 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
372 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
373 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
374 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
375 B	OK	SHS 200x10	S355	38.75	38.75	0.16	10 ULS Wind Leading + Snow + T	0.00
376 B	OK	RHS 400x300x16	S355	267.64	333.67	0.44	10 ULS Wind Leading + Snow + T	0.02
377 B	OK	RHS 400x300x16	S355	267.64	333.67	0.32	10 ULS Wind Leading + Snow + T	0.02
378 B	OK	SHS 200x10	S355	42.63	42.63	0.12	3 ULS	0.02
379 B	OK	SHS 200x10	S355	42.63	42.63	0.12	3 ULS	0.02
380 B	OK	SHS 200x10	S355	42.63	42.63	0.08	3 ULS	0.01
381 B	OK	SHS 200x10	S355	42.63	42.63	0.08	3 ULS	0.01
382 B	OK	SHS 200x10	S355	42.63	42.63	0.06	3 ULS	0.01
383 B	OK	SHS 200x10	S355	42.63	42.63	0.06	3 ULS	0.01
384 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.01
385 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.01
386 B	OK	SHS 200x10	S355	42.63	42.63	0.02	3 ULS	0.00
387 B	OK	SHS 200x10	S355	42.63	42.63	0.03	3 ULS	0.00
388 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.00
389 B	OK	SHS 200x10	S355	42.63	42.63	0.04	12 ULS 6.10 G+Q+Snow+Tempd	0.00
390 B	OK	SHS 200x10	S355	42.63	42.63	0.02	9 ULS Wind Leading + Snow + Tempd0	0.00
391 B	OK	SHS 200x10	S355	42.63	42.63	0.05	10 ULS Wind Leading + Snow + T	0.00
392 B	OK	SHS 200x10	S355	42.63	42.63	0.04	3 ULS	0.01
393 B	OK	SHS 200x10	S355	42.63	42.63	0.04	12 ULS 6.10 G+Q+Snow+Tempd	0.00
394 B	OK	SHS 200x10	S355	42.63	42.63	0.05	3 ULS	0.01
395 B	OK	SHS 200x10	S355	42.63	42.63	0.05	3 ULS	0.01
396 B	OK	RHS 400x300x16	S355	22.33	27.83	0.16	3 ULS	-
397 B	OK	RHS 400x300x16	S355	22.33	27.83	0.16	12 ULS 6.10 G+Q+Snow+Tempd	-
398 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.01

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
399 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
400 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
401 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
402 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
403 B	OK	SHS 200x10	S355	38.75	38.75	0.05	3 ULS	0.00
404 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
405 B	OK	SHS 200x10	S355	38.75	38.75	0.05	10 ULS Wind Leading + Snow + T	0.00
406 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
407 B	OK	SHS 200x10	S355	38.75	38.75	0.19	10 ULS Wind Leading + Snow + T	0.01
408 B	OK	RHS 400x300x16	S355	267.64	333.67	0.42	3 ULS	0.01
409 B	OK	RHS 400x300x16	S355	267.64	333.67	0.44	3 ULS	0.01
411 B	OK	SHS 200x10	S355	60.19	60.19	0.16	3 ULS	0.01
412 B	OK	SHS 200x10	S355	60.19	60.19	0.08	3 ULS	0.01
413 B	OK	SHS 200x10	S355	60.19	60.19	0.03	12 ULS 6.10 G+Q+Snow+Tempd	0.00
414 B	OK	SHS 200x10	S355	60.19	60.19	0.05	3 ULS	0.01
415 B	OK	SHS 200x10	S355	60.19	60.19	0.10	3 ULS	0.00
416 B	OK	SHS 200x10	S355	60.93	60.93	0.18	3 ULS	0.01
417 B	OK	SHS 200x10	S355	60.19	60.19	0.23	3 ULS	0.00
418 B	OK	SHS 200x10	S355	60.19	60.19	0.31	3 ULS	0.01
419 B	OK	SHS 200x10	S355	60.19	60.19	0.33	3 ULS	0.01
420 B	OK	SHS 200x10	S355	42.63	42.63	0.21	3 ULS	0.01
422 B	OK	SHS 200x10	S355	60.19	60.19	0.31	3 ULS	0.01
423 B	OK	SHS 200x10	S355	60.19	60.19	0.31	3 ULS	0.01
424 B	OK	SHS 200x10	S355	60.19	60.19	0.23	3 ULS	0.01
425 B	OK	SHS 200x10	S355	60.93	60.93	0.18	3 ULS	0.01
426 B	OK	SHS 200x10	S355	60.19	60.19	0.10	3 ULS	0.01
427 B	OK	SHS 200x10	S355	60.19	60.19	0.06	3 ULS	0.00
428 B	OK	SHS 200x10	S355	60.19	60.19	0.03	12 ULS 6.10 G+Q+Snow+Tempd	0.01
429 B	OK	SHS 200x10	S355	60.19	60.19	0.07	3 ULS	0.00
430 B	OK	SHS 200x10	S355	60.19	60.19	0.15	12 ULS 6.10 G+Q+Snow+Tempd	0.01
433 B	OK	SHS 200x10	S355	57.51	57.51	0.03	3 ULS	0.01
434 B	OK	SHS 200x10	S355	57.51	57.51	0.03	3 ULS	0.01
435 B	OK	SHS 200x10	S355	57.51	57.51	0.05	10 ULS Wind Leading + Snow + T	0.01
436 B	OK	SHS 200x10	S355	57.51	57.51	0.04	21 ULS WL	0.01
437 B	OK	SHS 200x10	S355	57.51	57.51	0.07	10 ULS Wind Leading + Snow + T	0.01
438 B	OK	SHS 200x10	S355	58.28	58.28	0.06	21 ULS WL	0.01
439 B	OK	SHS 200x10	S355	57.51	57.51	0.08	10 ULS Wind Leading + Snow + T	0.00
440 B	OK	SHS 200x10	S355	57.51	57.51	0.08	21 ULS WL	0.00
441 B	OK	SHS 200x10	S355	57.51	57.51	0.08	21 ULS WL	0.02
444 B	OK	SHS 200x10	S355	77.50	77.50	0.10	3 ULS	0.01
445 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00
446 B	OK	SHS 200x10	S355	77.50	77.50	0.07	8 TEMP d10	0.00
447 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00
448 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00
449 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00
450 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00
451 B	OK	SHS 200x10	S355	77.50	77.50	0.06	8 TEMP d10	0.00

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
452 B	OK	SHS 200x10	S355	77.50	77.50	0.10	3 ULS	0.01
453 B	OK	SHS 200x10	S355	77.50	77.50	0.09	21 ULS WL	0.01
454 B	OK	RHS 400x300x16	S355	228.00	284.24	0.30	10 ULS Wind Leading + Snow + T	0.01
456 B	OK	SHS 200x10	S355	42.63	42.63	0.17	3 ULS	0.02
458 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
460 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
462 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
464 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.00
466 B	OK	SHS 200x10	S355	42.63	42.63	0.12	3 ULS	0.00
468 B	OK	SHS 200x10	S355	42.63	42.63	0.12	3 ULS	0.01
470 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
472 B	OK	SHS 200x10	S355	42.63	42.63	0.16	3 ULS	0.02
474 B	OK	RHS 400x300x16	S355	22.33	27.83	0.27	3 ULS	-
476 B	OK	SHS 200x10	S355	77.50	77.50	0.22	3 ULS	0.00
477 B	OK	SHS 200x10	S355	77.50	77.50	0.20	3 ULS	0.00
478 B	OK	SHS 200x10	S355	77.50	77.50	0.19	3 ULS	0.00
479 B	OK	SHS 200x10	S355	77.50	77.50	0.19	3 ULS	0.00
480 B	OK	SHS 200x10	S355	77.50	77.50	0.19	3 ULS	0.00
481 B	OK	SHS 200x10	S355	77.50	77.50	0.18	3 ULS	0.00
482 B	OK	SHS 200x10	S355	77.50	77.50	0.18	3 ULS	0.00
483 B	OK	SHS 200x10	S355	77.50	77.50	0.19	3 ULS	0.00
484 B	OK	SHS 200x10	S355	77.50	77.50	0.19	3 ULS	0.00
485 B	OK	SHS 200x10	S355	77.50	77.50	0.14	3 ULS	0.02
486 B	OK	RHS 400x300x16	S355	228.00	284.24	0.53	3 ULS	0.01
488 B	OK	SHS 200x10	S355	60.93	60.93	0.49	3 ULS	0.01
489 B	OK	SHS 200x10	S355	60.93	60.93	0.42	3 ULS	0.02
490 B	OK	SHS 200x10	S355	60.93	60.93	0.28	3 ULS	0.02
491 B	OK	SHS 200x10	S355	60.93	60.93	0.19	12 ULS 6.10 G+Q+Snow+Tempd	0.02
492 B	OK	SHS 200x10	S355	60.93	60.93	0.06	3 ULS	0.02
493 B	OK	SHS 200x10	S355	60.93	60.93	0.06	3 ULS	0.02
494 B	OK	SHS 200x10	S355	60.93	60.93	0.21	3 ULS	0.02
495 B	OK	SHS 200x10	S355	60.93	60.93	0.30	3 ULS	0.02
496 B	OK	SHS 200x10	S355	60.93	60.93	0.45	3 ULS	0.02
497 B	OK	SHS 200x10	S355	60.93	60.93	0.51	3 ULS	0.02
498 B	OK	SHS 200x10	S355	42.63	42.63	0.18	3 ULS	0.02
510 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
511 B	OK	SHS 200x10	S355	88.89	88.89	0.03	3 ULS	0.00
512 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
513 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
514 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
515 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
516 B	OK	SHS 200x10	S355	88.89	88.89	0.04	9 ULS Wind Leading + Snow + Tempd0	0.00
517 B	OK	SHS 200x10	S355	88.89	88.89	0.04	3 ULS	0.00
518 B	OK	SHS 200x10	S355	88.89	88.89	0.05	10 ULS Wind Leading + Snow + T	0.00
519 B	OK	SHS 200x10	S355	88.89	88.89	0.04	21 ULS WL	0.00
533 B	OK	RHS 400x300x16	S355	228.00	284.24	0.28	10 ULS Wind Leading + Snow + T	0.01
535 B	OK	SHS 200x10	S355	42.63	42.63	0.19	3 ULS	0.03

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
537 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
539 B	OK	SHS 200x10	S355	42.63	42.63	0.14	3 ULS	0.01
541 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
543 B	OK	SHS 200x10	S355	42.63	42.63	0.14	3 ULS	0.00
545 B	OK	SHS 200x10	S355	42.63	42.63	0.10	3 ULS	0.00
547 B	OK	SHS 200x10	S355	42.63	42.63	0.12	3 ULS	0.01
549 B	OK	SHS 200x10	S355	42.63	42.63	0.10	3 ULS	0.01
551 B	OK	SHS 200x10	S355	42.63	42.63	0.16	3 ULS	0.02
553 B	OK	RHS 400x300x16	S355	22.33	27.83	0.25	3 ULS	-
565 B	OK	RHS 400x300x16	S355	228.00	284.24	0.56	3 ULS	0.01
578 B	OK	SHS 200x10	S355	60.93	60.93	0.51	3 ULS	0.00
579 B	OK	SHS 200x10	S355	60.93	60.93	0.46	3 ULS	0.02
580 B	OK	SHS 200x10	S355	60.93	60.93	0.32	3 ULS	0.02
581 B	OK	SHS 200x10	S355	60.93	60.93	0.22	3 ULS	0.02
582 B	OK	SHS 200x10	S355	60.93	60.93	0.07	3 ULS	0.02
583 B	OK	SHS 200x10	S355	60.93	60.93	0.05	3 ULS	0.02
584 B	OK	SHS 200x10	S355	60.93	60.93	0.17	3 ULS	0.02
585 B	OK	SHS 200x10	S355	60.93	60.93	0.27	3 ULS	0.02
586 B	OK	SHS 200x10	S355	60.93	60.93	0.41	3 ULS	0.02
587 B	OK	SHS 200x10	S355	60.93	60.93	0.48	3 ULS	0.01
599 B	OK	IPE 450	S355	23.27	104.40	0.40	3 ULS	0.00
601 B	OK	IPE 450	S355	23.27	104.40	0.26	10 ULS Wind Leading + Snow + T	0.00
602 B	OK	IPE 450	S355	23.27	104.40	0.48	10 ULS Wind Leading + Snow + T	0.00
603 B	OK	IPE 450	S355	23.27	104.40	0.54	12 ULS 6.10 G+Q+Snow+Tempd	0.00
604 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.00
605 B	OK	SHS 200x10	S355	42.63	42.63	0.07	3 ULS	0.01
606 B	OK	SHS 200x10	S355	42.63	42.63	0.07	3 ULS	0.01
607 B	OK	SHS 200x10	S355	38.75	38.75	0.04	3 ULS	0.00
608 B	OK	SHS 200x10	S355	38.75	38.75	0.06	8 TEMP d10	0.01
609 B	OK	SHS 200x10	S355	42.63	42.63	0.11	3 ULS	0.01
610 B	OK	SHS 200x10	S355	42.63	42.63	0.10	3 ULS	0.01
611 B	OK	SHS 200x10	S355	38.75	38.75	0.08	10 ULS Wind Leading + Snow + T	0.00
615 Beam_615	OK	IPE 450	S355	27.60	123.83	0.14	10 ULS Wind Leading + Snow + T	0.03
616 B	OK	IPE 450	S355	27.60	123.83	0.12	10 ULS Wind Leading + Snow + T	0.01
617 Beam_617	OK	IPE 450	S355	27.60	123.83	0.14	10 ULS Wind Leading + Snow + T	0.05
618 Beam_618	OK	IPE 450	S355	27.60	123.83	0.11	10 ULS Wind Leading + Snow + T	0.01
619 B	OK	IPE 450	S355	27.60	123.83	0.18	10 ULS Wind Leading + Snow + T	0.07
620 B	OK	IPE 450	S355	27.60	123.83	0.21	10 ULS Wind Leading + Snow + T	0.07
621 B	OK	IPE 450	S355	27.60	123.83	0.09	21 ULS WL	0.03
622 B	OK	IPE 450	S355	27.60	123.83	0.16	10 ULS Wind Leading + Snow + T	0.06
637 B	OK	IPE 450	S355	11.36	50.99	0.06	10 ULS Wind Leading + Snow + T	0.01
640 B	OK	IPE 450	S355	11.36	50.99	0.08	9 ULS Wind Leading + Snow + Tempd0	0.01
641 B	OK	IPE 450	S355	11.36	50.99	0.11	9 ULS Wind Leading + Snow + Tempd0	0.01
644 B	OK	IPE 450	S355	11.36	50.99	0.10	10 ULS Wind Leading + Snow + T	0.01
652 B	OK	IPE 450	S355	11.36	50.99	0.05	8 TEMP d10	-
654 B	OK	IPE 450	S355	11.36	50.99	0.05	8 TEMP d10	-
655 B	OK	IPE 450	S355	11.36	50.99	0.06	8 TEMP d10	-

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(uy)
657 B	OK	IPE 450	S355	11.36	50.99	0.06	8 TEMP d10	-
659 B	OK	IPE 450	S355	35.17	157.82	0.23	10 ULS Wind Leading + Snow + T	0.05
660 B	OK	IPE 450	S355	35.17	157.82	0.15	10 ULS Wind Leading + Snow + T	0.02
661 B	OK	IPE 450	S355	35.17	157.82	0.33	10 ULS Wind Leading + Snow + T	0.07
662 B	OK	IPE 450	S355	35.17	157.82	0.42	9 ULS Wind Leading + Snow + Tempd0	0.02
663 B	OK	IPE 450	S355	11.36	50.99	0.10	10 ULS Wind Leading + Snow + T	0.00
665 B	OK	IPE 450	S355	11.36	50.99	0.07	8 TEMP d10	0.01
666 B	OK	IPE 450	S355	12.45	55.84	0.10	9 ULS Wind Leading + Snow + Tempd0	0.01
667 B	OK	IPE 450	S355	25.43	114.11	0.14	21 ULS WL	0.01
668 B	OK	IPE 450	S355	11.36	50.99	0.10	9 ULS Wind Leading + Snow + Tempd0	0.00
669 B	OK	IPE 450	S355	11.36	50.99	0.11	21 ULS WL	0.01
670 B	OK	IPE 450	S355	11.36	50.99	0.12	9 ULS Wind Leading + Snow + Tempd0	0.00
671 B	OK	IPE 450	S355	12.45	55.84	0.10	21 ULS WL	0.01
672 B	OK	IPE 450	S355	11.36	50.99	0.14	21 ULS WL	0.02
673 B	OK	IPE 450	S355	12.45	55.84	0.15	10 ULS Wind Leading + Snow + T	0.00
674 B	OK	IPE 450	S355	25.43	114.11	0.21	10 ULS Wind Leading + Snow + T	0.02
675 B	OK	IPE 450	S355	11.36	50.99	0.16	10 ULS Wind Leading + Snow + T	0.01
676 B	OK	IPE 450	S355	11.36	50.99	0.19	21 ULS WL	0.02
677 B	OK	IPE 450	S355	11.36	50.99	0.23	10 ULS Wind Leading + Snow + T	0.01
678 B	OK	IPE 450	S355	12.45	55.84	0.15	10 ULS Wind Leading + Snow + T	0.01
679 B	OK	IPE 450	S355	11.36	50.99	0.20	9 ULS Wind Leading + Snow + Tempd0	0.00
1034 B	OK	IPE 450	S355	13.53	60.70	0.25	10 ULS Wind Leading + Snow + T	0.01
1035 B	OK	IPE 450	S355	40.04	179.67	0.55	10 ULS Wind Leading + Snow + T	0.02
1037 B	OK	IPE 450	S355	13.53	60.70	0.38	10 ULS Wind Leading + Snow + T	0.01
1038 B	OK	IPE 450	S355	40.04	179.67	0.82	21 ULS WL	0.02
1039 B	OK	IPE 450	S355	23.27	104.40	0.17	10 ULS Wind Leading + Snow + T	0.03
1040 B	OK	IPE 450	S355	23.27	104.40	0.12	3 ULS	0.05
1041 B	OK	IPE 450	S355	23.27	104.40	0.15	10 ULS Wind Leading + Snow + T	0.02
1042 B	OK	IPE 450	S355	23.27	104.40	0.09	3 ULS	0.03
1043 B	OK	IPE 450	S355	25.43	114.11	0.12	10 ULS Wind Leading + Snow + T	0.02
1044 B	OK	IPE 450	S355	25.43	114.11	0.24	10 ULS Wind Leading + Snow + T	0.02
1060 B	OK	SHS 200x10	S355	60.19	60.19	0.32	3 ULS	0.01
1061 B	OK	SHS 200x10	S355	60.19	60.19	0.29	3 ULS	0.01
1062 B	OK	SHS 200x10	S355	60.19	60.19	0.21	3 ULS	0.00
1063 B	OK	SHS 200x10	S355	60.19	60.19	0.20	3 ULS	0.00
1064 B	OK	SHS 200x10	S355	60.19	60.19	0.27	3 ULS	0.01
1065 B	OK	SHS 200x10	S355	60.19	60.19	0.31	3 ULS	0.02
1066 B	OK	SHS 200x10	S355	57.51	57.51	0.07	10 ULS Wind Leading + Snow + T	0.00
1067 B	OK	SHS 200x10	S355	57.51	57.51	0.02	21 ULS WL	0.00
1068 B	OK	SHS 200x10	S355	57.51	57.51	0.03	10 ULS Wind Leading + Snow + T	0.01

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
8 TEMP d10	0.00	5 WIND1	-	-	-	-
8 TEMP d10	0.03	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.01	4 SLS	-	-	-	-
-	-	-	0.30	18 SLS LL	0.10	8 TEMP d10

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
-	-	-	0.24	18 SLS LL	0.10	8 TEMP d10
-	-	-	0.18	5 WIND1	0.11	8 TEMP d10
-	-	-	0.19	5 WIND1	0.11	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.10	8 TEMP d10
-	-	-	0.21	5 WIND1	0.11	8 TEMP d10
-	-	-	0.21	5 WIND1	0.11	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.10	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.10	8 TEMP d10
-	-	-	0.29	18 SLS LL	0.10	8 TEMP d10
8 TEMP d10	0.01	4 SLS	-	-	-	-
5 WIND1	0.02	18 SLS LL	-	-	-	-
5 WIND1	0.06	4 SLS	-	-	-	-
8 TEMP d10	0.00	17 SLS DL	-	-	-	-
5 WIND1	0.06	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
5 WIND1	0.01	5 WIND1	-	-	-	-
18 SLS LL	0.08	4 SLS	-	-	-	-
18 SLS LL	0.03	4 SLS	-	-	-	-
8 TEMP d10	0.01	4 SLS	-	-	-	-
8 TEMP d10	0.01	4 SLS	-	-	-	-
5 WIND1	0.03	18 SLS LL	-	-	-	-
8 TEMP d10	0.02	4 SLS	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.00	18 SLS LL	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.05	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
5 WIND1	0.02	5 WIND1	-	-	-	-
18 SLS LL	0.03	4 SLS	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
8 TEMP d10	0.02	17 SLS DL	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
18 SLS LL	0.04	4 SLS	-	-	-	-
18 SLS LL	0.03	4 SLS	-	-	-	-
-	-	-	0.28	18 SLS LL	0.09	18 SLS LL
4 SLS	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	4 SLS	-	-	-	-
18 SLS LL	0.00	4 SLS	-	-	-	-
5 WIND1	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.02	18 SLS LL	-	-	-	-
5 WIND1	0.11	4 SLS	-	-	-	-
5 WIND1	0.11	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	5 WIND1	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.03	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	18 SLS LL	-	-	-	-
5 WIND1	0.11	4 SLS	-	-	-	-
5 WIND1	0.11	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
18 SLS LL	0.01	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
-	-	-	0.32	18 SLS LL	0.37	4 SLS
-	-	-	0.31	18 SLS LL	0.37	4 SLS
-	-	-	0.11	18 SLS LL	0.02	8 TEMP d10
-	-	-	0.11	18 SLS LL	0.02	8 TEMP d10
-	-	-	0.43	18 SLS LL	0.13	4 SLS
8 TEMP d10	0.03	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.01	4 SLS	-	-	-	-
-	-	-	0.24	18 SLS LL	0.12	18 SLS LL
-	-	-	0.19	5 WIND1	0.12	18 SLS LL
-	-	-	0.19	5 WIND1	0.10	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.11	18 SLS LL
-	-	-	0.21	5 WIND1	0.11	18 SLS LL
-	-	-	0.21	5 WIND1	0.10	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.09	8 TEMP d10
-	-	-	0.24	18 SLS LL	0.09	8 TEMP d10
8 TEMP d10	0.01	4 SLS	-	-	-	-
8 TEMP d10	0.08	4 SLS	-	-	-	-
8 TEMP d10	0.00	17 SLS DL	-	-	-	-
4 SLS	0.08	4 SLS	-	-	-	-
8 TEMP d10	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.00	17 SLS DL	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
8 TEMP d10	0.01	4 SLS	-	-	-	-
8 TEMP d10	0.02	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	5 WIND1	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
5 WIND1	0.00	4 SLS	-	-	-	-
8 TEMP d10	0.00	18 SLS LL	-	-	-	-
8 TEMP d10	0.00	18 SLS LL	-	-	-	-
-	-	-	0.09	18 SLS LL	0.02	8 TEMP d10
-	-	-	0.08	18 SLS LL	0.02	8 TEMP d10
8 TEMP d10	0.04	4 SLS	-	-	-	-
8 TEMP d10	0.05	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.00	4 SLS	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
8 TEMP d10	0.02	18 SLS LL	-	-	-	-
5 WIND1	0.14	4 SLS	-	-	-	-
5 WIND1	0.14	4 SLS	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
4 SLS	0.00	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
-	-	-	0.39	18 SLS LL	0.51	4 SLS
-	-	-	0.39	18 SLS LL	0.51	4 SLS
5 WIND1	0.01	4 SLS	-	-	-	-
5 WIND1	0.02	18 SLS LL	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
5 WIND1	0.02	18 SLS LL	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.02	18 SLS LL	-	-	-	-
5 WIND1	0.03	18 SLS LL	-	-	-	-
18 SLS LL	0.14	4 SLS	-	-	-	-
18 SLS LL	0.14	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.17	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
5 WIND1	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	18 SLS LL	-	-	-	-
-	-	-	0.27	5 WIND1	0.55	4 SLS
5 WIND1	0.11	4 SLS	-	-	-	-
18 SLS LL	0.11	4 SLS	-	-	-	-
5 WIND1	0.12	4 SLS	-	-	-	-
18 SLS LL	0.11	4 SLS	-	-	-	-
18 SLS LL	0.12	4 SLS	-	-	-	-
5 WIND1	0.11	4 SLS	-	-	-	-
4 SLS	0.12	4 SLS	-	-	-	-
4 SLS	0.11	4 SLS	-	-	-	-
18 SLS LL	0.10	4 SLS	-	-	-	-
18 SLS LL	0.06	18 SLS LL	-	-	-	-
18 SLS LL	0.17	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
18 SLS LL	0.03	4 SLS	-	-	-	-
4 SLS	0.05	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.06	4 SLS	-	-	-	-
4 SLS	0.05	4 SLS	-	-	-	-
5 WIND1	0.04	4 SLS	-	-	-	-
18 SLS LL	0.17	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
-	-	-	0.25	5 WIND1	0.56	4 SLS
18 SLS LL	0.17	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.03	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
5 WIND1	0.01	4 SLS	-	-	-	-
5 WIND1	0.06	4 SLS	-	-	-	-
18 SLS LL	0.04	4 SLS	-	-	-	-
18 SLS LL	0.05	4 SLS	-	-	-	-
5 WIND1	0.07	4 SLS	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-
4 SLS	0.00	18 SLS LL	-	-	-	-
4 SLS	0.01	5 WIND1	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.02	18 SLS LL	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
18 SLS LL	0.00	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
18 SLS LL	0.00	4 SLS	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
5 WIND1	0.00	5 WIND1	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
8 TEMP d10	0.01	18 SLS LL	-	-	-	-
8 TEMP d10	0.01	18 SLS LL	-	-	-	-
8 TEMP d10	0.01	5 WIND1	-	-	-	-
8 TEMP d10	0.01	5 WIND1	-	-	-	-
-	-	-	0.27	18 SLS LL	0.03	8 TEMP d10
-	-	-	0.08	4 SLS	0.03	8 TEMP d10
-	-	-	0.10	18 SLS LL	0.03	8 TEMP d10
-	-	-	0.08	8 TEMP d10	0.03	8 TEMP d10
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
18 SLS LL	0.02	18 SLS LL	-	-	-	-
8 TEMP d10	0.03	18 SLS LL	-	-	-	-

Case (uy)	Ratio(uz)	Case (uz)	Ratio(vx)	Case (vx)	Ratio(vy)	Case (vy)
8 TEMP d10	0.00	5 WIND1	-	-	-	-
8 TEMP d10	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.01	5 WIND1	-	-	-	-
8 TEMP d10	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	5 WIND1	-	-	-	-
8 TEMP d10	0.01	18 SLS LL	-	-	-	-
4 SLS	0.00	5 WIND1	-	-	-	-
18 SLS LL	0.01	5 WIND1	-	-	-	-
8 TEMP d10	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
5 WIND1	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	5 WIND1	-	-	-	-
18 SLS LL	0.00	18 SLS LL	-	-	-	-
18 SLS LL	0.00	4 SLS	-	-	-	-
18 SLS LL	0.01	18 SLS LL	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
5 WIND1	0.01	5 WIND1	-	-	-	-
5 WIND1	0.02	5 WIND1	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
18 SLS LL	0.02	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
5 WIND1	0.02	4 SLS	-	-	-	-
18 SLS LL	0.01	4 SLS	-	-	-	-
4 SLS	0.01	18 SLS LL	-	-	-	-
4 SLS	0.01	4 SLS	-	-	-	-
4 SLS	0.02	4 SLS	-	-	-	-

Appendice G: Calcoli del tetto del Terminal a guscio

Eckersley O'Callaghan

eocengineers.com

Project Name		
Section		
Project No.	Page	Date
Revision	By	Checked

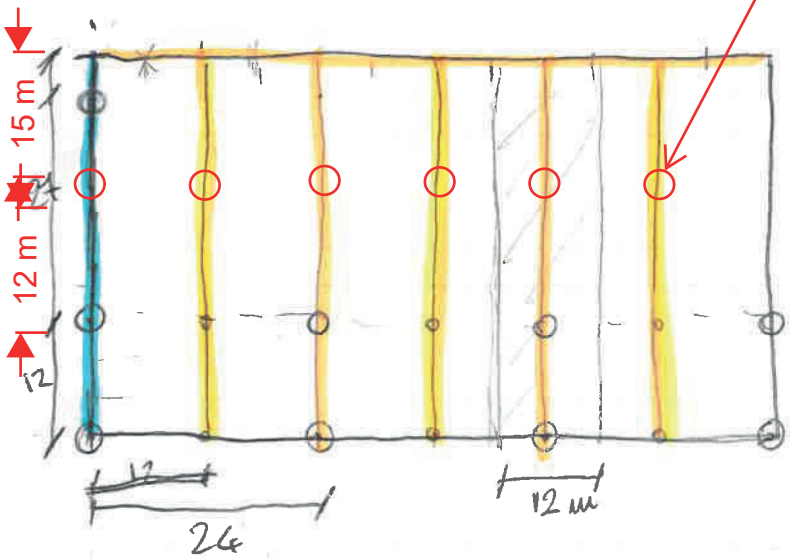
Wang Buildings

Roof.

Update loads. 1 KPa DL, 2 KPa SDL = 3

0.6 LL + 1.4 Wind + 0.8 snow ≈ 2.8 KPa

COLUMN ALL THE WAY TO ROOF



DL = ~~3.21~~ KPa

SDL = ~~1.75~~ 2 KPa

LL = 0.6 KPa

} sizes n similar

PFS 10x8 x 6x12 x 6x2

I = ~~1474~~ 2 x 10^4 mm^4

Edge: I = ~~814900 x 10^4 mm^4~~

Internal Beams: trib width = 12 m

Add 0.75 KPa for steelwork? Sec Beams = 13 KN/m x12 = 156

SLS loads: Dead: (3.21 + 1.75) x 15 = 57 KN/m

Live: 0.6 x 12 = 10.8 KN/m

ULS

DEFLECTION = (3.5+0.6)*15^4 *(5/384 EI)
E = 205 000 MPa

L/360 = 42 mm (L= 15 m for longest beam)

VEd = 507 kN
MEd = 1899 kN

According to Blue Book Can use 762 x 267 x 134 beam, assuming secondary beams provide lateral restraint every 4 meters.

Design Deflection driven

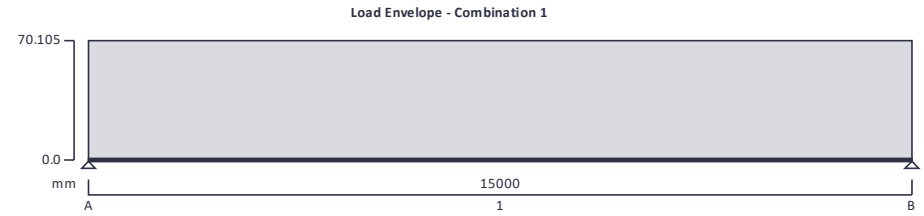
Euorpean equivalent size = HE 800 M
I = 442590x10^4

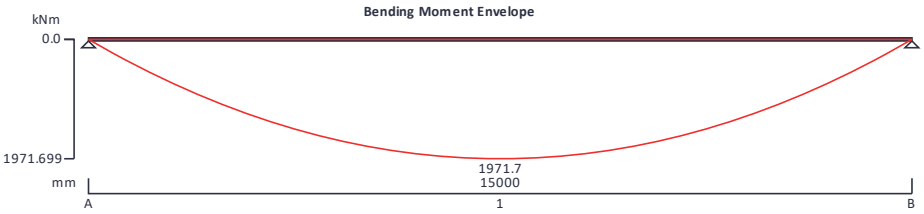
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

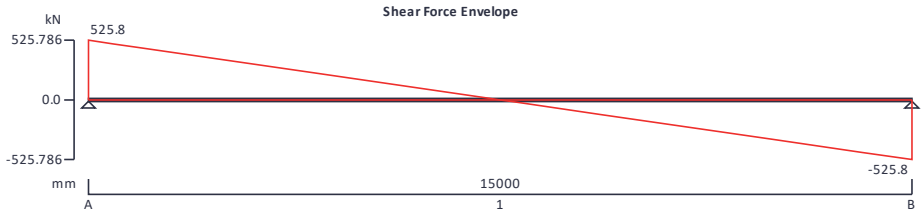
15 m Long, Trib area 12 m

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	42 - Permanent full UDL 42 kN/m
	7.2 - Variable full UDL 7.2 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

15 m Long, Trib area 12 m

Analysis results

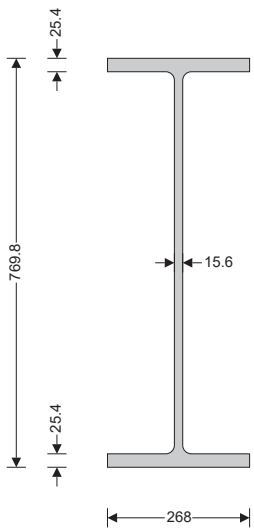
Maximum moment	M_{max} = **1971.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **525.8** kN	V_{min} = **-525.8** kN
Deflection	δ_{max} = **9.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **525.8** kN	R_{A_min} = **525.8** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **329.5** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **54** kN	
Maximum reaction at support B	R_{B_max} = **525.8** kN	R_{B_min} = **525.8** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **329.5** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **54** kN	

Section details

Section type	**UKB 762x267x197 (Tata Steel Advance)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	t = max(t_f, t_w) = **25.4** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5 15 m Long, Trib area 12 m

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.83}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d = \mathbf{686}$ mm

$c / t_w = 53.3 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{109.7}$ mm

$c / t_f = 5.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{719}$ mm

Shear area factor $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) = \mathbf{525.8}$ kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{12685}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{2526.6}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(abs(M_{s1_max}), abs(M_{s1_min})) = \mathbf{1971.7}$ kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{2472.5}$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{41.7}$ mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{9.419}$ mm

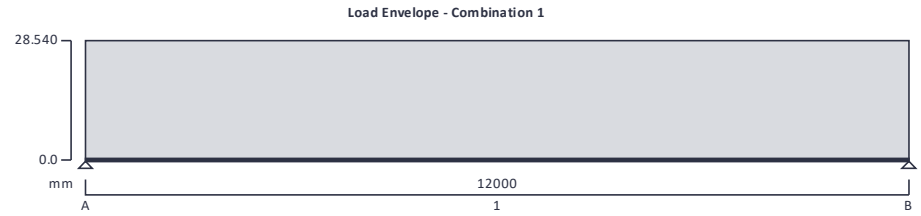
PASS - Maximum deflection does not exceed deflection limit

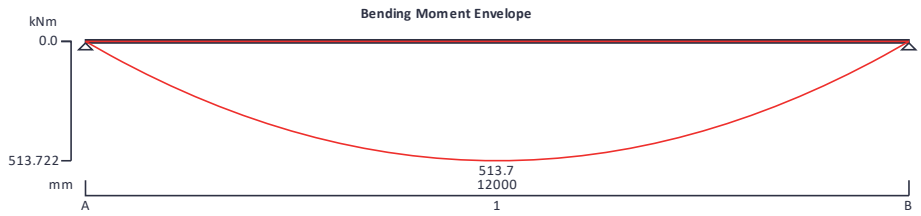
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

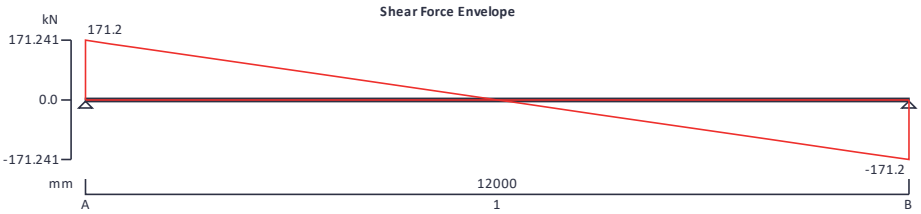
12 m Long, Trib area 4 m

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	DL - Permanent full UDL 10.5 kN/m
	wind snow LL - Live full UDL 8.4 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Dead × 1.35
		Live × 1.50
		Permanent × 1.35
		Dead × 1.35
		Live × 1.50

	Support B	Permanent × 1.35
		Dead × 1.35
		Live × 1.50

Analysis results

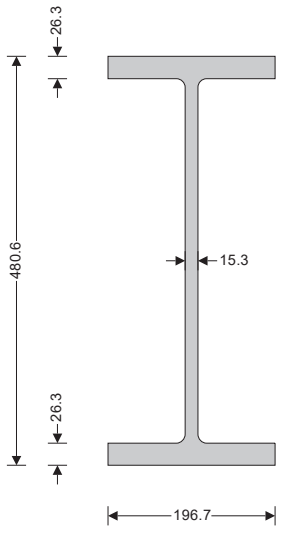
Maximum moment	M_{max} = **513.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **171.2** kN	V_{min} = **-171.2** kN
Deflection	δ_{max} = **16.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **171.2** kN	R_{A_min} = **171.2** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **70.8** kN	
Unfactored live load reaction at support A	R_{A_Live} = **50.4** kN	
Maximum reaction at support B	R_{B_max} = **171.2** kN	R_{B_min} = **171.2** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **70.8** kN	
Unfactored live load reaction at support B	R_{B_Live} = **50.4** kN	

Section details

Section type	**UKB 457x191x133 (Tata Steel Advance)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **26.3** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.83**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d$ = **407.6** mm

$c / t_w = 32.3 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **80.5** mm

$c / t_f = 3.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **428** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **171.2** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **7577** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **1509.2** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min}))$ = **513.7** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **1059.1** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to dead and live loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **33.3** mm

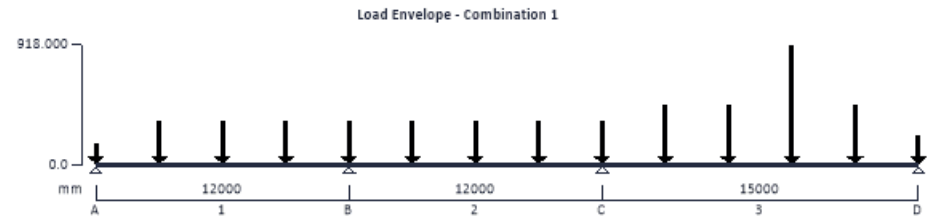
Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **16.917** mm

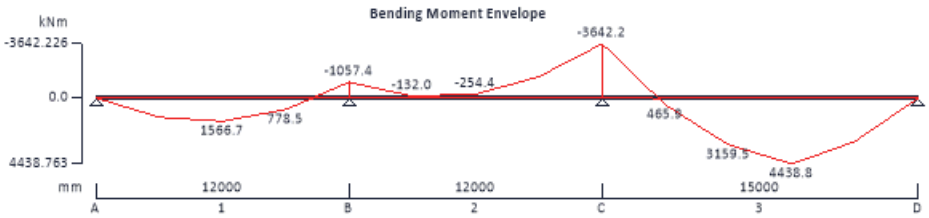
PASS - Maximum deflection does not exceed deflection limit

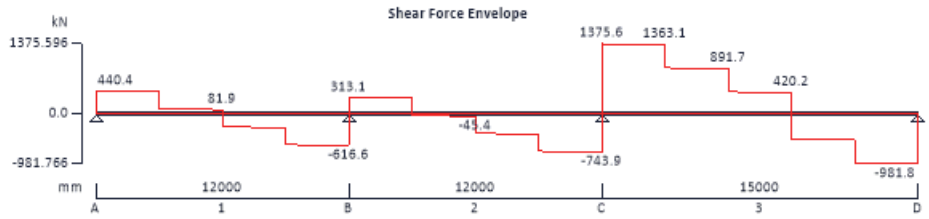
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Ter Beam - Permanent point load 340 kN at 3000 mm
	Permanent point load 340 kN at 6000 mm
	Permanent point load 340 kN at 9000 mm
	Permanent point load 340 kN at 15000 mm
	Permanent point load 340 kN at 18000 mm
	Permanent point load 340 kN at 21000 mm

Permanent point load 340 kN at 24000 mm
Permanent point load 340 kN at 27000 mm
Permanent point load 340 kN at 30000 mm
Permanent point load 340 kN at 33000 mm
Permanent point load 340 kN at 12000 mm
Permanent point load 170 kN at 0 mm
Permanent point load 340 kN at 33000 mm
Permanent point load 340 kN at 36000 mm
Permanent point load 170 kN at 39000 mm

Load combinations

Load combination 1	Support A	Permanent × 1.00
		Variable × 1.00
		Permanent × 1.00
		Variable × 1.00
	Support B	Permanent × 1.00
		Variable × 1.00
		Permanent × 1.00
		Variable × 1.00
	Support C	Permanent × 1.00
		Variable × 1.00
		Permanent × 1.35
		Variable × 1.50
	Support D	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **4438.8** kNm	M_{min} = **-3642.2** kNm
Maximum moment span 1	M_{s1_max} = **1566.7** kNm	M_{s1_min} = **-1057.4** kNm
Maximum moment span 2	M_{s2_max} = **-132** kNm	M_{s2_min} = **-3642.2** kNm
Maximum moment span 3	M_{s3_max} = **4438.8** kNm	M_{s3_min} = **-3642.2** kNm
Maximum shear	V_{max} = **1375.6** kN	V_{min} = **-981.8** kN
Maximum shear span 1	V_{s1_max} = **440.4** kN	V_{s1_min} = **-616.6** kN
Maximum shear span 2	V_{s2_max} = **313.1** kN	V_{s2_min} = **-743.9** kN
Maximum shear span 3	V_{s3_max} = **1375.6** kN	V_{s3_min} = **-981.8** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Deflection span 1	δ_{s1_max} = **0** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0** mm	δ_{s2_min} = **0** mm
Deflection span 3	δ_{s3_max} = **0** mm	δ_{s3_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **610.4** kN	R_{A_min} = **610.4** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **593.2** kN	
Maximum reaction at support B	R_{B_max} = **1269.7** kN	R_{B_min} = **1269.7** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **1372.7** kN	
Maximum reaction at support C	R_{C_max} = **2459.5** kN	R_{C_min} = **2459.5** kN
Unfactored permanent load reaction at support C	$R_{C_Permanent}$ = **2025** kN	
Maximum reaction at support D	R_{D_max} = **1211.3** kN	R_{D_min} = **1211.3** kN
Unfactored permanent load reaction at support D	$R_{D_Permanent}$ = **889.2** kN	

Section details

Section type **HE 1000 B (Arcelor)**

Steel grade **S355**

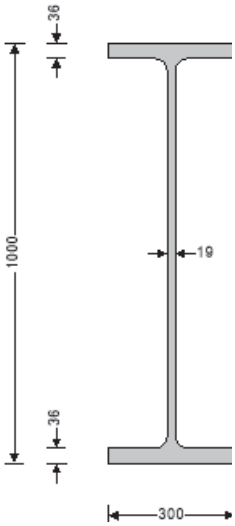
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w) =$ **36.0** mm

Nominal yield strength $f_y =$ **345** N/mm^2

Nominal ultimate tensile strength $f_u =$ **470** N/mm^2

Modulus of elasticity $E =$ **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections $\gamma_{M0} =$ **1.00**

Resistance of members to instability $\gamma_{M1} =$ **1.00**

Resistance of tensile members to fracture $\gamma_{M2} =$ **1.10**

Lateral restraint

Span 1 has full lateral restraint

Span 2 has full lateral restraint

Span 3 has full lateral restraint

Effective length factors

Effective length factor in major axis $K_y =$ **1.000**

Effective length factor in minor axis $K_z =$ **1.000**

Effective length factor for torsion $K_{LT.A} =$ **1.000**

$K_{LT.B} =$ **1.000**

$K_{LT.C} =$ **1.000**

$K_{LT.D} =$ **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **0.83**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d =$ **868** mm

$c / t_w = 55.4 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **110.5** mm

$c / t_f = 3.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **928** mm

Shear area factor $\eta =$ **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **1375.6** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **21249** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **4232.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 3 major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s3_max}), \text{abs}(M_{s3_min})) =$ **4438.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **5125** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s3} / 360 =$ **41.7** mm

Maximum deflection span 3 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0** mm

PASS - Maximum deflection does not exceed deflection limit

Appendice H: Calcoli del tetto principale

Di seguito sono riportate le forze interne massime negli elementi principali del tetto per le strategie MJ Opzione 3 & Opzione 4.Il dimensionamento Preliminare degli eleemnti é stato eseguito secondo le forze del caso limite ultimo peggiore di un elemento tipico.

Option 3 Results

Secondary Beams

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	1022	626	2107	810	6599	1321
Bar	3675	3569	3718	3685	3725	3569
Case	100 (C)	103 (C)	100 (C)	100 (C)	100 (C)	103 (C)
MIN	-3749	-611	-2094	-826	-6984	-1184
Bar	2793	3623	3702	3735	3631	3570
Case	103 (C)	103 (C)	100 (C)	100 (C)	100 (C)	103 (C)

MC Branches

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	13765	128	359	67	463	1038
Bar	130	81	82	260	79	80
Case	303 (C) (SRSS)	303 (C) (SRSS)	303 (C) (SRSS)	300 (C) (SRSS)	100 (C)	303 (C) (SRSS)
MIN	2092	-89	-314	0	-2715	-1067
Bar	98	74	73	67	82	74
Case	108 (C)	303 (C) (SRSS)	303 (C) (SRSS)	100 (C)	303 (C) (SRSS)	303 (C) (SRSS)

Vienyard Beams

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	1951	430	1431	53	1161	2604
Bar	3816	4421	697	1692	1735	3882
Case	303 (C) (SRSS)	303 (C) (SRSS)	303 (C) (SRSS)	301 (C) (CQC)	103 (C)	303 (C) (SRSS)
MIN	-3053	-216	-1279	-27	-8281	-1336
Bar	1571	3881	2073	3882	4559	3880
Case	303 (C) (SRSS)	100 (C)	303 (C) (SRSS)	303 (C) (SRSS)	303 (C) (SRSS)	100 (C)

MC Ties

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	826	2626	86	32	0	0
Bar	545	496	496	496	496	496
Case	104 (C)	301 (C) (CQC)	301 (C) (CQC)	303 (C) (SRSS)	109 (C)	301 (C) (CQC)
MIN	-5322	-94	-54	0	0	0
Bar	333	517	496	331	496	496
Case	303 (C) (SRSS)	103 (C)	109 (C)	100 (C)	301 (C) (CQC)	106 (C)

MC Columns

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	48015	6858	5591	1115	22297	56802
Bar	2421	52	57	2548	2389	52
Case	303 (C) (SRSS)	303 (C) (SRSS)	300 (C) (SRSS)	300 (C) (SRSS)	303 (C) (SRSS)	303 (C) (SRSS)
MIN	8366	-2331	-4063	-725	-33664	-13939
Bar	6	2421	2389	2389	52	2421
Case	107 (C)	100 (C)	100 (C)	303 (C) (SRSS)	300 (C) (SRSS)	100 (C)

Option 4 Results

Secondary Beams

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	2895	1481	1984	673	3672	2984
Bar	2855	4731	4803	4881	3461	4748
Case	301 (C) (SRSS)	301 (C) (SRSS)	301 (C) (SRSS)	110 (C)	110 (C)	301 (C) (SRSS)
MIN	-3649	-627	-1893	-609	-7936	-1420
Bar	2793	4748	3737	2824	4803	4731
Case	103 (C)	103 (C)	105 (C)	110 (C)	301 (C) (SRSS)	103 (C)

MC Branches

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	8213	179	271	0	665	2298
Bar	197	89	70	67	79	68
Case	301 (C) (SRSS)	301 (C) (SRSS)	105 (C)	100 (C)	100 (C)	301 (C) (SRSS)
MIN	2099	-174	-270	0	-2176	-1968
Bar	113	68	89	67	70	70
Case	107 (C)	105 (C)	105 (C)	100 (C)	105 (C)	105 (C)

Vienyard Beams

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	1204	452	908	21	1002	2925
Bar	4523	1644	1594	1646	1528	1648
Case	301 (C) (SRSS)	301 (C) (SRSS)	105 (C)	301 (C) (SRSS)	105 (C)	301 (C) (SRSS)
MIN	-2779	-79	-894	-15	-5442	-915
Bar	955	1636	4017	3898	3880	889
Case	103 (C)	110 (C)	105 (C)	110 (C)	105 (C)	103 (C)

MC Ties

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	784	1393	78	20	0	0
Bar	512	496	517	496	517	496
Case	110 (C)	301 (C) (SRSS)	110 (C)	110 (C)	110 (C)	301 (C) (SRSS)
MIN	-3508	-541	-21	0	0	0
Bar	343	517	500	331	496	496
Case	105 (C)	110 (C)	109 (C)	100 (C)	301 (C) (SRSS)	106 (C)

MC Columns

	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
MAX	32994	5512	4577	4085	21090	33791
Bar	2421	21	2290	29	2389	2546
Case	100 (C)	301 (C) (SRSS)	301 (C) (SRSS)	301 (C) (SRSS)	100 (C)	301 (C) (SRSS)
MIN	7492	-2252	-4095	-3510	-20497	-13147
Bar	6	2421	2389	2282	2288	2421
Case	107 (C)	100 (C)	100 (C)	105 (C)	301 (C) (SRSS)	100 (C)

Key:

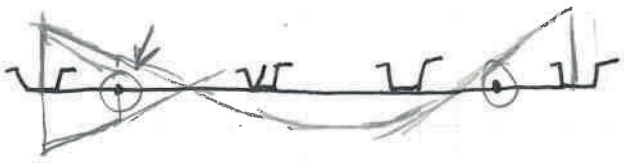
100 (C)	1.35 DL + 1.5 LL
103 (C)	1.35 DL + 1.5 TLmax + 1.05 LL
106 (C)	1.00 DL + 1.5 TLmin
107 (C)	1.00 DL + 1.50 WL East
108 (C)	1.00 DL + 1.50 WL West
109 (C)	1.35 DL + 1.5 TLmax + 1.05 WL East
300 (C) (SRSS)	DL+SX+0.6LL
301 (C) (CQC)	DL+SY+0.6LL
303 (C) (SRSS)	DL+(SX+0.3SY)+0.6LL

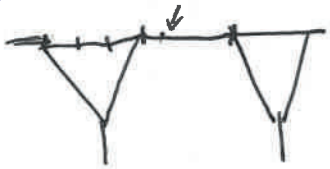
SECONDARY BEAMS

Max

- Axial tension (1.35DL + 1.5SL2 + 1.05LL) 102 3675

Occurring @ contraflexure point near MC supporting cantilever





$F_x = 1126$ KN @ point

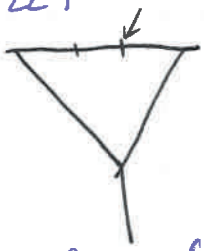
$M_y = -7378$ KNm (@ point - 3973)

$F_z = -1931$ KN (@ point - 1852)

Other values ≤ 100 KN or KNm

1MPa tension negligible

- Axial compression (1.35DL + 1.5 TL max + 1.05 LL)



Small shear & more

$F_x = -3769$ KN

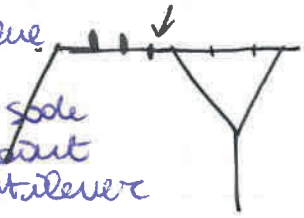
$M_y = 2623$ KNm

Mod column (control part of MJs)

Stress OK

- Shear Major axis 102 2257

Use absolute value



Mod 2 cols on side contraflexure point near front cantilever

$F_z = 2257$ KN

$M_y = -5166$ KNm

With shear moment loading

Minor shears & Torsion values OK do not check for now

Torsion only on front 2 columns on corners - mid sec beam

Max

- Moment major axis 102

Sagging
Mid beam 2nd col from left, front row.

$M_y = 7047$ KNm

$F_x = -2662$ KN

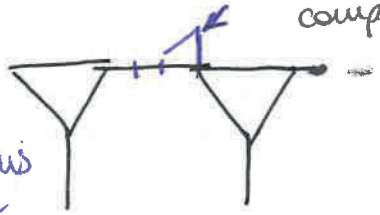
$F_z = 2138$ KN (@ point 1363)

$A_s = 18 \phi 25$ @ Bottom

Axial & shear small in comparison

- Moment major axis 102

Hogging
Front row of columns next to cantilever



$M_y = -7418$ KNm

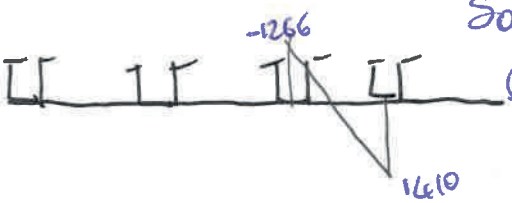
$F_x = 1101$ KN

$F_z = 1926$ KN

$A_s \sim A_s$ bottom — lots espace

Axial & shear small in

- Moment Minor axis



Sagging & hogging
left column sec row

$M_z = 1610$ KNm, $M_z = -1266$ KNm

$M_y = -2232$ KNm, $M_z = 1698$ KNm

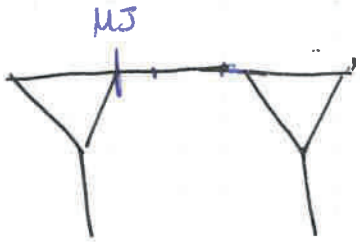
$F_z = 1069$ KN

$F_x = -1909$

VINEYARD BEAMS.

- Max Axial - Tension.[102]

Near Mz location.



$F_x = 1000$ KN →

$F_z = -768$ KN @ Point

$M_y = -2738$ KNm @ Point

1 MPa tension → small

- Max Axial - Compression.[103]

2nd last column

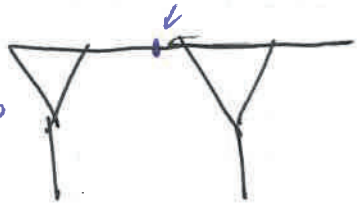
$F_x = -2768$ KN

$M_y = -799$ KNm (29 @ Point - ok)

$F_z = 500$ KN

- Max shear (Abs values)[102]

behind 1st row of column



$F_z = 996$ KN

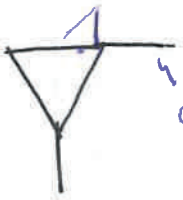
$M_y = -5215$ hogging

$M_z = -423$

- Max Major axis moment

Hogging

1st row of Mc



cantilever

$M_y = -5865$ KNm

$F_z = 806$ KN

$F_x = 85$ KN

- Max Major axis moment

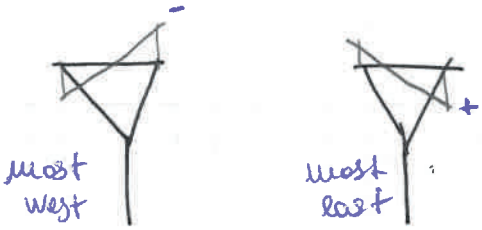
Sagging

$M_y = 1973$ KNm

$F_x = -1136$ KN - compression

$F_z = 536$ KN

- Max Minor axis moment



$M_z = -1465$ KNm $= 1607$ KNm

$F_z = 780$ KN

$M_y = -5653$ KNm $= -4398$ KNm

Non-typical
- special case.

<u>Off the shelf sizing</u>

Eckersley O'Callaghan

eocengineers.com

Project Name		
Section		
Project No.	Page	Date
Revision	By	Checked

Barachit beam.

Reinforcement needed:

$A_s = \dfrac{M_{Ed}}{0.9d \, f_{yd}}$ (d ~ 1000)

$f_{cd} = 28.33$

$d \sim 880$

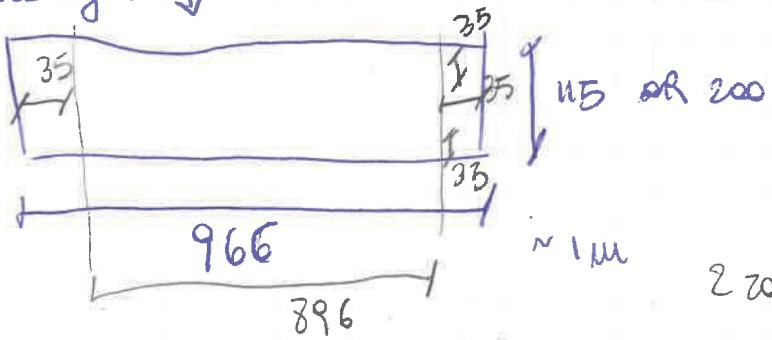
$f_{yd} = 435$ MPa

Secondary: $M_{Ed} = 5865$ hogging

150 — 110

15 K mm² OR 11 K mm² (10 Tie)

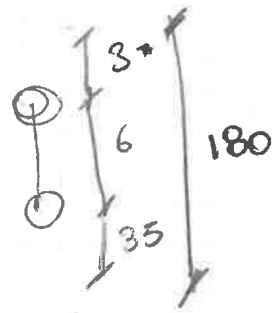
Primary: ↓



$\phi 25 = 4.91$

31 22 bars

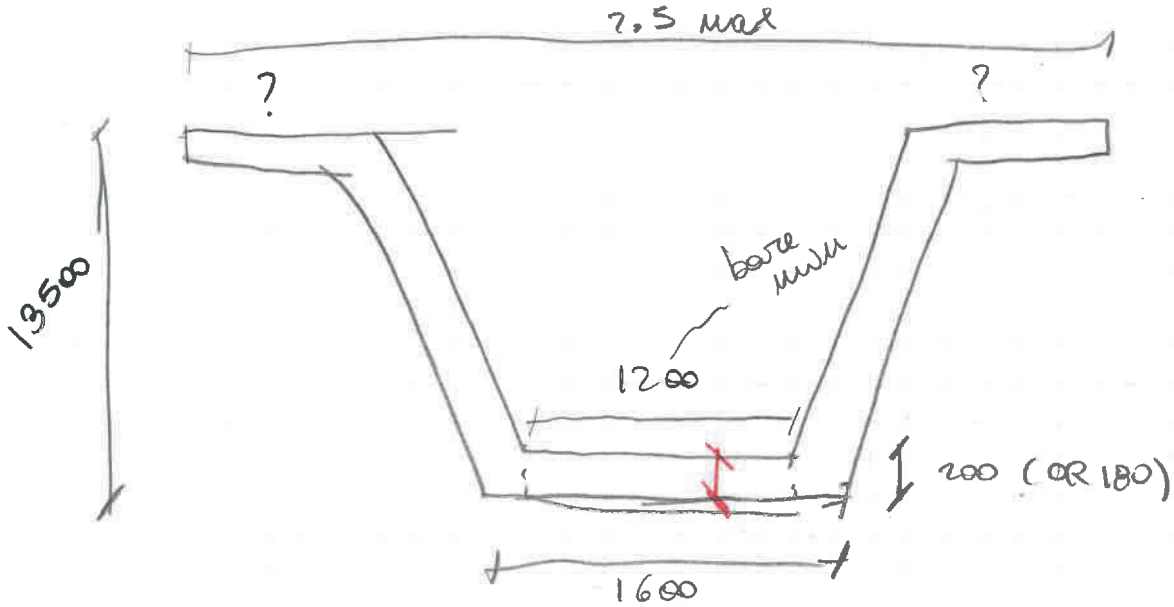
2 rows 25 bars ⇒



90 mm spacing

11 bars per row

11 bars per row 90 mm spacing = 1175 m



MEGA COLUMN DESIGN

SLENDERNESS: (5.8.3.2)

radius of gyration

$\lambda = l_0/i$ (5.14) $= \frac{10.6}{0.979} = 10.83$

Same EI as in (5.17)

$i_y = \sqrt{\frac{I}{A}} = \sqrt{\frac{1.917}{2}} = 0.979$

(5.8.3.1) (5.13N)

$\lambda_{lim} = 20ABC/\sqrt{n}$

$e_i = l_0/400 = 10.83/400 = 0.027m$

$\therefore e_i N_{ed} = 894.6$ kNm

$\therefore M_{02_y} = 21635 + 895 = 22530$ kNm

$M_{01_y} = 9073 + 895 = 9968$ kNm

$\therefore \lambda = 2.74/0.979 = 2.798$

5.8.3.2 (6): Members with varying cross section, the criterion in 5.8.3.1 should be checked with the effective length based on the buckling load:

$l_0 = \pi\sqrt{EI/N_B}$ (5.17)

$= 10.6m$

EI: representative bending stiffness

N_B: buckling load expressed in terms of EI

$N_B = n\pi^2 EI/L^2 = 709711409$ o

4.22×10^{10} N/m² $n=2$

$\rightarrow E = 42244$ MPa

$I = 1.917m^4$

$A_c = 2m^2$

$N_{ed} = 33{,}030$kN

$f_{cd} = 53.33$MPa

$r_m = 9968/22530 = 0.44$

$\gamma_{ef} = 0.8$

$A = 0.86$

$B = 1.1$

$C = 1.258$

$n = 0.31$

$\lambda_{lim} = \frac{20\times0.86\times1.1\times1.258}{\sqrt{0.31}} = 42.75$