THIS FOLDER WAS CHECKED BY ARUP ON

21.3.03

The checking process was to ensure that 3 identical sets of each Volume of the Operating and Maintenance Manuals existed

5-7 CARLTON GARDENS
LONDON SW1

OPERATING & MAINTENANCE INSTRUCTIONS
for the
BUILDING FABRIC

VOLUME 5
5.2.2 – Section 3 Part 2

Collated By :
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MASTER INDEX

Volume 1 Mechanical Services

MASTER INDEX – *cont.*

Volume 2 *BMS*

Volume 3 *Electrical Services*

***MASTER INDEX** – cont.*

Volume 4 *Lifts*

Volume 5 *Building Fabric*

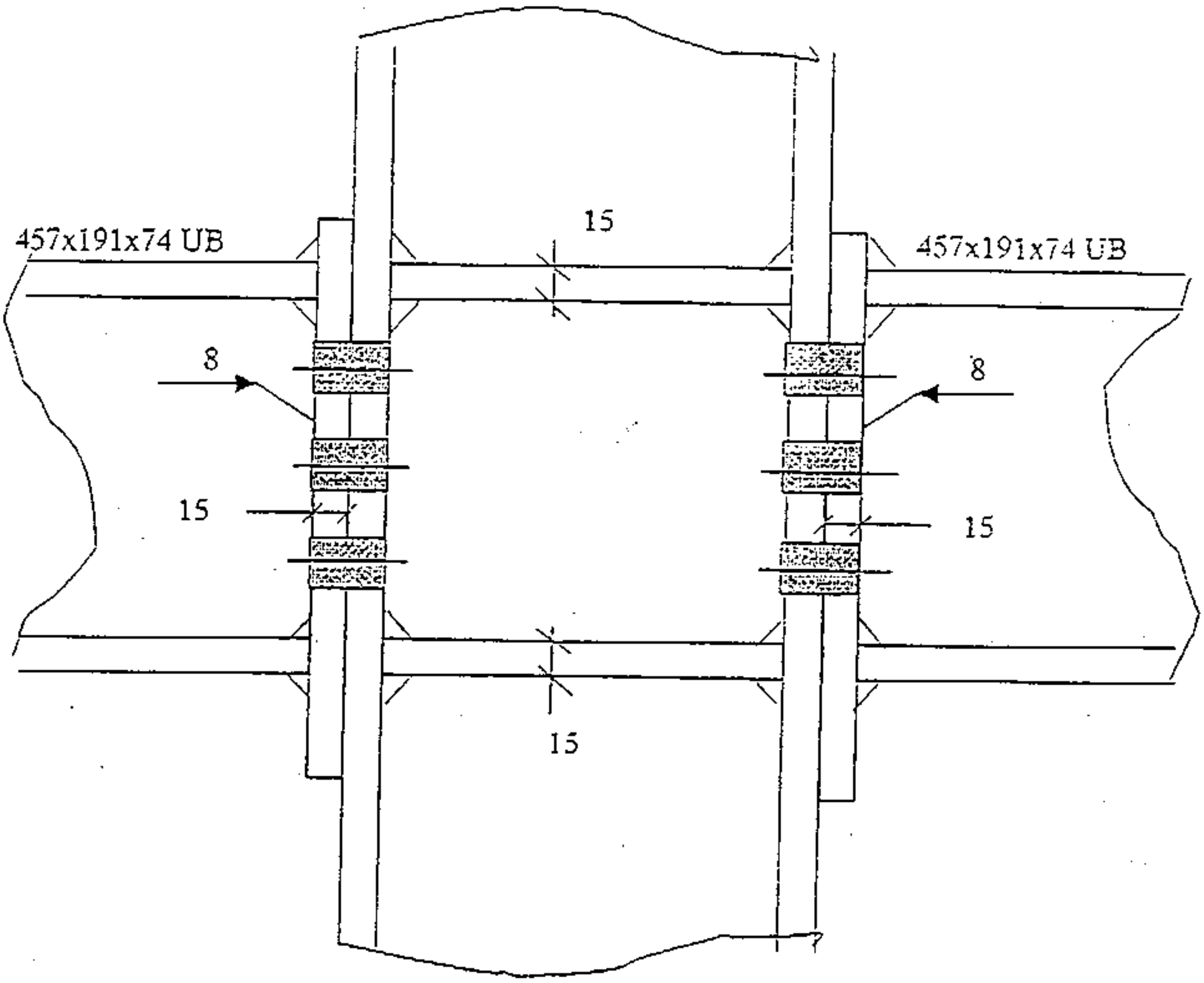
MASTER INDEX – *cont.*

Volume 5 Building Fabric – *cont.*

B

2

Connection Ref C41



All welds 12mm c.f.w. O.N.O

Calculation Sheet

Connection Ref 41

Axial = 2000kN (c)

Shear = 345kN

Agpy = 95.0 x 100 x 0.275 = 2612kN

Adopt 15thk end plate

Web welds = 8mm

Flange welds = 12mm

Shear - 6 bolts

Shear/bolt = 345 / 6 = 57.5kN

Minimum T of web of column = 11.5mm (assuming columns are grade 43)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B3-LG4 TO BEAM B2-LG

SERIES SHEET 1 OF 1

SHT. No C41A REV. A

BY LS DATE 08/98

EX. DATE

(CN02)

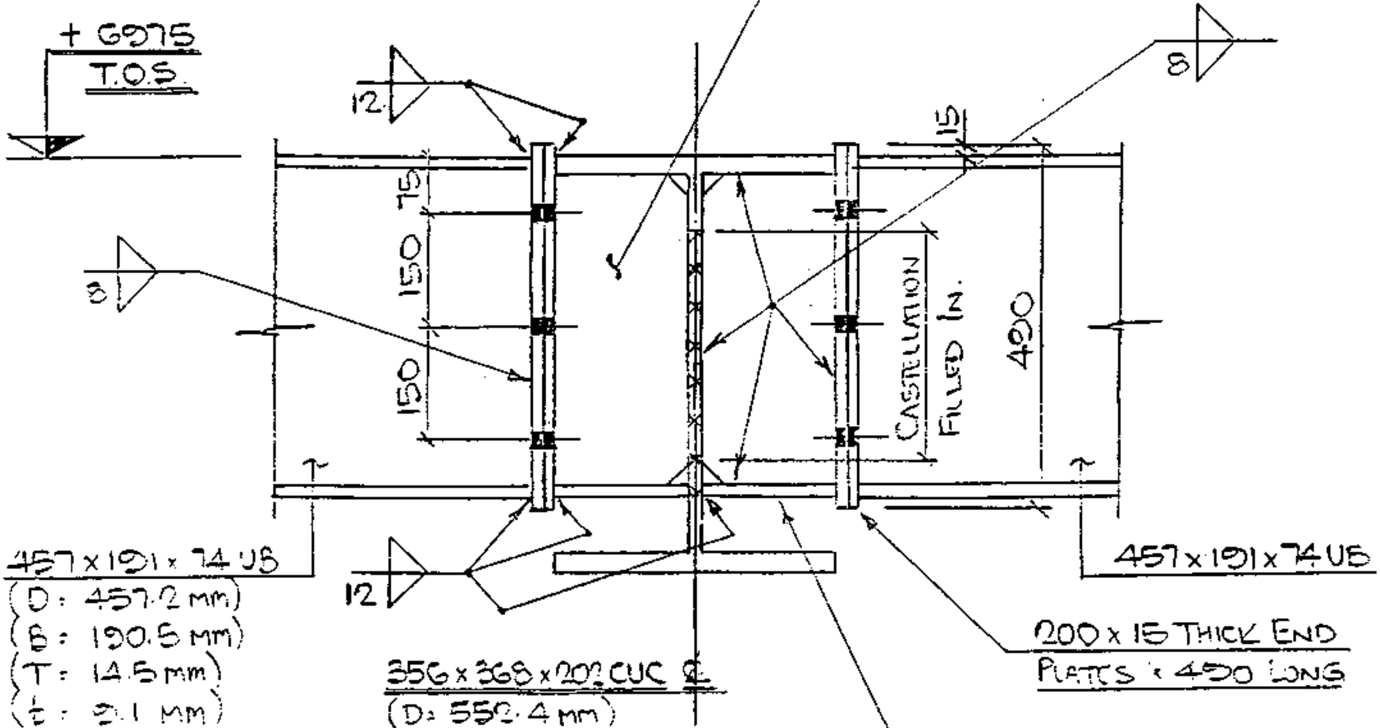
BEAM B3-LG4 TO BEAM B2-LG

457 x 191 x 74 UB TO 356 x 368 x 202 CUC

LOADINGS: —

END SHEAR = 345 kN

AXIAL LOAD = 2000 kN (COMPRESSION)



M20 BOLTS (GRADE 8.8)
@ 90 HORIZONTAL CENTRES

NOTE!

ANY CASTELLATION DIRECTLY IN LINE WITH CONNECTION MUST BE FILLED IN TO ALLOW PASSAGE OF COMPᴺ LOAD THROUGH WEB

(A) REFERENCE CALCULATIONS: —

SHEET No: — C41

REV 'A' 14/02/98 KB CALCULATION REFERENCE AMENDED

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B3-LG+ & B5-LG+ TO COLUMN CG

SERIES SHT. 1 OF 2

SHT. No. C41B REV. A

BY KB DATE 06/5

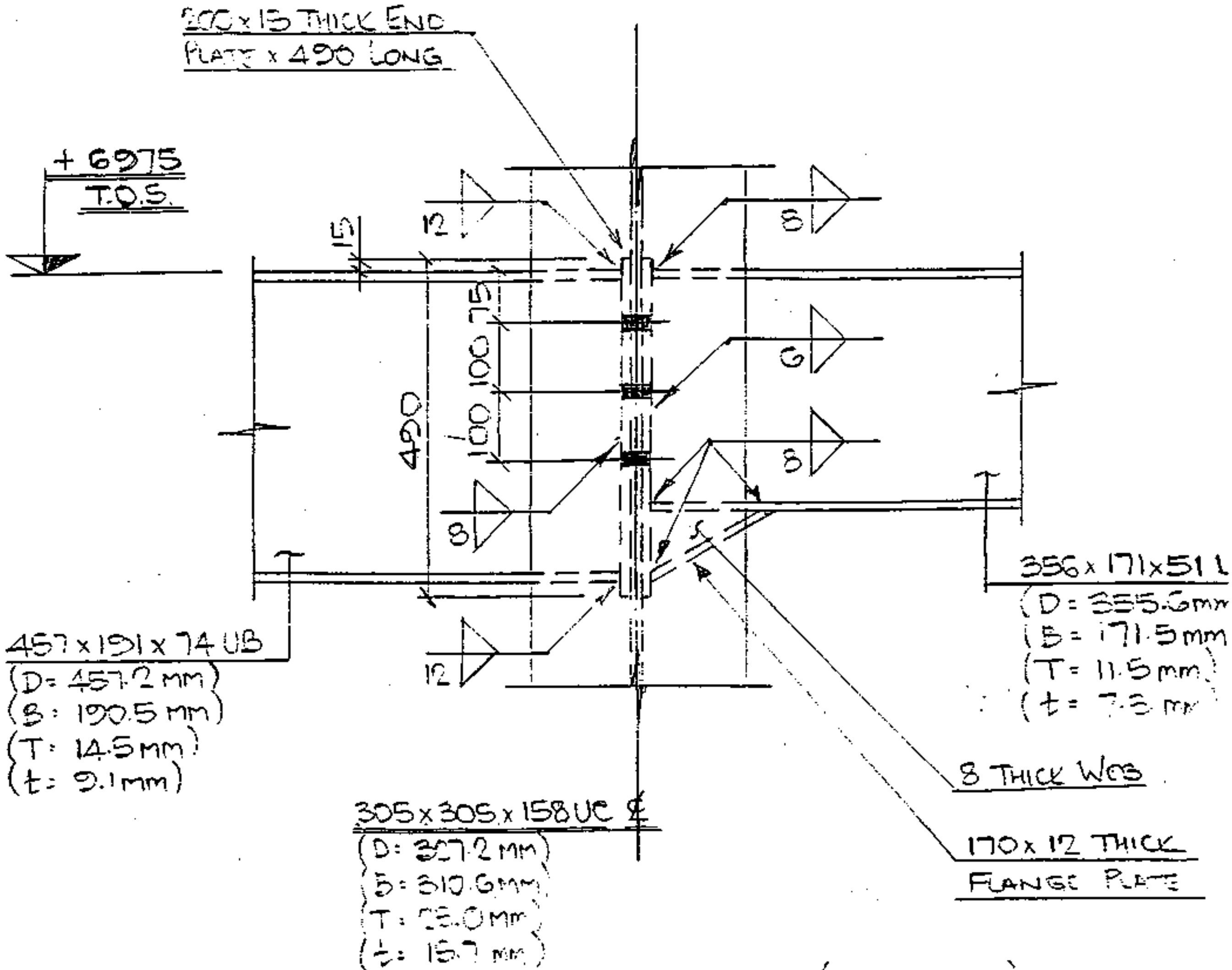
EX. DATE

(CN02)

REV	DATE	DESCRIPTION	BY
REV 'A'	14/08/98	REFERENCE CALCULATIONS ADDED; BEAM REACTIONS AMENDED	KB

BEAM B3-LG+ & B5-LG+ TO COLUMN CG

457 x 191 x 74 UB & 356 x 171 x 51 UB TO 305 x 305 x 158 UC



M20 BOLTS (GRADE 8.8)

6 LEG FILLET WELD (U.N.O.)

(A) REFERENCE CALCULATIONS :— SHEET NOS C41 & C43

(A) END REACTIONS :—

BEAM B3-LG+ :— END SHEAR = 345 kN
AXIAL LOAD = 2000 kN (COMP)

BEAM B5-LG+ :— END SHEAR = 200 kN
AXIAL LOAD = 2000 kN (COMP)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS	SERIES SHEET 1 OF 4
SUBJECT CONNECTION DESIGN	SHT. No. C41C REV.
BEAM 53-LG+ TO BEAM 54-LG	BY KB DATE 07/93
	EX. DATE

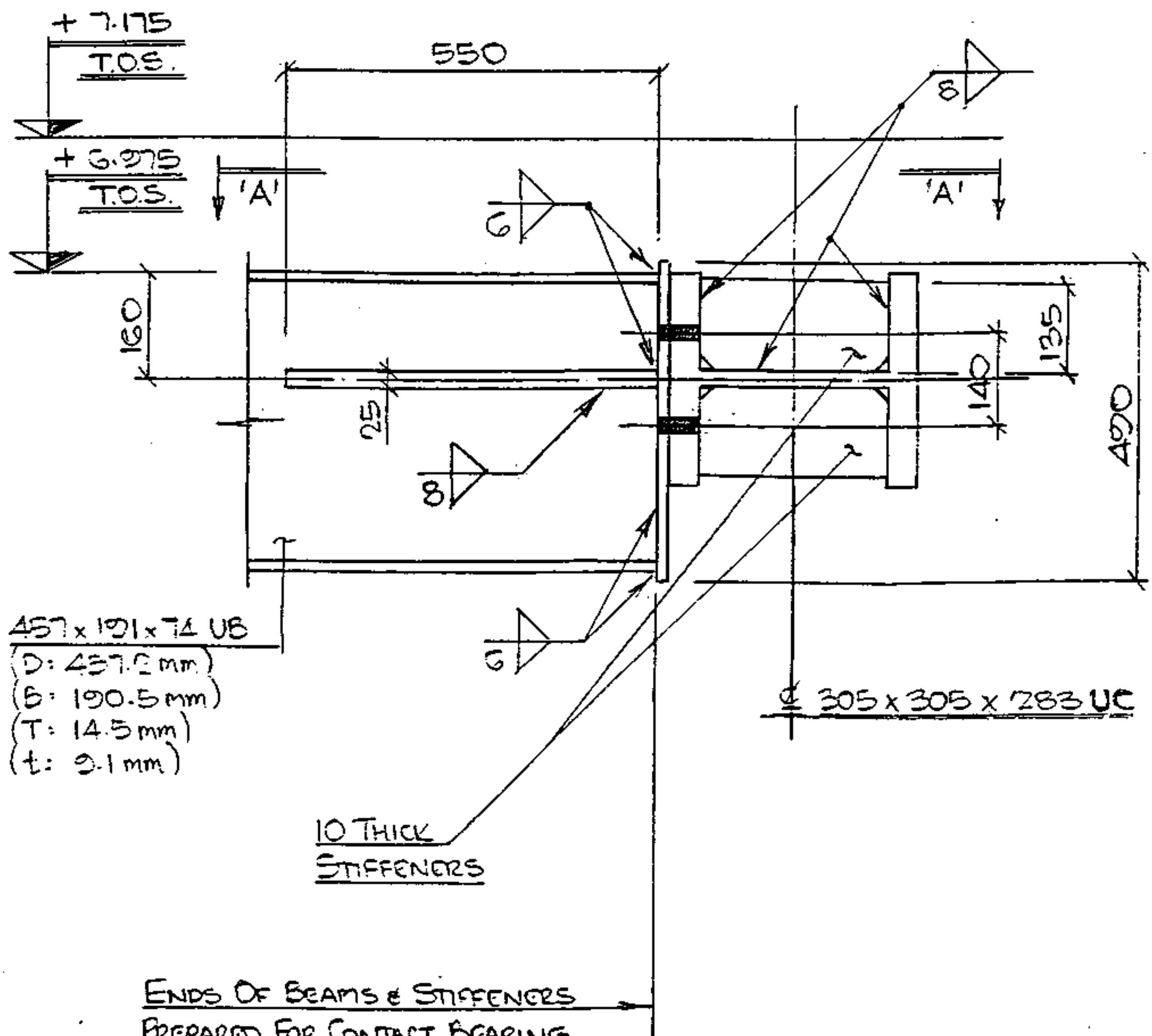
BEAM 53-LG+ TO BEAM 54-LG

457 x 191 x 74 UB TO 305 x 305 x 283 UC

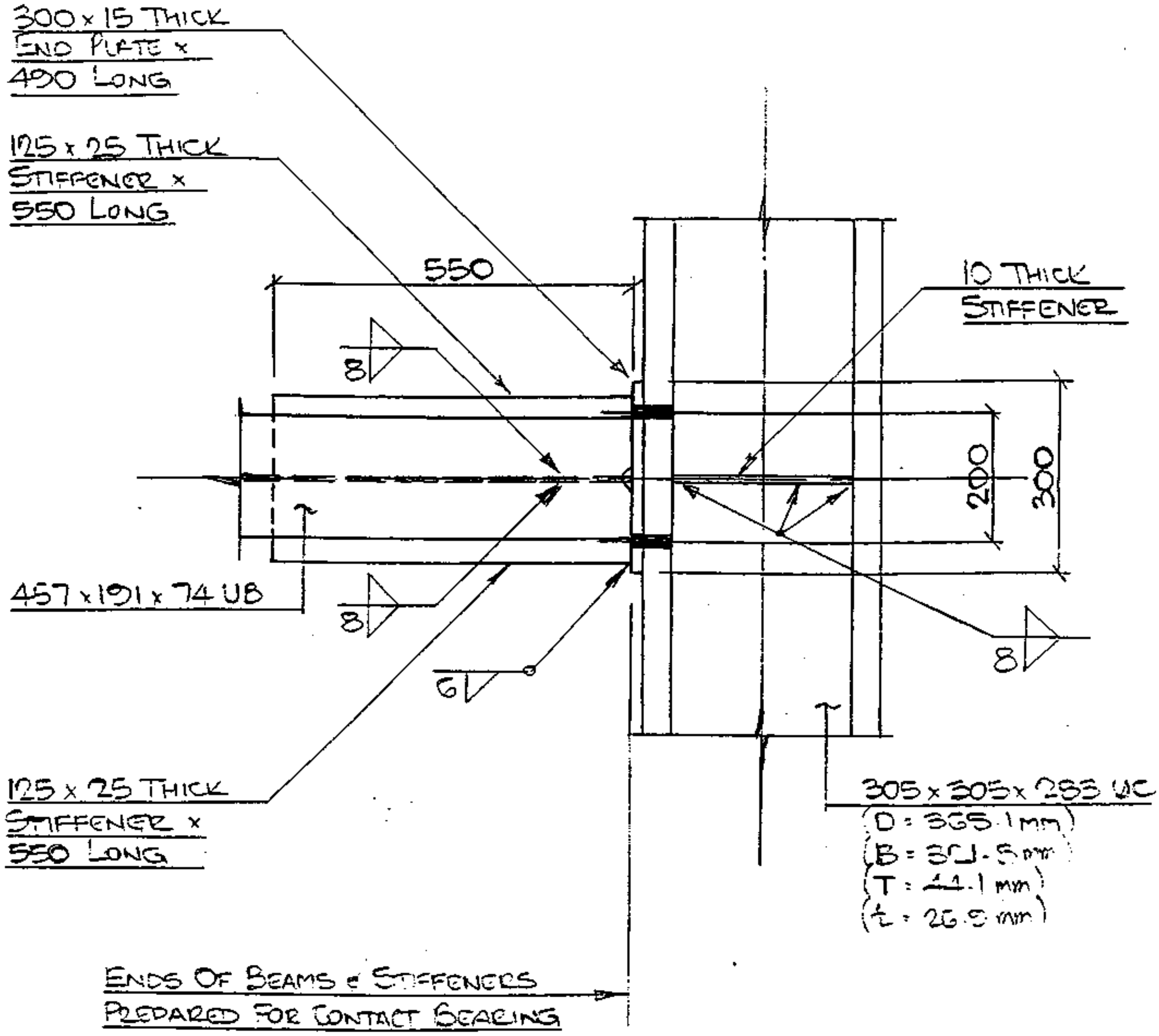
END REACTIONS :—

SHEAR = 345 kN

AXIAL = 2000 kN (COMPRESSION) ~ IN 457 UB SECTION.



M20 BOLTS (GRADE 8.8)



M20 BOLTS (GRADE 8.8)

VIEW ON 'A-A'

DESIGN OF UB WEB STIFFENER

AXIAL LOAD = 2000 KN (COMPN)

BEARING AREA REQD

= 2000 x 10³ ÷ 355 = 5634 mm²

ASSUME 25 THICK STIFFENER

MIN STIFFENER WIDTH REQD = 0.5 x 5634 ÷ 25 = 113 mm → SAY 125 mm

MIN STIFFENER LENGTH REQD

$$= \frac{2000 \times 1000}{2 \times 355 \times 0.6 \times 9.1} = 516 \text{ mm} \longrightarrow 550 \text{ mm}$$

USE :— 125 x 25 THICK STIFFENERS x 550 LONG

DESIGN OF WELD FOR STIFFENER TO UB WEB

SHEAR LOAD PER STIFFENER = 1000 KN

LEVER ARM = 125 − {0.5 x 115} = 67.5 mm

BENDING MOMENT = 1000 x 67.5 = 67500 KNmm

STIFFENER LENGTH = 550 mm

EFFECTIVE WELD LENGTH = 550 − (2 x 10) = 530 mm

WELD AREA = 2 x 530 = 1060 mm²

Z = 2 x 530² ÷ 6 = 93633 mm³

f_s = 1000 ÷ 1060 = 0.95 KN/mm²

f_m = 67500 ÷ 93633 = 0.72 KN/mm²

f_{RES} : $\sqrt{0.95^2 + 0.72^2}$ = 1.20 KN/mm^2

WELD STRENGTH (GRADE 50) = 255 N/mm^2

WELD SIZE REQ^D = $\dfrac{1.20 \times 10^3}{0.7 \times 255}$ = 6.7 mm ⟶ 8 LEG F.W. (MIN^m)

USE :— 8 LEG FILLET WELD

Kvaerner Cleveland Bridge Ltd

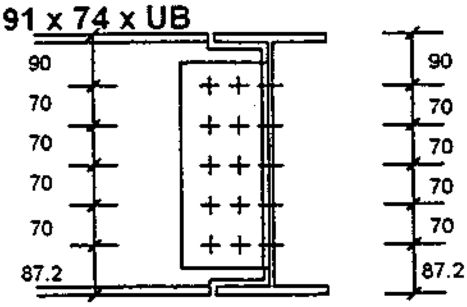
O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C42/1 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

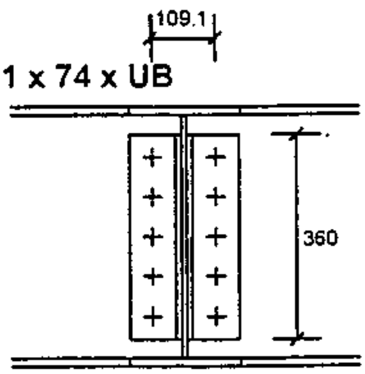
Details of Connection Ref:- Connection C42

Connected Beam 457 x 191 x 74 x UB



All Bolts M20 Grade

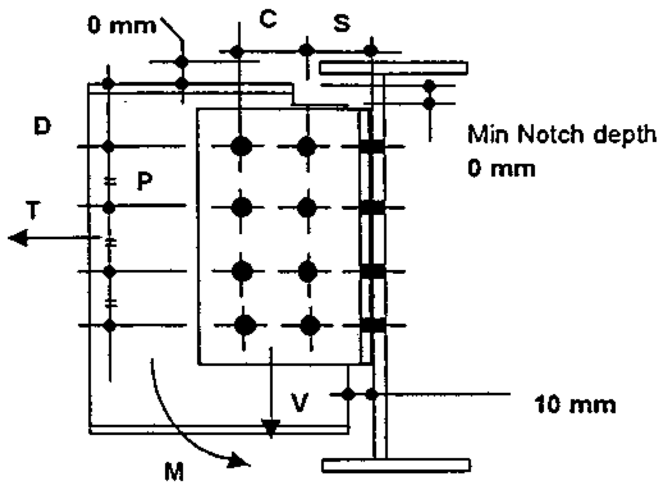
Supporting Beam 457 x 191 x 74 x UB



Cleats - 150 x 90 x 10 RSA

Calculation Sheet

Connection Ref:- **Connection C42**



Supporting Beam Loaded on Both sides		
Bolt Diameter	20	mm
Bolt Grade	8.8	
Angle grade	50	
Top beam to bolt D	90	OK
Bolt Centres C	50	mm
Bolt Pitch P	70	mm
Column offset S	50	mm
Shear Force V	430	kN
Tie Force T	75	kN
Bending Moment M	32.25	kNm
% Carried by bolt Group	100%	
Web Thickness	9.1	mm
Compensation plate Thk	0	mm
Angle Length	360	mm OK L>0.6D

Cleat long leg	150 mm	Bolt hole size			22 mm
Cleat short leg	90 mm				
Angle leg thickness	10 mm				
Connected Beam	457 x 191 x 74 x UB	Grade	50	py	355.0 N/mm2
Supporting Beam	457 x 191 x 74 x UB	Grade	50	py	355.0 N/mm2
Top Notch Length	102 mm				
Top Notch Depth	26 mm				
Bottom Notch Length	102 mm	OK			
Bottom Notch Depth	26 mm				

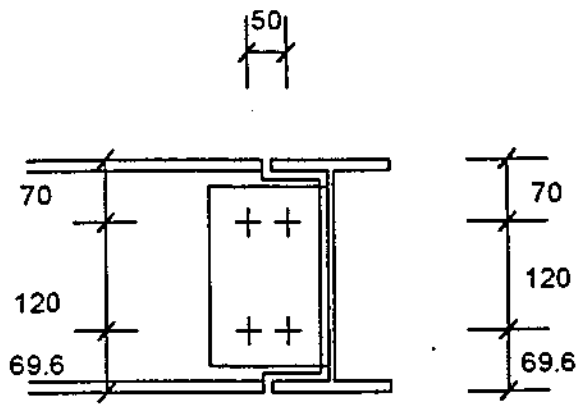
Bolt Eccentricities from centroid

ex	25 mm	ey1	70 mm
		ey2	140 mm
I of Bolt Group	104250 mm^3		
Fvv	43 kN	Fth	7.5 kN
Fmv	7.73 kN	Fmh	43.31 kN

Resultant force in bolt	71.80 kN	
Double Shear Capacity of Bolt	183.8 kN	
Bearing capacity in section	100.1 kN	Bolt Bearing Governs
Bearing capacity in Angles	220 kN	Bolt Bearing Governs
Limiting Value	100.1 kN	**Bolt Capacity adequate**

Connected Beam 254 x 146 x 43 x UB Grade 50

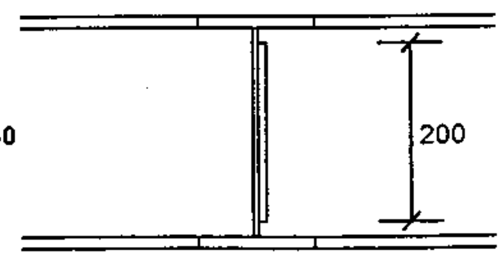
Supporting Beam 254 x 146 x 43 x UB Grade 50



All Bolts M20 Grade 8.8

Weld to supporting Member to be 6 mm Fillet weld
All Bolts M20 Grade 8.8

Fin Plates - 200 Lng x 150 x 10 Plt Grade 50



Details of Connection Ref:- Connection C43

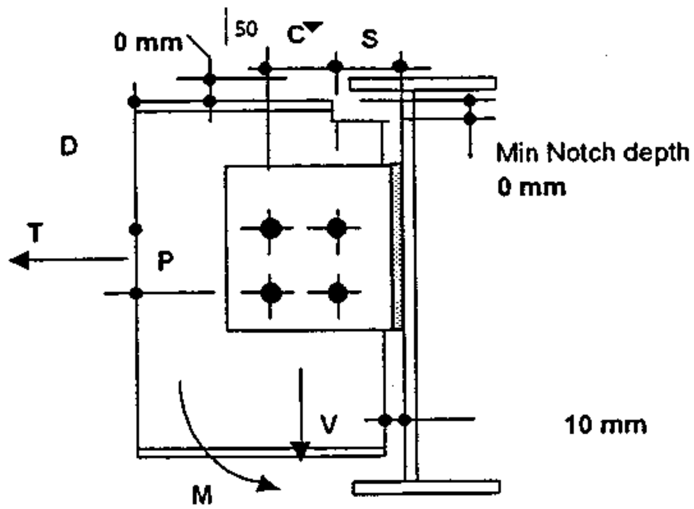
Kvaerner Cleveland Bridge Ltd

O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C43/2 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Connection Ref:-	**Connection C43**	
Supporting Beam Loaded on Both sides		
Bolt Diameter	20 mm	
Bolt Grade	8.8	
Fin Plate grade	50	
Top beam to bolt D	**70 OK**	
Bolt Centres C	50 mm	
Bolt Pitch P	120 mm	
Column offset S	50 mm	
Shear Force V	**140 kN**	
Tie Force T	**75 kN**	
Bending Moment M	**10.5 kNm**	
% Carried by bolt Group	**100%**	
Web Thickness	7.3 mm	
Compensation plate Thk	0 mm	
Fin Plate Length	**200 mm OK L>0.6D**	
Fin Plate Width	**150 mm**	Bolt hole size 22 mm



Fin Plate thickness	10 mm			
Connected Beam	254 x 146 x 43 x UB	Grade	50 py	355.0 N/mm2
Supporting Beam	254 x 146 x 43 x UB	Grade	50 py	355.0 N/mm2
Top Notch Length	82 mm			
Top Notch Depth	22 mm			
Bottom Notch Length	82 mm	OK		
Bottom Notch Depth	22 mm			

Bolt Eccentricities from centroid

ex	25 mm	ey1	60 mm

I of Bolt Group	16900 mm^3		
Fvv	35 kN	Fth	18.75 kN
Fmv	15.53 kN	Fmh	37.28 kN
Resultant force in bolt			75.45 kN
Shear Capacity of Bolt			91.9 kN
Bearing capacity in section			80.3 kN Bolt Bearing Governs
Bearing capacity in Fin Plate			220 kN Bolt Bearing Governs
Limiting Value		80.3 kN	**Bolt Capacity adequate**

Kvaerner Cleveland Bridge Ltd

O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C43/3 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Check Fin Plate Grade of Fin Plate = 50
Shear Area of Fin Plate 1800 mm^2
Nett Area of Fin Plate 1360 mm^2
Shear Capacity = 0.6xpy*Av = 383.4 kN Applied Shear =140 kN
but not exceeding 0.5xUs*Avnett = 333.2 kN **Limiting Value =333.2 OK**

Check Web Shear
Shear Area of Web 1416 mm^2
Nett Area of Web 1095 mm^2
Shear Capacity = 0.6xpy*Av = 301.7 kN Applied Shear =140 kN
but not exceeding 0.5xUs*Avnett = 268.3 kN **Limiting Value =268.3 OK**

Check Block Failure
Av1 1226.4 mm^2 Applied Shear =140 kN
Ateff 255.5 mm^2 **Limiting Value =323.8 OK**

Check Max Notch Length
Flat Plate Remaining

Flange Area	0	Stem Area	1573.9
Flange y	0	Stem y	108 mm
y bar	107.8	Flange h 107.8	Stem h 0.0
Flange I	0	Stem I	6096603
I Section	6096603	Z min Section	56555
Mcx of notched Section	20.1 kNm	D/tw	35.56
Maximum Notch Length	143 mm	**Notch Length OK**	

Check weld

Weld Size	6 mm	*Weld carries shear and tension only*	
Weld Capacity	0.903 kN/mm		
Total Weld Length	376 mm	Z of Weld	11781.3333 mm^2
Tension load per mm	0.199 kN/mm	BM load	0.000 kN/mm
Shear load per mm	0.372 kN/mm		
Resultant Load / mm	0.422 kN/mm	**Weld Strength Adequate**	

Check Tie capacity of Fin Plate
Tie Capacity of Fin Plate Tension = Le*tw*py = 553.80 kN Bolt Capacity in shear
Tie Capacity of Web in Tension = $L_e*t_w*p_y$ = 549.40 kN 367.6 kN
Tie Capacity of Web in Bearing = $L_e*t_w*p_y$ = 401.50 kN
Applied Tie Force 75.0 kN **Tie Force < Limiting Value - OK**

Check Moment Capacity of fin plate		Applied Shear	140.0 kN
I Nett of Plate	506.49 cm^4	Shear Capacity	333.2 kN
Z nett of plate	50.65 cm^3	Snett of plate	73.6 cm^3
Moment at 1st bolt line	7.00 kNm	Moment capacity	21.58 kNm

Plate Adequate in Bending
For Connection onto a Column instead of a beam
Note:- Web bending on supporting beam/column due to tie force has not been checked.

Kvaerner Cleveland Bridge Ltd

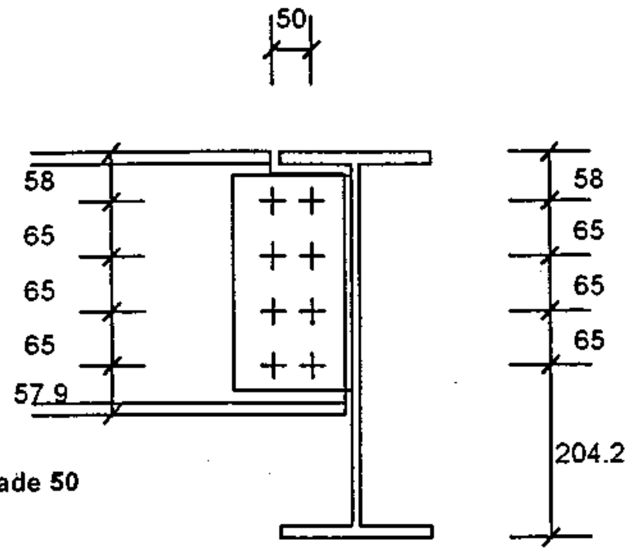
O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C44/1 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Details of Connection Ref:- Connection C44

Connected Beam 305 x 165 x 54 x UB Grade 50



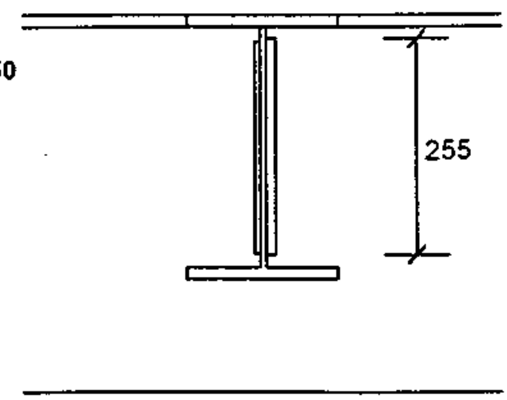
Supporting Beam 457 x 191 x 74 x UB Grade 50

- Compensation plate req'd 6 mm thk

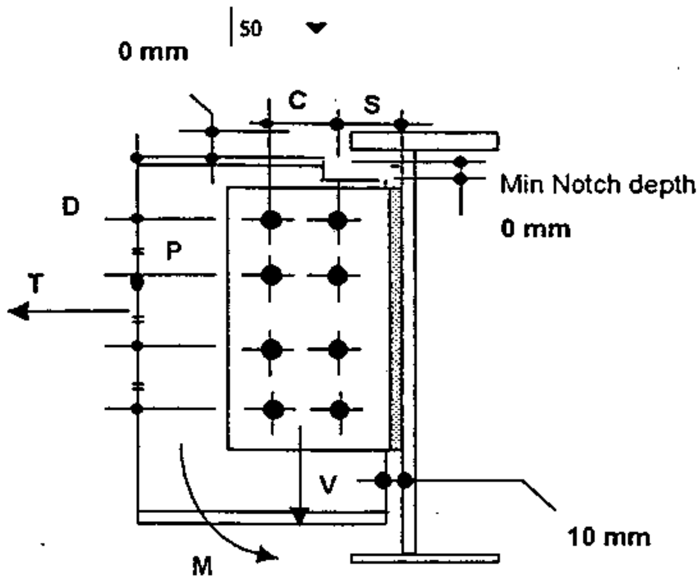
All Bolts M20 Grade 8.8

Weld to supporting Member to be 6 mm Fillet weld
All Bolts M20 Grade 8.8

Fin Plates - 255 Lng x 150 x 12 Plt Grade 50



Connection Ref:-	**Connection C44**			
Supporting Beam Loaded on Both sides				
Bolt Diameter	20 mm			
Bolt Grade	8.8			
Fin Plate grade	50			
Top beam to bolt D	58 OK			
Bolt Centres C	50 mm			
Bolt Pitch P	65 mm			
Column offset S	50 mm			
Shear Force V	350 kN			
Tie Force T	75 kN			
Bending Moment M	26.25 kNm			
% Carried by bolt Group	100%			
Web Thickness	7.7 mm			
Compensation plate Thk	6 mm			
Fin Plate Length	255 mm OK L>0.6D			
Fin Plate Width	150 mm	Bolt hole size		22 mm



' - Compensation plate req'd 6 mm thk

Fin Plate thickness	12 mm '			
Connected Beam	305 x 165 x 54 x UB	Grade	50 py	355.0 N/mm2
Supporting Beam	457 x 191 x 74 x UB	Grade	50 py	355.0 N/mm2
Top Notch Length	102 mm			
Top Notch Depth	26 mm			
Bottom Notch Length	0 mm	OK		
Bottom Notch Depth	0 mm			

Bolt Eccentricities from centroid

ex	25 mm	ey1	32.5 mm
		ey2	97.5 mm
I of Bolt Group	48500 mm³		
Fvv	43.75 kN	Fth	9.375 kN
Fmv	13.53 kN	Fmh	52.77 kN

Resultant force in bolt	84.52 kN	
Shear Capacity of Bolt	91.9 kN	
Bearing capacity in section with comp. plt.	120.56 kN	Edge Distance Governs
Bearing capacity in Fin Plate	99 kN	Edge distance Governs
Limiting Value	91.9 kN	**Bolt Capacity adequate**

Compensation plate to be grade 50

Calculation Sheet

Check Fin Plate Grade of Fin Plate = 50

Shear Area of Fin Plate	2754 mm^2		
Nett Area of Fin Plate	1698 mm^2		
Shear Capacity = 0.6xpy*Av =	586.6 kN	Applied Shear =350 kN	
but not exceeding 0.5xUs*Avnett =	416.0 kN	**Limiting Value =416.0 OK**	

Check Web Shear

Shear Area of Web	3359 mm^2		
Nett Area of Web	2153 mm^2		
Shear Capacity = 0.6xpy*Av =	715.5 kN	Applied Shear =350 kN	
but not exceeding 0.5xUs*Avnett =	527.6 kN	**Limiting Value =527.6 OK**	

Check Block Failure

Av1	2973.7 mm^2	Applied Shear =350 kN
Ateff	458.5 mm^2	**Limiting Value =745.7 OK**

Check Max Notch Length **(Compensation plate ignored)**

T Section Remaining

Flange Area	2285		Stem Area		2088.2
Flange y	7		Stem y		149 mm
y bar	74.9	Flange h	68.0	Stem h	74.4
Flange I	35742		Stem I		12799074
I Section	34976096		Z min Section		166527
Mcx of notched Section	59.12 kNm		D/tw		40.38
Maximum Notch Length	169 mm		**Notch Length OK**		

Check weld

Weld Size	6 mm	*Weld carries shear and tension only*	
Weld Capacity	0.903 kN/mm		
Total Weld Length	486 mm	Z of Weld	19683 mm^2
Tension load per mm	0.154 kN/mm	BM load	0.000 kN/mm
Shear load per mm	0.720 kN/mm		
Resultant Load / mm	0.737 kN/mm	**Weld Strength Adequate**	

Check Tie capacity of Fin Plate

Tie Capacity of Fin Plate Tension = Le*tw*py =	711.42 kN	Bolt Capacity in shear
Tie Capacity of Web in Tension = $L_e*t_w*p_y$ =	609.02 kN	735.2 kN
Tie Capacity of Web in Bearing = $L_e*t_w*p_y$ =	847.00 kN	

Applied Tie Force 75.0 kN **Tie Force < Limiting Value - OK**

Check Moment Capacity of fin plate		Applied Shear	350.0 kN
I Nett of Plate	1096.18 cm^4	Shear Capacity	416.0 kN
Z nett of plate	85.97 cm^3	Snett of plate	126.4 cm^3
Moment at 1st bolt line	17.50 kNm	Moment capacity	17.80 kNm

Moment Capacity reduced due to High Shear Condition Plate OK

For Connection onto a Column instead of a beam

Note:- Web bending on supporting beam/column due to tie force has not been checked.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B13-G & B5-G TO COLUMN C2

(RAINWATER PIPE) (CN13)

SERIES SHEET 1 OF 3

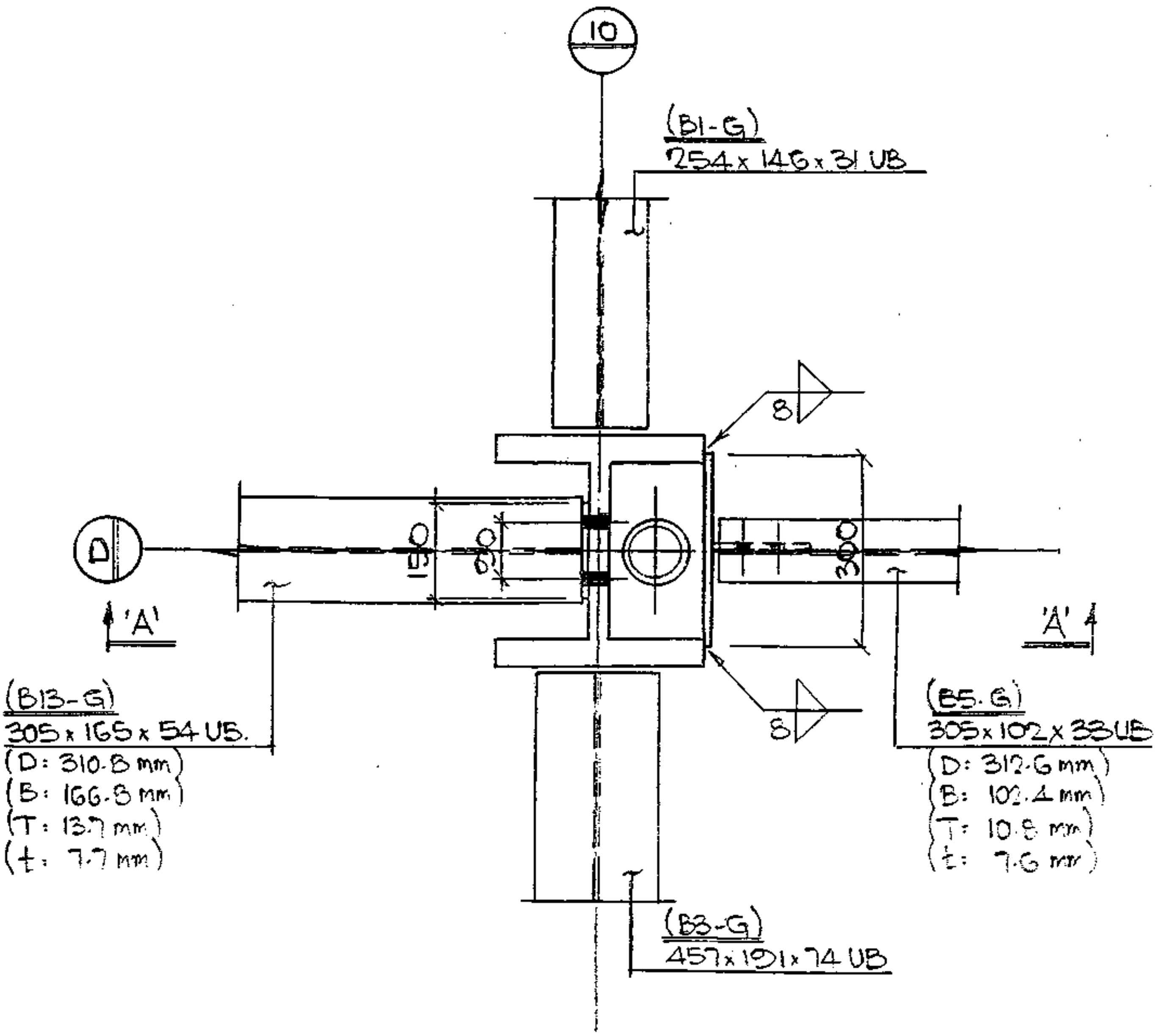
SHT. No. C44A REV.

BY LE DATE 07/98

EX. DATE

BEAM B13-G & B5-G TO COLUMN C2 ~ RAINWATER PIPE @ D/10

305 x 165 x 54 UB & 305 x 102 x 33 UB TO 305 x 305 x 283 UC

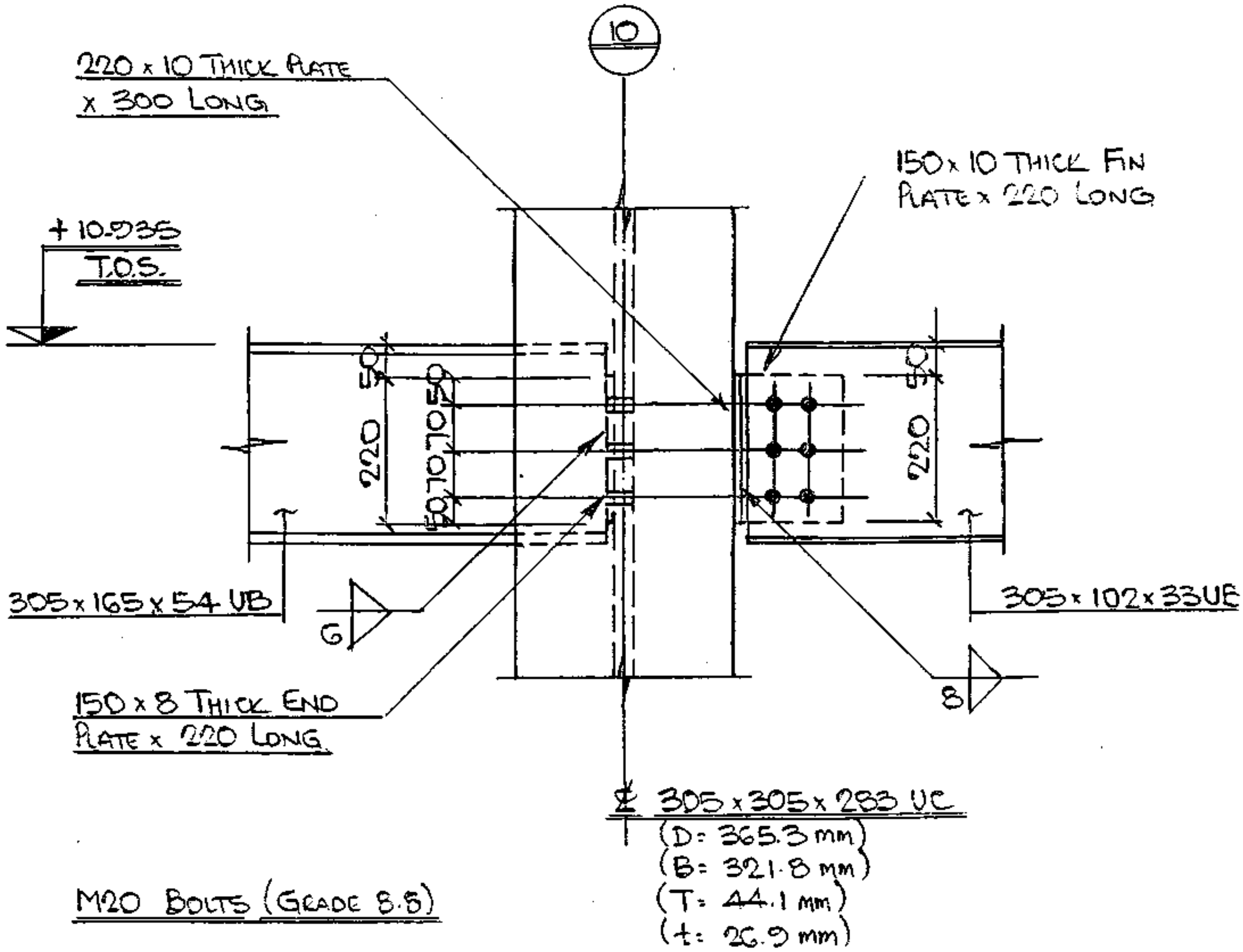


END REACTIONS —

B13-G : 350 kN

B5-G : 177 kN

(CN13)



View On 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

SERIES SHEET 3 OF 3

SHT. No. C44A REV.

BY KS DATE 07/05

EX. DATE

(CN13)

CHECK PLATE ACROSS TOES OF FLANGES

END REACTION : 350 KN

TRY : — 10 THICK PLATE

A_v : 0.9 x 10 x 220 = 1980 mm²

P_y : 355 N/mm²

P_v = 0.6 x 355 x 1980 ÷ 10³

= 421 KN > (0.5 x 350 KN) OK

BENDING MOMENT : 350 x 300 ÷ 4 = 26250 KNmm

Z = 10 x 220² ÷ 6 : 80666 mm³

Mc = P_y x Z

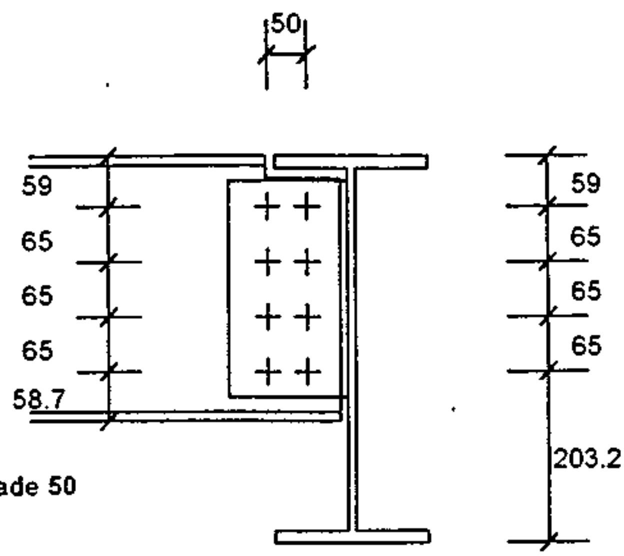
= 355 x 80666 ÷ 10³

= 28636 KNmm > 26250 KNmm OK

USE : — 10 THICK PLATE

Details of Connection Ref:- Connection C45

Connected Beam 305 x 102 x 33 x UB Grade 50



Supporting Beam 457 x 191 x 74 x UB Grade 50

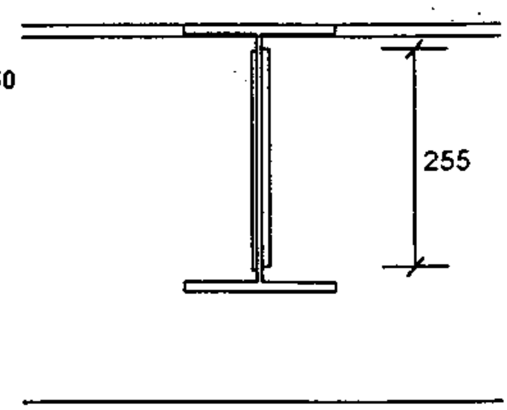
- Compensation plate req'd 6 mm thk

All Bolts M20 Grade 8.8

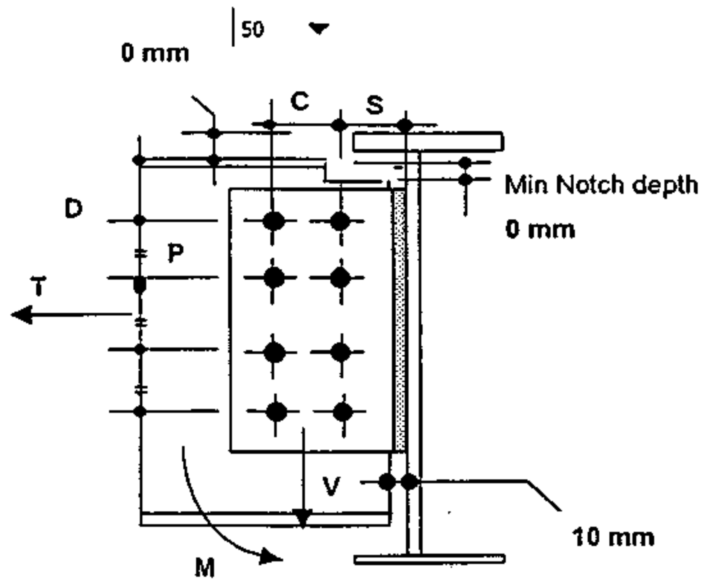
Weld to supporting Member to be 6 mm Fillet weld

All Bolts M20 Grade 8.8

Fin Plates - 255 Lng x 150 x 10 Plt Grade 50



Calculation Sheet



Connection Ref:-	**Connection C45**	
Supporting Beam Loaded on Both sides		
Bolt Diameter	20 mm	
Bolt Grade	8.8	
Fin Plate grade	50	
Top beam to bolt D	**59 OK**	
Bolt Centres C	50 mm	
Bolt Pitch P	65 mm	
Column offset S	50 mm	
Shear Force V	**246 kN**	
Tie Force T	**75 kN**	
Bending Moment M	**18.45 kNm**	
% Carried by bolt Group	**100%**	
Web Thickness	6.6 mm	
Compensation plate Thk	6 mm	
Fin Plate Length	**255 mm OK L>0.6D**	
Fin Plate Width	**150** mm	Bolt hole size 22 mm

' - Compensation plate req'd 6 mm thk

Fin Plate thickness	10 mm '			
Connected Beam	305 x 102 x 33 x UB	Grade	50 py	355.0 N/mm2
Supporting Beam	457 x 191 x 74 x UB	Grade	50 py	355.0 N/mm2
Top Notch Length	102 mm			
Top Notch Depth	26 mm			
Bottom Notch Length	0 mm	OK		
Bottom Notch Depth	0 mm			

Bolt Eccentricities from centroid

ex	25 mm	ey1	32.5 mm
		ey2	97.5 mm
I of Bolt Group	48500 mm^3		
Fvv	30.75 kN	Fth	9.375 kN
Fmv	9.51 kN	Fmh	37.09 kN

Resultant force in bolt	61.48 kN	
Shear Capacity of Bolt	91.9 kN	
Bearing capacity in section with comp. plt.	114.345 kN	Edge Distance Governs
Bearing capacity in Fin Plate	82.5 kN	Edge distance Governs
Limiting Value	82.5 kN	**Bolt Capacity adequate**

Compensation plate to be grade 50

Calculation Sheet

Check Fin Plate	Grade of Fin Plate =		50	
Shear Area of Fin Plate	2295 mm^2			
Nett Area of Fin Plate	1415 mm^2			
Shear Capacity = 0.6xpy*Av =		488.8 kN	Applied Shear =246 kN	
but not exceeding 0.5xUs*Avnett =		346.7 kN	**Limiting Value =346.7 OK**	
Check Web Shear				
Shear Area of Web	3096 mm^2			
Nett Area of Web	1988 mm^2			
Shear Capacity = 0.6xpy*Av =		659.5 kN	Applied Shear =246 kN	
but not exceeding 0.5xUs*Avnett =		487.0 kN	**Limiting Value =487.0 OK**	
Check Block Failure				
Av1	2736.0 mm^2		Applied Shear =246 kN	
Ateff	420 mm^2		**Limiting Value =685.7 OK**	

Check Max Notch Length **(Compensation plate ignored)**

T Section Remaining

Flange Area	1106		Stem Area	1820.9
Flange y	5		Stem y	149 mm
y bar	94.6	Flange h 89.2	Stem h	54.2
Flange I	10750		Stem I	11550952
I Section	25700510		Z min Section	133777
Mcx of notched Section	47.49 kNm		D/tw	47.38
Maximum Notch Length	193 mm		**Notch Length OK**	

Check weld

Weld Size	6 mm	*Weld carries shear and tension only*	
Weld Capacity	0.903 kN/mm		
Total Weld Length	486 mm	Z of Weld	19683 mm^2
Tension load per mm	0.154 kN/mm	BM load	0.000 kN/mm
Shear load per mm	0.506 kN/mm		
Resultant Load / mm	0.529 kN/mm	**Weld Strength Adequate**	

Check Tie capacity of Fin Plate

Tie Capacity of Fin Plate Tension = Le*tw*py =	592.85 kN	Bolt Capacity in shear
Tie Capacity of Web in Tension = L_e*t_w*p_y =	525.77 kN	735.2 kN
Tie Capacity of Web in Bearing = L_e*t_w*p_y =	726.00 kN	

Applied Tie Force 75.0 kN **Tie Force < Limiting Value - OK**

Check Moment Capacity of fin plate		Applied Shear	246.0 kN
I Nett of Plate	913.48 cm^4	Shear Capacity	346.7 kN
Z nett of plate	71.65 cm^3	Snett of plate	105.4 cm^3
Moment at 1st bolt line	12.30 kNm	Moment capacity	27.16 kNm

Moment Capacity reduced due to High Shear Condition Plate OK

For Connection onto a Column instead of a beam

Note:- Web bending on supporting beam/column due to tie force has not been checked.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

B5-LG# TO B1-LG

SERIES SHT 1 OF 4

SHT. No C47A REV.

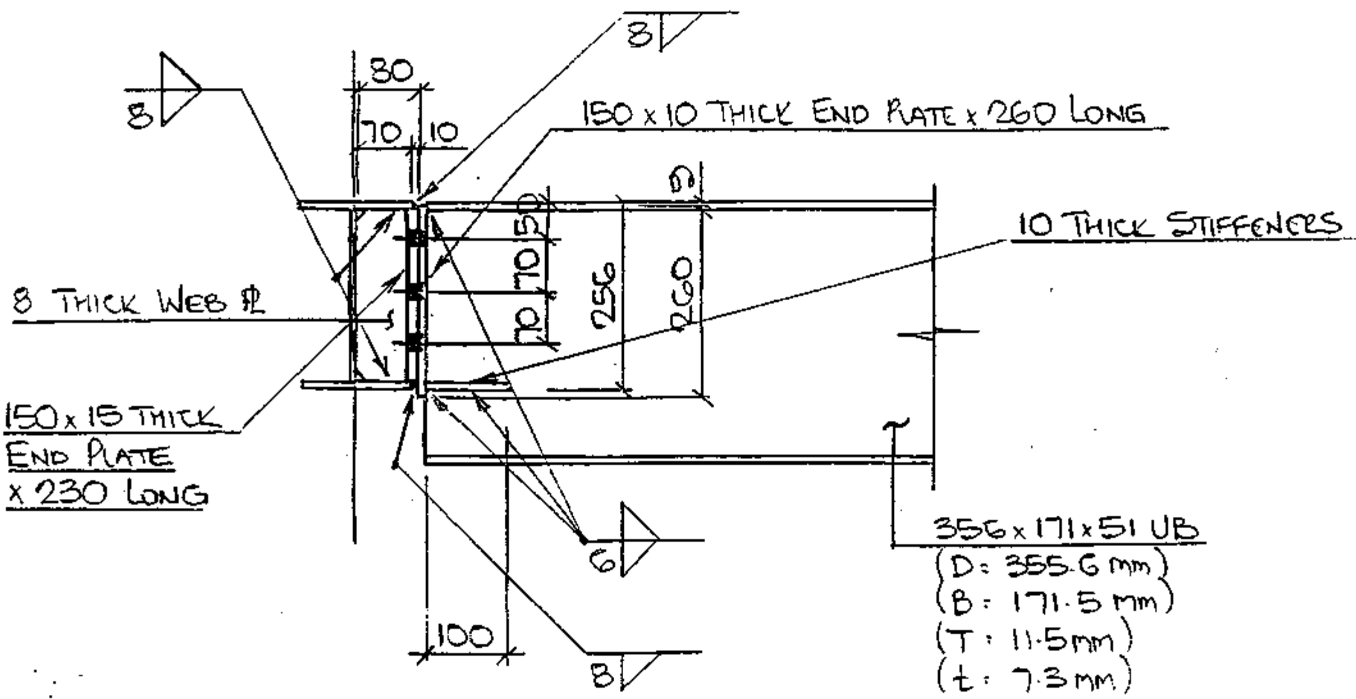
BY KB DATE 06/99

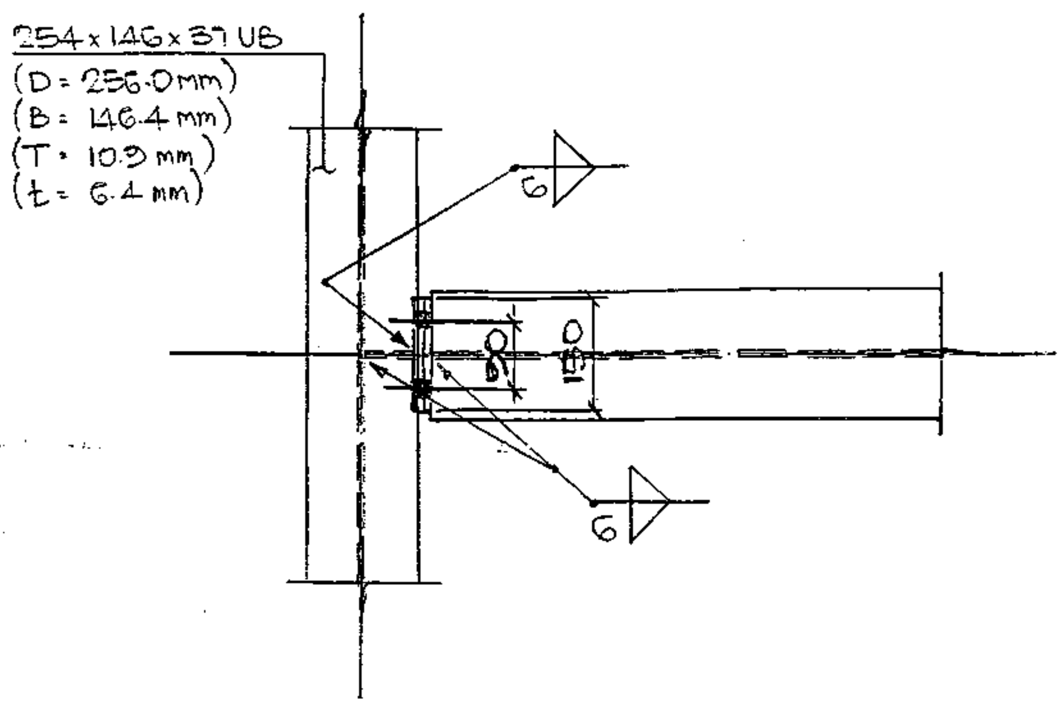
EX. DATE

BEAM B5-LG # TO BEAM B1-LG

356 x 171 x 51 UB TO 254 x 146 x 37 UB

END SHEAR = 400 KN.





Kvaerner Cleveland Bridge Ltd.

SERIES SHT. 2 OF 4

O/No. 325 JOB CARLTON GARDENS

SHT. No. C47A REV.

SUBJECT CONNECTION DESIGN

BY KB DATE 06/98

EX. DATE

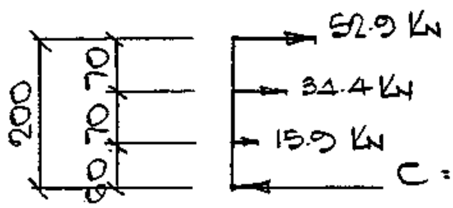
<u>DESIGN OF BOLTS</u>

SHEAR LOAD : 400 KN

LEVER ARM : 80 mm

BENDING MOMENT : 400 x 80 = 32000 KN mm

<u>TRY : — 6 No ~ M20 BOLTS (GRADE 8.8)</u>



<u>TAKE MOMENTS ABOUT ℄ OF BOTTOM FLANGE</u>

$C = 2 \times (52.9 + 34.4 + 15.9) = 206.4$ KN

$I_{BOLTS} = 2 \times (60^2 + 130^2 + 200^2) = 121000 \text{ mm}^4$

$Z = 121000 \div 200 = 605 \text{ mm}^3$

$f_s = 400 \div 6 = 66.7$ KN

$f_t = 32000 \div 605 = 52.9$ KN

$P_s = 91.8$ KN $>$ 66.7 KN OK

$P_T = 110.0$ KN $>$ 52.9 KN OK

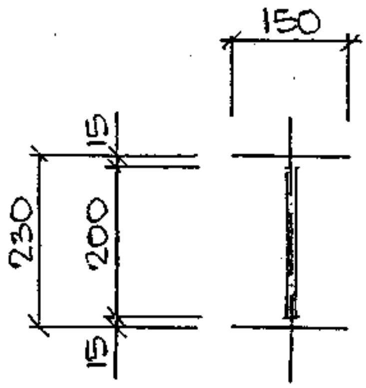
<u>COMBINED LOAD : —</u>

$$\frac{f_s}{P_s} + \frac{f_t}{P_T} \leq 1.40$$

$$\frac{66.7}{91.8} + \frac{52.9}{110.0} = 1.21 < 1.40 \text{ OK}$$

<u>USE : — 6 No ~ M20 BOLTS (GRADE 8.8)</u>

DESIGN OF WELD FOR END PLATE TO 254 x 146 UB



CONSIDER WELD OF UNIT LEG LENGTH

AREA = 2 x (150 + 200) = 700 mm²

$I_{xx} = \left(\frac{2 \times 150}{12}\right) + \left(\frac{2 \times 200^3}{12}\right) + (2 \times 150 \times 115^2) = 5300858 \text{ mm}^4$

$Z_{xx} = 5300858 \div 115 = 46094 \text{ mm}^3$

$f_s = 400 \div 700 = 0.58 \text{ KN/mm}^2$

$f_m = 400 \times 80 \div 46094 = 0.70 \text{ KN/mm}^2$

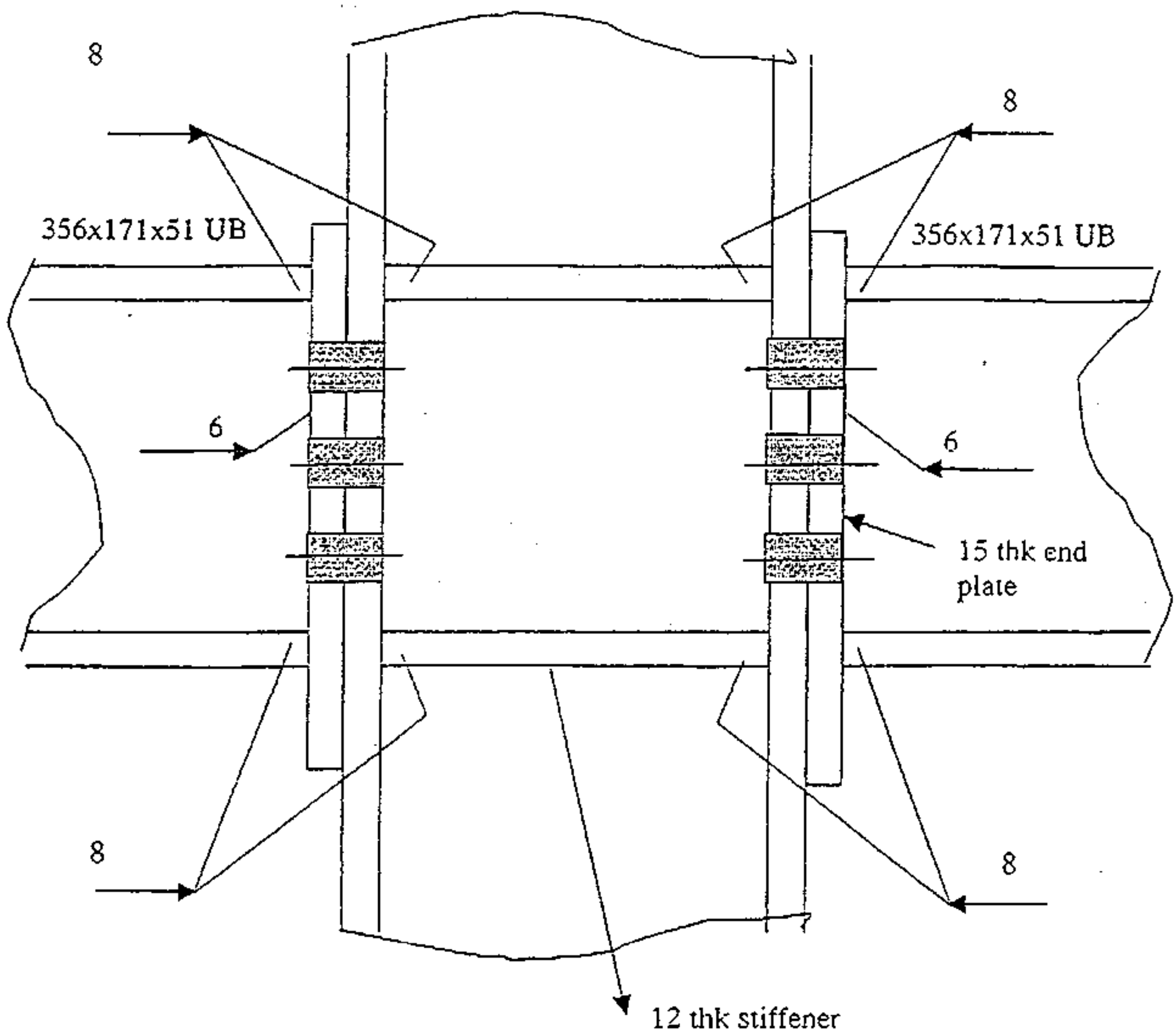
$f_{res} = \sqrt{0.58^2 + 0.70^2} = 0.91 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 50) = 255 N/mm²

WELD SIZE REQD = $\frac{0.91 \times 10^3}{0.70 \times 255}$ = 5.1 mm ⟶ 6 LEG F.W. (MINm)

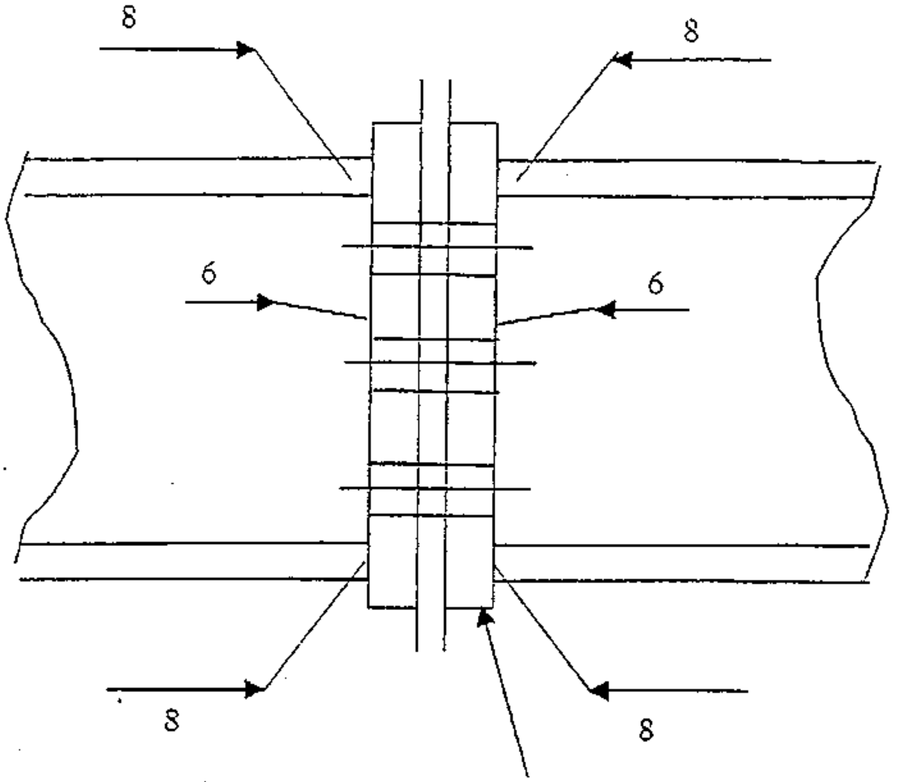
USE: — 6 LEG FILLET WELD

Connection Ref C48 & C49



Calculation Sheet

For compression through web of column End plate on either side of web should line up



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

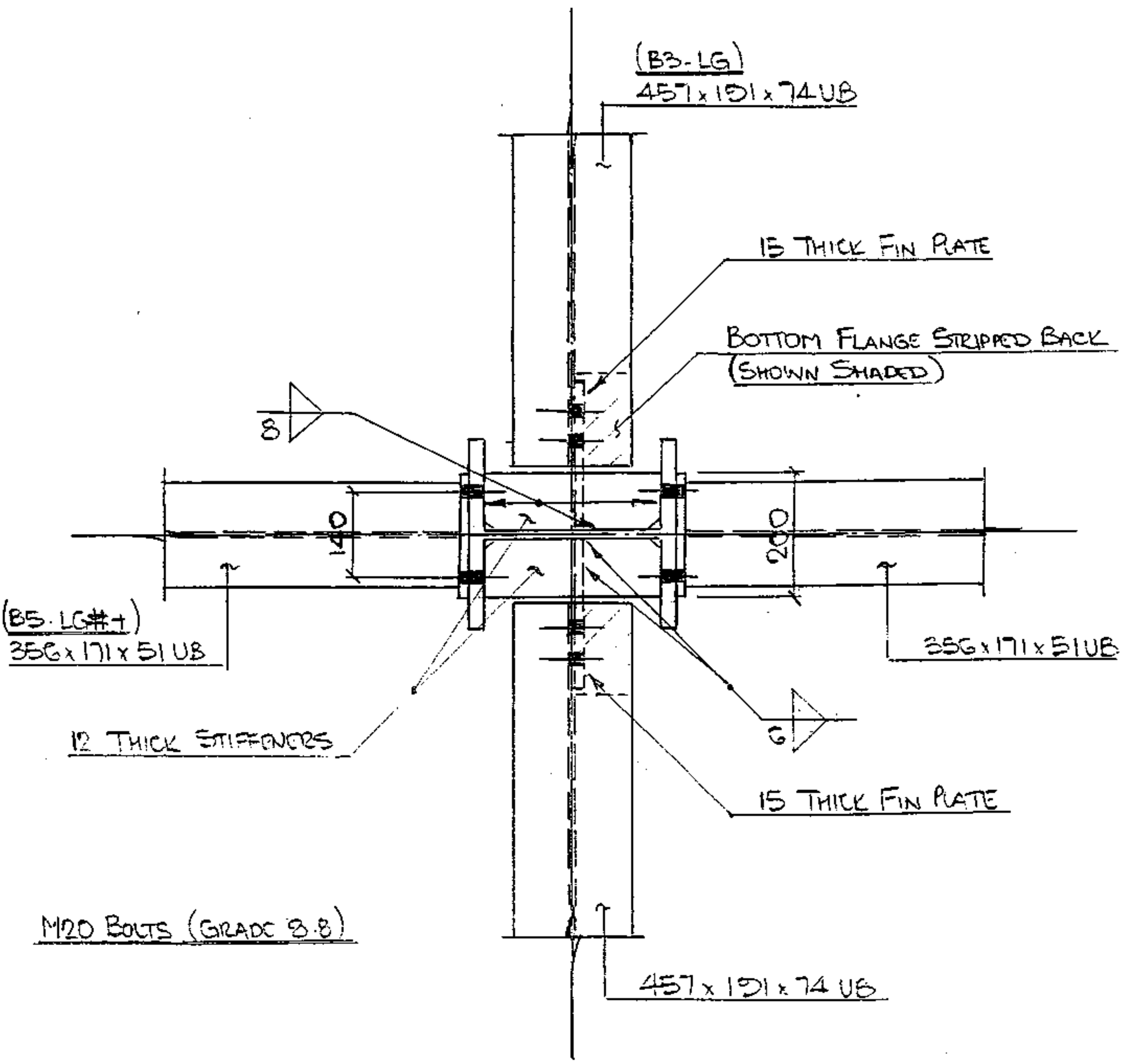
A SERIES SHEET No 6 OF 10

SHT. No. C48 REV. A

BY KB DATE 06/98

EX. DATE

(CN02)



VIEW ON 'A-A'

REV 'A' 14/08/98 SHEET No REVISED 2 OF 6 → 6 OF 10 KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

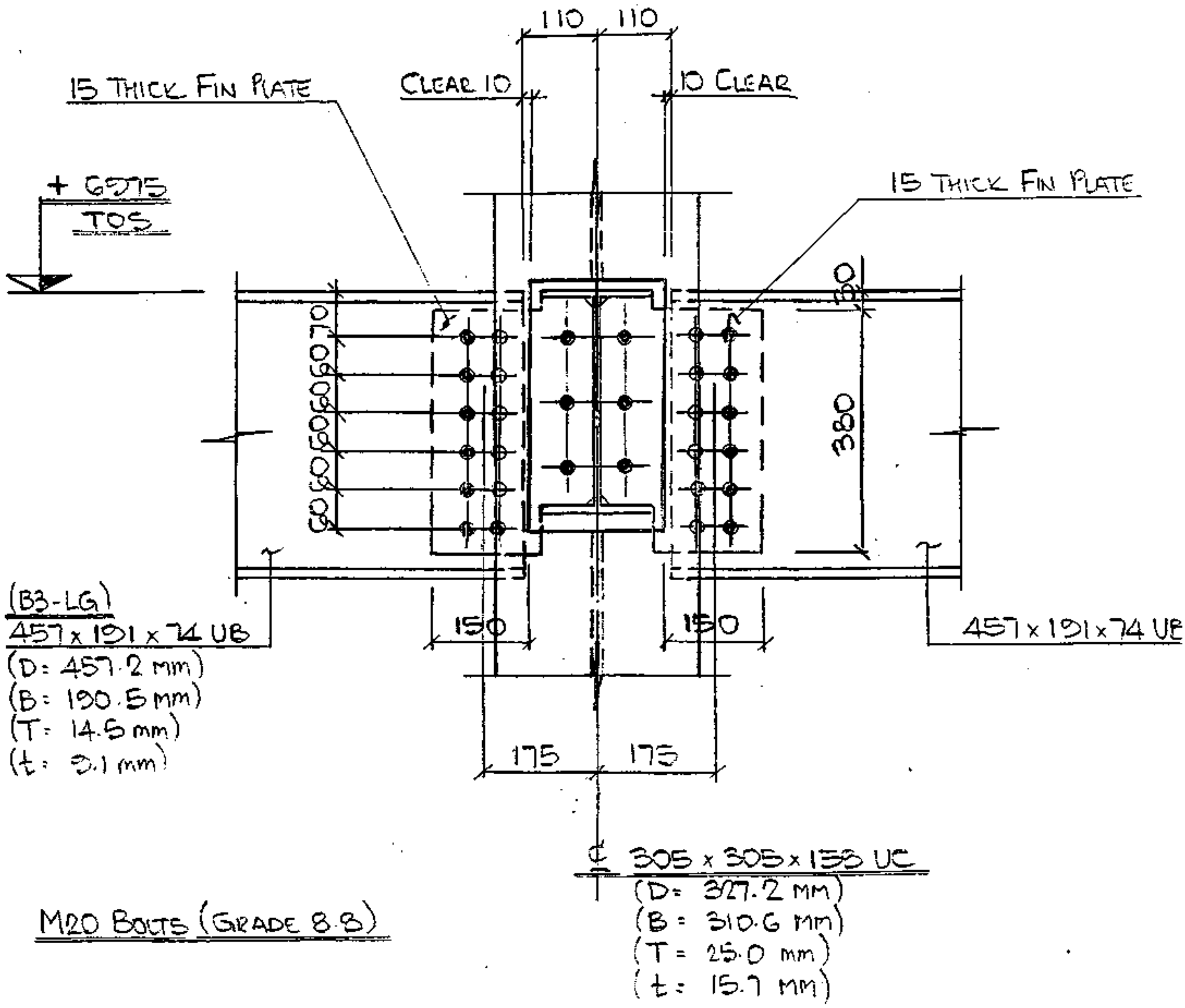
SERIES SHEET No 7 OF 10

SHT. No. C48 REV. A

BY KB DATE 06/98

EX. DATE

(CN02)



VIEW ON 'B-B'

REV 'A' 14/08/98 SHEET No REVISED 3 of 6 → 7 of 10 KB

KB

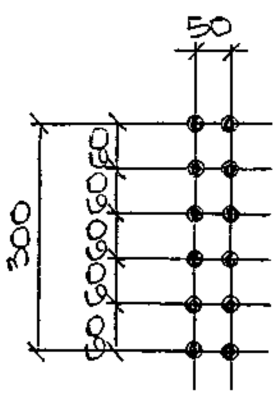
DESIGN OF BOLTS

TRY :- 12 No ~ M20 BOLTS (GRADE 8.8)

END SHEAR = 345 KN.

LEVER ARM = 100 + 50 + 25 = 175 mm

BENDING MOMENT = 345 x 175 = 60375 KNmm



$I_{xx} = 4 \times (30^2 + 90^2 + 150^2) = 126000 \text{ mm}^4$

$I_{yy} = 12 \times 25^2 = 7500 \text{ mm}^4$

$I_p = 126000 + 7500 = 133500 \text{ mm}^4$

$\bar{r} = \sqrt{25^2 + 150^2} = 152.0 \text{ mm}$

$Z_p = 133500 \div 152.0 = 878 \text{ mm}^3$

$f_s = 345 \div 12 = 28.8$ KN

$f_m = 60375 \div 878 = 68.8$ KN

$f_m (V_c) = 68.8 \times 25.0 \div 152.0 = 11.3$ KN

$f_m (H_c) = 68.8 \times 150.0 \div 152.0 = 67.9$ KN

$f_{RES} = \sqrt{(28.8 + 11.3)^2 + 67.9^2} = 78.9$ KN

BOLT SHEAR ; $P_s = 245 \times 375 \div 10^3 = 91.9$ KN

BOLT BEARING ; $P_{bb} = 1035 \times 20 \times 9.1 \div 10^3 = 188.3$ KN

PLY BEARING ; P_{bs}

(i) $d \times t \times p_{bs} = 20 \times 9.1 \times 550 \div 10^3 = 100.1$ KN

(ii) $\frac{1}{2} \times e \times t \times p_{bs} = \frac{1}{2} \times 40 \times 9.1 \times 550 \div 10^3 = 100.1$ KN

REV 'A' 14/08/98 SHEET No REVISED 4 OF 6 — 8 OF 10

Kvaerner Cleveland Bridge Ltd.

SERIES Sheet No 10 of 10
O/No. 325 JOB Carlton Gardens
SHT. No. C48 REV. A
SUBJECT Connection Design
BY KB DATE 06/98
EX. ... DATE ...
(CN02)

$A_V = 0.9 \times 380 \times 10 = 3420\ mm^2$

$A_{VNET} = 3420 - (6 \times 10 \times 22) = 2100\ mm^2$

$0.6 \times P_y \times A_V = 0.6 \times 355 \times 3420 \div 10^3 = 728\ KN$

$0.5 \times U_s \times A_{VNET} = 0.5 \times 490 \times 2100 \div 10^3 = 514\ KN$

$P_V = \underline{514\ KN > 345\ KN}$ OK

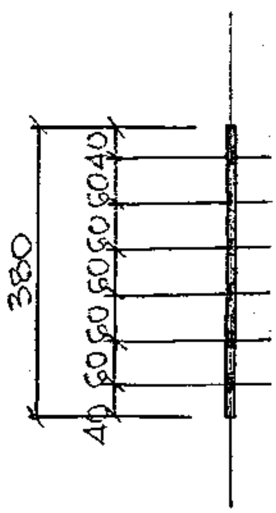
$A_{V1} = (40 + \{60 \times 5\}) \times 10 = 3400\ mm^2$

$A_{TEFF} = (100 - \{2.5 \times 22\}) \times 10 = 450\ mm^2$

$P_{VB} = (0.6 \times P_y \times A_{V1}) + (0.5 \times U_s \times A_{TEFF})$

$= (0.6 \times 355 \times 3400 \div 10^3) + (0.5 \times 490 \times 450 \div 10^3)$

$P_{VB} = \underline{834\ KN > 345\ KN}$ OK



$I_{XX} = 10 \times 380^3 \div 12 = 45726666\ mm^4$

$I_{NET} = 45726666 - \left(\frac{6 \times 10 \times 22^3}{12}\right)$

$- (2 \times 10 \times 22 \times \{30^2 + 90^2 + 150^2\})$

$= 31813426\ mm^4$

$Z = 31813426 \div 190 = 167439\ mm^3$

$M_C = 355 \times 167439 \div 10^3$

$= \underline{59440\ KNmm < 60375\ KNmm}$ FAILS

$\underline{\text{Plate Thickness Req}^d}$ $= 10.0 \times 60375 \div 59440 = 10.2\ mm \rightarrow 15\ mm$

Use :- 15 Thick Fin Plate

KB

Sheet No Revised 6 of 6 → 10 of 10

Rev 'A' 14/08/98

REV 'A' 14/08/98 CALCULATION REFERENCE AMENDED KB

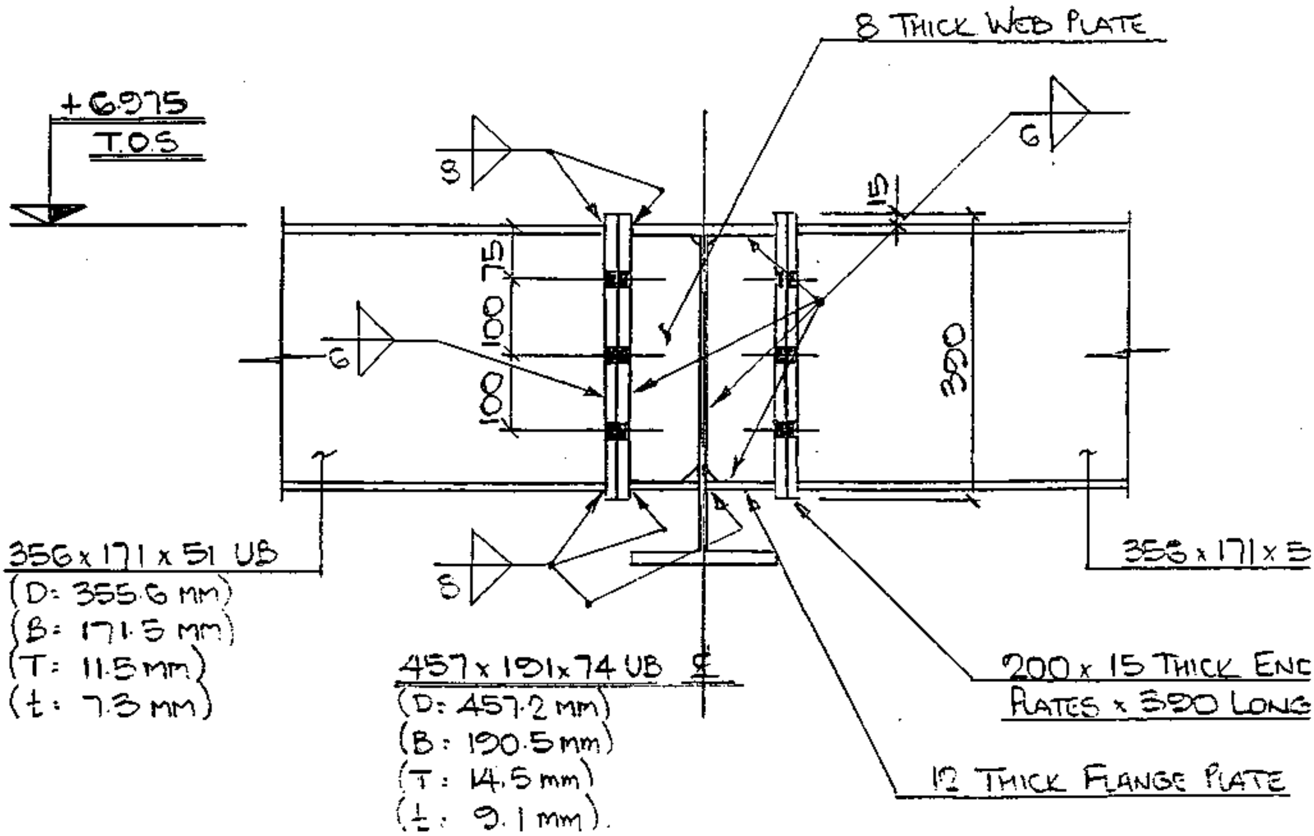
BEAM B5-LG+ TO BEAM B3-LG

356 x 171 x 51 UB TO 457 x 191 x 74 UB

LOADINGS :—

END SHEAR : 200 KN.

AXIAL LOAD : 2000 KN (COMPRESSION)



M20 BOLTS (GRADE 8.8)
@ 90 HORIZONTAL CENTRES

A REFERENCE CALCULATIONS : —

SHEET NOS C48 / SERIES SHEET NOS 1 OF 10 TO 4 OF 10 (INCLUSIVE)

DESIGN OF UB WEB STIFFENER

AXIAL LOAD : 2000 KN (COMP^N)

BEARING AREA REQ^D

: $2000 \times 10^3 \div 355 = 5634\ mm^2$.

ASSUME 25 THICK STIFFENER

MIN STIFFENER WIDTH REQ^D = $0.5 \times 5634 \div 25 = 113$ MM
→ SAY 125 mm

MIN STIFFENER LENGTH REQ^D

$$= \frac{2000 \times 1000}{2 \times 355 \times 0.6 \times 7.3} = 643\ mm \longrightarrow 650\ mm.$$

USE : — 125 x 25 THICK STIFFENERS x 650 LONG

DESIGN OF WELD FOR STIFFENER TO UB WEB

SHEAR LOAD PER STIFFENER = 1000 KN.

LEVER ARM = $125 - \{0.5 \times 115\} = 67.5$ mm

BENDING MOMENT = $1000 \times 67.5 = 67500$ KN mm

STIFFENER LENGTH = 650 MM

EFFECTIVE WELD LENGTH = $650 - (2 \times 10) = 630$ MM.

WELD AREA = $2 \times 630 = 1260\ MM^2$

$Z = 2 \times 630^2 \div 6 = 132300\ mm^3$

$f_s = 1000 \div 1260 = 0.80\ KN/mm^2$

$f_m = 67500 \div 132300 = 0.51\ KN/mm^2$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

SHT. No. C49A REV.

BY KB DATE 07/98

EX. DATE

$$f_{RES} = \sqrt{0.80^2 + 0.51^2} = 0.95 \ kN/mm^2$$

WELD STRENGTH (GRADE 50) = 255 kN/mm^2

$$\text{WELD SIZE REQ}^{D} = \frac{0.95 \times 10^3}{0.70 \times 255} = 5.4\,mm \longrightarrow 6 \text{ LEG F.W. (MIN}^{M}\text{)}$$

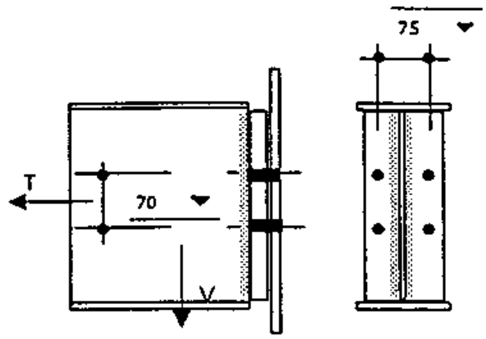
USE :— 6 LEG FILLET WELD

Kvaerner Cleveland Bridge Ltd

O/No. : M347	Job : Carlton Gardens	Series :	
Subject :	Connection design	Sht. No. : C50/1	Rev. :
		By : AJC	Date : May 98
		EX : JL	Date : May 98

Calculation Sheet

Connection Ref:- C50



Bolt Diameter	20 mm
Bolt Grade	8.8
Plate grade	50
Bolt Centres C	75 mm
Bolt Pitch P	70 mm
Shear Force V	260 kN
Tie Force T	75 kN
Plate Thickness	10 mm
Web Thickness	8.6 mm
Bolt hole size	22 mm
Section Grade	43
Plate Depth	250 mm
Plate Width	170 mm

Section Reference
254 x 254 x 73 x UC
Grade 43

Bolt Loads

Shear Force per bolt	65 kN
Tension per bolt	18.75 kN

Shear Capacity of Bolt	91.9 kN
Tension Capacity of Bolt	110 kN

Shear Force / Capacity Ratio	0.71	**<1.0, OK**
Tension Force / Capacity Ratio	0.17	**<1.0, OK**
Combined Ratio	0.88	**<1.4, OK**

Bearing capacity in plate	110 kN	
Bearing capacity Ratio	0.84	**<1.0, OK**

Bolt Capacity adequate

Check weld — Assume Grade 43 weld strength

Weld carries shear and tension only

Weld Size	6 mm
Weld Capacity	0.903 kN/mm
Total Weld Length	476 mm
Tension load per mm	0.158 kN/mm
Shear load per mm	0.546 kN/mm
Resultant Load / mm	0.568 kN/mm

Allowable weld stress
215 N/mm^2

Weld Strength Adequate

Shear Capacity of Beam Web

Shear capacity of web = $0.6 \times py \times Av_{web}$ 319.3 kN

Shear capacity of beam web adequate

Check capacity of end plate

Shear capacity of end plate = $0.6 \times py \times Av_{plate}$ 479.3 kN

Shear capacity through bolt line = $0.5 \times Us \times Av_{net}$ 454.2 kN

Plastic modulus of plate	6.3 cm^3
Allowable stress	355 N/mm^2
Moment capacity	2.22 kNm
Moment at face of web	1.25 kNm

Applied Shear 260 kN

Shear capacity of plate adequate

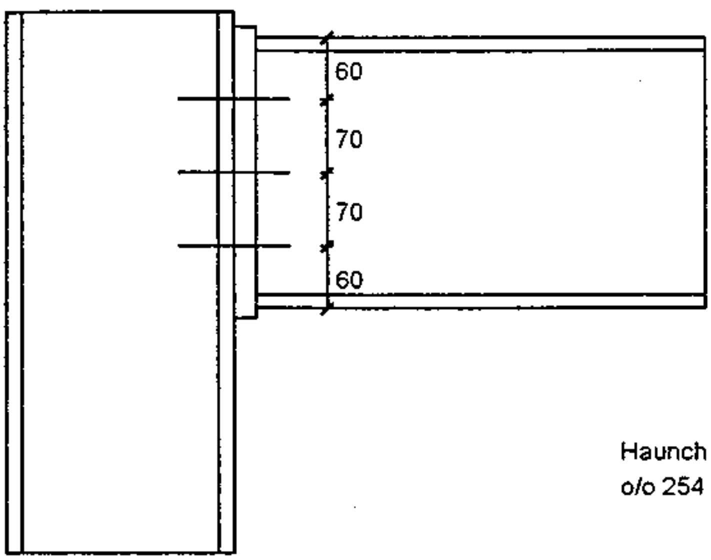
Plate Adequate In Bending

Design checks on capacity of section by original designers

James Lupton Consultants - Inhouse Design Software
Connection Reference C51

Rafter Section 254 * 146 * 43 UB

Stanchion Section 203 * 203 * 60 UC



Haunch size : 0mm x 0mm
o/o 254 * 146 * 43 UB

Scale 1/7.5

Stiffeners :

End Plate :
200 x 20 thk (minimum)
6 No M24 Gr 8.8 Bolts @ 90 mm Crs
Welds :- 10mm Fw Flgs -- 6mm Fw Web

Calculation Sheet

James Lupton Consultants - Inhouse Design Software
Ref: **C51**
Moment Connection Design with reference to Constrado Monograph- Plastic Design of Low Rise Frames by Horne & Morris - Pages 206 to 219 & BS5950
:Release 98/1

Loading Information

Moment kNm	70.00	Shear kN	177.00	Tens. kN	75
Moment kNm	-35.00	Shear kN	177.00	Tens. kN	75

Section Information

Rafter Section	254 * 146 * 43 UB	Steel grade	50
Stanchion Section	203 * 203 * 60 UC	Rafter Pitch	0.00
Haunch Section			

No Haunch to Connection

Bolt Data
6 No M24 Grade 8.8 Bolts at 90 crs.

Modulus blt grp +ve Mom 54.54
Modulus blt grp -ve Mom 54.54

		Positive Bending		Negative Bending	
Bolt Row	**Pitch**	**Lever ar**	**Tension**	**Lever ar**	**Tension**
q	60	187.30	140.84	47.30	28.71
r	70	117.30	92.88	117.30	52.69
s	70	47.30	44.91	187.30	76.67
t	0	0.00	0.00	0.00	0.00
u	0	0.00	0.00	0.00	0.00
v	0	0.00	0.00	0.00	0.00
w	0	0.00	0.00	0.00	0.00
x	0	0.00	0.00	0.00	0.00

Positive Bending :	**Total Tension**	557.26	kN
	Flange comp.	482.26	kN
Negative Bending :	**Total Tension**	316.13	kN
	Flange comp.	241.13	kN

Worst Bolt values (+ve Bending)

	Maximum value	**Allowable value**	
Tension	140.84 kN	159.00 kN	- **Tension adequate**
Shear	29.50 kN	132.00 kN	- **Shear adequate**
Bearing	29.50 kN	156.77 kN	- **Bearing adequate**
Interaction ft/pt + fs/ps =		1.11	- **Interaction adequate**

Worst Bolt values (-ve Bending)

	Maximum value	**Allowable value**	
Tension	76.67 kN	159.00 kN	- **Tension adequate**
Shear	29.50 kN	132.00 kN	- **Shear adequate**
Bearing	29.50 kN	156.77 kN	- **Bearing adequate**
Interaction ft/pt + fs/ps =		0.71	- **Interaction adequate**

Calculation Sheet

Ref: C51

Moment Connection Design with reference to Constrado Monograph- Plastic Design of Low Rise Frames by Horne & Morris - Pages 206 to 219 & BS5950

Welds - End Plate to Beam

Refer to Clause 6.6.5.1. of BS5950

Min weld reqd.	8.18	Use	10	fw - top flange
Min weld reqd.	5.16	Use	6	fw - web
Min weld reqd.	8.98	Use	10	fw - bottom flange

End Plate Bending (+ve Bending)

Web T	7.3	v'=	47.3	m''=	35.35
Row q\ dq=	95.00	Te min	17.558		
Row r\ dr=	70.00	Te min	16.610		
Row s\ ds=	95.00	Te min	12.955		
Row t\ dt=	0.00	Te min	0.000		

Adopt min 20 mm thk. * 200 End Plate.

Column Flange Bending

Flange T	14.2	m =	30.15	n'=	57.600
Col Web T	9.3	n =	55	w =	47.473

Flange Capacities

	Unstiff.	v	Stiffened	Force	
row q to r	557.70	26.00	926.56	467.44	No Stiffener Required
row r to s	602.74	26.00	924.17	275.58	No Stiffener Required
row s to t	0.00	0.00	0.00	0.00	
row t to u	0.00	0.00	0.00	0.00	
row u to v	0.00	0.00	0.00	0.00	
row v to w	0.00	0.00	0.00	0.00	
row w to x	0.00	0.00	0.00	0.00	

Web Compression at U/S Haunch (+ve Bending)

Applied Compression	=	482.26	kN	
Slendereness	=	41.95		
Compression stress	=	296.10	N/mm^2	
Web buckling capacity	=	667.22	kN	
Web bearing capacity	=	510.74	kN	
Maximum outstand	=	95.00	mm	)
As for bearing	=	1086.79	mm^2	)
Min ts bearing	=	0.00	mm	) **Calculations**
Try stiffeners		6 thick x	110	)
Iyy Effective section	=	6052657	mm^4	) **not**
Nett area	=	4779.6	mm^2	)
Nett ry	=	35.59	mm	) **Applicable**
Net slenderness	=	3.56	mm	)
pc buckling	=	355	N/mm^2	)
Buckling capacity	=	1696.76	kN	)
				)

No Compression Stiffener required

Calculation Sheet

Ref: C51

Moment Connection Design with reference to Constrado Monograph- Plastic Design of Low Rise Frames by Horne & Morris - Pages 206 to 219 & BS5950

Web Compression at top of column (-ve Bending)

Applied Compression	=	241.13	kN		
Slendereness	=	41.95			
Compression stress	=	296.10	N/mm^2		
Web buckling capacity	=	447.47	kN		
Web bearing capacity	=	391.89	kN		
Maximum outstand	=	95	mm	)	
As for bearing	=	543.40	mm^2	)	
Min ts bearing	=	0.00	mm	)	Calculations
Try stiffeners		6 thick x 65		)	
Iyy Effective section	=	1365264	mm^4	)	not
Nett area	=	2742.3	mm^2	)	
Nett ry	=	22.31	mm	)	Applicable
Net slenderness	=	5.68	mm	)	
pc buckling	=	355.00	N/mm^2	)	
Buckling capacity	=	973.52	kN	)	

No Compression Stiffener required

Stanchion Web Shear capacity

Applied Shear force	=	0.00	kN		
Web capacity	=	415.20	kN		
Theta	=	61.94	deg.	)	Calculations
Area required	=	0.00	mm^2	)	not
Maximum outstand	=	95	mm	)	Applicable
Minimum thickness	=	0.00	mm	)	

No Shear Stiffener is Required

Column Flange Stiffeners

Max force	116.86	kN	)	
Stiff' Force	93.49	kN	)	
Max eff wid	65.7	mm	)	Calculations
Min thickness	5.81	mm	)	
Try thk =	6	mm	)	not
Width =	80	mm	)	
Min Flange weld	3.99 Adopt weld	6 mm	)	Applicable
Shear to web	93.49	kN	)	
Moment =	3772.25	kNmm	)	
Weld F/mm =	0.91	kN/mm	)	

Column Flange Stiffeners are not Required

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS
SUBJECT CONNECTION DESIGN
BEAM B1-G & BEAM B8-G TO COLUMN C11
COMBINED CONNECTION C38A / C52

SERIES SHEET 1 OF 3
SHT. No. C52A REV.
BY .... DATE ....
EX. .... DATE ....

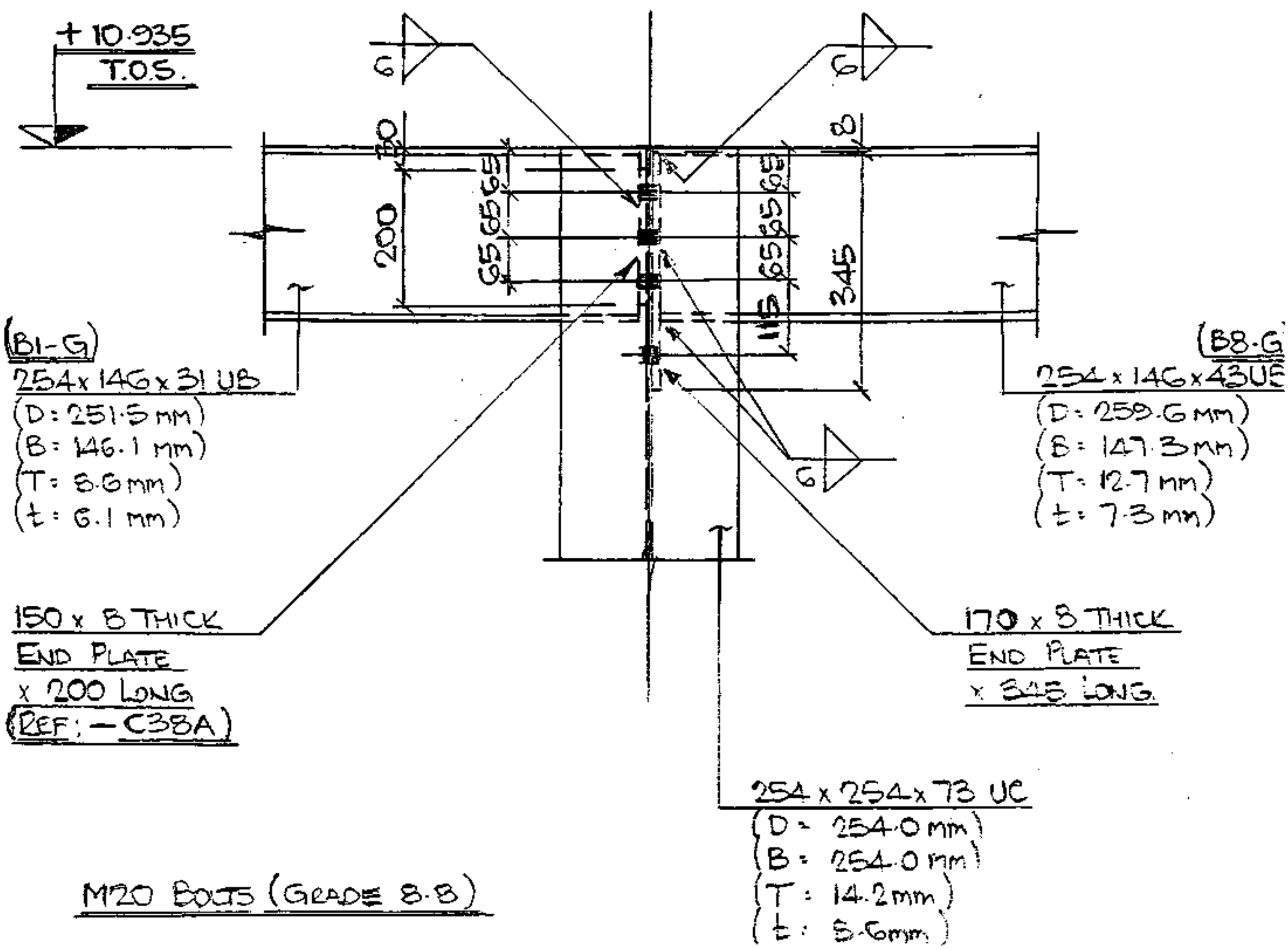
BEAM B1-G & BEAM B8-G TO COLUMN C11

254 x 146 x 31 UB & 254 x 146 x 43 UB TO 254 x 254 x 73 UC

END REACTIONS :—

BEAM B1-G : 177 KN

BEAM B8-G : 400 KN



M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

SHT. No. C52A REV.

BY KB DATE 06/98

EX. DATE

COMBINED CONNECTION C38A / C52

CHECK BOLTS

TRY :- M20 BOLTS (GRADE 8.8)

BOLT SHEAR, P_s = $375 \times 245 \div 10^3$ = 91.8 KN

BOLT BEARING ; P_{bb} = $1035 \times 20 \times 8 \div 10^3$ = 165.6 KN

PLY BEARING ; P_{bs}

a) $d \times t \times p_{bs}$ = $20 \times 8 \times 550 \div 10^3$ = 88.0 KN

b) $\frac{1}{2} \times e \times t \times p_{bs}$ = $\frac{1}{2} \times 35 \times 8 \times 550 \div 10^3$ = 77.0 KN.

FOR 8 THICK END PLATE & 35 MINIMUM EDGE DISTANCE

BOLT CAPACITY = 77.0 KN.

BEAM B1-G

END REACTION = 177 KN

No OF BOLTS = 6

SHEAR / BOLT = $177 \div 6$

= 29.5 KN < 77.0 KN OK

BEAM B8-G

END REACTION = 400 KN

No OF BOLTS = 8

SHEAR / BOLT = 50 KN < 77.0 KN. OK

CHECK COLUMN WEB BEARING

MAXIMUM BOLT LOAD = 29.5 + 50.0 = 79.5 KN

COMBINED CONNECTION C38A / C52

COLUMN WEB BEARING CAPACITY

$= 550 \times 20 \times 8.6 \div 10^3$

$= 94.6$ KN $>$ 79.5 KN OK

BOLTS SATISFACTORY

CHECK :— BEAM 58-6 (254 x 146 x 43 UB)

a) END PLATE ~ TRY :— 8 THICK x 345 LONG

SHEAR LOAD PER BOLT LINE $= 0.5 \times 400 = 200$ KN

$A_V = 0.9 \times 345 \times 8 = 2484 \text{ mm}^2$

$A_{VNET} = 2484 - (4 \times 22 \times 8) = 1780 \text{ mm}^2$

$0.6 \times p_y \times A_V = 0.6 \times 355 \times 2484 \div 10^3 = 529$ KN

$0.5 \times U_s \times A_{VNET} = 0.5 \times 490 \times 1780 \div 10^3 = 436$ KN

$P_V = 436$ KN $>$ 200 KN OK

END PLATE SATISFACTORY

CHECK WELD

BY INSPECTION :—

WITH SHEET No :— C52/1

USE :— 6 LEG FILLET WELD (FULL PROFILE)

Kvaerner Cleveland Bridge Ltd

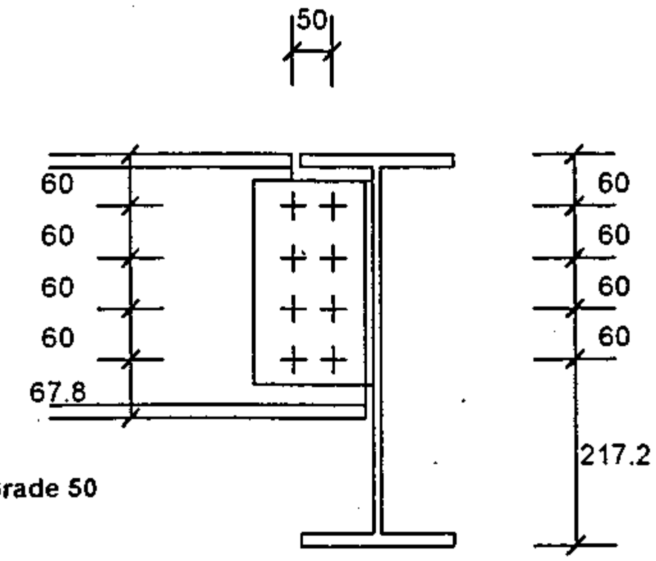
O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C53/1 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Details of Connection Ref:- Connection C53

Connected Beam 305 x 305 x 97 x UC Grade 50



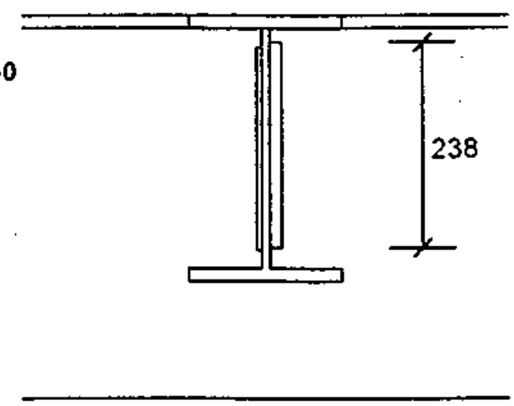
Supporting Beam 457 x 191 x 74 x UB Grade 50

- Compensation plate req'd 6 mm thk

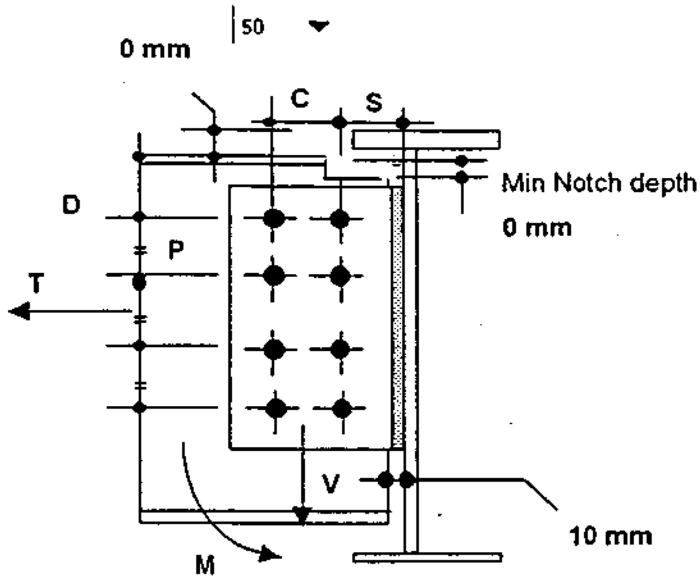
All Bolts M20 Grade 8.8

Weld to supporting Member to be 6 mm Fillet weld
All Bolts M20 Grade 8.8

Fin Plates - 238 Lng x 150 x 15 Plt Grade 50



Calculation Sheet



Connection Ref:-	**Connection C53**		
Supporting Beam Loaded on Both sides		**0 mm**	
Bolt Diameter	20 mm		
Bolt Grade	8.8		
Fin Plate grade	50		
Top beam to bolt D	**60 OK**		
Bolt Centres C	50 mm		
Bolt Pitch P	60 mm		
Column offset S	50 mm		
Shear Force V	340 kN		
Tie Force T	75 kN		
Bending Moment M	25.5 kNm		
% Carried by bolt Group	**100%**		
Web Thickness	9.9 mm		
Compensation plate Thk	6 mm		
Fin Plate Length	**238 mm OK L>0.6D**		
Fin Plate Width	**150 mm**	Bolt hole size	22 mm

' - Compensation plate req'd 6 mm thk

Fin Plate thickness	15 mm '			
Connected Beam	305 x 305 x 97 x UC	Grade	50 py	355.0 N/mm2
Supporting Beam	457 x 191 x 74 x UB	Grade	50 py	355.0 N/mm2
Top Notch Length	102 mm			
Top Notch Depth	26 mm			
Bottom Notch Length	0 mm	OK		
Bottom Notch Depth	0 mm			

Bolt Eccentricities from centroid

ex	25 mm	ey1	30 mm
		ey2	90 mm
I of Bolt Group	42250 mm^3		
Fvv	42.5 kN	Fth	9.375 kN
Fmv	15.09 kN	Fmh	54.32 kN
Resultant force in bolt			85.87 kN
Shear Capacity of Bolt			91.9 kN
Bearing capacity in section with comp. plt.			148.665 kN Edge Distance Governs
Bearing capacity in Fin Plate			119.625 kN Edge distance Governs
Limiting Value		91.9 kN	**Bolt Capacity adequate**

Compensation plate to be grade 50

Kvaerner Cleveland Bridge Ltd

O/No. : M347 Job : Carlton Gardens
Subject : Connection design

Series :
Sht. No. : C53/3 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Check Fin Plate Grade of Fin Plate = 50

Shear Area of Fin Plate	3213 mm^2		
Nett Area of Fin Plate	1893 mm^2		
Shear Capacity = 0.6xpy*Av =	684.4 kN	Applied Shear =340 kN	
but not exceeding 0.5xUs*Avnett =	463.8 kN	**Limiting Value =463.8 OK**	

Check Web Shear

Shear Area of Web	3880 mm^2		
Nett Area of Web	2481 mm^2		
Shear Capacity = 0.6xpy*Av =	826.5 kN	Applied Shear =340 kN	
but not exceeding 0.5xUs*Avnett =	607.9 kN	**Limiting Value =607.9 OK**	

Check Block Failure

Av1	3274.2 mm^2	Applied Shear =340 kN
Ateff	535.5 mm^2	**Limiting Value =828.6 OK**

Check Max Notch Length **(Compensation plate ignored)**

T Section Remaining

Flange Area	4694		Stem Area		2637.4
Flange y	8		Stem y		149 mm
y bar	58.4	Flange h	50.7	Stem h	90.2
Flange I	92768		Stem I		15597558
I Section	49213665		Z min Section		220282
Mcx of notched Section	78.20 kNm		D/tw		31.09
Maximum Notch Length	230 mm		**Notch Length OK**		

Check weld

Weld Size	6 mm	*Weld carries shear and tension only*	
Weld Capacity	0.903 kN/mm		
Total Weld Length	452 mm	Z of Weld	17025.3333 mm^2
Tension load per mm	0.166 kN/mm	BM load	0.000 kN/mm
Shear load per mm	0.752 kN/mm		
Resultant Load / mm	0.770 kN/mm	**Weld Strength Adequate**	

Check Tie capacity of Fin Plate

Tie Capacity of Fin Plate Tension = Le*tw*py =	798.75 kN	Bolt Capacity in shear
Tie Capacity of Web in Tension = $L_e{}^*t_w{}^*p_y$ =	745.07 kN	735.2 kN
Tie Capacity of Web in Bearing = $L_e{}^*t_w{}^*p_y$ =	1089.00 kN	

Applied Tie Force 75.0 kN **Tie Force < Limiting Value - OK**

Check Moment Capacity of fin plate

		Applied Shear	340.0 kN
I Nett of Plate	1085.84 cm^4	Shear Capacity	463.8 kN
Z nett of plate	91.25 cm^3	Snett of plate	133.2 cm^3
Moment at 1st bolt line	17.00 kNm	Moment capacity	31.56 kNm

Moment Capacity reduced due to High Shear Condition Plate OK

For Connection onto a Column instead of a beam

Note:- Web bending on supporting beam/column due to tie force has not been checked.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B13-5 & BEAM B5-5 TO COLUMN C2

SERIES SHEET 1 OF 3

SHT. No. C53A REV.

BY KB DATE 09/98

EX. DATE

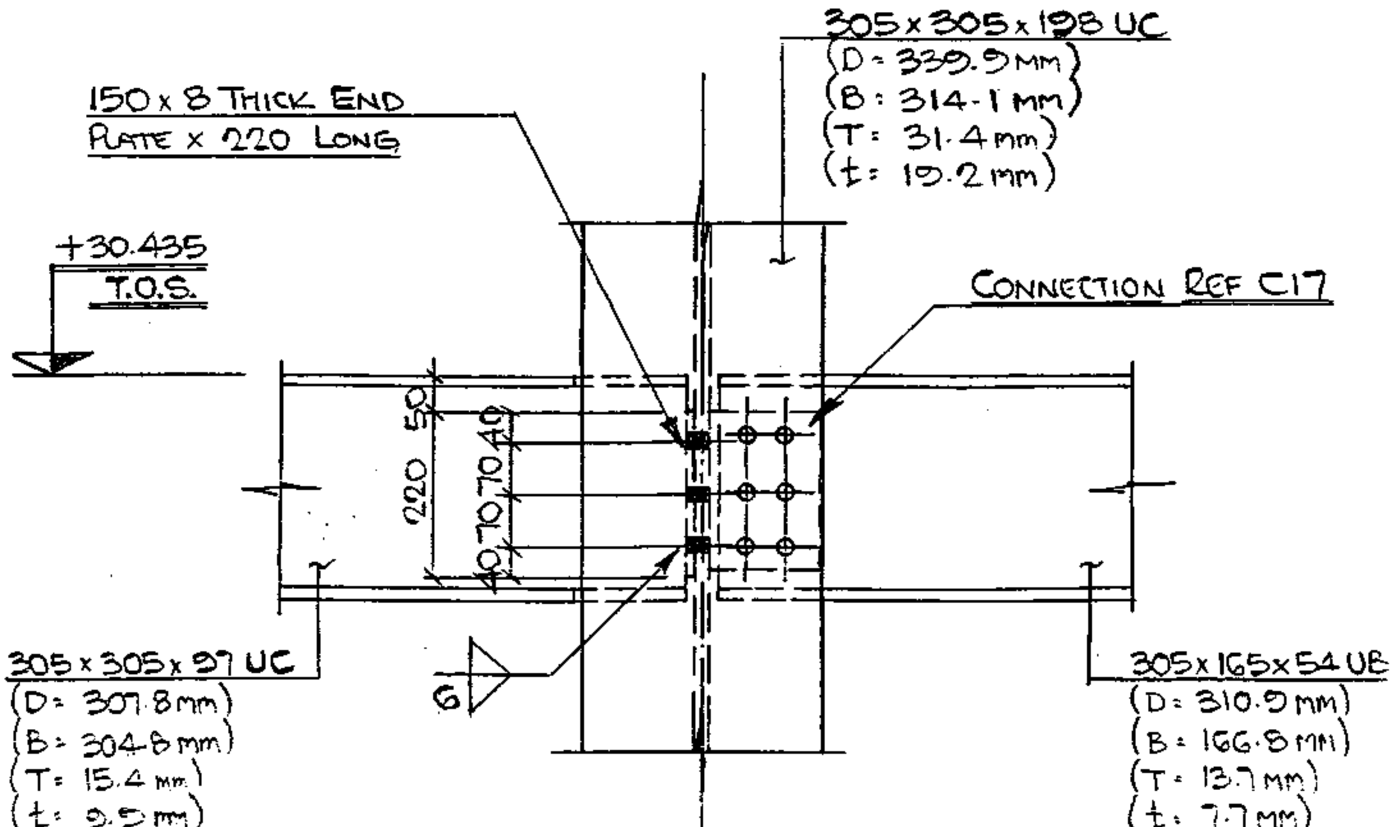
BEAM B13-5 & BEAM B5-5 TO COLUMN C2

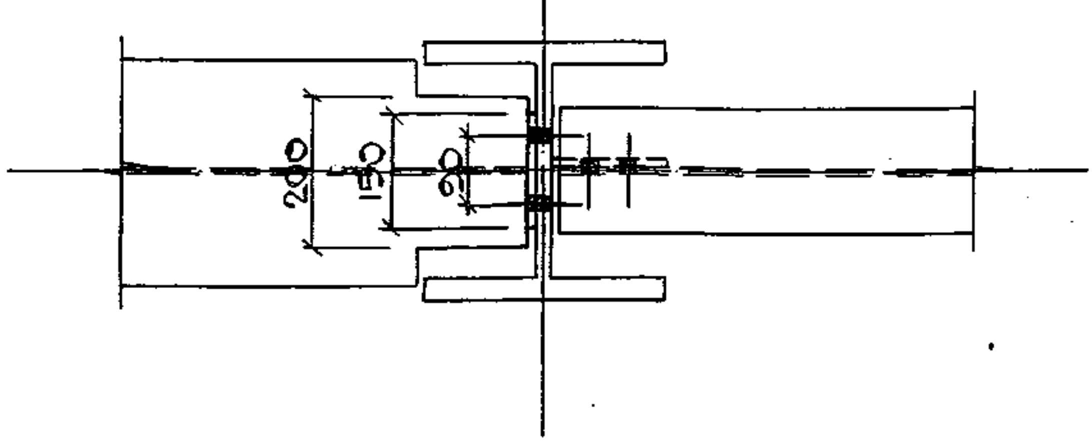
305 x 305 x 97 UC & 305 x 165 x 54 UB TO 305 x 305 x 198 UC

END REACTIONS : —

BEAM B13-5 = 340 KN

BEAM B5-5 = 234 KN





DESIGN OF BOLTS

TRY: — 6 No ~ M20 BOLTS (GRADE 8.8)

END REACTION : 340 KN

LOAD PER BOLT = 340 ÷ 6 = 56.7 KN

BOLT SHEAR ; P_s = $245 \times 375 \div 10^3$ = 91.8 KN

BOLT BEARING ; P_{bb} = $1035 \times 20 \times 8 \div 10^3$ = 165.6 KN

PLY BEARING ; P_{bs}

(a) $d \times t \times p_{bs}$ = $20 \times 8 \times 550 \div 10^3$ = 88.0 KN

(b) $\frac{1}{2} \times e \times t \times p_{bs}$: $\frac{1}{2} \times 40 \times 8 \times 550 \div 10^3$ = 88.0 KN

P_{bs} = 88.0 KN

BOLT CAPACITY : 88.0 KN > 56.7 KN OK

USE : — 6 No ~ M20 BOLTS (GRADE 8.8)

DESIGN OF END PLATE

TRY : — 8 THICK PLATE

END REACTION : 340 KN

SHEAR LOAD PER LINE OF BOLTS = 0.5 × 340 = 170 KN

A_v = $0.9 \times 8 \times 220$ = 1584 mm^2

A_{vnet} = $1584 - (3 \times 8 \times 22)$ = 1056 mm^2

$0.6 \times P_y \times A_v$ = $0.6 \times 355 \times 1584 \div 10^3$ = 337 KN

$0.5 \times U_s \times A_{vnet}$: $0.5 \times 490 \times 1056 \div 10^3$ = 258 KN

P_v = 258 KN > 170 KN OK

$A_{VI} = 180 \times 8 = 1440\ mm^2$

$A_{TEFF} = (30 - \{0.5 \times 22\}) \times 8 = 152\ mm^2$

$P_{VB} = (0.6 \times P_y \times A_{VI}) + (0.5 \times U_S \times A_{TEFF})$

$= (0.6 \times 355 \times 1440 \div 10^3) + (0.5 \times 450 \times 152 \div 10^3)$

$= \underline{343\ KN > 170\ KN}$ OK

<u>USE :— 150 x 8 THICK END PLATE x 220 LONG</u>

<u>DESIGN OF WELD FOR END PLATE TO BEAM WEB</u>

LENGTH OF END PLATE = 220 mm

EFFECTIVE WELD LENGTH = $220 - (2 \times 10) = 200\ mm$

LOAD PER mm RUN OF WELD : $340 \div (2 \times 200) = 0.85\ KN/mm$

WELD STRENGTH (GRADE 50) : $255\ N/mm^2$

WELD SIZE REQ^D : $\dfrac{0.85 \times 10^3}{0.7 \times 255} = \underline{4.8\ mm} \longrightarrow$ <u>6 LEG F.W.</u> (MIN^M)

<u>USE :— 6 LEG FILLET WELD</u>

<u>CHECK BEAM WEB AT END PLATE</u>

SHEAR LOAD = 340 KN

EFFECTIVE WEB DEPTH = 220 mm

$A_V = 0.9 \times 220 \times 9.9 = 1960\ mm^2$

$P_V = 0.6 \times 355 \times 1960 \div 10^3$

$= \underline{417\ KN > 340\ KN}$ OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B9-G TO BEAM B3-G

SERIES SHEET 1 OF 1

SHT. No. C54 REV.

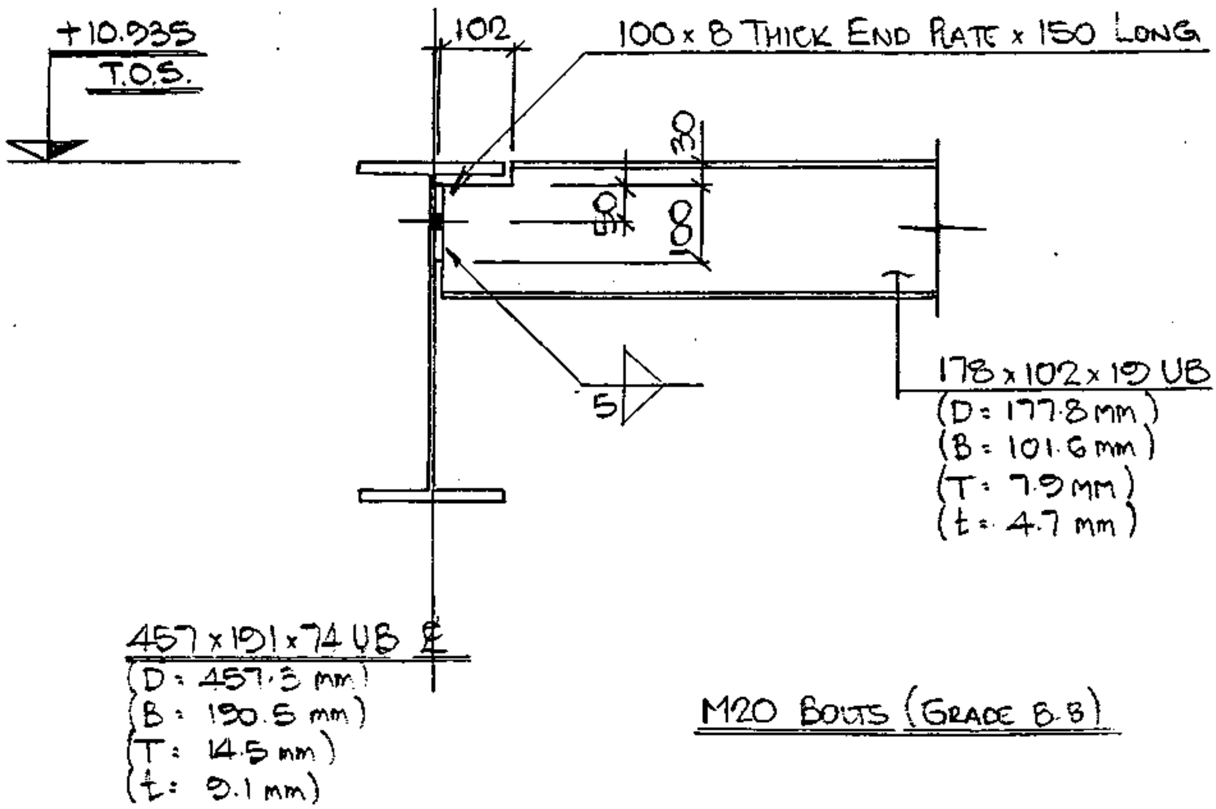
BY KB DATE 07/98

EX. DATE

BEAM B9-G TO BEAM B3-G

178 x 102 x 19 UB TO 457 x 191 x 74 UB

END REACTION = 50 KN



M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM BG-3 TO COLUMN C3

SERIES SHEET 1 OF 1

SHT. No. C55 REV.

BY KB DATE 07/98

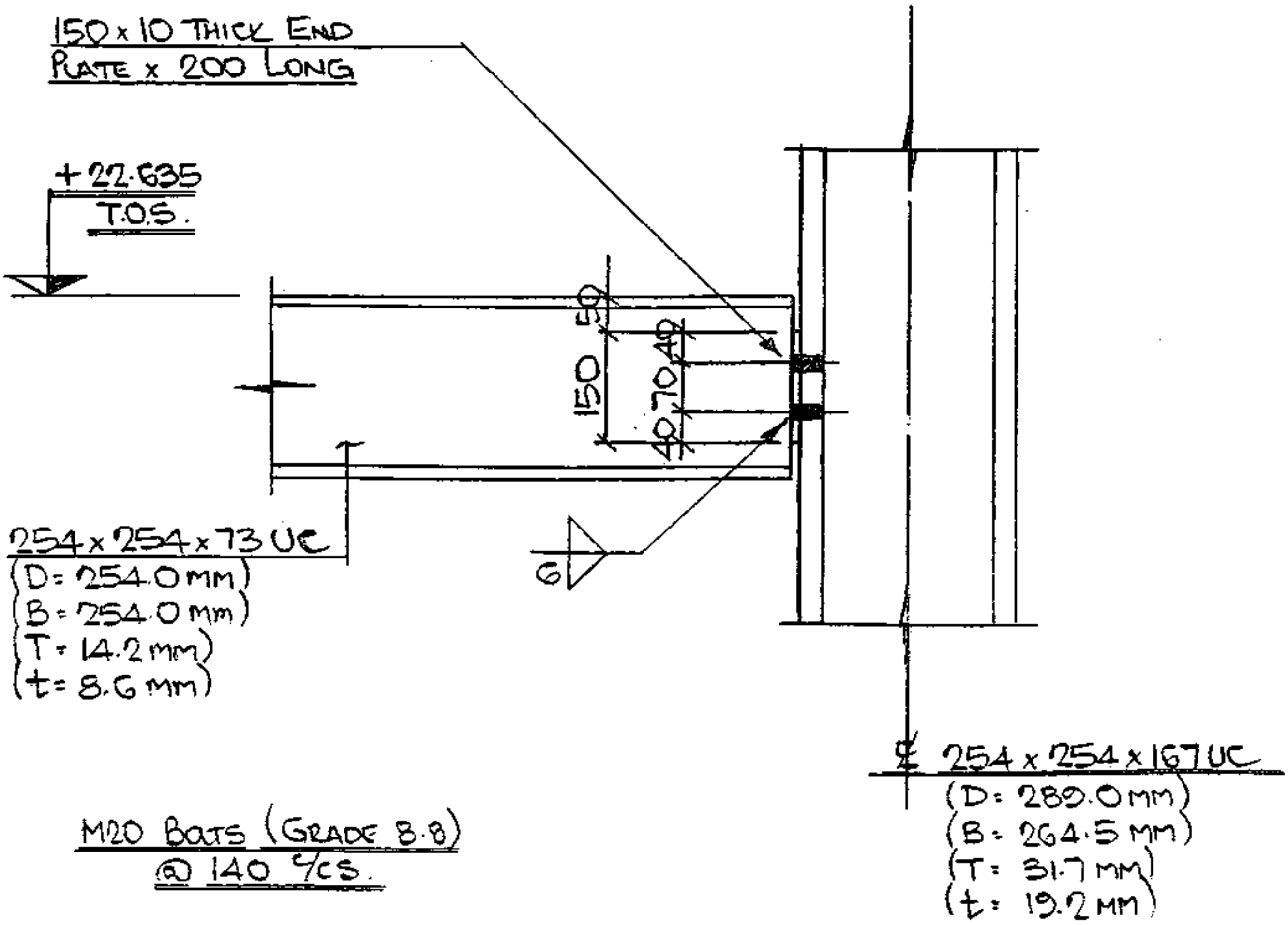
EX. DATE

(CN~22)

BEAM BG-3 TO COLUMN C3

254 x 254 x 73 UC TO 254 x 254 x 167 UC

END REACTION = 150 KN.



M20 BOLTS (GRADE 8.8)
@ 140 C/CS.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B4-3 TO BEAM B11-3

(CN ~ 22)

SERIES SHEET 1 OF 1

SHT. No. CS6 REV.

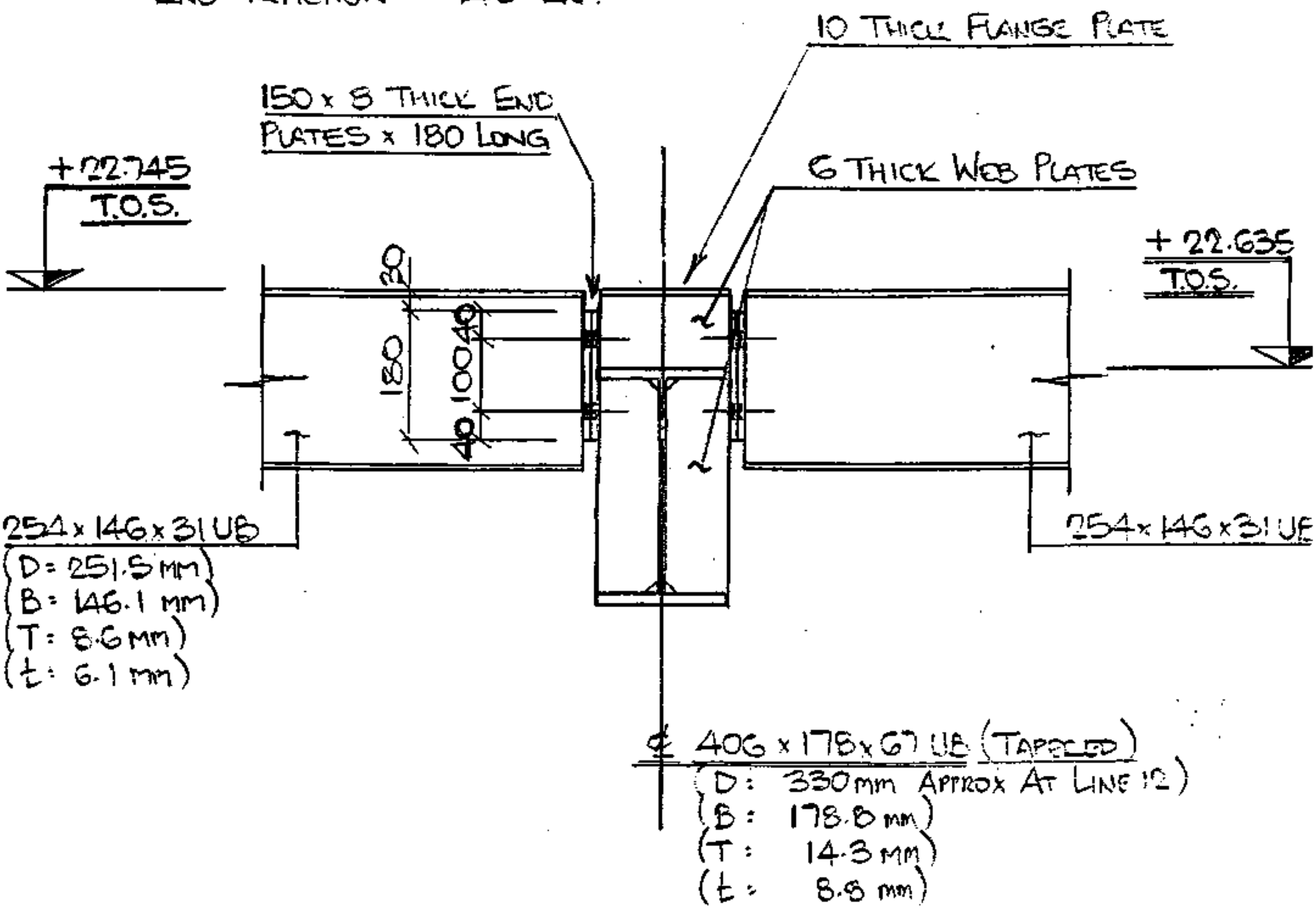
BY KB DATE 03/93

EX. DATE

BEAM B4-3 TO BEAM B11-3

254 x 146 x 31 UB TO 406 x 178 x 67 UB (TAPERED)

END REACTION = 140 KN.



M20 BOLTS (GRADE 8.8)

6 LEG FILLET WELD (U.N.O.)

REV A | 24/09/98 | BEAM B1-4 ~ T.O.S. LEVEL AMENDED 26.535 → 26.630 | CN-44 | KB

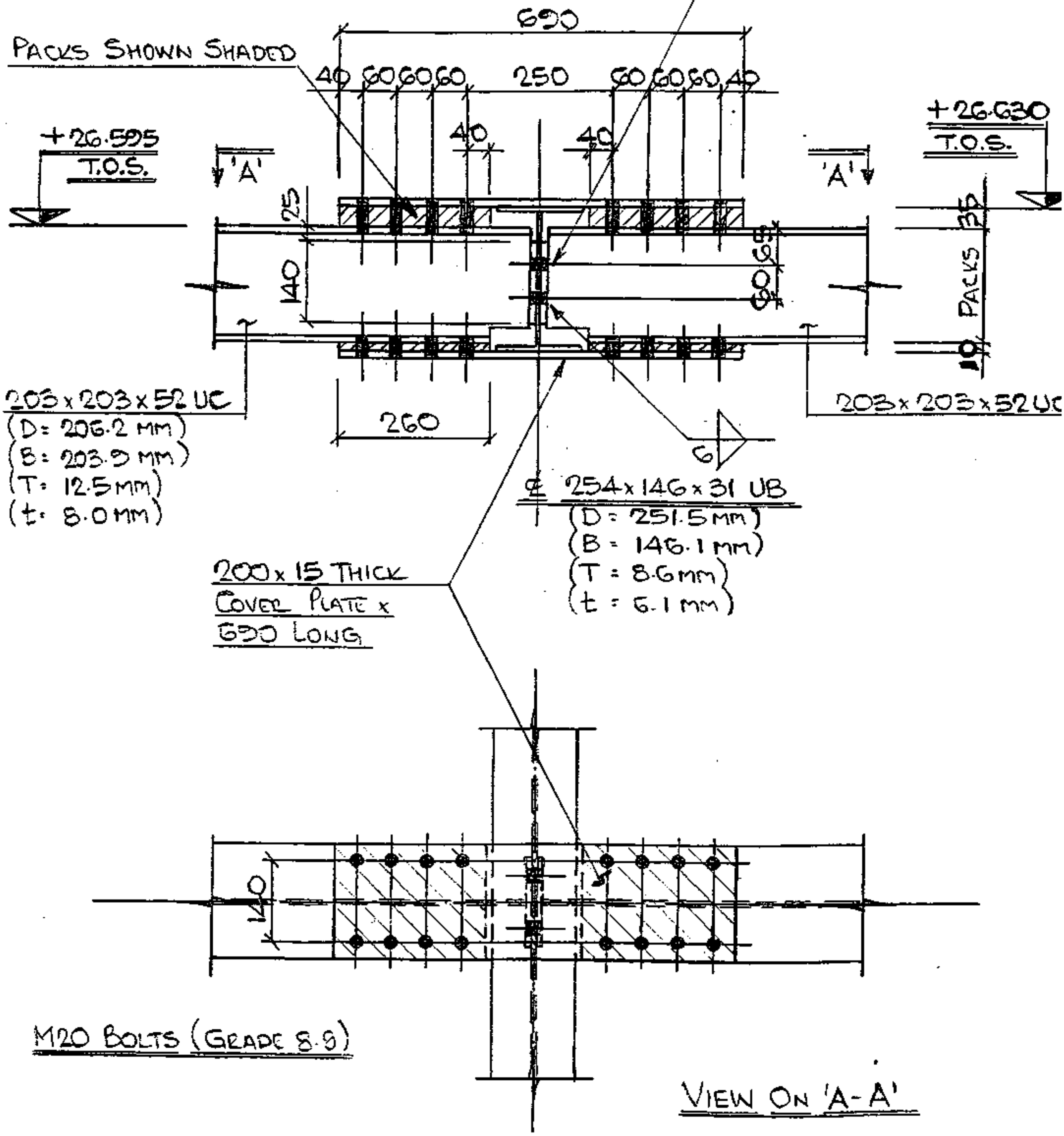
BEAMS B6-4 TO BEAM B1-4

203 x 203 x 52 UC TO 254 x 146 x 31 UB

END REACTIONS : –

SHEAR : 100 KN

MOMENT : 150 KNM



M20 BOLTS (GRADE 8.8)

VIEW ON 'A-A'

REV A | 24/09/98 | BEAM B1-4 ~ T.O.S. LEVEL AMENDED 26.535 ⟶ 26.630 | CN-44 | KB

DESIGN OF FLANGE SPLICE CONNECTIONS

(a) BOLTS

MOMENT : 150 KNm

LEVER ARM = 206.2 mm

FLANGE FORCE : 150 ÷ 0.2062 = 728 KN

SHEAR VALUE M20 BOLT (GRADE 8.8) : 91.8 KN.

No OF BOLTS REQ^D : 728 ÷ 91.8 : 7.9 ⟶ 8 No BOLTS

(b) COVER PLATES.

FLANGE FORCE = 728 KN

P_y = 355 N/mm^2

NET TENSION AREA REQ^D : $728 \times 10^3 \div 355$ = 2050 mm^2

ADD HOLES : $2 \times 15 \times 22$ = 660 MM^2

2710 mm^2

PLATE WIDTH : 200 mm

THICKNESS REQ^D : 2710 ÷ 200 : 13.6 mm ⟶ 15 THICK

USE : — 200 x 15 THICK COVER PLATES

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B6-4 TO BEAM B1-4

(CN ~ 24)

SERIES SHEET 1 OF 2

SHT. No. C57 REV.

BY KB DATE 07/98

EX. DATE

* SUPERSEDED *

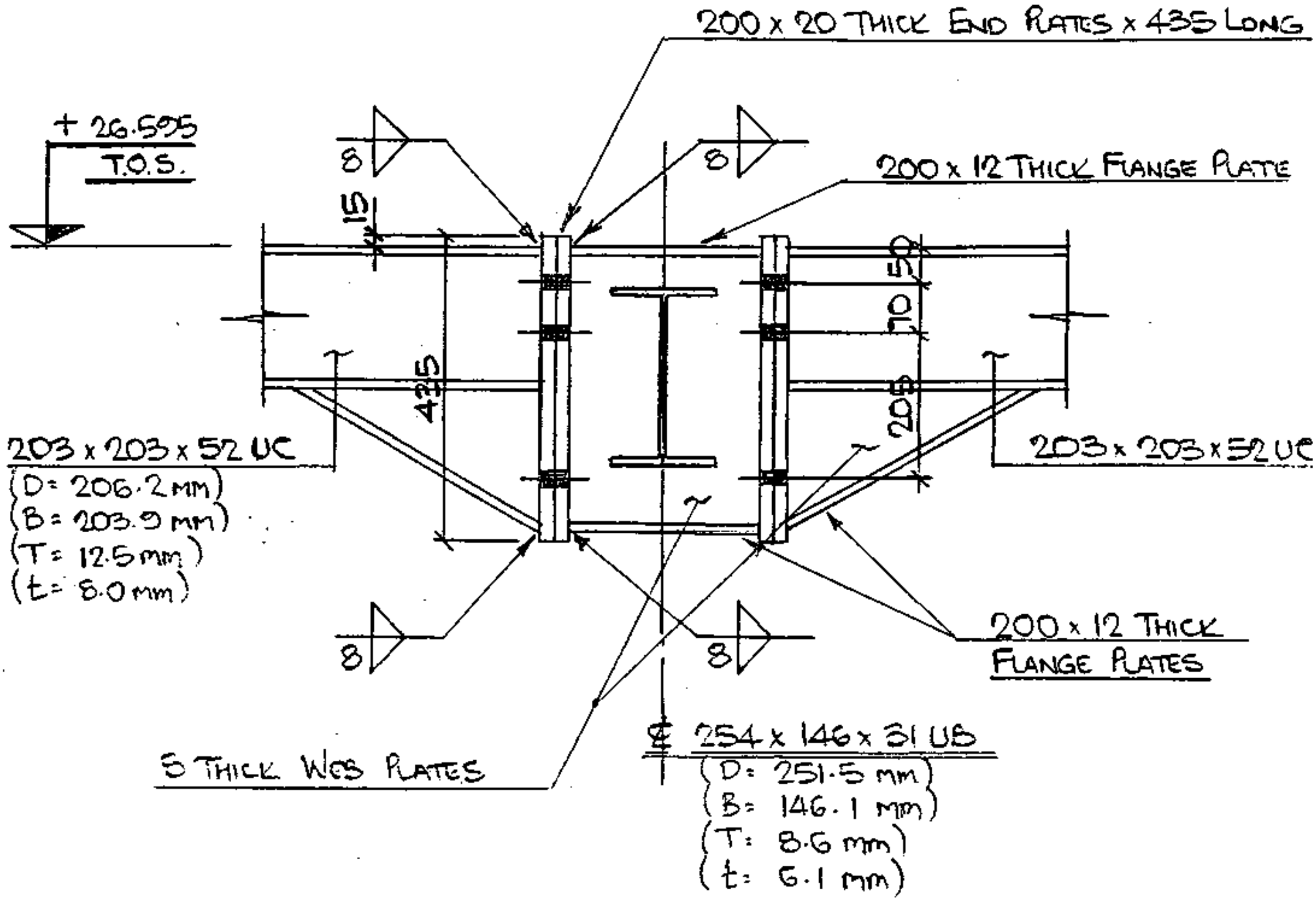
BEAM B6-4 TO BEAM B1-4

203 x 203 x 52 UC TO 254 x 146 x 31 UB

END REACTIONS :—

SHEAR : 100 KN

MOMENT : 150 KNM



6 LEG FILLET WELD (U.N.O.)

M20 BOLTS (GRADE 8.8)

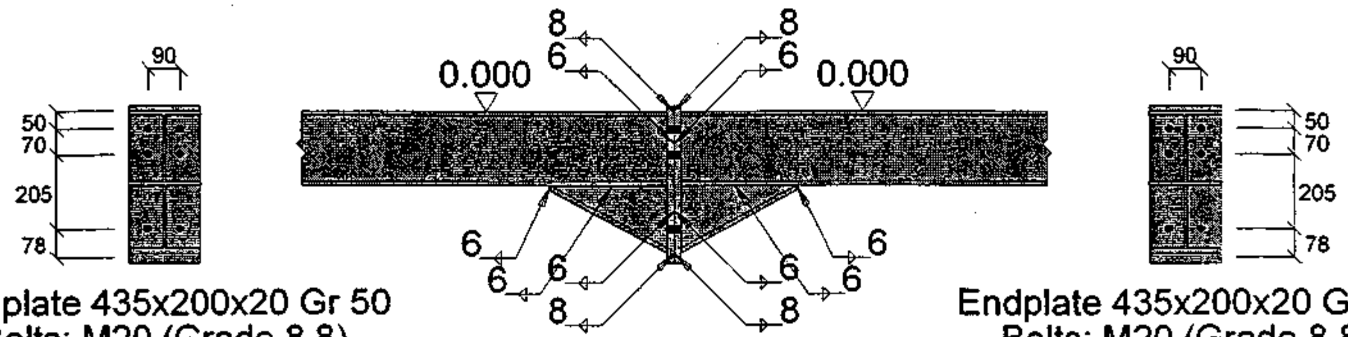
Job : 325
Project : Carlton Gardens
Structure : 4th Floor
Calcs by : K.B.
Page No. : 1

SHEET 2 OF 2
~~C57~~

31 Jul 1998

* SUPERSEDED *

203UC52 to 203UC52



Endplate 435x200x20 Gr 50
Bolts: M20 (Grade 8.8)

Endplate 435x200x20 Gr 50
Bolts: M20 (Grade 8.8)

UC 203x203x52 Gr 50 (Slope 0.0deg)
Haunch: Built up plate Gr 50
Length = 0.340m, Depth = 0.300m
B/F Flange 200x12mm thk, Web 8mm thk

UC 203x203x52 Gr 50 (Slope 0.0deg)
Haunch: Built up plate Gr 50
Length = 0.340m, Depth = 0.300m
B/F Flange 200x12mm thk, Web 8mm thk

Design Summary

Design Combination	Face	Status	Moment Capacity	Shear Capacity	Beam Web Capacity	Weld Checks	Stiffener Checks
Load Case 1	Left	Pass	0.96	0.30	0.00	Pass	n/a

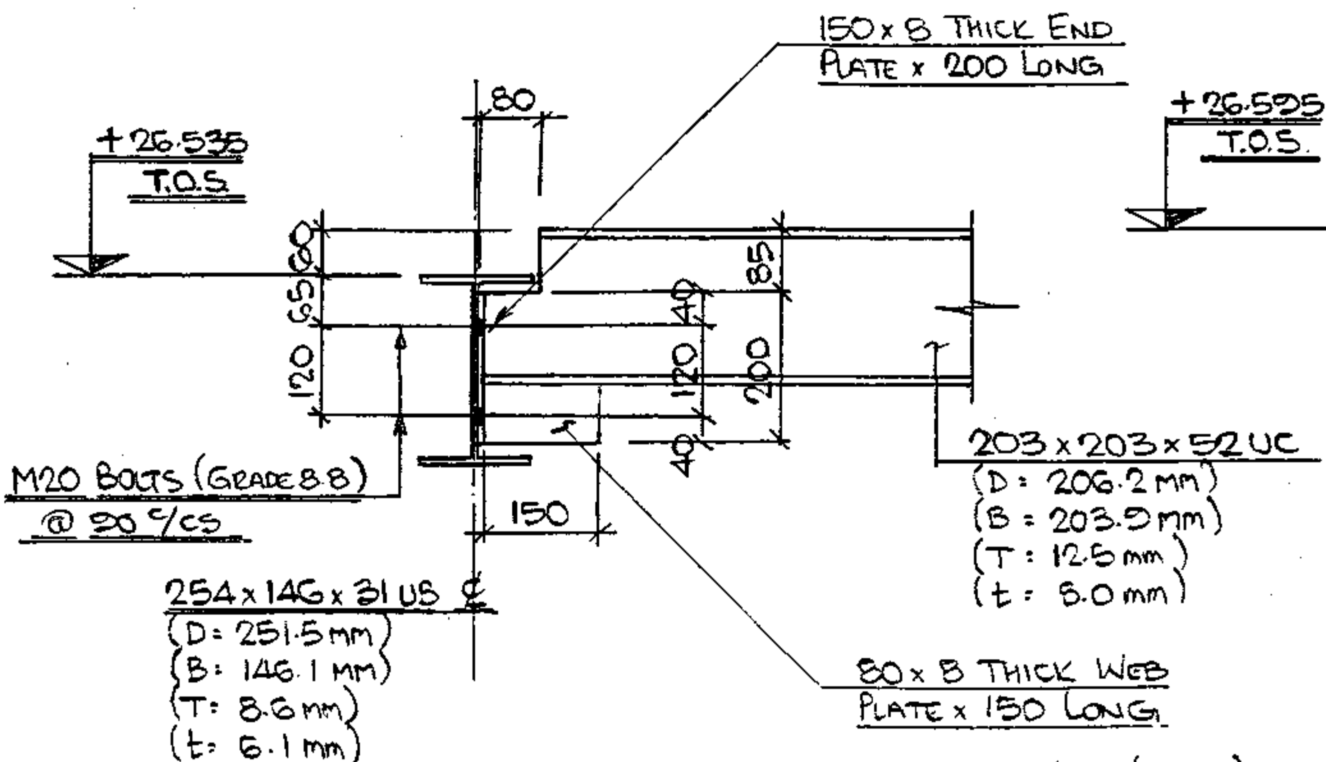
Design Forces Summary

Design Combination	Face	M (kNm)	V (kN)	N (kN)	Mse (kNm)	Factor
Load Case 1	Left	150.0	100.0	0.0	0.0	1.0

BEAM B6-4 TO BEAM B1-4

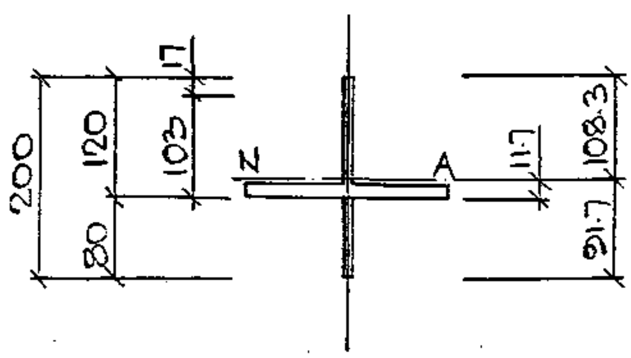
203 x 203 x 52 UC TO 254 x 146 x 31 UB

END REACTION = 150 KN



6 LEG FILLET WELD (U.N.O.)

CHECK 203 x 203 x 52 UC SECTION AT END OF NOTCH



AREA OF SECTION

$(0.5 \times 66.40) + (1.70 \times 0.80)$

$+ (8.0 \times 0.80)$

$= 33.20 + 1.36 + 6.40 = \underline{40.96\ cm^2}$

TO FIND NEUTRAL AXIS ~ TAKE MOMENTS ABOUT U/S BOTTOM FLANGE

$40.96\bar{x} = (33.20 \times 17.6) + (1.36 \times 111.5) - (6.40 \times 40.0)$

$\bar{x} = 479.96 \div 40.96 = \underline{11.7\ mm}$

(CN~24)

$$I_{NA} = 2031000 + \left(\frac{8.0 \times 17^3}{12}\right) + \left(\frac{8.0 \times 80^3}{12}\right) + (3320 \times 5.9^2)$$

$$+ (136 \times 99.8^2) + (640 \times 51.7^2) = 5556392 \text{ mm}^4$$

$$Z = 5556392 \div 108.3 = 51305 \text{ mm}^3$$

$$M_C = 355 \times 51305 \div 10^3 = 18213 \text{ KNmm}$$

$$BM = 150 \times 80 = \underline{12000 \text{ KNmm} < 18213 \text{ KNmm}} \text{ OK}$$

Shear Area; $A_V = 0.9 \times 200 \times 8.0 = 1440 \text{ mm}^2$

$$P_V = 0.6 \times 355 \times 1440 \div 10^3$$

$$= \underline{306 \text{ KN} > 150 \text{ KN}} \text{ OK}$$

<u>Section Satisfactory</u>

BEAMS B1-5 ; B6-5 & COLUMN C7 TO BEAM B7-5

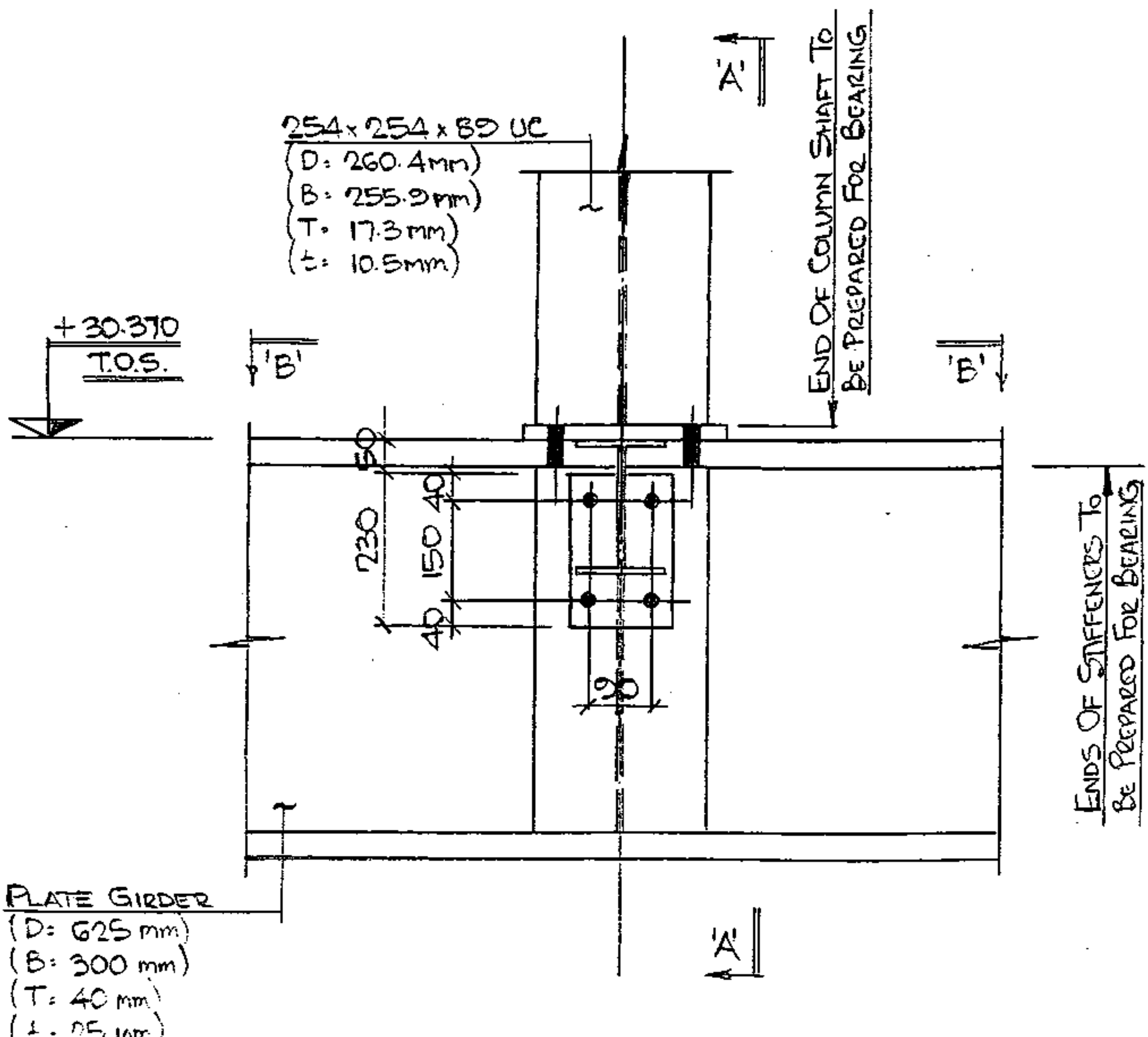
254 x 146 x 31 UB ; 203 x 133 x 25 UB & 254 x 254 x 89 UC TO 625 x 300 x 25 x 40 PLATE GIRDER

END REACTIONS :—

BEAM B1-5 = 164 KN.

BEAM B6-5 = 123 KN.

COLUMN C7 = 1430 KN.



M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

SERIES Sheet 2 of 7

O/No. 325 JOB Carlton Gardens

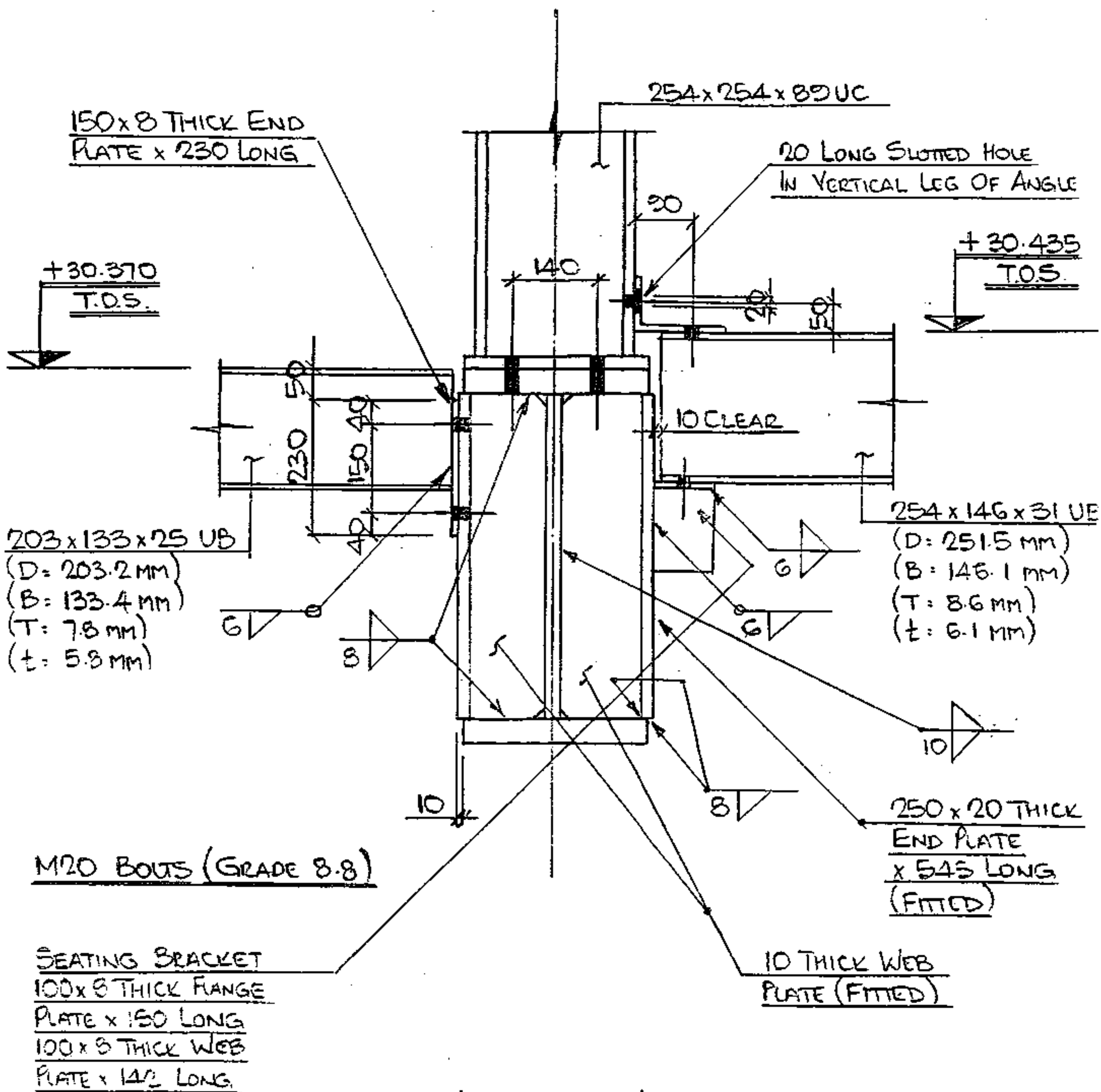
SHT. No. C59 REV.

SUBJECT Connection Design

BY KB DATE 08/92

EX. DATE

(CN ~ 30)



View On 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

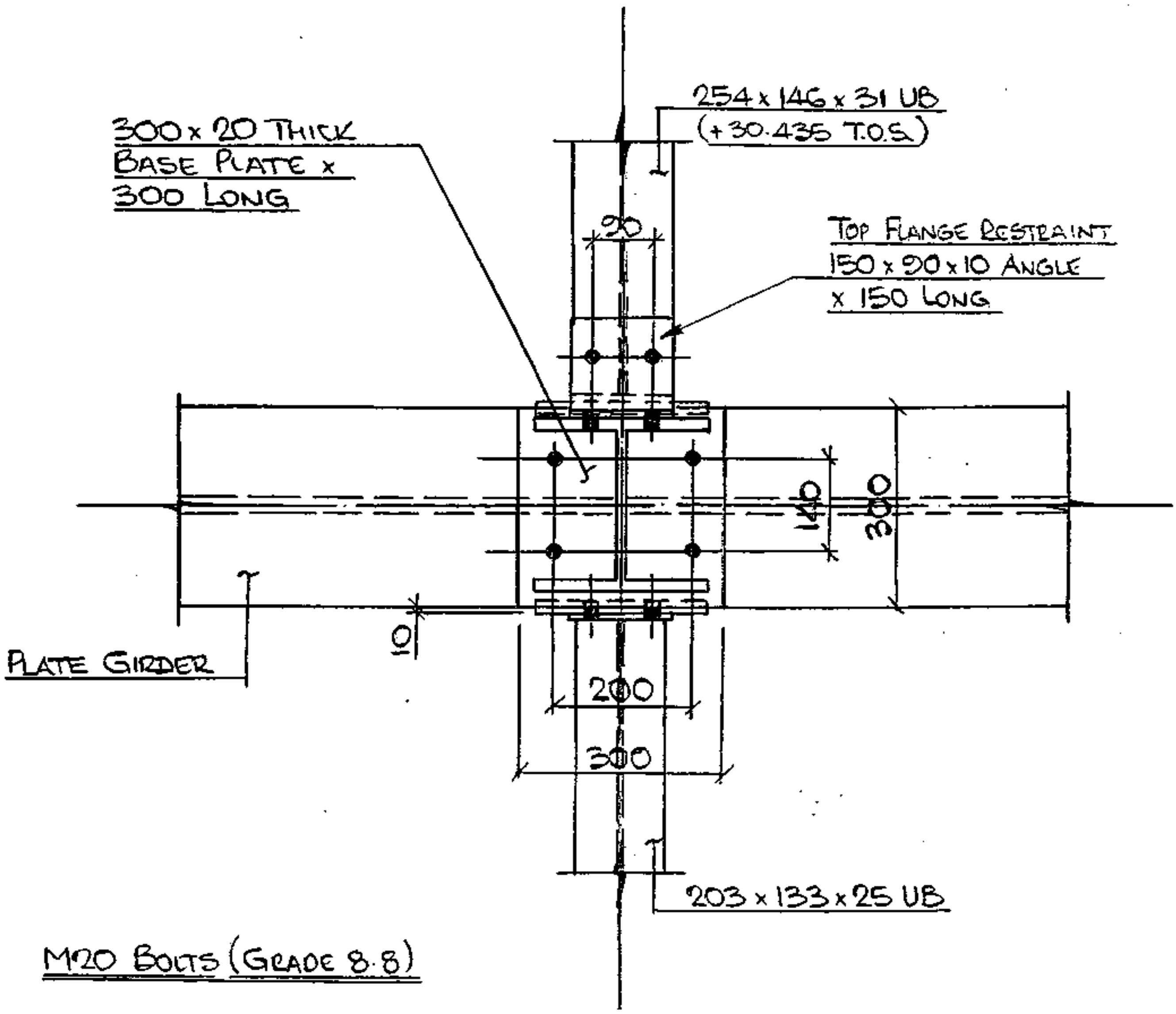
SERIES SHEET 3 OF 7

SHT. No. C59 REV.

BY KB DATE 08/98

EX. DATE

(CN~30)



VIEW ON 'B-B'

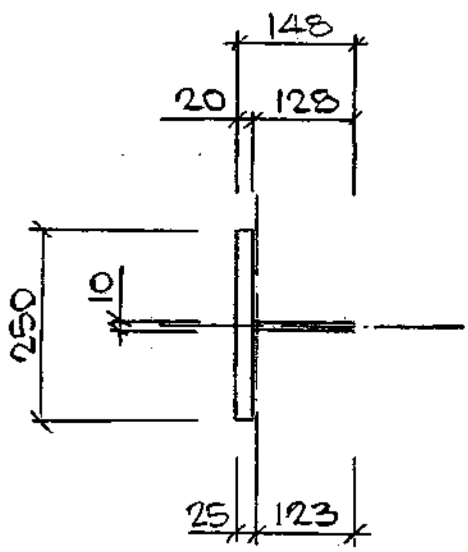
DESIGN OF WELD FOR STIFFENER TO PLATE GIRDER WEB.

WEB DEPTH : $625 - (2 \times 40) = 545$ MM

EFFECTIVE WELD LENGTH : $545 - (2 \times \{10 + 8\}) = 509$ mm ⟶ SAY 500

WELD LENGTH : $2 \times 500 = 1000$ mm^2

$Z = 2 \times 500^2 \div 6 = 83333$ mm^3.



AREA OF SECTION

$A = (250 \times 20) + (128 \times 10)$

$= 5000 + 1280 = \underline{6280 \text{ mm}^2}$

TO FIND NEUTRAL AXIS ~
TAKE MOMENTS ABOUT L.H. SIDE

$6280\bar{x} = (5000 \times 10) + (1280 \times 84)$

$\bar{x} = 157520 \div 6280 = \underline{25.0 \text{ mm}}$

SHEAR LOAD = $(0.5 \times 1430) + 164 = 879$ KN

BENDING MOMENT : $(715 \times 123) + (164 \times 148) = 112217$ KN mm

$f_s = 879 \div 1000 = 0.88$ KN/mm^2

$f_m = 112217 \div 83333 = 1.35$ KN/mm^2

$f_{RES} = \sqrt{0.88^2 + 1.35^2} = 1.57$ KN/mm^2

WELD STRENGTH (GRADE 50) = 255 N/mm^2

WELD SIZE REQD : $\dfrac{1.57 \times 10^3}{0.70 \times 255} = 8.8$ MM ⟶ 10 LEG F.W.

USE : — 10 LEG FILLET WELD

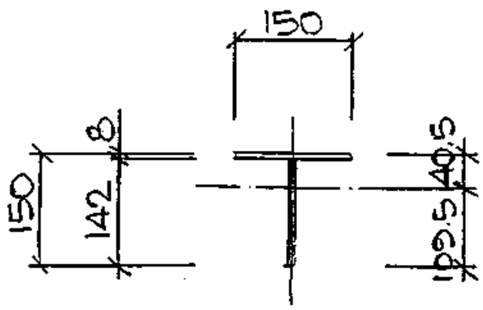
(CN ~ 30)

DESIGN OF SEATING BRACKET

END REACTION : 164 KN

LEVER ARM = 50 mm

BENDING MOMENT : 164 x 50 = 8200 KNMM



TRY : – 8 THICK PLATES

SECTION AREA

(150 x 8) + (142 x 8)

: 1200 + 1136 = 2336 mm²

TO FIND NEUTRAL AXIS ~ TAKE MOMENTS ABOUT TOP

$2336\bar{x} : (1200 \times 4) + (1136 \times 79)$

$\bar{x} : 94544 \div 2336 : 40.5\ mm$

SHEAR AREA ; Av : $0.9 \times 142 \times 8 : 1022\ mm^2$

Pv : $0.6 \times 355 \times 1022 \div 1000$

: 217 KN > 164 KN OK

$$I_{xx} : \left(\frac{150 \times 8^3}{12}\right) + \left(\frac{8 \times 142^3}{12}\right) + (1200 \times 36.5^2) + (1136 \times 38.5^2)$$

$= 5197794\ mm^4$

$Z_{xx} = 5197794 \div 109.5 : 47468\ mm^3$

$M_c : 355 \times 47468 \div 10^3$

: 16851 KN mm > 8200 KN mm OK

BRACKET SATISFACTORY

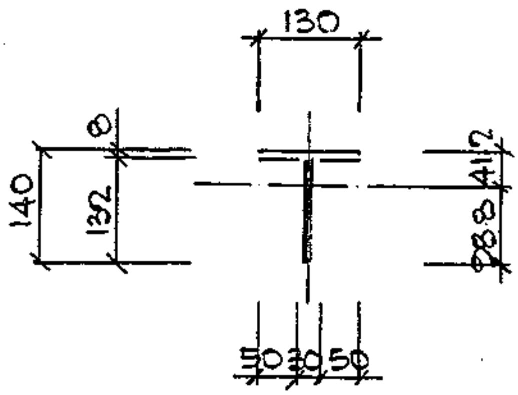
(CN ~ 30)

DESIGN OF WELD FOR SEATING BRACKET TO END PLATE

END REACTION = 164 KN

LEVER ARM = 50 MM

BENDING MOMENT = 164 × 50 = 8200 KNmm



CONSIDER WELD OF UNIT LEG LENGTH

WELD AREA = $130 + (2 \times 50) + (2 \times 132)$

$= \underline{494 \text{ mm}^2}$

TO FIND NEUTRAL AXIS ~ TAKE MOMENTS ABOUT TOP

$494\bar{x} = (100 \times 8) + (264 \times 74)$

$\bar{x} = 20336 \div 494 = \underline{41.2 \text{ mm.}}$

$I_{xx} = \left(\frac{130}{12}\right) + \left(\frac{2 \times 50}{12}\right) + \left(\frac{2 \times 132^3}{12}\right) + (130 \times 41.2^2) + (100 \times 33.2^2) + (264 \times 32.8^2)$

$= 998260 \text{ mm}^4$

$Z_{xx} = 998260 \div 98.8 = 10104 \text{ mm}^3$

$f_s = 164 \div 494 = 0.34 \text{ KN/mm}^2$

$f_m = 8200 \div 10104 = 0.82 \text{ KN/mm}^2$

$F_{RES} = \sqrt{0.34^2 + 0.82^2} = 0.89 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 50) = 255 N/mm²

WELD SIZE REQ^D = $\dfrac{0.89 \times 10^3}{0.70 \times 255}$ = 5.0 mm ⟶ USE :- 6 LEG FILLET WELD

(CN~30)

DESIGN OF WELD FOR FLANGE PLATE TO WEB PLATE

SHEAR/mm OF WELD : $\frac{S \times A \times \bar{y}}{I}$

$= \frac{164 \times 1200 \times 36.5}{5197794} = 1.38$ KN/mm

2 No RUNS OF WELD = $0.5 \times 1.38 = 0.69$ KN/mm PER WELD

WELD STRENGTH (GRADE 50) = 255 N/mm^2

WELD SIZE REQD = $\frac{0.69 \times 10^3}{0.70 \times 255} = 3.9$mm ⟶ 6 LEG F.W (MIN^M)

USE :– 6 LEG FILLET WELD

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B1-5 & COLUMN C7 TO BEAM B9-5

(CN-30)

SERIES SHEET 1 OF 6

SHT. No. C59A REV.

BY KB DATE 08/98

EX. DATE

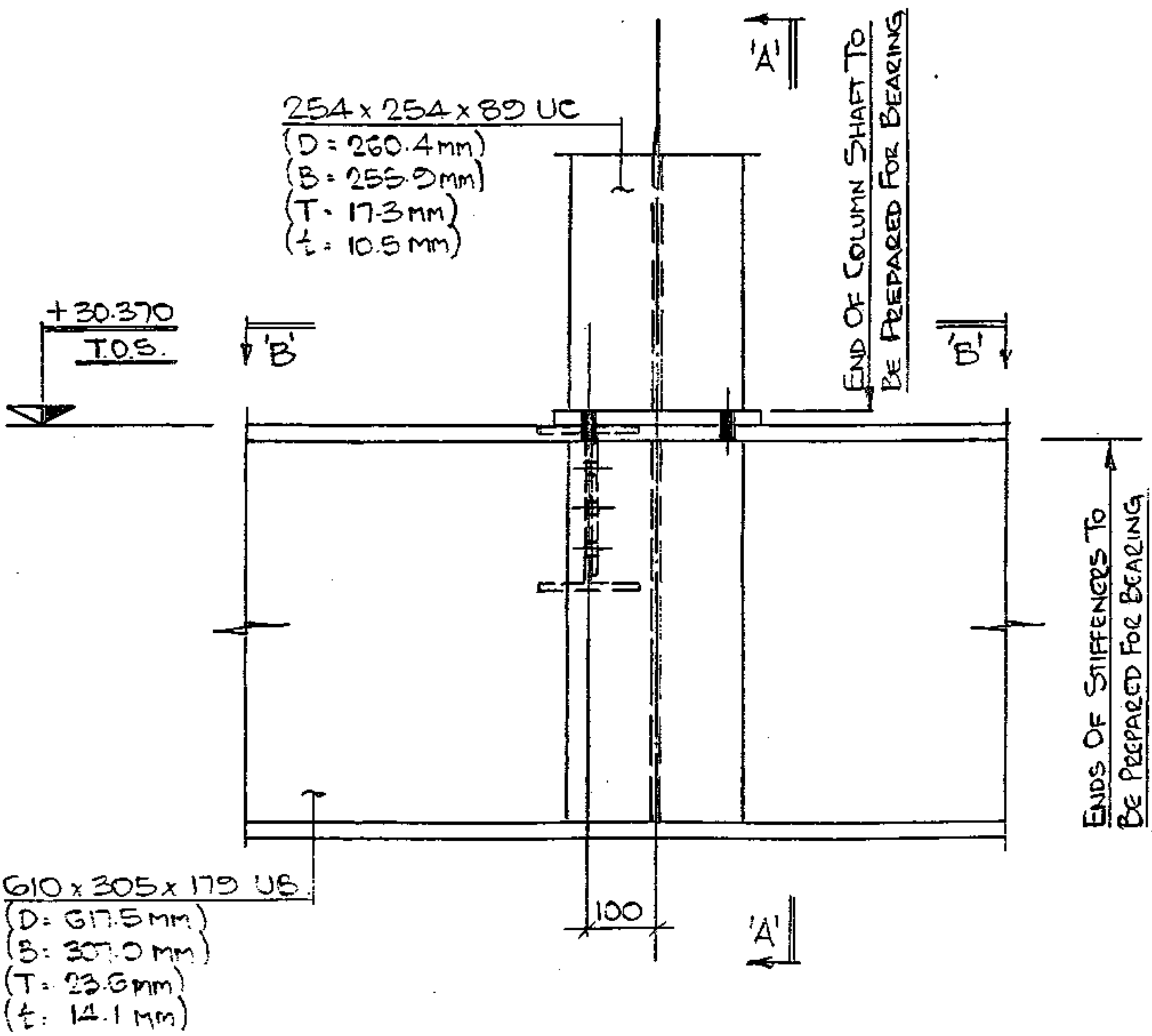
BEAM B1-5 & COLUMN C7 TO BEAM B9-5

254 x 146 x 31 UB & 254 x 254 x 89 UC TO 610 x 305 x 179 UB

END REACTIONS :-

BEAM B1-5 = 164 KN.

COLUMN C7 = 1430 KN

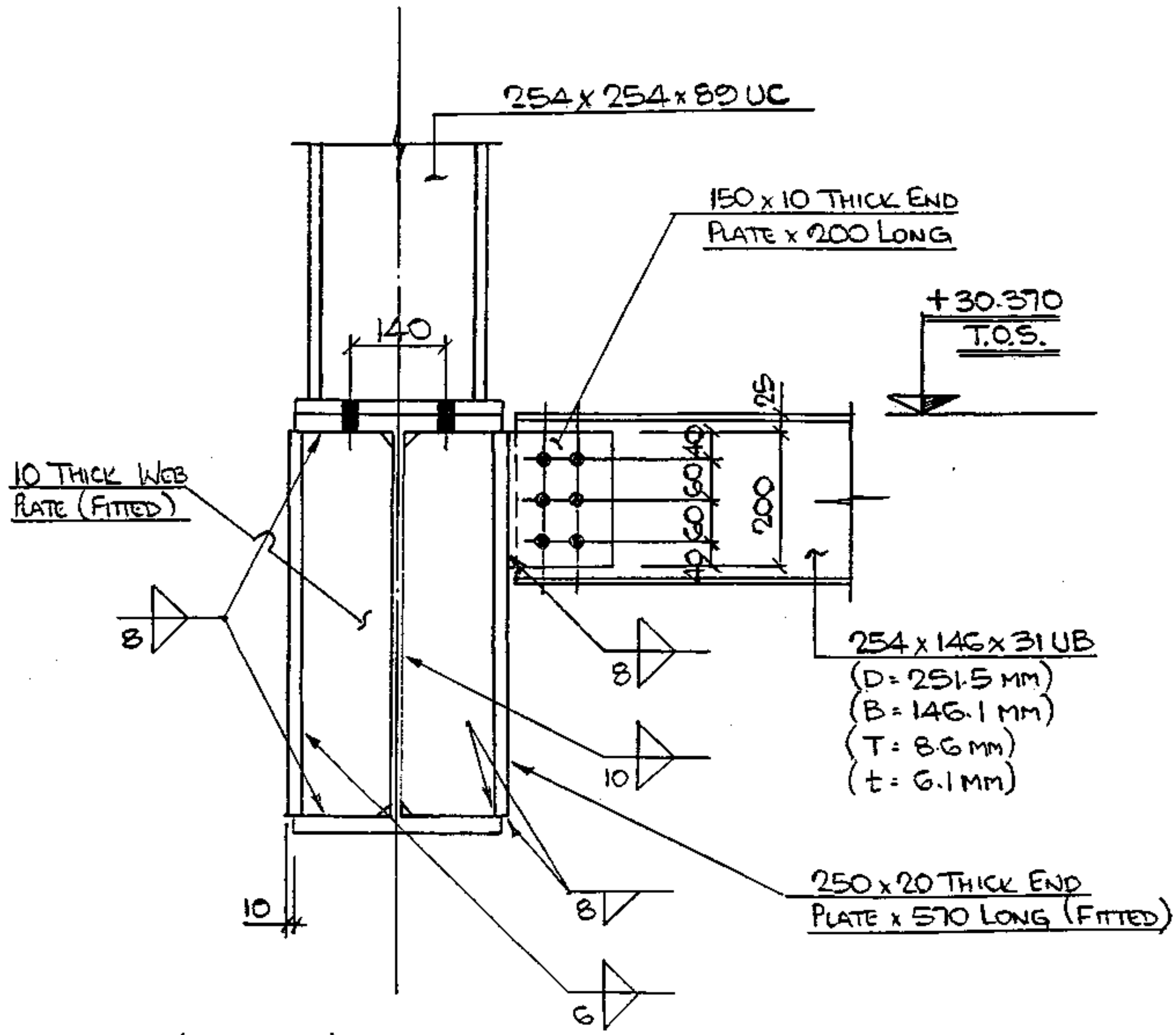


M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

SERIES SHEET 2 OF 6
O/No. 325 JOB CARLTON GARDENS
SHT. No. C59A REV.
SUBJECT CONNECTION DESIGN
BY KB DATE 08/98
EX. DATE

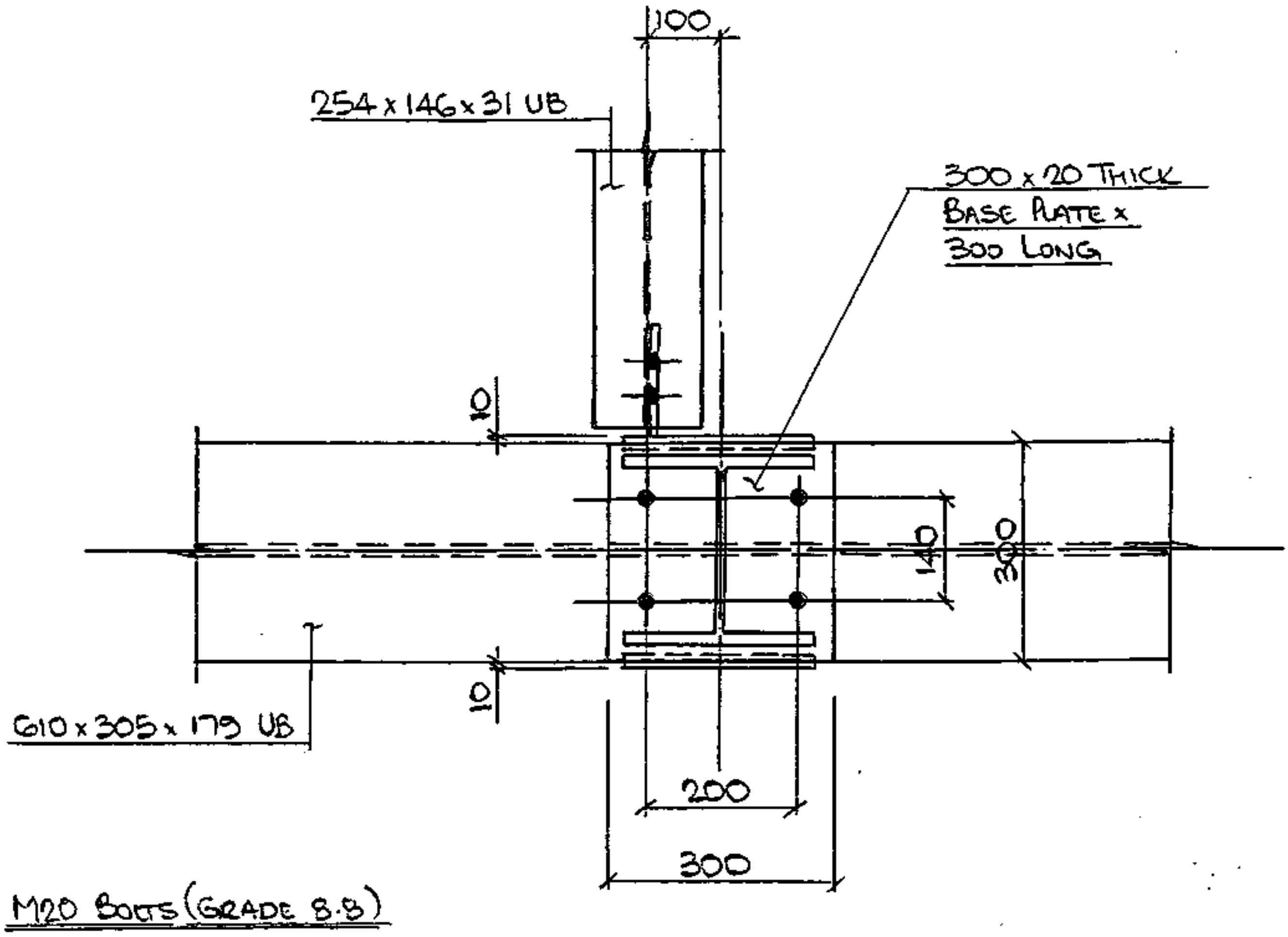
(CN ~ 3C)



M20 BOLTS (GRADE 8.8)

VIEW ON 'A-A'

(CN-30)

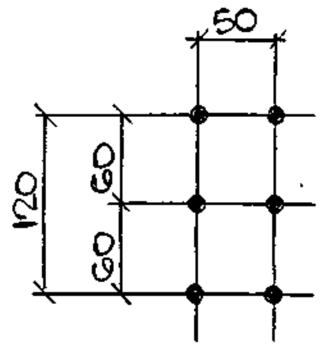


VIEW ON 'B-B'

(CN ~ 30)

DESIGN OF BOLTS

TRY : — 6 No ~ M20 BOLTS (GRADE 8.8)



$I_{xx} = 4 \times 60^2 = 14400 \text{ mm}^4$

$I_{yy} = 6 \times 25^2 = 3750 \text{ mm}^4$

$I_p = 14400 + 3750 = 18150 \text{ mm}^4$

$r = \sqrt{60^2 + 25^2} = 65.0 \text{ mm}$

$Z = 18150 \div 65.0 = 279 \text{ mm}^3$

END REACTION = 164 KN

LEVER ARM = 75 mm

BENDING MOMENT = 164 x 75 = 12300 KN mm

$f_s = 164 \div 6 = 27.4$ KN

$f_m = 12300 \div 279 = 44.1$ KN

$f_m (V_c) = 44.1 \times 25.0 \div 65.0 = 17.0$ KN

$f_m (H_c) = 44.1 \times 60.0 \div 65.0 = 40.7$ KN

$f_{res} = \sqrt{(27.4 + 17.0)^2 + 40.7^2} = 60.3$ KN

BOLT SHEAR ; $P_s = 245 \times 375 \div 10^3 = 91.8$ KN

BOLT BEARING ; $P_{bb} = 1035 \times 20 \times 6.1 \div 10^3 = 126.2$ KN

PL BEARING ; P_{bs}

(a) $d \times t \times p_{bs} = 20 \times 6.1 \times 550 \div 10^3 = 67.1$ KN

(b) $\frac{1}{2} \times e \times t \times p_{bs} = \frac{1}{2} \times 40 \times 6.1 \times 550 \div 10^3 = 67.1$ KN

$P_{bs} = 67.1$ KN

(CN~30)

BOLT CAPACITY = 67.1 KN > 60.3 KN OK

USE :— 6 No ~ M20 BOLTS (GRADE 8.8)

CHECK WELD FOR FIN PLATE TO FLANGE

LENGTH OF FIN PLATE = 200 mm

EFFECTIVE WELD LENGTH = $200 - (2 \times 10) = 180$ mm

LOAD PER MM RUN OF WELD : $164 \div (2 \times 180) = 0.46$ KN/mm²

WELD STRENGTH (GRADE 50) = 255 N/mm²

WELD SIZE REQ^{D} = $\dfrac{0.46 \times 10^3}{0.70 \times 255} = 2.6$ mm → 6 LEG F.W (MIN^{M}/)

USE :— 8 LEG FILLET WELD

CHECK :— 10 THICK FIN PLATE

a) SHEAR CAPACITY

$A_V = 0.9 \times 200 \times 10 = 1800$ mm²

$A_{VNET} = 1800 - (3 \times 22 \times 10) = 1140$ mm²

$0.6 \times P_y \times A_V = 0.6 \times 355 \times 1800 \div 10^3 = 383$ KN

$0.5 \times U_S \times A_{VNET} = 0.5 \times 490 \times 1140 \div 10^3 = 279$ KN

P_V: 279 KN > 164 KN OK

$A_{V1} = 160 \times 10 = 1600$ mm²

$A_{TEFF} = (100 - \{2.5 \times 22\}) \times 10 = 450$ mm²

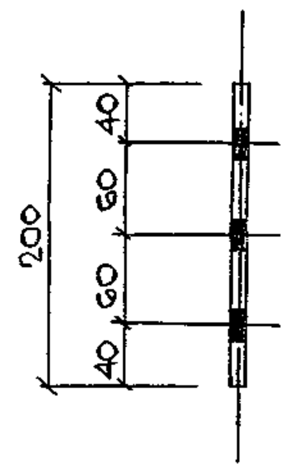
(CN-30)

$P_{VB} = (0.6 \times P_y \times A_{VI}) + (0.5 \times U_S \times A_{TEFF})$

$= (0.6 \times 355 \times 1600 \div 1000) + (0.5 \times 490 \times 450 \div 10^3)$

$P_{VB} = \underline{451\ KN > 164\ KN}$ OK

b) MOMENT CAPACITY



$I_{XX} = 10 \times 200^3 \div 12 = 6666666\ mm^4$

$I_{NET} = 6666666 - \left(\frac{3 \times 10 \times 22^3}{12}\right) - (2 \times 10 \times 22 \times 60^2)$

$= 5056046\ mm^4$

$Z = 5056046 \div 100 = 50560\ mm^3$

$M_C = 355 \times 50560 \div 10^3$

$= \underline{17948\ KNmm > 12300\ KNmm}$ OK

USE : — 10 THICK FIN PLATE

REV 'A'	28/09/98	SHEAR & MOMENT REACTIONS AMENDED FAX REF 54015/ED ~ 28.09.98	KB

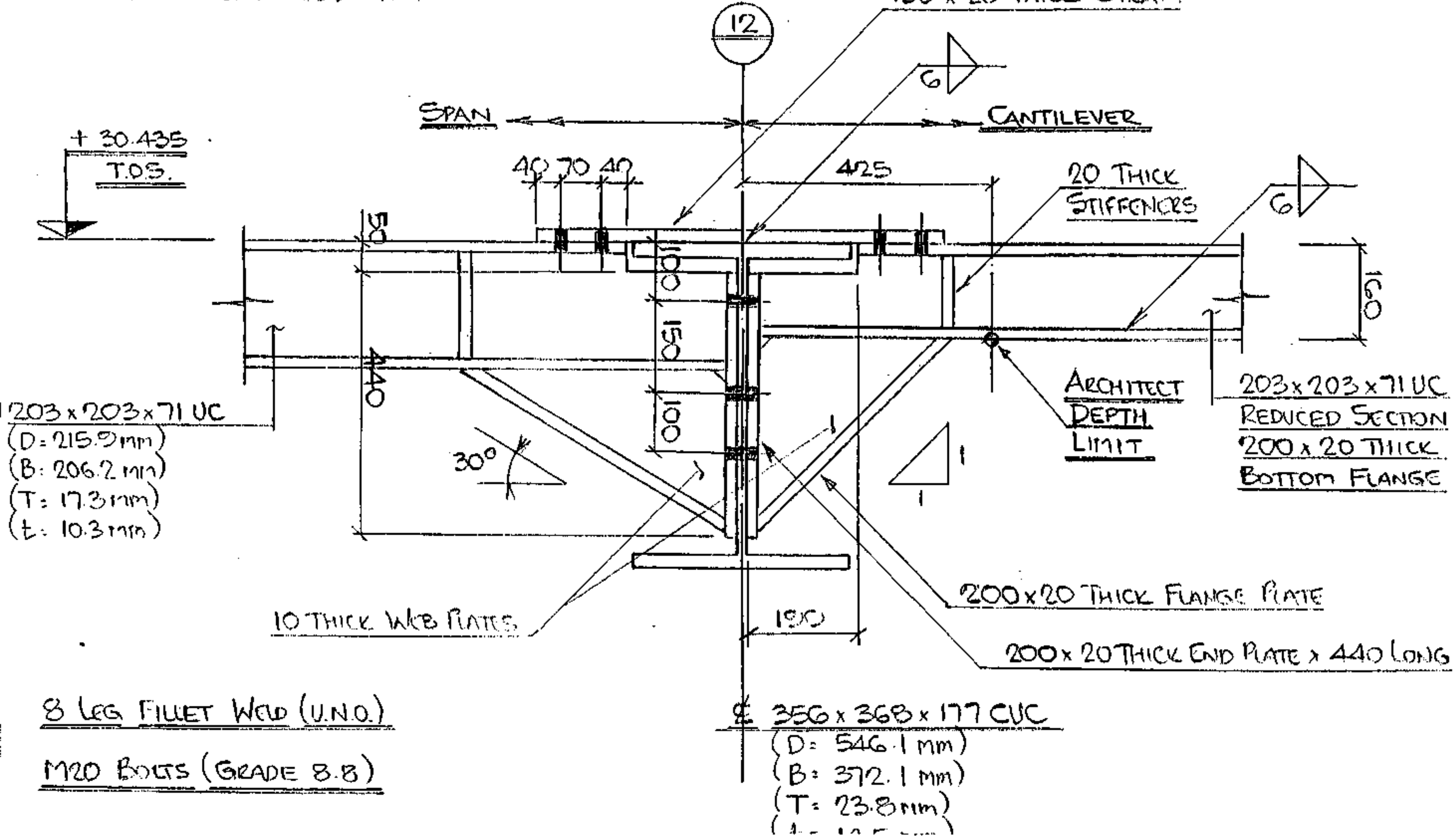
BEAMS B14-5 TO BEAM B2-5

203 x 203 x 71 UC TO 356 x 368 x 177 CUC

END REACTIONS : —

A SHEAR : 125 KN

A MOMENT : 180 KNM



8 LEG FILLET WELD (U.N.O.)

M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAMS B14-5 TO BEAM B2-5

(CN ~ 30)

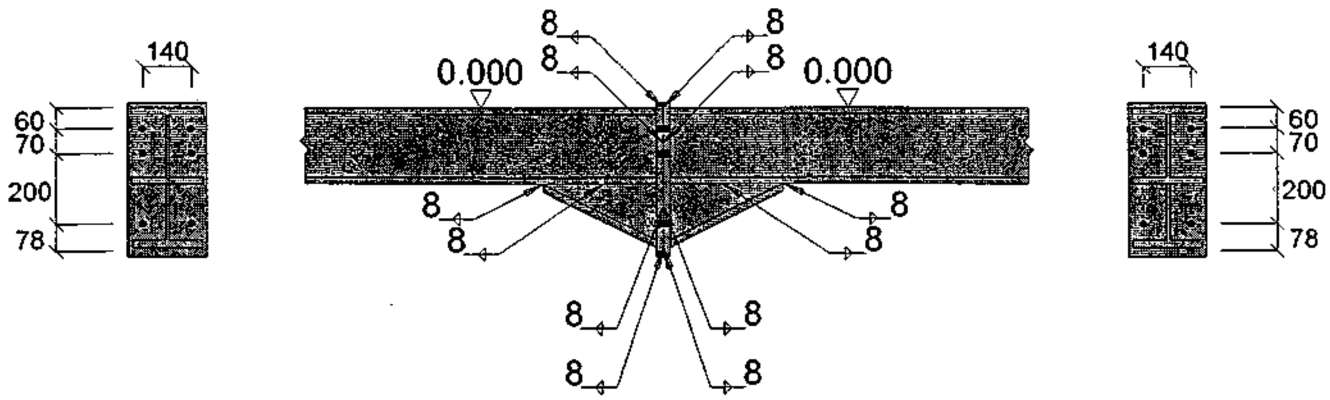
SERIES SHEET 1 OF 3

SHT. No. C60 REV. A

BY KB DATE 09/98

EX .... DATE ....

203UC71 to 203UC71



Endplate 440x225x20 Gr 50
Bolts: M20 (Grade 8.8)

Endplate 440x225x20 Gr 50
Bolts: M20 (Grade 8.8)

UC 203x203x71 Gr 50 (Slope 0.0deg)
Haunch: Built up plate Gr 50
Length = 0.340m, Depth = 0.300m
B/F Flange 200x20mm thk, Web 10mm thk

UC 203x203x71 Gr 50 (Slope 0.0deg)
Haunch: Built up plate Gr 50
Length = 0.340m, Depth = 0.300m
B/F Flange 200x20mm thk, Web 10mm thk

Design Summary

Design Combination	Face	Status	Moment Capacity	Shear Capacity	Beam Web Capacity	Weld Checks	Stiffener Checks
Load Case 1	Left	Pass	1.00	0.35	0.00	Pass	n/a

Design Forces Summary

Design Combination	Face	M (kNm)	V (kN)	N (kN)	Mse (kNm)	Factor
Load Case 1	Left	150.0	115.0	0.0	0.0	1.0

(CN-30)

DESIGN OF TOP FLANGE TENSION STRAP ~ BOTTOM FLANGES IN BEARING

A MOMENT = 180 KNm

LEVER ARM = 500 mm (APPROX)

A FLANGE FORCE = 180.00 ÷ 0.500 = 360 KN (TENSION)

P_y = 345 N/mm² ~ T > 16 mm (ASSUMED)

NET TENSION AREA REQ^D

A = 360 × 10³ ÷ 345 = 1044 mm²

ADD 2 No HOLES @ 22 DIA

A = 1044 + (2 × 22 × 20) = 1924 mm²

ASSUMING 20 THICK PLATE

A WIDTH REQ^D = 1924 ÷ 20 = 96.2 mm

USE: — 150 × 20 THICK STRAP.

DESIGN OF BOLTS

A FLANGE FORCE = 360 KN

SHEAR VALUE ~ M20 BOLTS (GRADE 8.8) = 91.8 KN

A No OF BOLTS REQ^D = 360 ÷ 91.8 = 3.9 → SAY 4 No BOLTS

USE: — 4 No ~ M20 BOLTS (GRADE 8.8)

REV 'A' 28/09/98 SHEAR & MOMENT REACTIONS AMENDED FAX REF 5401S/ED ~ 28/09/98 KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B1-5 & BEAM B14-5 TO BEAM B2-5

(CN – 30)

SERIES SHEET 1 OF 1

SHT. No. C60A REV.

BY KB DATE 09/98

EX. DATE

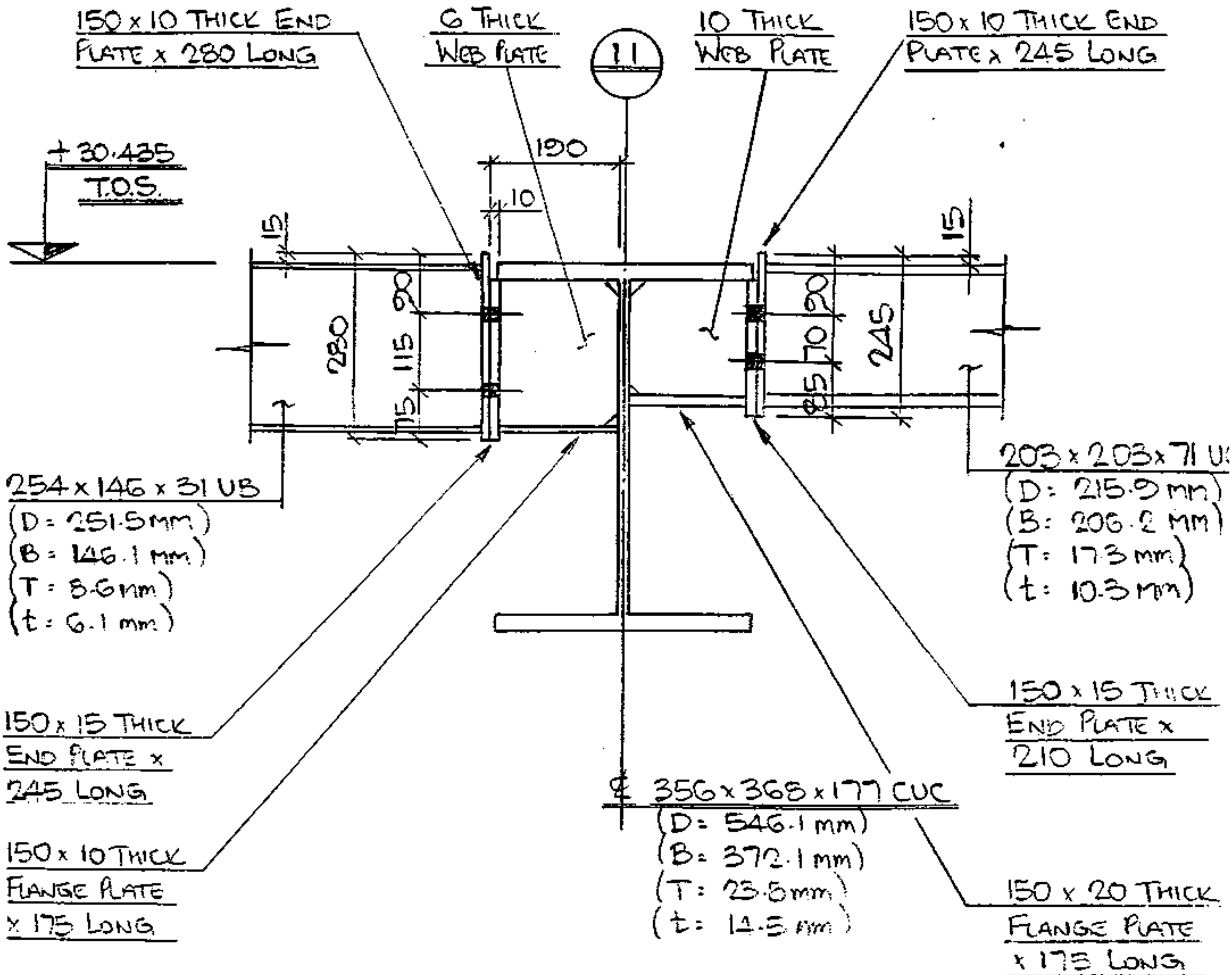
BEAM B1-5 & BEAM B14-5 TO BEAM B2-5

254 x 146 x 31 UB & 203 x 203 x 71 UC TO 356 x 368 x 177 CUC

END REACTIONS : –

BEAM B1-5 = 164 KN

BEAM B14-5 = 234 KN

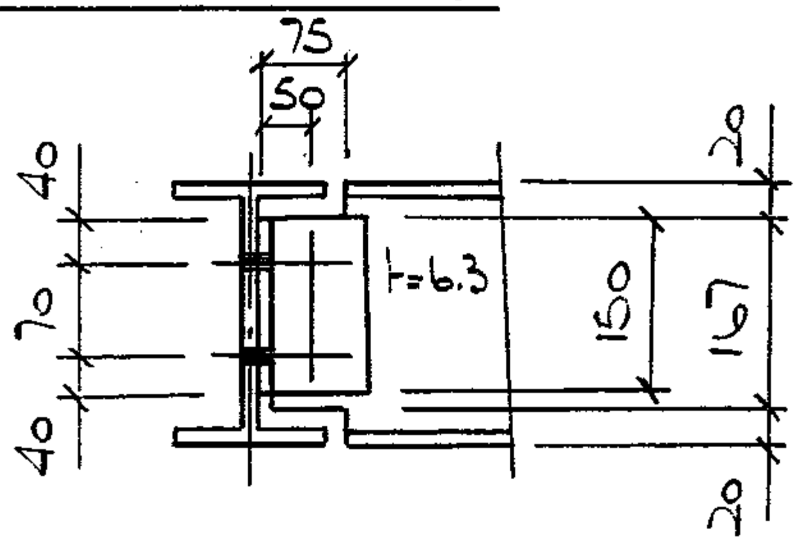


M20 BOLTS (GRADE 8.8)

6 LEG FILLET WELD (U.N.O.)

CONN CA203 (203×133×30UB - 70kN)

203UB to 203UB



2N° 90×90×10 Angles
6N° M20 Gr8.8 Bolts

i) Bolts

Supported Beam $f_v = 70/2 = 35kN$

$f_h = 70\times50/70 = 50kN$

$f_r = \sqrt{(35^2+50^2)} = \underline{61kN} < 2\times92k$

Check bearing on beam web

Allowable bearing $= 6.3\times20\times550\times10^{-3}$

$= \underline{69kN} > 61kN$

Supporting Beam $f_v = 70/4 = \underline{17.5kN} < 61kN$

$\therefore$ bolts ok

CONN CA203 cont

iii) Check Beam for Notch

a. Shear $A_v = 0.9 \times 167 \times 6.3 = 947\,mm^2$

$A_{V net} = 947 - (2 \times 22 \times 6.3) = 670\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 947 = \underline{202\,kN} > 70\,kN$

$P_{V net} = 0.5 \times 490 \times 10^{-3} \times 670 = \underline{164\,kN} > 70\,kN$

b. Bending $Z = 6.3 \times 167^2/6 = 29\,283\,mm^3$

$M = 70 \times 75 \times 10^{-3} = \underline{5.25\,kNm}$

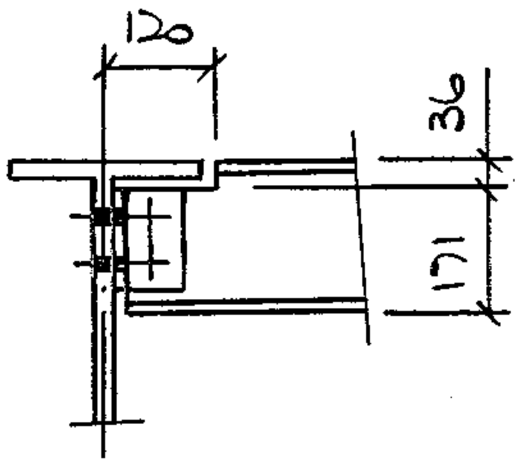
$P_b = 29\,283 \times 355 \times 10^{-6}$

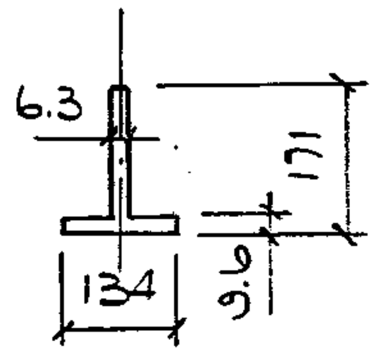
$= \underline{10.4\,kNm} > 5.25\,kNm$

$\therefore$ Section OK

CONN CA203 cont.

Check Beam Notch into larger UB
ie 610 × 229 × 140 UB





$A = 2303\ mm^2$

$\bar{x} = 43mm$

$I_{NA} = 636\ cm^4$

a. Shear $A_v = 0.9 \times 171 \times 6.3 = 970\ mm^2$

$A_{V_{net}} = 970 - (2 \times 22 \times 6.3) = 693 mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 970 = \underline{207 kN} > 70 kN$

$P_{V_{net}} = 0.5 \times 490 \times 10^{-3} \times 693 = \underline{170 kN} > 70 kN$

b. Bending $M = 70 \times 120 \times 10^{-3} = \underline{8.4\ kNm}$

$P_b = 636 \times 10^4 / (171 - 43) \times 355 \times 10^{-6}$

$= 17.6 kNm > 8.4 kNm$

$\therefore$ Section OK

Kvaerner Cleveland Bridge Ltd. SERIES SH 1 OF 2

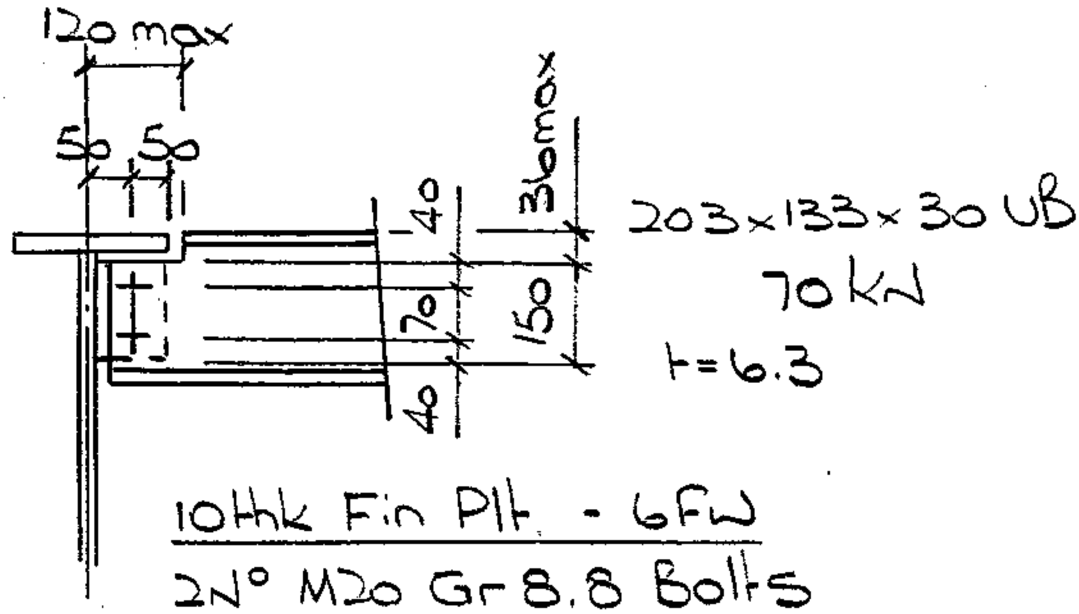
O/No. 325 JOB CARLTON GARDENS SHT. No. C61A REV.

SUBJECT Connection Design BY KM DATE Oct. 98

EX. DATE

CN17

Conn. for 203x133x30 UB (B12-6)



10thk Fin Plt - 6FW

2N° M20 Gr 8.8 Bolts

i) Bolts

$F_v = 70/2 = 35\,kN$

$F_h = 70 \times 50/70 = 50\,kN$

$F_r = \sqrt{(35^2 + 50^2)} = \underline{61\,kN} < 92\,kN$

Check bearing on beam web / Fin Plt

Allowable bearing $= 6.3 \times 20 \times 550 \times 10^{-3}$

$= \underline{69\,kN} > 61\,kN$

$\therefore$ Bolts OK

ii) Fin Plt.

a. Shear

$A_V = 0.9 \times 150 \times 10 = 1350\,mm^2$

$A_{Vnet} = 1350 - (2 \times 22 \times 10) = 910\,mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 1350 = \underline{288\,kN} > 70\,kN$

$P_{Vnet} = 0.5 \times 490 \times 10^{-3} \times 910 = \underline{223\,kN} > 70\,kN$

b. Bending

$Z = 10 \times 150^2 / 6 = 37\,500\,mm^3$

$M = 70 \times 50 \times 10^{-3} = \underline{3.5\,kNm}$

$P_b = 37\,500 \times 355 \times 10^{-6} = \underline{13.3\,kNm} > 3.5\,kNm$

$\therefore$ Plt OK

iii) Fin Plt. Weld

$P_V = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = 321\,kN > 70\,kN$

$\therefore$ Weld OK

Beam Notch as Conn. C61

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 2

SHT. No. C62 REV.

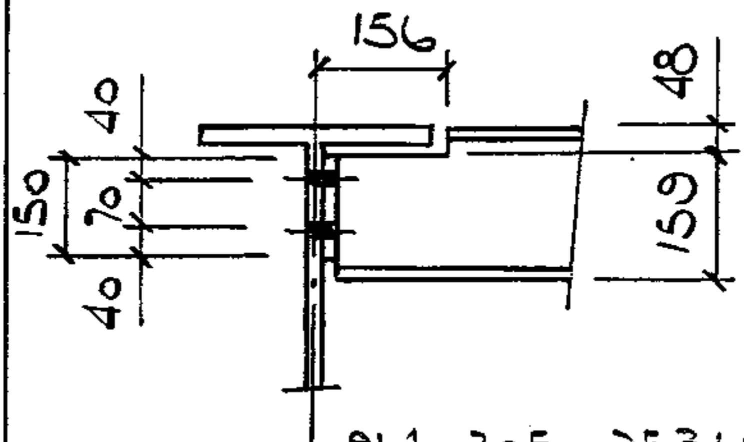
BY KM DATE Aug 98

EX. DATE

CN17

CONN EA203

203×133×30UB to 914×305×253UB



203×133×30UB · 70kN

150×8thk End Plt.

6FW

4N° M20 Gr 8.8 Bolts

i) Bolts $F_v = {}^{70}/_4 = \underline{17.5\,kN} < 92\,kN$

Allowable bearing on end plt. $= 8 \times 20 \times 550 \times 10^{-3}$

$= \underline{88\,kN} > 17.5\,kN$

$\therefore$ Bolts OK

ii) End Plt

$A_v = 0.9 \times 150 \times 8 = 1080\,mm^2$

$A_{v_{net}} = 1080 - (2 \times 22 \times 8) = 728\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1080 = \underline{230\,kN} > {}^{70}/_2\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 728 = \underline{178\,kN} > {}^{70}/_2\,kN$

$\therefore$ End Plt. OK

CONN EA203 cont

iii/ End Plt. Weld 6mm FW

$P_V = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321\,kN} > 70\,kN$

$\therefore$ Weld ok

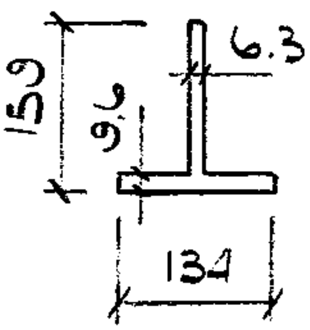
iv/ Check web behind End Plt for Shear

$A_V = 0.9 \times 150 \times 6.3 = 850\,mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 850 = \underline{181\,kN} > 70\,kN$

$\therefore$ Web ok

v/ Check beam for Notch



$A = 2228\,mm^2$

$\bar{x} = 38\,mm$

$I_{NA} = 554\,cm^4$

Bending

$M = 70 \times 156 \times 10^{-3} = 10.9\,kNm$

$P_b = 554 \times 10^4 / (159 - 38) \times 355 \times 10^{-6} = \underline{16.3\,kNm} > 10.9\,kNm$

$\therefore$ Section ok

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 1

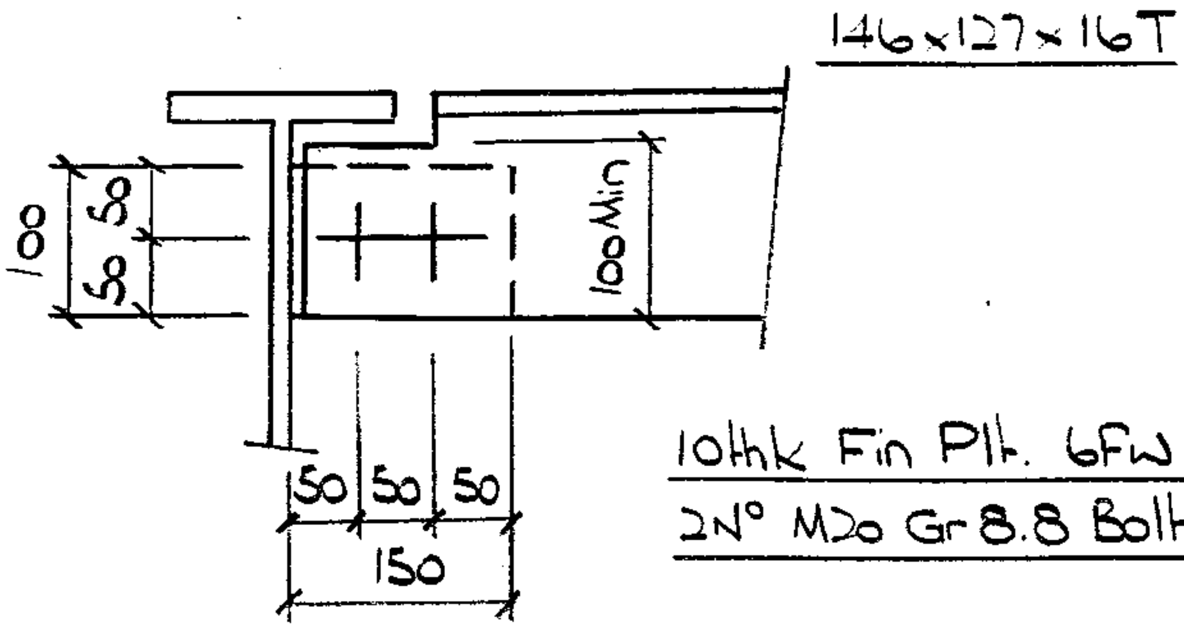
SHT. No. C63 REV.

BY KM DATE Aug 98

EX. DATE

CN17

CONN FA146 (146×127×16 T)



10thk Fin Plt. 6FW

2N° M20 Gr 8.8 Bolts

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 1

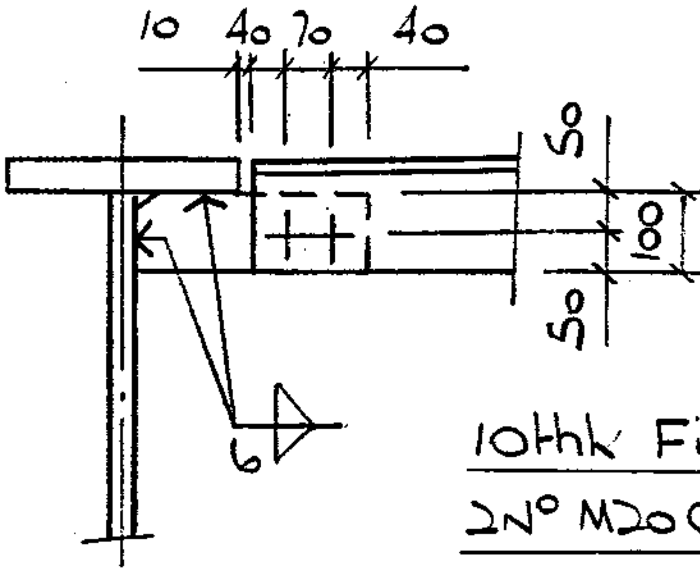
SHT. No. C63A REV.

BY KM DATE Aug 98

EX. DATE

CN17

CONN FB146 (146×127×16T)

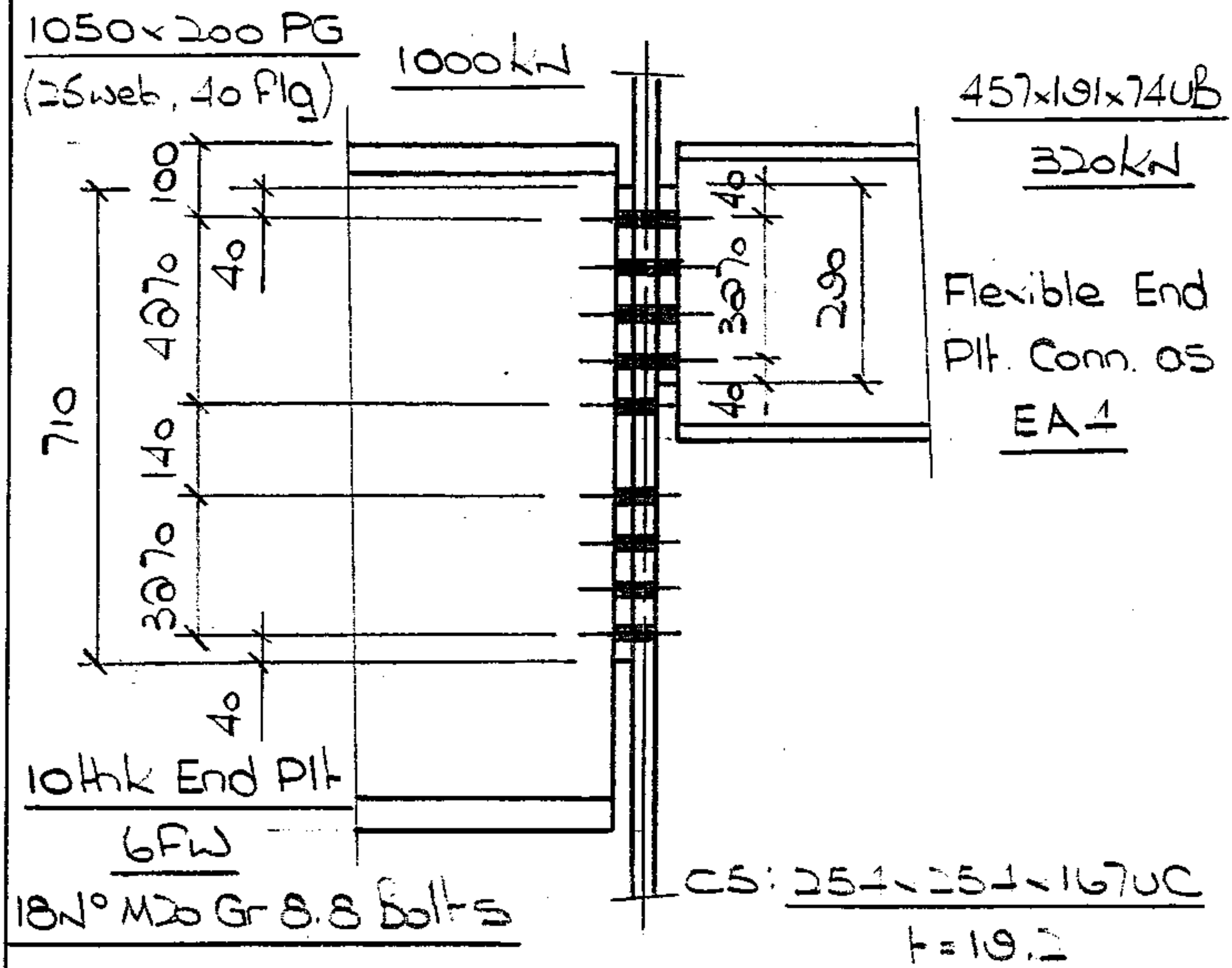


10thk Fin Plt

2N° M20 Gr8.8 Bolts

CONN F6/1

B5·6 to C5 to B3·6



1050 PG. Conn.

i) Bolts $F_v = {}^{1000}/_{18} = \underline{56\,kN} < 92\,kN$

Allowable bearing $= 10 \times 20 \times 550 \times 10^{-3}$

on end plt $= \underline{110\,kN} > 56\,kN$

CONN F6/1 cont.

Bolts cont.

Check bearing on Col. web.

Load per bolt $= {}^{1000}/_{18} + {}^{320}/_{8} = \underline{96\,kN} < 2 \times 92\,kN$

Allowable bearing $= 19.2 \times 20 \times 550 \times 10^{-3}$

$= \underline{211\,kN} > 96\,kN$

$\therefore$ Bolts OK

ii) End Plt.

$A_v = 0.9 \times 710 \times 10 = 6390\,mm^2$

$A_{v_{net}} = 6390 - (9 \times 22 \times 10) = 4410\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 6390 = \underline{1361\,kN} > {}^{1000}/_{2}\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 4410 = \underline{1080\,kN} > {}^{1000}/_{2}\,kN$

$\therefore$ End Plt OK

iii) End Plt Weld 6FW

$P_v = 2 \times 710 \times 4.2 \times 255 \times 10^{-3}$

$= \underline{1520\,kN} > 1000\,kN$

$\therefore$ Weld OK

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O/No. 325 JOB CARLTON GARDENS
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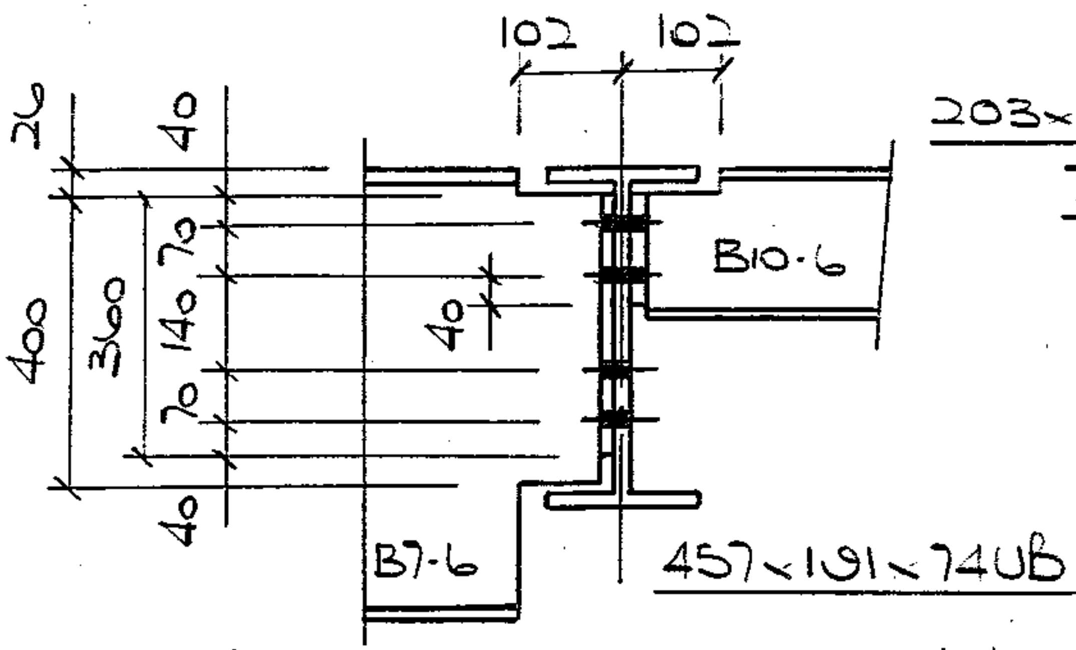
CONN F6/2

B7·6 to B3·6 to B10·6



Cellular Beam 200kN

% 406×140×46UB

200×10thk End Plts

6FW

8N° M20 Gr 8.8 bolts

Beam B10-6

i) Bolts $\quad F_v = 70/4 = \underline{17.5kN} < 92kN$

Allowable bearing on end plt $= 10\times20\times550\times10^{-3}$

$= \underline{110kN} > 17.5kN$

ii) End Plt.

$A_v = 0.9\times150\times10 = 1350mm^2$

$A_{v_{net}} = 1350 - (2\times22\times10) = 910mm^2$

$P_v = 0.6\times355\times10^{-3}\times1350 = \underline{271kN} > 70/2\ kN$

$P_{v_{net}} = 0.5\times490\times10^{-3}\times910 = \underline{223kN} > 70/2\ kN$

$\therefore$ End Plt OK

CONN F6/2 cont.

Beam B10·6 cont

iii/ Check web behind End Plt for shear

$A_V = 0.9 \times 150 \times 6.3 = 850\ mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 850 = \underline{181\ kN} > 70\ kN$

$\therefore$ Web OK

iv/ End Plt. Weld 6FW

$P_V = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321\ kN} > 70\ kN$

$\therefore$ Weld OK

v/ Check beam for Notch

Beam OK as CONN. CA203

Beam B7-6

i/ Bolts $F_V = 200/8 = \underline{25\ kN} < 92\ kN$

Allowable bearing on end plt $= \underline{110\ kN} > 25\ kN$

Check bearing on 457UB web

Load per bolt $= 70/4 + 200/8 = \underline{42.5\ kN}$

Allowable bearing $= 9.1 \times 20 \times 550 \times 10^{-3}$

$= \underline{100\ kN} > 42.5\ kN$

$\therefore$ Bolts OK

CONN F6/2 cont.
Beam B7-6 cont.

ii/ **End Plt**

$A_v = 0.9 \times 360 \times 10 = 3240\ mm^2$

$A_{v_{net}} = 3240 - (4 \times 22 \times 10) = 2360\ mm^2$

$P_v = 0.6 \times 335 \times 10^{-3} \times 3240 = \underline{690\ kN} > 200/2\ kN$

$P_{v_{net}} = 0.6 \times 490 \times 10^{-3} \times 2360 = \underline{694\ kN} > 200/2\ kN$

$\therefore$ End Plt OK

iii/ **Check web behind End Plt for shear**

$A_v = 0.9 \times 360 \times 6.9 = 2235\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2235 = \underline{476\ kN} > 200\ kN$

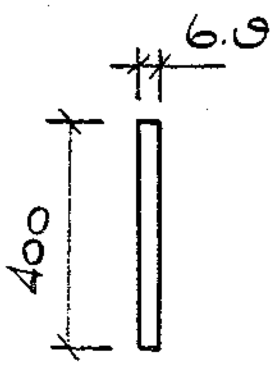
$\therefore$ Web OK

iv/ **End Plt. Weld** 6FW

$P_v = 2 \times 360 \times 4.2 \times 255 \times 10^{-3} = \underline{771\ kN} > 200\ kN$

$\therefore$ Weld OK

v/ **Check beam for Notch**



$Z = 6.9 \times 400^2/6 = 184\ 000\ mm^3$

$M = 200 \times 102 \times 10^{-3} = \underline{20.4\ kNm.}$

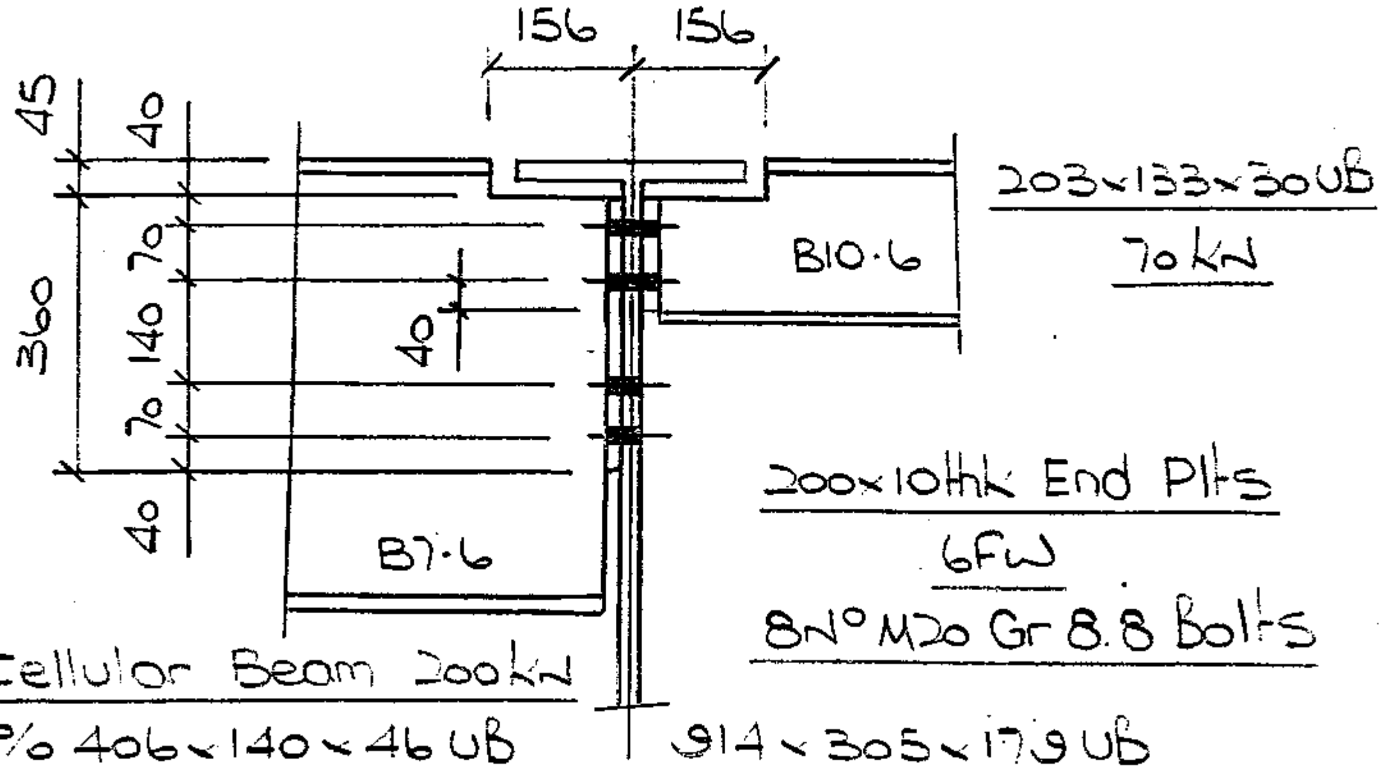
$P_b = 184\ 000 \times 355 \times 10^{-6}$

$= \underline{65\ kNm} > 20.4\ kNm$

$\therefore$ Section OK

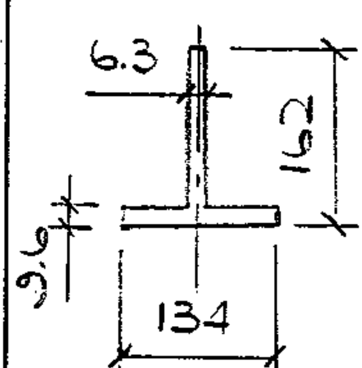
CONN F6/4

B7·6 to B9·6 to B10·6



Conn. design similar to Conn F6/2

Check 203UB for Notch



$A = 2247 mm^2$, $\bar{x} = 39 mm$, $I_{NA} = 547 cm^4$

Shear

$A_v = 0.9 \times 162 \times 6.3 = \underline{918 mm^2}$

$P_v = 0.6 \times 355 \times 10^{-3} \times 918 = \underline{196 kN} > 70 kN$

Bending

$M = 70 \times 156 \times 10^{-3} = \underline{10.9 kNm}$

$P_b = 547 \times 10^4 / (162 - 39) \times 355 \times 10^{-6} = \underline{15.8 kNm} > 10.9 kNm$

∴ Section OK

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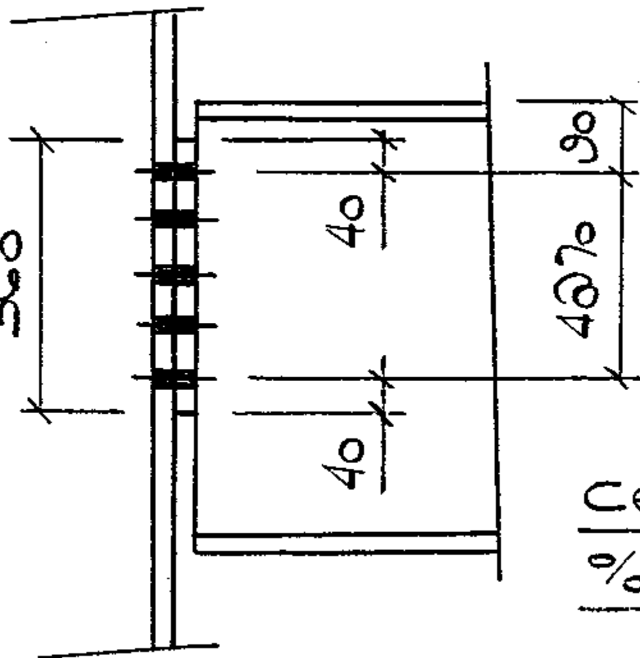
CONN F6/3

B7·6 to C2



Cellular Beam 200kN

% 406×140×46 UB

200×10thk End Plts

6FW

10N° M20 Gr 8.8 Bolts

Conn design similar to Conn F6/2

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SHT. No. C67 REV.

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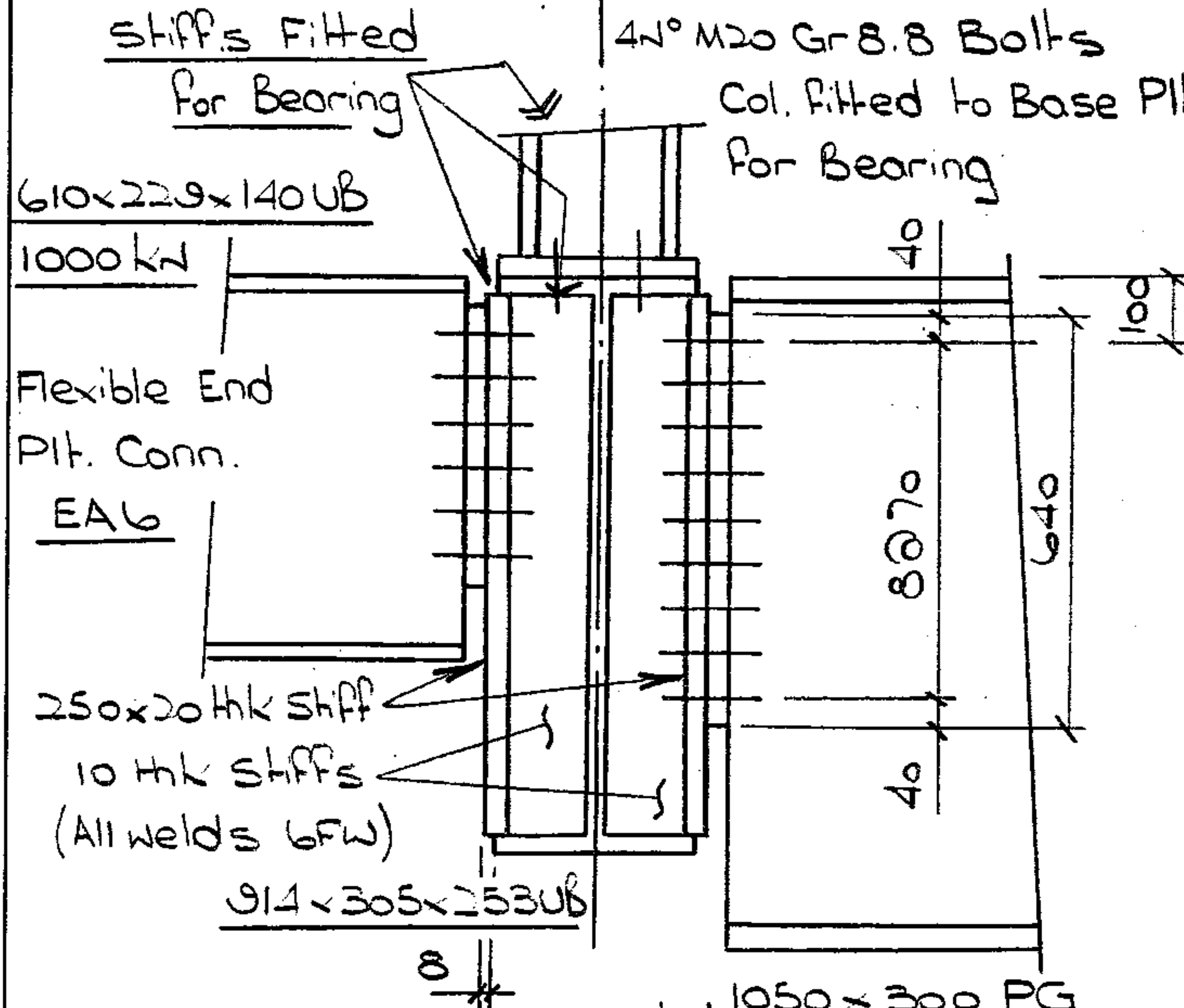
CONN F6/5

B8-6 to B9-6 to B6-6

254x89UC F_c = 2463kN

20thk Base Plt. - 6FW

4No M20 Gr8.8 Bolts



1250kN 1050x300 PG

(25web, 30flg)

200x10thk End Plt

6FW

18No M20 Gr8.8 Bolts

CONN F6/5 cont

1050 PG. Conn

i) Bolts $F_v = {}^{1250}/_{18} = \underline{69 kN} < 92 kN$

Allowable bearing on end plt $= 10 \times 20 \times 550 \times 10^{-3}$
$= \underline{110 kN} > 69 kN$

$\therefore$ Bolts OK

ii) End Plt

$A_v = 0.9 \times 640 \times 10 = 5760 mm^2$

$A_{vnet} = 5760 - (9 \times 22 \times 10) = 3780 mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 5760 = \underline{1226 kN} > {}^{1250}/_{2} kN$

$P_{vnet} = 0.5 \times 490 \times 10^{-3} \times 3780 = \underline{926 kN} > {}^{1250}/_{2} kN$

$\therefore$ End Plt. OK

iii) End Plt Weld 6fw

$P_v = 2 \times 640 \times 4.2 \times 255 \times 10^{-3}$
$= \underline{1370 kN} > 1250 kN$

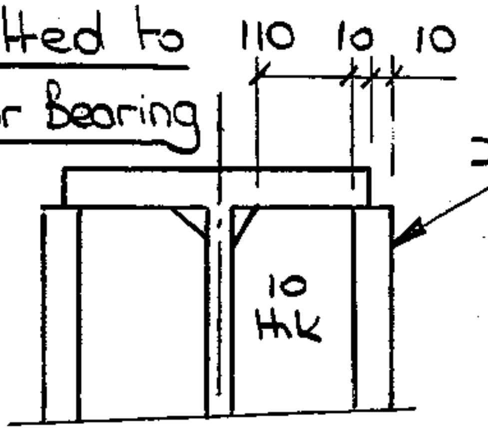
$\therefore$ Weld OK

CONN F6/5 cont.

Check stiffs for bearing

stiffs fitted to
Top Flg for Bearing

$F_c = \underline{2463\,kN}$



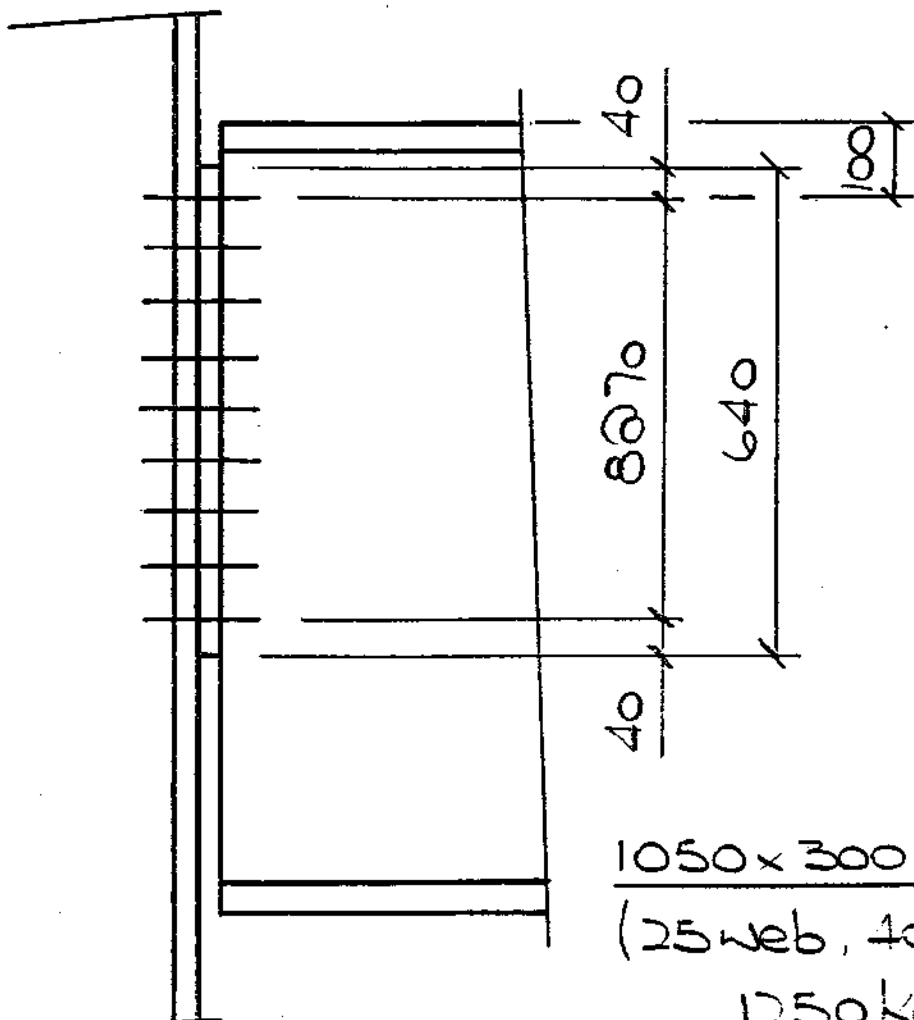
Bearing Area
$= 2 \times 250 \times 10 + 2 \times 110 \times 10$
$= 7200\,mm^2$

$P_b = 7200 \times 355 \times 10^{-3} = \underline{2556\,kN} > 2463\,kN$

$\therefore$ stiffs OK

CONN F6/8

B6-6 to C2



1050 x 300 PG
(25 web, 40 flg)
1250 kN

305 x 305 x 198 UC

200 x 10 thk End Plt
6 FW
18 N° M20 Gr 8.8 Bolts

Conn design as CONN F6/5

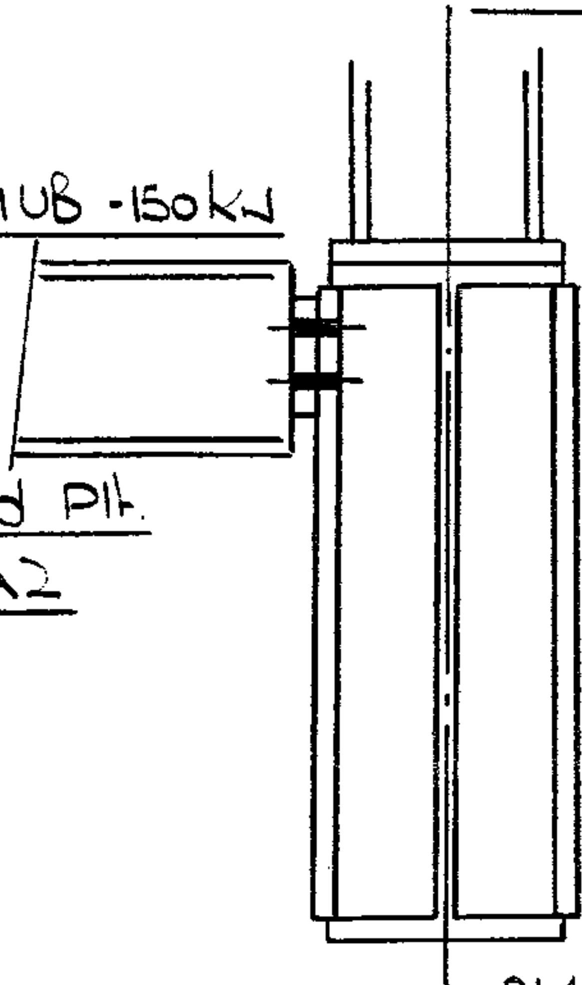
CONN F6/12

B1-6 to B9-6

254 x 89 UC

B1·6

254 x 146 x 31 UB - 150 kN



Flexible End Plt.

Conn. EA2

914 x 305 x 253 UB

Details as CONN F6/5 uno.

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CONN F6/6

254 x 89 UC

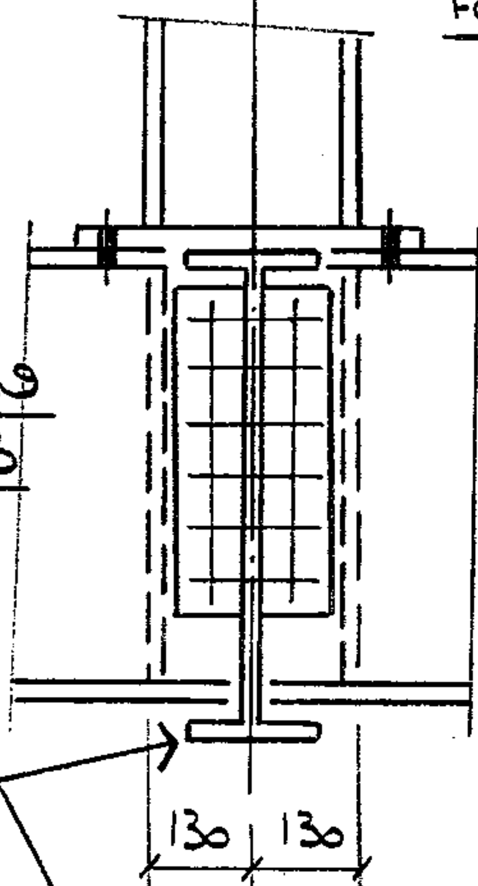
Fc = 2463 kN

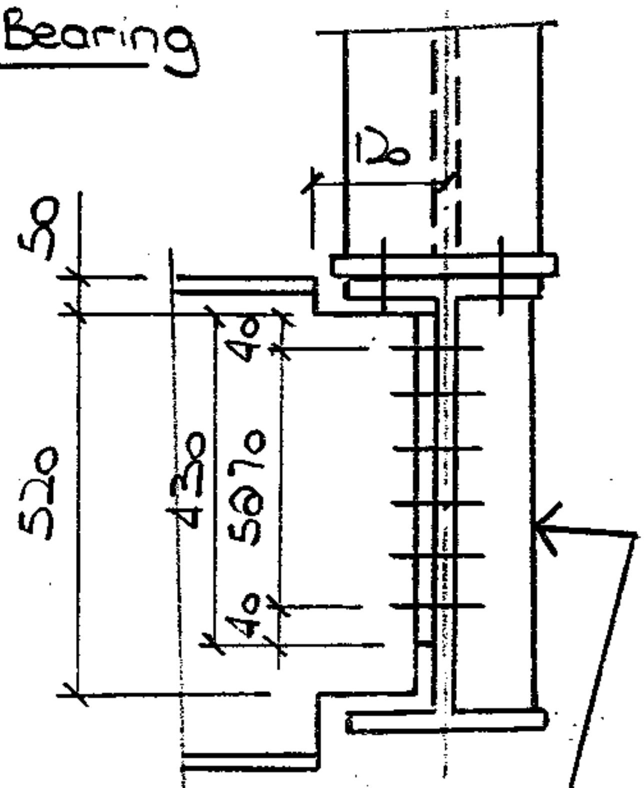
20thk Base Plt. - 6FW - 4N° M20 Gr8.8 Bolts

Col. fitted to Base Plt for Bearing

610x229 x140 UB

t = 13.1





Cellular Beam - 550kN

% 406x178x74UB
457 x 152 x 82UB

D=628

200x10thk End Plt

6FW

12N° M20 Gr 8.8 Bolts

2N° 100x20thk Stiffs

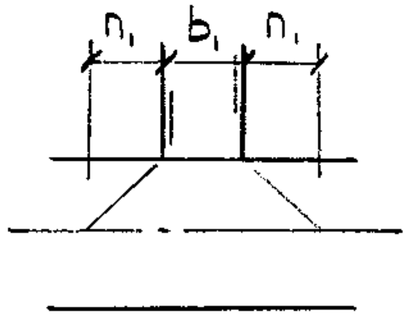
8mm FW

Stiffs fitted to Top Flg of 610UB for Bearing

Conn F6/6 cont.

Check 610UB for Web Buckling & Bearing

Web Buckling



610×229×140UB

$P_W = (b_1 + n_1)\, t .\, p_c$

$b_1 = 260 + 20 = 280\text{mm}$

$n_1 = {}^{616}/_{2} = 308\text{mm} \quad t = 13.1$

Slenderness of beam web

$\lambda = 2.5\,{}^{d}/_{t} = 2.5 \times {}^{547}/_{13.1} = 104$

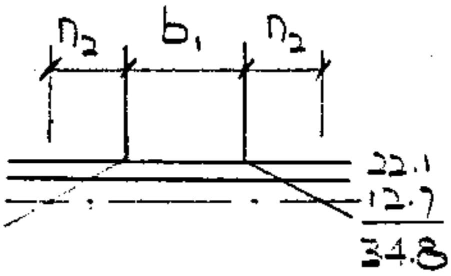
$p_y = 355\ \text{N/mm}^2$

From Table 27(c) $p_c = 133\ \text{N/mm}^2$

$\therefore P_W = (280 + 2 \times 308) \times 13.1 \times 133 \times 10^{-3}$

$= \underline{1561\text{kN}} < 2463\text{kN}$

Web Bearing



$P_B = (b_1 + n_2)\, t .\, p_{yw}$

$b_1 = 280\text{mm}, \; n_2 = 2.5 \times 34.8 = 87\text{mm}$

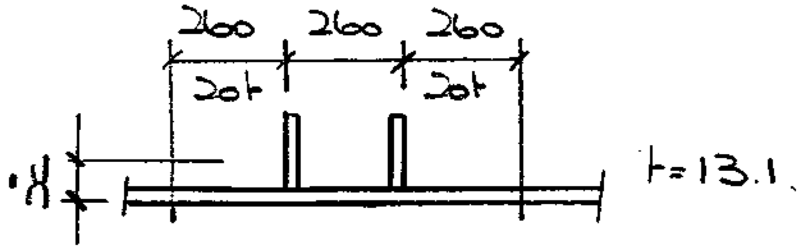
$t = 13.1\text{mm}, \quad p_{yw} = 355\ \text{N/mm}^2$

$\therefore P_B = (280 + 2 \times 87) \times 13.1 \times 355 \times 10^{-3}$

$= \underline{2111\text{kN}} < 2463\text{kN}$

CONN F6/6 cont.

Check 610UB + Stiffs for Web Buckling



100×20thk Stiff.s

Area = 780 + 13.1 + 2×100×20 = 14 218 mm^2

$$\bar{x} = \frac{13.1/2 \times 780 \times 13.1 + 2 \times 100 \times 20 \times 63.1}{14\,218} = 22\,\text{mm}$$

$$I_{NA} = 2 \times \frac{20 \times 100^3}{12} + 2 \times 20 \times 100 \times 41.1^2 + 780 \times 13.1 \times 15.45^2$$

$$= 12\,529\,235\,\text{mm}^4$$

$$r = \sqrt{\frac{12\,529\,235}{14\,218}} = 29.7\,\text{mm}$$

$$\lambda = \frac{0.7 \times 610}{29.7} = 14 \qquad p_y = 355\,\text{N/mm}^2$$

From Table 27 (c) $p_c = 355\,\text{N/mm}^2$

$$\therefore P_w = 14\,218 \times 355 \times 10^{-3}$$
$$= \underline{5047\,\text{kN}} > 2463\,\text{kN} \qquad \therefore \text{OK}$$

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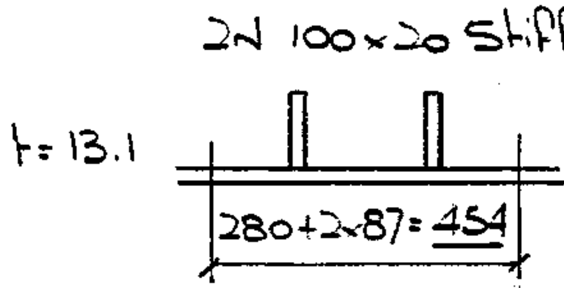
CONN F6/6 cont.

Check 610UB + stiffs for Bearing

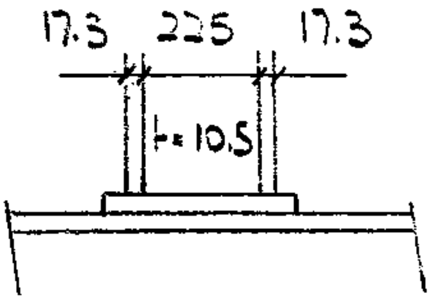


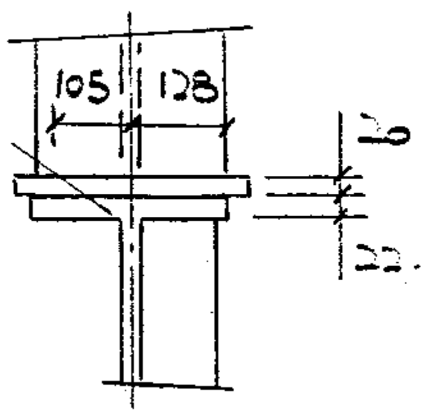
$Area = 454 \times 13.1 + 2 \times 100 \times 20 = 9947\ mm^2$

$\therefore P_B = 9947 \times 355 \times 10^{-3} = \underline{3531\ kN} > 2463\ kN$

$\therefore OK$

Check Col to Base for Bearing





$Area = 2 \times (105 + 128) \times 17.3 + 225 \times 10.5 = \underline{10\,424\ mm^2}$

$\therefore P_B = 10\,424 \times 355 \times 10^{-3} = \underline{3700\ kN} > 2463\ kN$

$\therefore OK$

CONN F6/6 cont.

Check Cellular Beam Conn.

i/ Bolts

$F_v = 550/12 = \underline{46\,kN} < 92\,kN$

Bearing on end plt / 610UB web $= 10 \times 20 \times 550 \times 10^{-3}$

$= \underline{110\,kN} > 46\,kN$

∴ Bolts OK

ii/ End Plt.

$A_v = 0.9 \times 430 \times 10 = 3870\,mm^2$

$A_{vnet} = 3870 - (6 \times 22 \times 10) = 2550\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 3870 = \underline{824\,kN} > 550/2\,kN$

$P_{vnet} = 0.5 \times 490 \times 10^{-3} \times 2550 = \underline{625\,kN} > 550/2\,kN$

∴ End Plt OK

iii/ End Plt. Weld - 6FW

$P_v = 2 \times 430 \times 4.2 \times 255 \times 10^{-3} = \underline{921\,kN} > 550\,kN$

∴ Weld OK

iv/ Check web behind End Plt for shear

$A_v = 0.9 \times 430 \times 9.7 = 3754\,mm^2$

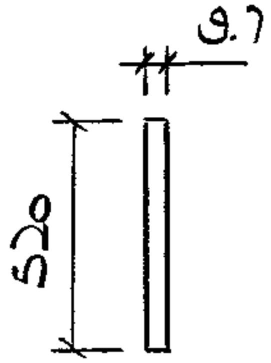
$P_v = 0.6 \times 355 \times 10^{-3} \times 3754 = \underline{800\,kN} > 550\,kN$

∴ Web OK

CONN F6/6 cont.

Cellular Beam Conn. cont.

v) Check Beam for Notch



$Z = 9.7 \times 520^2/6 = 437\,146\ mm^3$

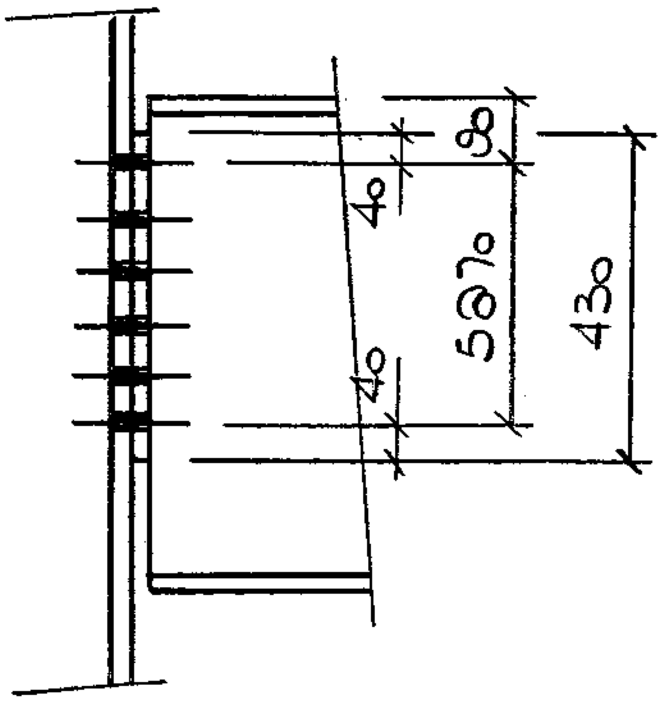
$M = 550 \times 120 \times 10^{-3} = \underline{66\ kNm}$

$P_b = 437\,146 \times 355 \times 10^{-6}$

$= \underline{155\ kNm} > 66\ kNm$

$\therefore$ Beam OK

CONN F6/7 B2.6 to C7

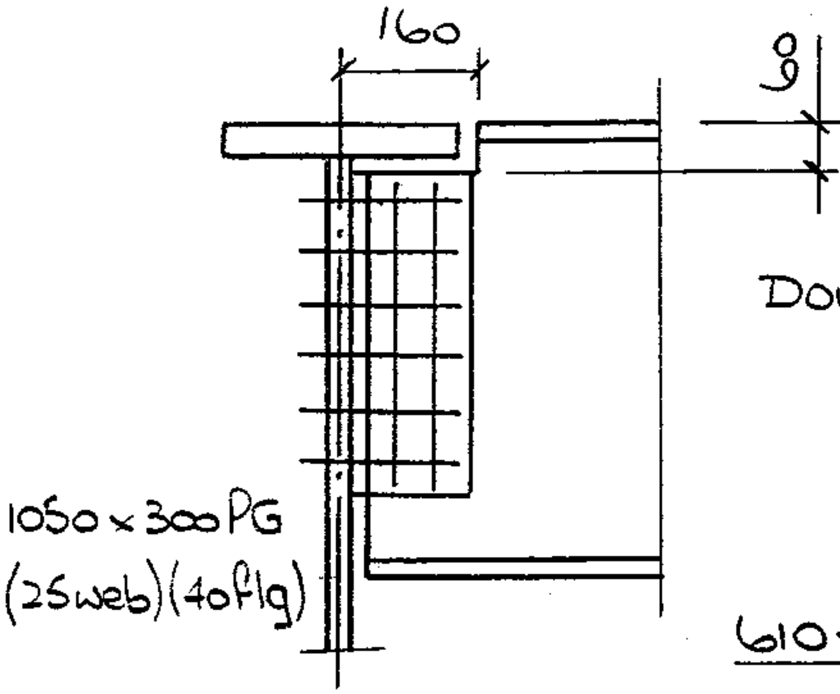


Cellular Beam
550kN
% 406×178×74UB
457×152×82UB
200×10thk End Plt
6FW
12N° M20 Gr 8.8
Bolts

254×89 UC

Conn Similar to CONN F6/6

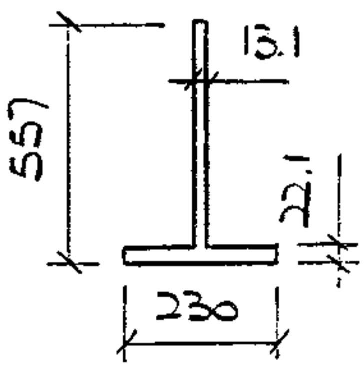
CONN F6/9
B8·6 to B6·6



Double Angle Conn.
CB6

610 x 229 x 140 UB
1000 kN

Check Beam for Notch



$A = 12\,090\ mm^2$ $\quad \bar{x} = 172\ mm$

$I_{NA} = 39\,557\ cm^4$

Shear

$A_V = 0.9 \times 557 \times 13.1 = 6\,567\ mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 6567 = \underline{1398\ kN} > 1000\ k$

Bending

$M = 1000 \times 160 \times 10^{-3} = \underline{160\ kNm}$

$P_b = 39\,557 \times 10^4 / (557 - 172) \times 355 \times 10^{-6}$

$= \underline{365\ kNm} > 160\ kNm \quad \therefore OK$

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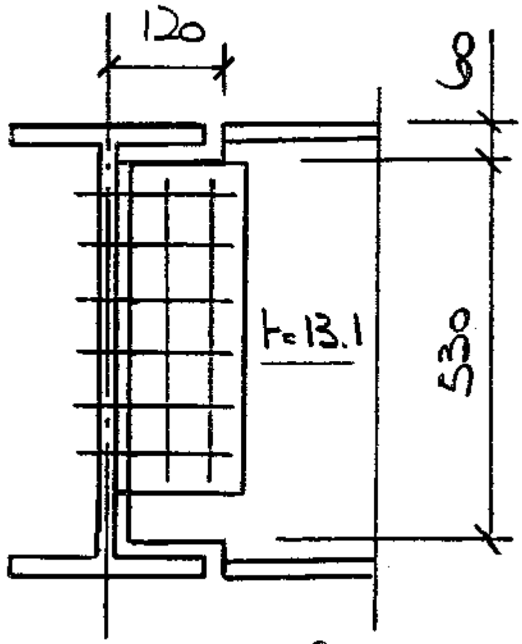
CONN F6/11

B8·6 to B8·6



610×229×140 UB

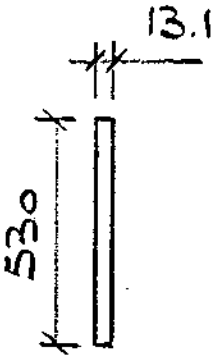
1000 kN

Double Angle Conn

CB6

610×229×140 UB

Check Beam for Notch



$Z = 13.1 \times 530^2 / 6 = 613\ 298\ mm^3$

Shear

$A_v = 0.9 \times 530 \times 13.1 = 6249\ mm^2$

$A_{v_{net}} = 6249 - (6 \times 22 \times 13.1) = 4520\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 6249 = \underline{1331\ kN} > 1000\ kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 4520 = \underline{1107\ kN} > 1000\ kN$

Bending

$M = 1000 \times 120 \times 10^{-3} = \underline{120\ kNm}$

$P_b = 613\ 298 \times 355 \times 10^{-6} = \underline{218\ kNm} > 120\ kNm$

$\therefore$ Section OK

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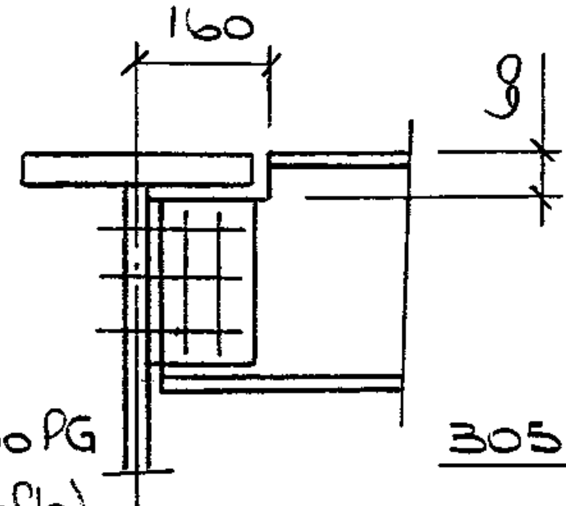
CONN F6/10

B4·6 to B6·6

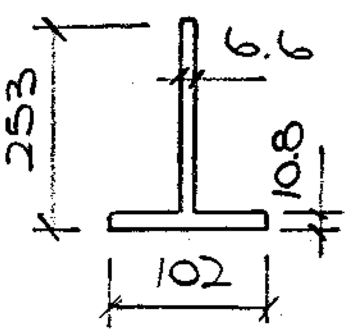


1050×300 PG
(25web)(40flg)

Double Angle Conn.
CB3

305×102×33UB
200kN

Check beam for Notch



$A = 2700\,mm^2 \quad \bar{x} = 80\,mm$

$I_{NA} = 1825\,cm^4$

Shear

$A_v = 0.9 \times 253 \times 6.6 = 1503\,mm^2$

$A_{vnet} = 1503 - (3 \times 22 \times 6.6) = 1067\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1503 = \underline{320\,kN} > 200\,kN$

$P_{vnet} = 0.5 \times 490 \times 10^{-3} \times 1067 = \underline{261\,kN} > 200\,kN$

Bending

$M = 200 \times 160 \times 10^{-3} = \underline{32\,kNm}$

$P_b = 1825 \times 10^4 / (253 - 80) \times 355 \times 10^{-6} = \underline{37\,kNm} > 32\,kNm$

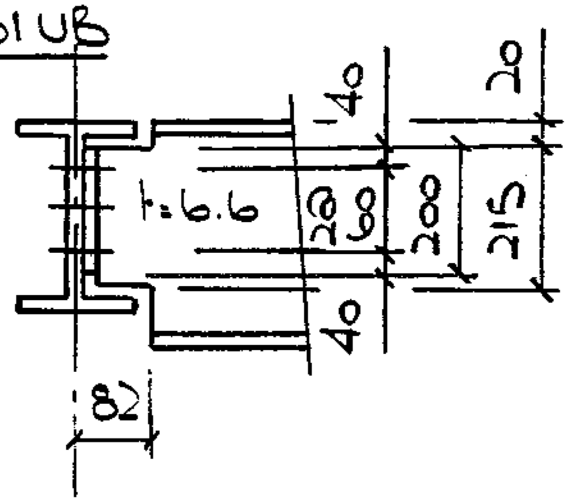
$\therefore$ Section ok

CONN F6/13

B4·6 to B1·6

254×146×31 UB

t=6.1



305×102×33 UB

200 kN

150×10thk End Plt

6FW

6N° M20 Gr 8.8 Bolts

i) Bolts

$F_v = 200/6 = \underline{33\,kN} < 92\,kN$

Allowable bearing on End Plt / 254 UB web $= 6.1 \times 20 \times 550 \times 10^{-3}$

$= \underline{67\,kN} > 33\,kN$

∴ Bolts ok

ii End Plt

$A_v = 0.9 \times 200 \times 10 = 1800\,mm^2$

$A_v = 1800 - (3 \times 22 \times 10) = 1140\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1800 = \underline{383\,kN} > 200/2\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 1140 = \underline{279\,kN} > 200/2\,kN$

∴ End Plt OK

CONN F6/13 cont.

iii/ Check Web behind End Plt for Shear

$A_v = 0.9 \times 215 \times 6.6 = 1277 \text{mm}^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1277 = \underline{272 \text{kN}} > 200 \text{kN}$

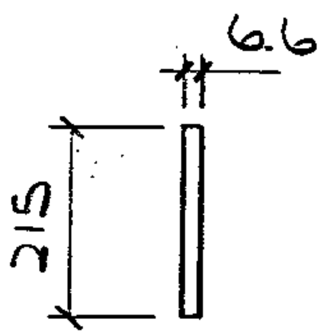
$\therefore$ Web OK

iv/ End Plt. Weld - 6FW

$P_v = 2 \times 200 \times 4.2 \times 255 \times 10^{-3} = \underline{428 \text{kN}} > 200 \text{kN}$

$\therefore$ Weld OK

v/ Check Beam for Notch



$Z = 6.6 \times 215^2/6 = 50\,848 \text{mm}^3$

$M = 200 \times 82 \times 10^{-3} = \underline{16.4 \text{kNm}}$

$P_b = 50\,848 \times 355 \times 10^{-6}$

$= \underline{18 \text{kNm}} > 16.4 \text{kNm}$

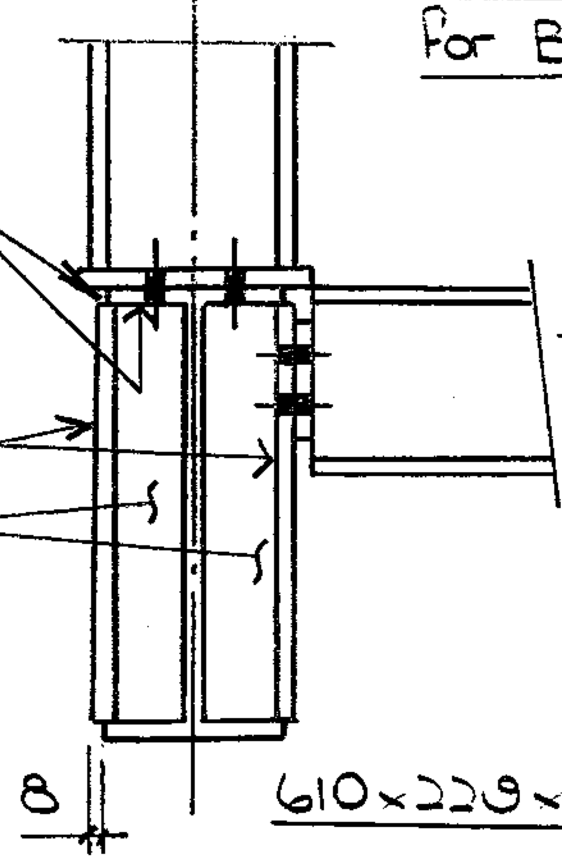
$\therefore$ Section OK

Conn F6/14
B1-6 to B8-6
+ Col. C8

254 × 89 UC $F_c = 2463$ kN
20thk Base Plt - 6FW
4N° M20 Gr. 8.8 Bolts
Col. fitted to Base Plt
For Bearing

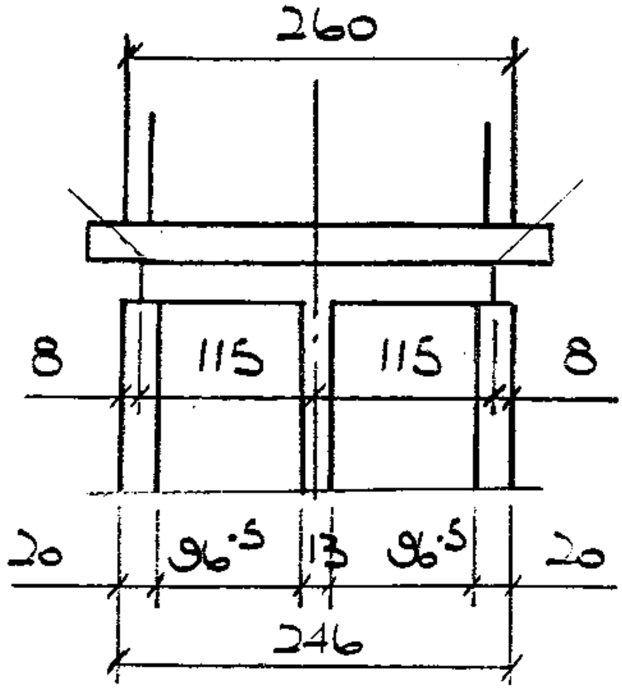
Stiff.s Fitted
for Bearing

250 × 20thk Stiff
15thk Stiff
All welds 10FW



254 × 146 × 31UB
150 kN
Flexible End Plt
Conn EA2

610 × 229 × 140 UB



CONN F6/14

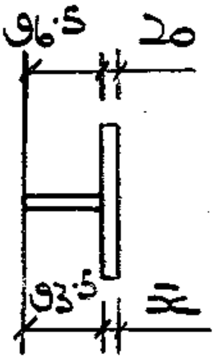
Check Bearing on Tee Stiff.

$$Area = 2 \times 250 \times 10 + (96.5 + 13 + 96.5) \times 15$$
$$= 8090\ mm^2$$

$$P_B = 8090 \times 355 \times 10^{-3} = \underline{2872\ kN} > 2463\ kN$$

$\therefore$ OK

Check Tee to Beam Web Weld - 10FW



$$A = 96.5 \times 15 + 250 \times 20 = 6448\ mm^2$$

$$\bar{x} = \frac{10 \times 250 \times 20 + 68.25 \times 96.5 \times 15}{6448} = 23\ mm$$

Ecc on weld = 93.5mm

Length of weld = 537mm

15thk web

250×20 flg

WELD

$$F_V = \frac{2463/2 \times 10^3}{2 \times 537 \times 7.0} = \underline{164\ N/mm^2}$$

$$M = 2463/2 \times 93.5 = 115\,145\ kNmm$$

$$Z = \frac{2 \times 7.0 \times 537^2}{6} = 672\,861\ mm^3$$

$$F_b = 115\,145 \times 10^3 / 672\,861 = \underline{171\ N/mm^2}$$

$$F_r = \sqrt{(164^2 + 171^2)} = \underline{237\ N/mm^2} < 255\ N/mm^2$$

$\therefore$ Weld OK

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SUBJECT Connection Design

SERIES SH 1 OF 1

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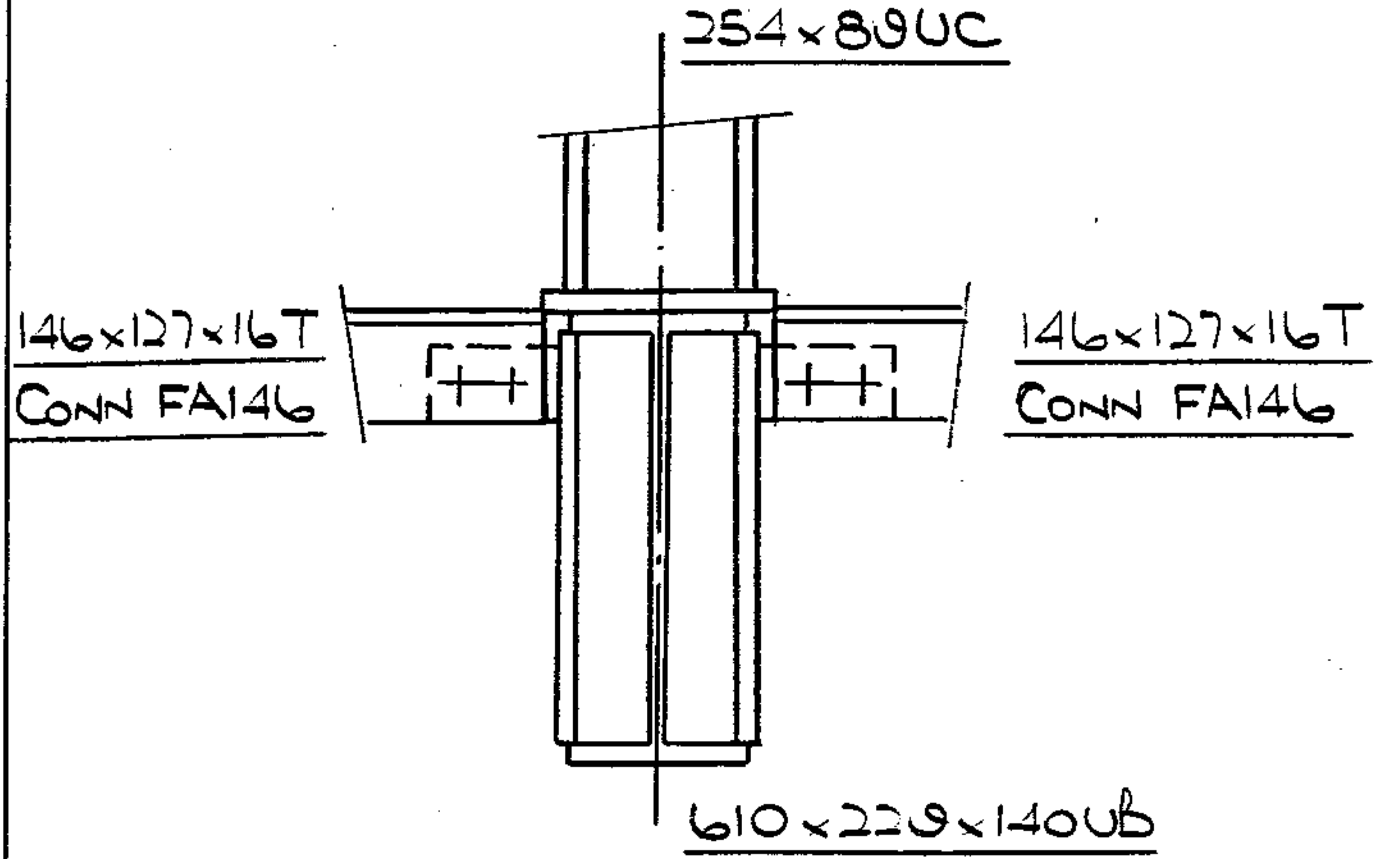
CONN F6/15

B11-6 to B8-6 + Col. C8



DETAILS AS CONN F6/14 UNO

Kvaerner Cleveland Bridge Ltd.

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SHT. No. C74 REV.

BY KM DATE Aug 98

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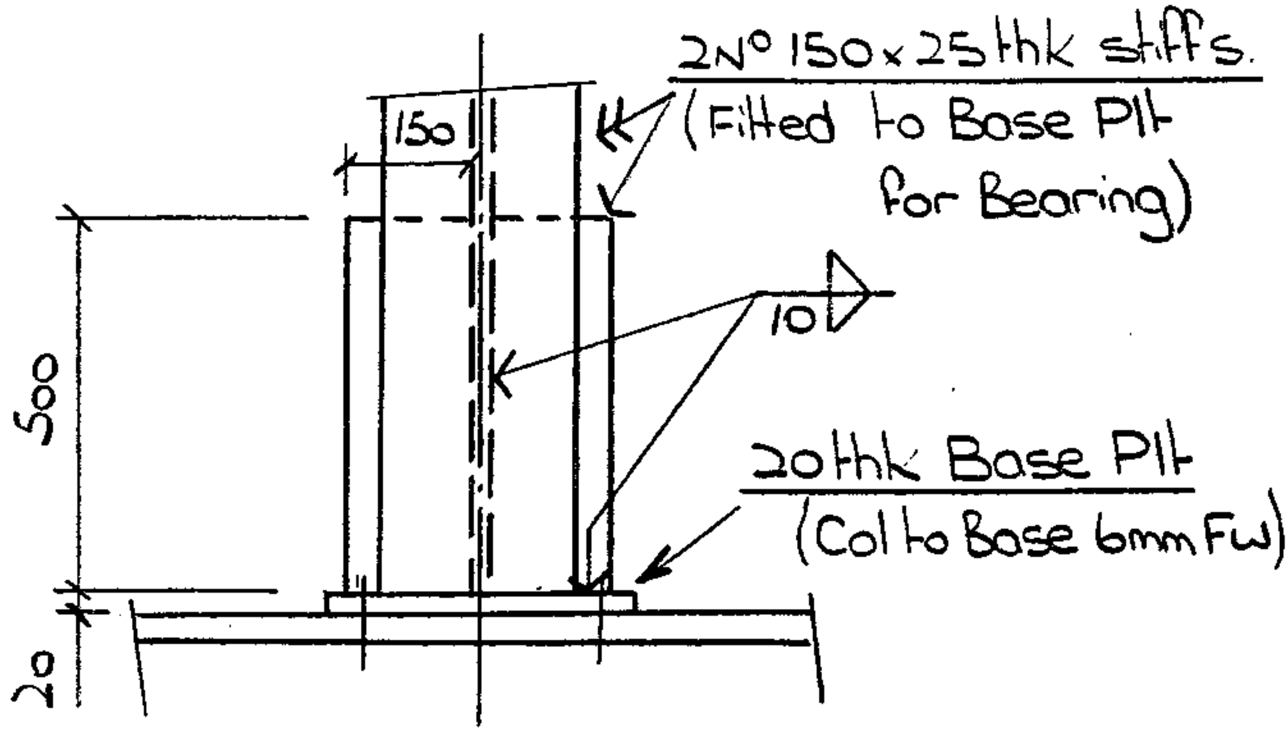
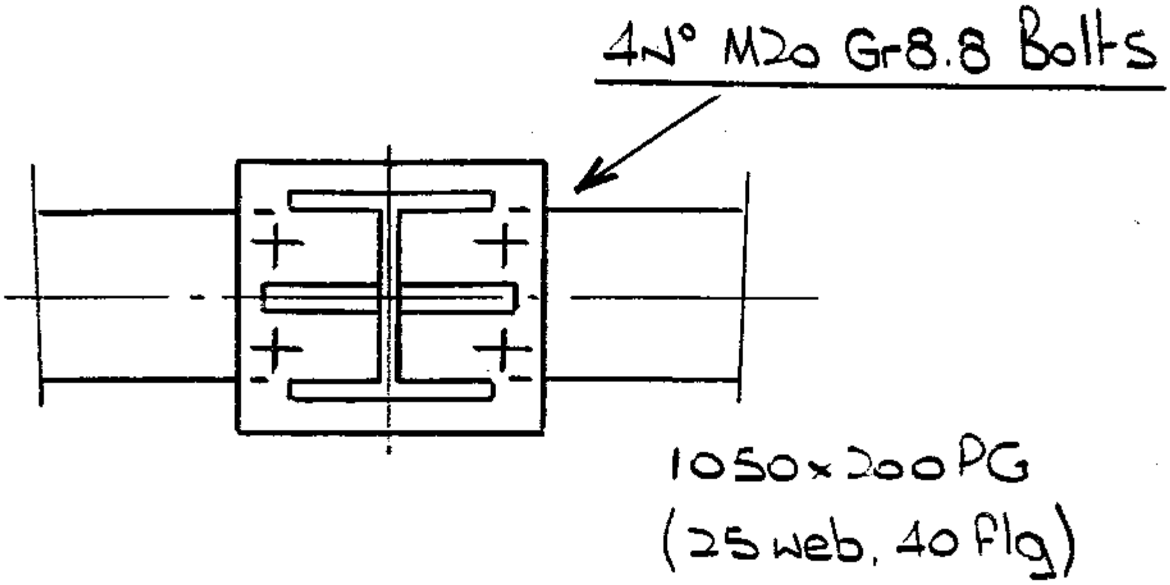
CN17

BASE B1

254 × 89 UC. Col. to 1050 × 200 PG

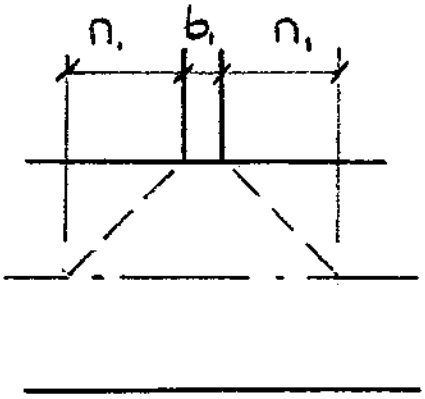




BASE B1 cont

Check 1050 PG for Web Buckling & Bearing

Web Buckling



1050 × 200 PG
(25 web, 40 flg)

$P_w = (b_1 + n_1)\, t .\, p_c$

$b_1 = 10.5 \quad n_1 = 1050/2 = 525 \quad t = 25$

Slenderness of beam web

$\lambda = 2.5\, d/t = 2.5 \times 970/25 = 97$

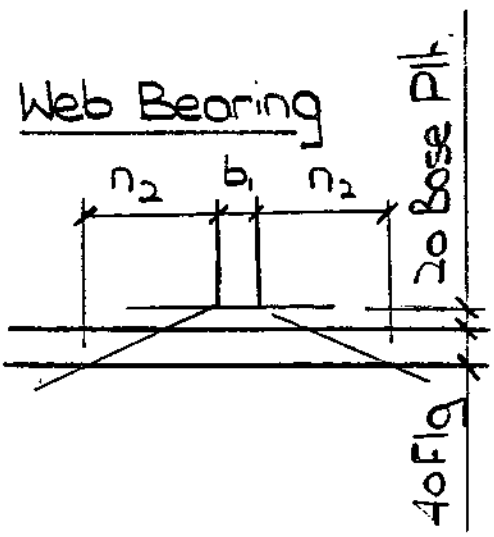
$p_y = 355\ N/mm^2$

From Table 27 (c) $p_c = 147\ N/mm^2$

$\therefore P_w = (10.5 + 2 \times 525) \times 25 \times 147 \times 10^{-3}$

$= \underline{3\,897\,kN} > 2463\,kN \quad \therefore OK$

Web Bearing



$P_B = (b_1 + n_2)\, t .\, p_{yw}$

$b_1 = 10.5\ ,\ n_2 = 2.5 \times (20 + 40) = 150$

$t = 25 \quad p_{yw} = 355\ N/mm^2$

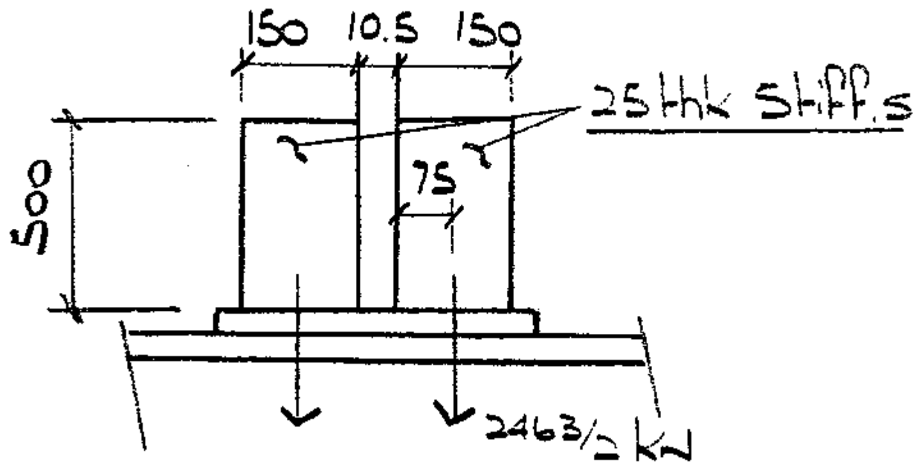
$\therefore P_B = (10.5 + 2 \times 150) \times 25 \times 355 \times 10^{-3}$

$= \underline{2756\,kN} > 2463\,kN \quad \therefore OK$

$\therefore$ No Stiffs Required

BASE B1 cont

Check Column Stiff's



Check Bearing

$P_B = (150 + 10 + 150) \times 25 \times 355 \times 10^{-3}$

$= \underline{2751 \text{ kN}} > 2463 \text{ kN}$

Check Stiff to Col web Weld 10mm FW

$F_V = \dfrac{2463/2 \times 10^3}{2 \times 7.0 \times 500} = \underline{176 \text{ N/mm}^2}$

$M = 2463/2 \times 75 = 92\,363 \text{ kNmm}$

$Z = \dfrac{2 \times 7.0 \times 500^2}{6} = 583\,333 \text{ mm}^3$

$F_b = 92\,363 \times 10^3 / 583\,333 = \underline{158 \text{ N/mm}^2}$

$F_r = \sqrt{(176^2 + 158^2)} = \underline{236 \text{ N/mm}^2} < 255 \text{ N/mm}^2$

$\therefore$ Weld OK

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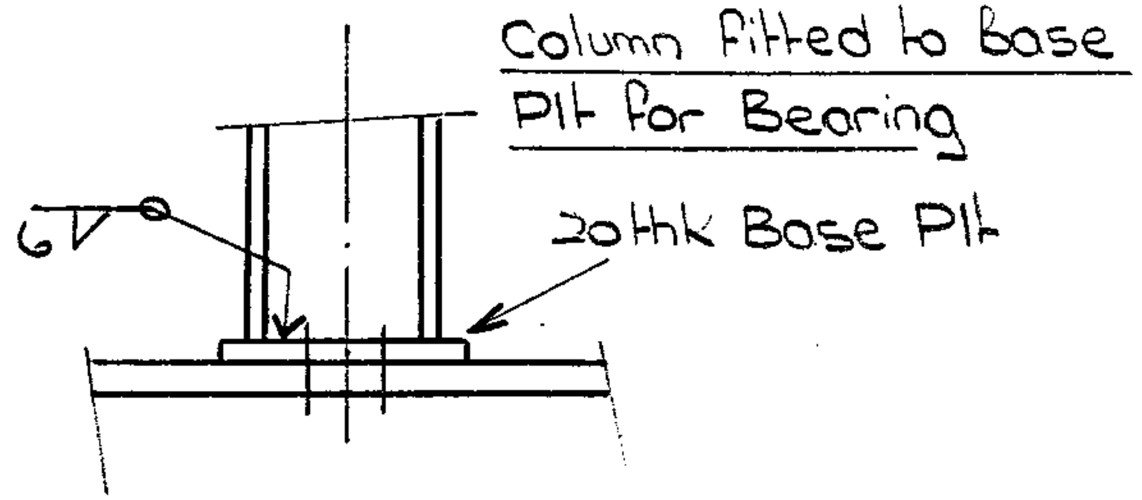
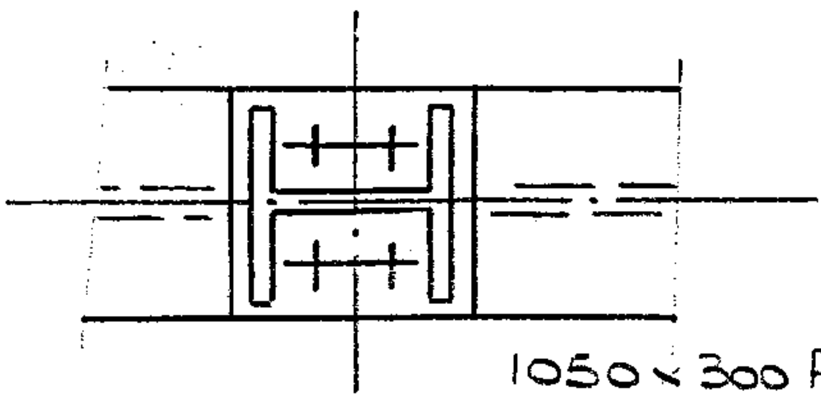
BASE B2

254 x 89 UC Col. to 1050 x 300 PG

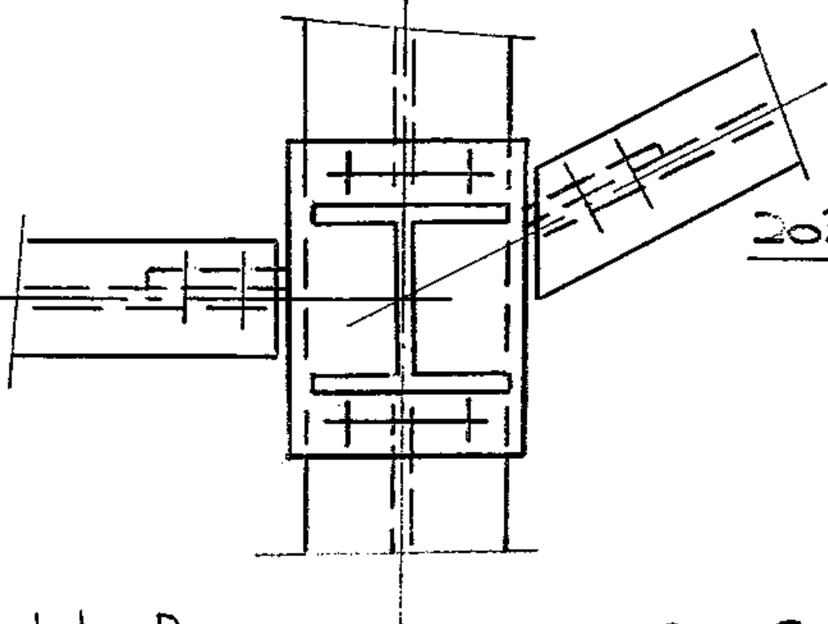


4 N° M20 Gr 8.8 Bolts



Conn of Col. C8 to B8-6 + Beams B12-6

610x229x140UB



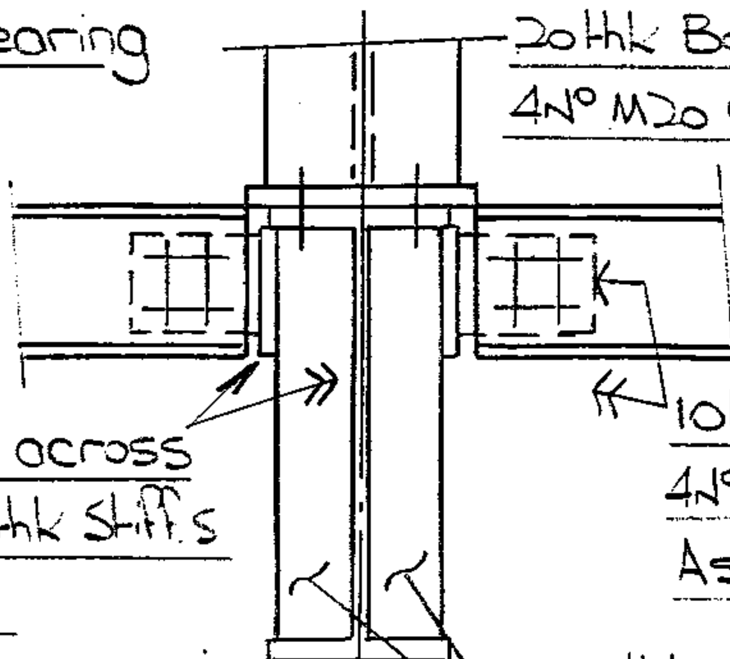
203x133 x30UB 70kN

203x133x30UB 70kN

Col. Fitted to Base Plt. for Bearing

254x89UC Fc=2463kN

20thk Base Plt. - 6FW
4N° M20 Gr8.8 Bolts



10thk Plts across toes of 20thk Stiff.s 6FW

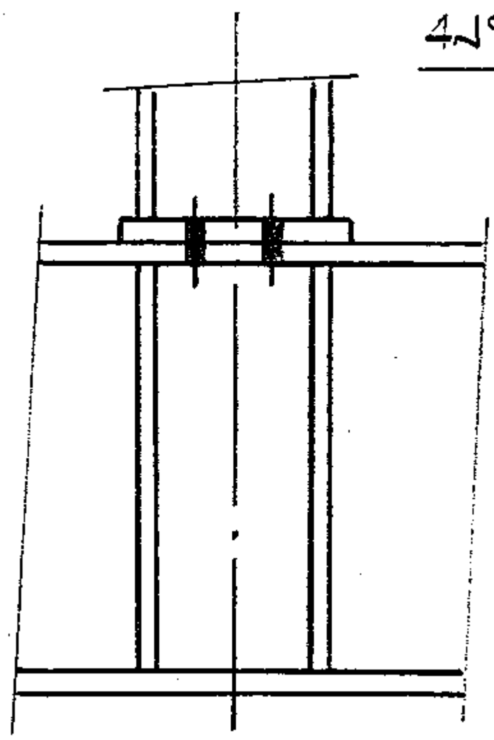
10thk Fin Plt - 6FW
4N° M20 Gr 8.8 Bolts
As Conn. C77

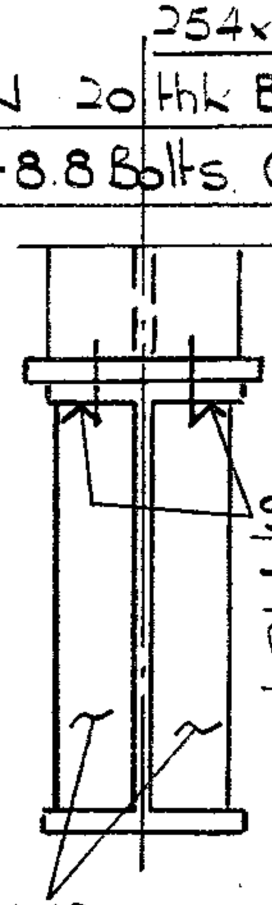
20thk Stiffs as Conn C74B
Fitted to Top Flg of UB for Bearing

BASE B4

254 x 89UC

F_c = 2463kN 20 thk Base Plt - 6FW
4N° M20 Gr 8.8 Bolts. Col. fitted to Base Plt for Bearing





Stiffs fitted to Top Flg of UB for Bearing

610 x 229 x 140 UB

4N° - 100 x 20 Stiffs - 6FW

i) Check Stiff to Beam Web Weld - 6FW

$$F_V = \frac{2463/4 \times 10^3}{2 \times 537 \times 4.2} = \underline{137\ N/mm^2}$$

$$M = 2463/4 \times 100/2 = 30\ 788\ kNmm$$

$$z = \frac{2 \times 4.2 \times 537^2}{6} = 403\ 716\ mm^3$$

$$F_b = 30\ 788 \times 10^3 / 403\ 716 = \underline{76\ N/mm^2}$$

$$F_r = \sqrt{(137^2 + 76^2)} = \underline{156\ N/mm^2} < 255\ N/mm^2$$

∴ Weld OK

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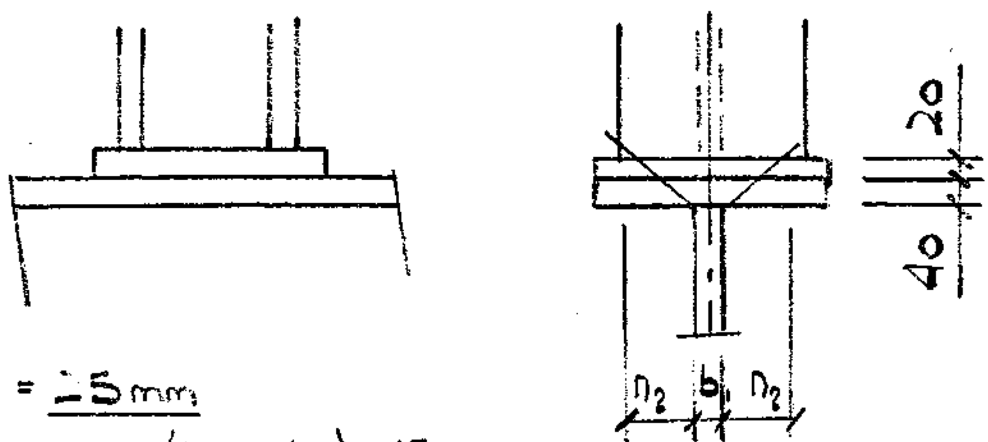
BASE B2 cont.

Check 1050PG for Web Buckling & Bearing

Web check similar to BASE B1 ∴ OK

Check Col Base for Bearing



b_1 = 25 mm

$n_2 = 2.5 \times (20+40) = 150$ mm

Effective flg width = 150 + 25 + 150 = 325 mm

∴ all of Col. area can be considered for Bearing

$A = 114\ cm^2$

$P_B = 114 \times 10^2 \times 355 \times 10^{-3} = 4047\ kN > 2463\ kN$

∴ OK

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SHT. No. C75 REV. A

BY KM DATE Sep98

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CN47

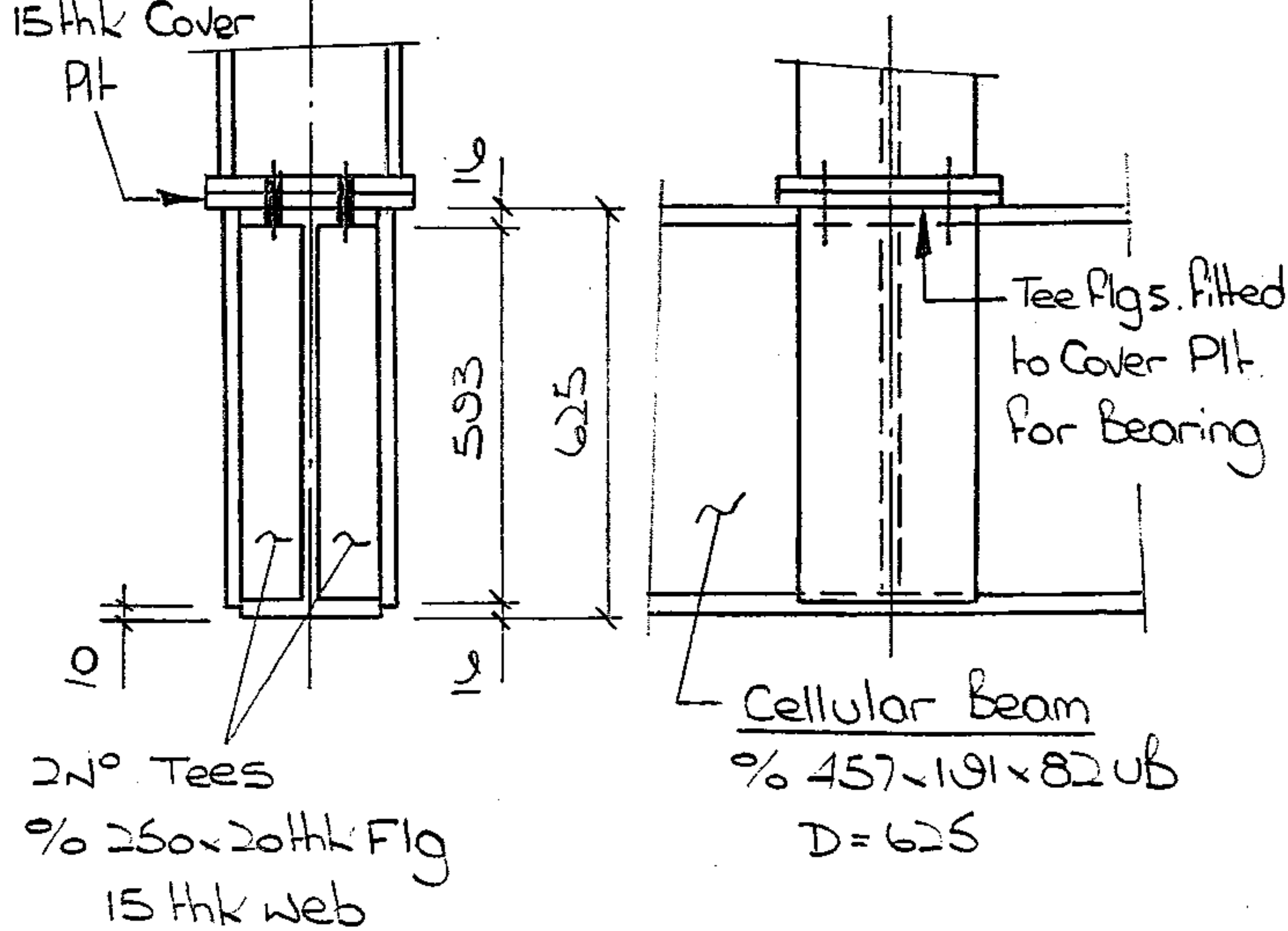
Rev A - Redrawn. Beam Section revised

254 x 89 UC Col. to B2.6 (Cellular Beam)

254 x 89 UC F_c = 2463 kN

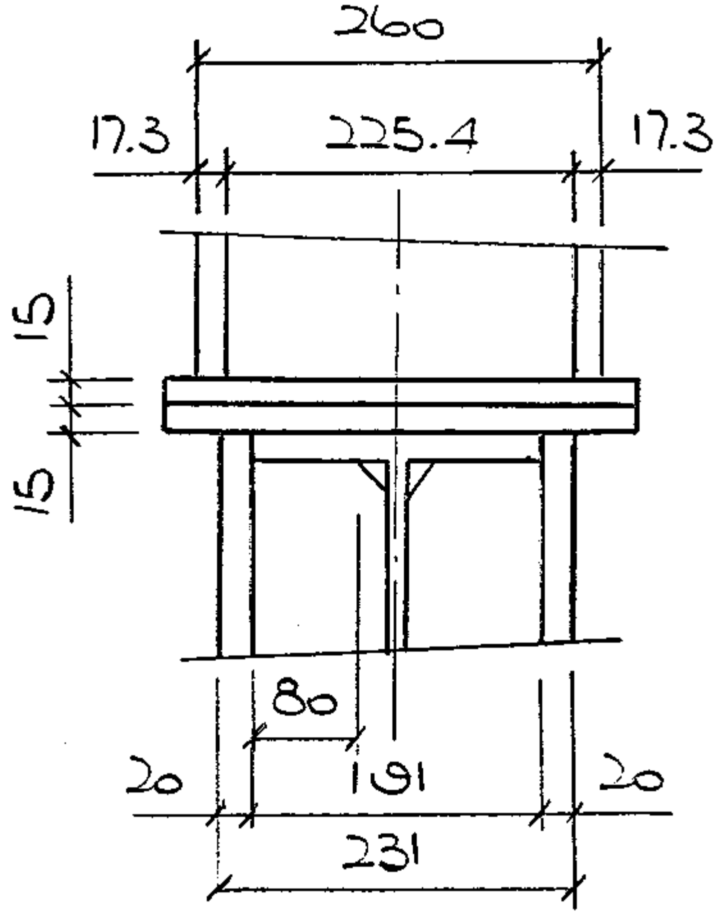
15 thk Base Plt. - 6FW - 4 N° M20 Gr 8.8 Bolts

Col. fitted to Base Plt. for Bearing



All welds 8FW uno

Col C8 to Beam B2-6 cont.



Check bearing on Col.

Consider Flgs only

Area = 256 × 17.3 × 2 = 8 858 mm^2

P_B = 8858 × 355 × 10^{-3} = 3145 kN > 2463 kN ∴ OK

Check bearing on Tee Stiff.s

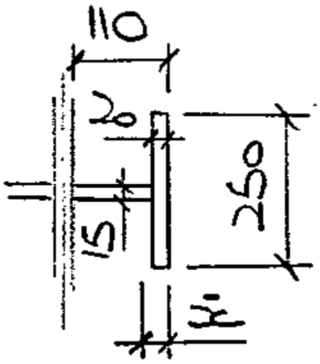
Consider Flgs only

Area = 250 × 20 × 2 = 10 000 mm^2 > 8858 mm^2

∴ OK

Col. C8 to Beam B2-6 cont.

Check Tee to Beam Weld - 8FW



$A = 90 \times 15 + 250 \times 20 = 6350\ mm^2$

$\bar{x} = \dfrac{10 \times 250 \times 20 + 65 \times 90 \times 15}{6350} = \underline{22\ mm}$

Weld 8FW

F_V (stiff - web) $= \dfrac{2463/2 \times 10^3}{2 \times 560 \times 5.6} = \underline{196\ N/mm^2} < 255\ N/mm^2$

F_H (stiff - flg) $= \dfrac{2463/2 \times (110 - 22)}{593} = 183\ kN$

$= \dfrac{183 \times 10^3}{2 \times 80 \times 5.6} = \underline{204\ N/mm^2} < 255\ N/mm^2$

$\therefore$ Weld OK

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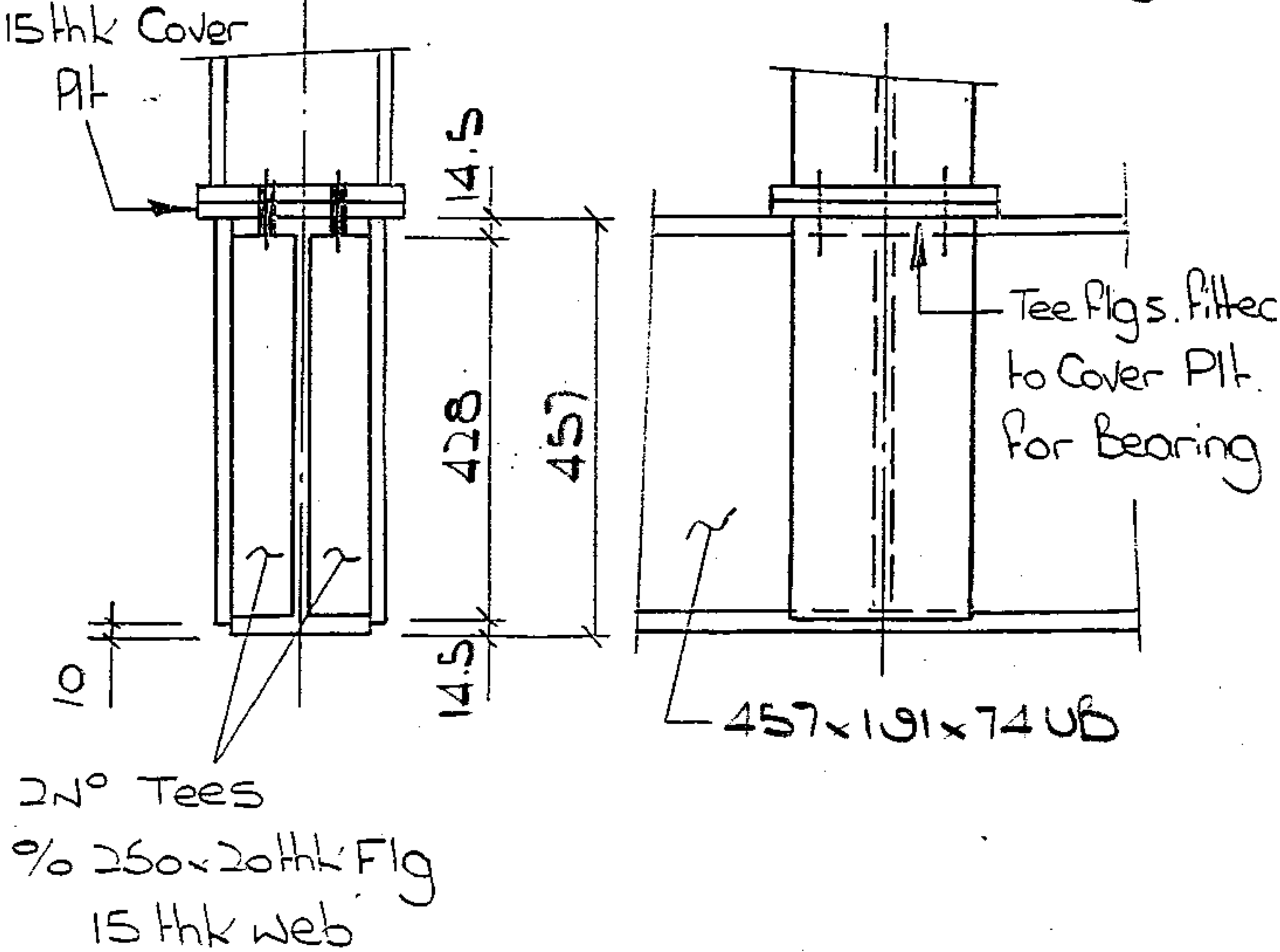
254 x 89 UC Col. to 457 x 191 x 74 UB

254 x 89 UC F_c = 2463 kN

15 thk Base Plt. - 6FW - 4No M20 Gr 8.8 Bolts

Col. fitted to Base Plt. for Bearing



All welds 10FW uno

Details as C75 uno

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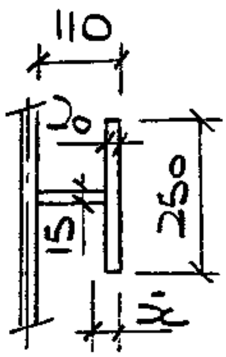
Col. C8 to 457 UB cont.

Check Tee to Beam Weld - 10FW



$A = 90 \times 15 + 250 \times 20 = 6350 mm^2$

$\bar{x} = \dfrac{10 \times 250 \times 20 + 65 \times 90 \times 15}{6350}$

$= \underline{22 mm}$

Weld 10FW

F_V (stiff - web) $= \dfrac{2463/2 \times 10^3}{2 \times 400 \times 7.0} = \underline{220 N/mm^2} < 255 N/mm^2$

F_H (stiff - flg) $= \dfrac{2463/2 \times (110-22)}{428} = 253 kN$

$= \dfrac{253 \times 10^3}{2 \times 80 \times 7.0} = \underline{226 N/mm^2} < 255 N/mm^2$

$\therefore$ Weld OK

Conn of Col. C12 to Beam B8-6

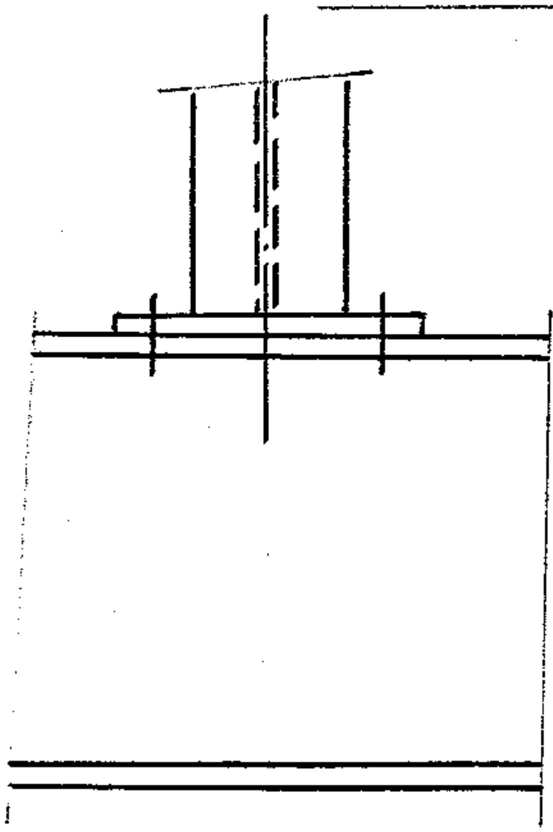
C12 203×203×52UC $t = 8.0$

$F_c = 100kN$

15thk Base Plt

6FW

4N° M20 Gr 8.8 Bolts



B8-6

610×229×140UB

$T = 22.1$

$r = 12.7$

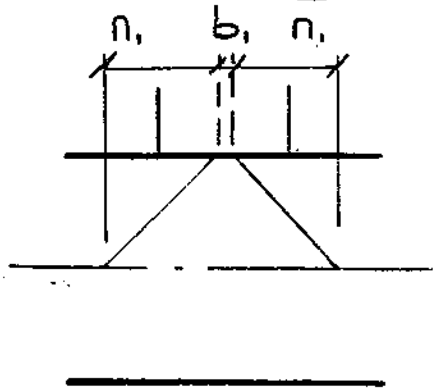
$t = 13.1$

$D = 616$

$d = 547$

Check 610UB for Web Buckling & Bearing

Web Buckling



$P_w = (b_1 + n_1)\,t\,p_c$

$b_1 = 8.0$, $n_1 = {}^{616}/_2 = 308$, $t = 13.1$

Slenderness of beam web

$\lambda = 2.5\,{}^{d}/_{t} = 2.5 \times {}^{547}/_{13.1} = 104$

$p_y = 355\,N/mm^2$

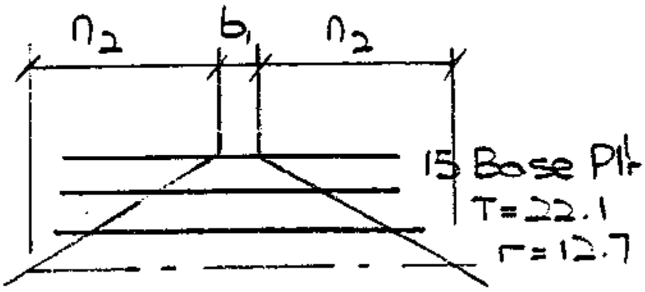
From Table 27(c) $p_c = 133\,N/mm^2$

$\therefore P_w = (8.0 + 2 \times 308) \times 13.1 \times 133 \times 10^{-3}$

$= \underline{1087\,kN} > 100kN \quad \therefore OK$

Conn of C12 to B8·6 cont.

Web Bearing



$P_B = (b_1 + n_2)t \cdot p_{yw}$

$b_1 = 8.0,\ n_2 = 2.5(15 + 22.1 + 12.7)$

$= 124.5$

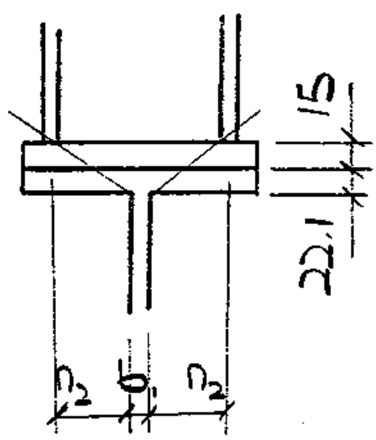
$t = 13.1 \quad p_{yw} = 355\ N/mm^2$

$\therefore P_B = (8 + 2 \times 124.5) \times 13.1 \times 355 \times 10^{-3}$

$= \underline{1195\ kN} > 100\ kN$

$\therefore$ OK

Check Col web for Bearing



$P_B = (b_1 + n_2)t \cdot p_{yw}$

$b_1 = 13.1 \quad n_2 = 2.5(22.1 + 15)$

$= 93\ mm$

$t = 8.0, \quad p_{yw} = 355\ N/mm^2$

$\therefore P_B = (13.1 + 2 \times 93) \times 8.0 \times 355 \times 10^{-3}$

$= \underline{565\ kN} > 100\ kN$

$\therefore$ OK

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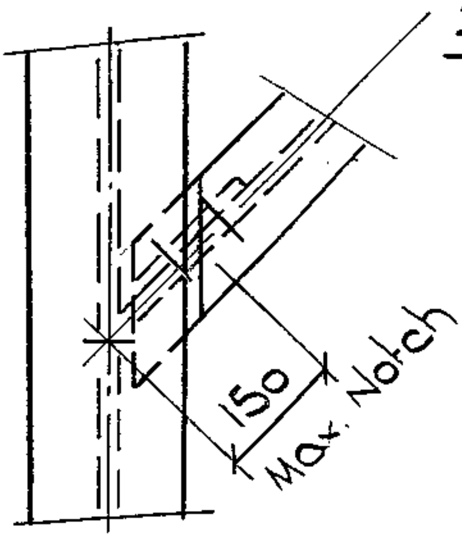
Conn of Beam B12-6 to B3-6 / B4-6

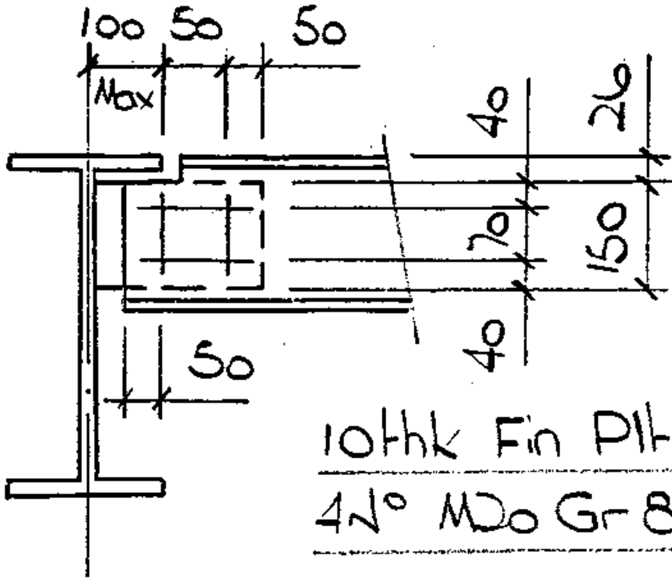
203x133x30 UB

70kN



457x191x74 UB

305x102x33 UB



10thk Fin Plt - 6FW

4No M20 Gr 8.8 Bolts

Beam B12-6 to B3-6/B4-6 cont.

i/ Bolts

$$Z = \frac{4 \times (35)^2}{35} = 140\,mm$$

$F_v = 70/4 = 17.5\,kN$

$F_h = 125 \times 70/140 = 62.5\,kN$

$F_r = \sqrt{(17.5^2 + 62.5^2)} = \underline{65\,kN} < 92\,kN$

Allowable bearing on fin plt / beam web $= 6.3 \times 20 \times 550 \times 10^{-3}$

$= \underline{69\,kN} > 65\,kN$ $\therefore$ Bolts OK

ii/ Fin Plt

a. Shear

$A_v = 0.9 \times 150 \times 10 = 1350\,mm^2$

$A_{v_{net}} = 1350 - (2 \times 22 \times 10) = 910\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1350 = \underline{288\,kN} > 70\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 910 = \underline{223\,kN} > 70\,kN$

b. Bending

$Z = 10 \times 150^2/6 = 37\,500\,mm^3$

$M = 125 \times 70 \times 10^{-3} = \underline{8.75\,kNm}$

$P_b = 37\,500 \times 355 \times 10^{-6} = \underline{13.3\,kNm} > 8.75\,kNm$

$\therefore$ Fin Plt. OK

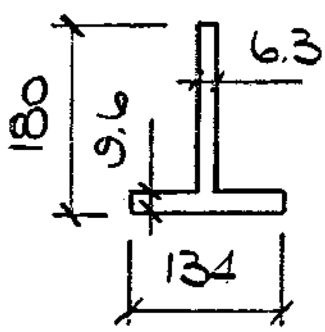
Beam B12-6 to B3-6 / B4-6 cont.

iii/ Fin Plt. Weld - 6FW

$P_v = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321 kN} > 70 kN$

$\therefore$ Weld OK

iv/ Check Beam for Notch



$A = 2360 mm^2$

$\bar{x} = 46 mm$

$I_{NA} = 734 cm^3$

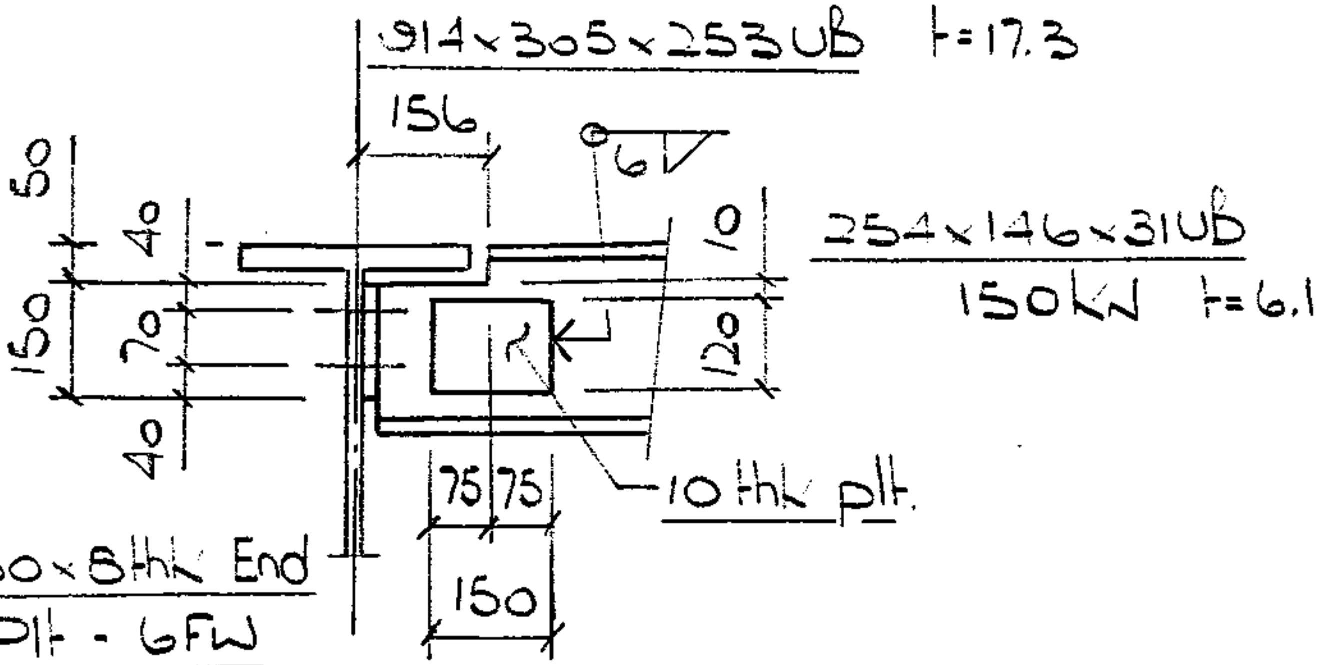
$M = 70 \times 150 \times 10^{-3} = \underline{10.5 kNm}$

$P_b = \frac{734 \times 10^4}{(180 - 46)} \times 355 \times 10^{-6}$

$= \underline{19 kNm} > 10.5 kNm$

$\therefore$ Beam OK

Conn of Beam B1-6 to B9-6



4N° M20 Gr8.8 Bolts

i) Bolts

$F_v = 150/4 = \underline{37.5kN} < 92kN$

Allowable bearing on End Plt / 914UB web $= 8 \times 20 \times 550 \times 10^{-3}$

$= \underline{88kN} > 37.5kN$

∴ Bolts OK

ii) End Plt.

$A_v = 0.9 \times 150 \times 8 = 1080\ mm^2$

$A_{vnet} = 1080 - (2 \times 22 \times 8) = 728\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1080 = \underline{230kN} > 150/2\ kN$

$P_{vnet} = 0.5 \times 490 \times 10^{-3} \times 728 = \underline{178kN} > 150/2\ kN$

∴ End Plt. OK

Beam B1-6 to B9-6 cont.

iii/ Check web behind End Plt. for Shear

$A_v = 0.9 \times 150 \times 6.1 = 824\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 824 = \underline{176\,kN} > 150\,kN$

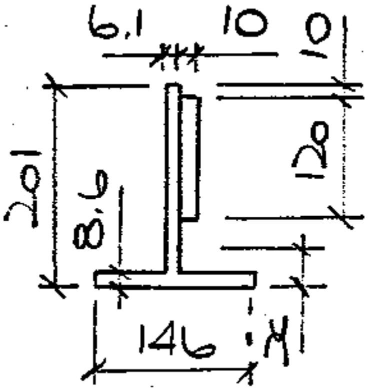
$\therefore$ Web OK

iv/ End Plt. Weld - 6FW

$P_v = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321\,kN} > 150\,kN$

$\therefore$ Weld OK

v/ Check Beam for Notch



$A = 192.4 \times 6.1 + 120 \times 10 + 146 \times 8.6$

$= 1174 + 1200 + 1256 = \underline{3630\,mm^2}$

$\bar{x} = \dfrac{1174 \times 104.8 + 1200 \times 131 + 1256 \times 4.3}{3630}$

$= \underline{79\,mm}$

$I_{NA} = 6.1 \times 201^3/12 = 4\,127\,972$

$10 \times 120^3/12 = 1\,440\,000$

$1174 \times 25.8^2 = 781\,146$

$1200 \times 52^2 = 3\,244\,800$

$1256 \times 74.7^2 = 7\,008\,593$

$16\,602\,511\,mm^4$

$M = 150 \times 156 \times 10^{-3} = \underline{23.4\,kNm}$

$P_b = {}^{16\,602\,511}/(201 - 79) \times 355 \times 10^{-6} = \underline{48.3\,kNm} > 23.4$

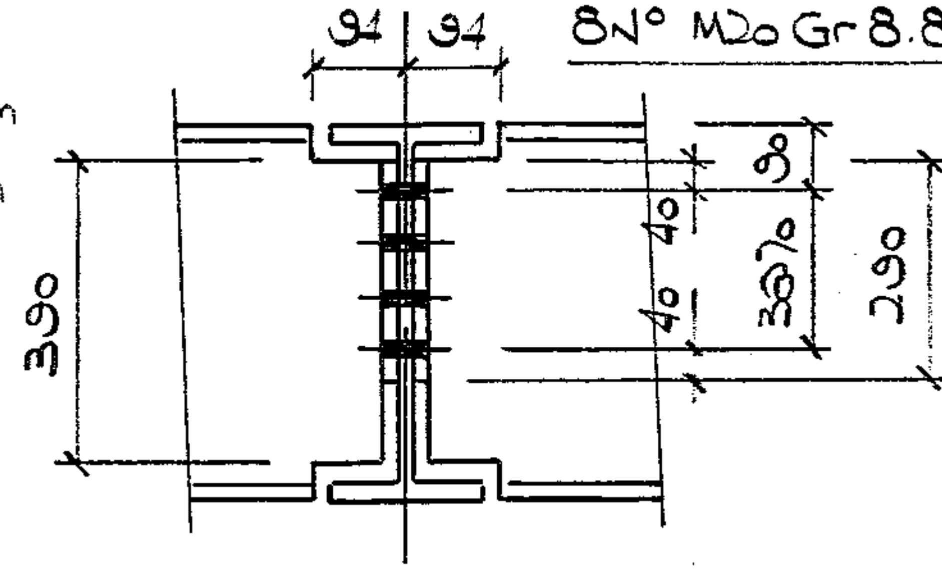
$\therefore$ Beam OK

CONN. R1

Cellular Beams B1·R, B4·R & B9·R — Beam to Beam

150×8 End Plt - 6FW
8N° M20 Gr 8.8 Bolts

D = 488mm
t = 8.0mm



Cellular Beam B1·R % 356×171×57UB 275kN
B4·R % 356×171×57UB 250kN
B9·R % 356×171×57UB 100kN

i/ Bolts

$F_v = 275/8 = \underline{34\ kN} < 92kN$

Allowable bearing on end plt $= 8 \times 20 \times 550 \times 10^{-3}$
$= \underline{88kN} > 34kN$

Check supporting web t = 8.0

Load per bolt $= 2 \times 275/8 = \underline{69kN}$

Allowable bearing $= 8 \times 20 \times 550 \times 10^{-3}$
$= \underline{88kN} > 69kN$

∴ Bolts ok

CONN. R1 cont.

ii/ End Plt.

$A_v = 0.9 \times 290 \times 8 = 2088\,mm^2$

$A_{V_{net}} = 2088 - (4 \times 22 \times 8) = 1384\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2088 = \underline{445\,kN} > {}^{275}/_{2}\,kN$

$P_{V_{net}} = 0.5 \times 490 \times 10^{-3} \times 1384 = \underline{339\,kN} > {}^{275}/_{2}\,kN$

$\therefore$ End Plt OK

iii/ End Plt. Weld - 6FW

$P_v = 2 \times 290 \times 4.2 \times 255 \times 10^{-3} = \underline{621\,kN} > 275\,kN$

$\therefore$ Weld OK

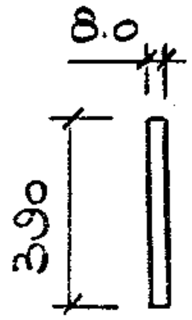
iv/ Check web behind End Plt for Shear

$A_v = 0.9 \times 290 \times 8.0 = 2088\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2088 = \underline{445\,kN} > 275\,kN$

$\therefore$ Web OK

v/ Check Beam for Notch



$Z = {}^{8.0 \times 390^2}/_{6} = 202\,800\,mm^2$

$M = 275 \times 94 \times 10^{-3} = \underline{26\,kNm}$

$P_b = 202\,800 \times 355 \times 10^{-6}$

$= \underline{72\,kNm} > 26\,kNm$

$\therefore$ Beam OK

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	BY KM DATE Sep 98
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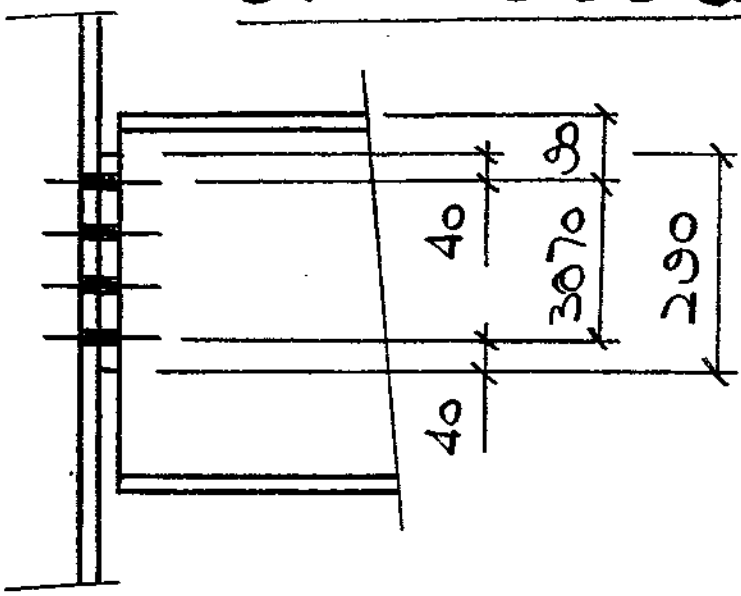
CONN R2

Cellular Beams B1-R, B4-R & B9-R   Beam to Column

150 × 8 End Plt - 6FW

8N° M20 Gr8.8 Bolts



Cellular Beam	B1·R	%356×171×57UB	275kN
	B4·R	%356×171×57UB	250kN
	B9·R	%356×171×57UB	100kN

Conn. as CONN R1

CONN R3

Cellular Beams B2-R

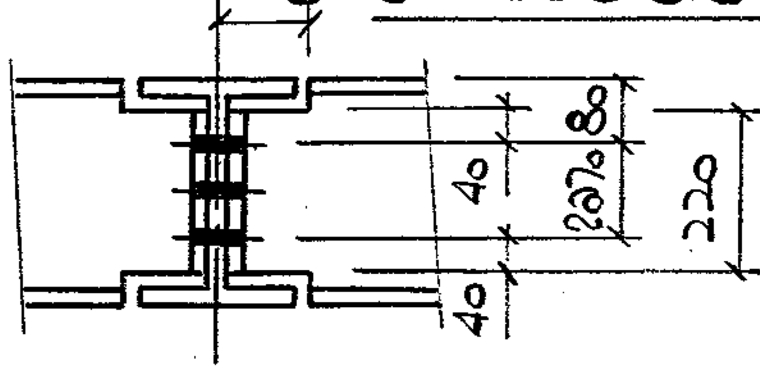
Beam to Beam

150×8 End Plt - 6FW

6N° M20 Gr 8.8 Bolts

D = 208mm

t = 13.0



Cellular Beam o/o 203×203×86UC 275kN

i) Bolts

$F_v = {}^{275}/_6 = \underline{46\text{kN}} < 92\text{kN}$

Allowable bearing on end plt $= 8 \times 20 \times 550 \times 10^{-3}$

$= \underline{88\text{kN}} > 46\text{kN}$

Check supporting web $\underline{t = 13.0}$

Load per bolt $= {}^{2\times275}/_6 = \underline{91\text{kN}}$

Allowable bearing $= 13 \times 20 \times 550 \times 10^{-3}$

$= \underline{143\text{kN}} > 91\text{kN}$

∴ Bolts ok

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SUBJECT Connection Design

SERIES SH 2 OF 2

SHT. No. C80 REV.

BY KM DATE Sep 98

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CONN R3 cont.

ii) End Plt.

$A_v = 0.9 \times 220 \times 8 = 1584 \text{ mm}^2$

$Av_{net} = 1584 - (3 \times 22 \times 8) = 1056 \text{ mm}^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1584 = \underline{337 \text{ kN}} > 275/2 \text{ kN}$

$P_{Vnet} = 0.5 \times 490 \times 10^{-3} \times 1056 = \underline{259 \text{ kN}} > 275/2 \text{ kN}$

$\therefore$ End Plt OK

iii) End Plt Weld - 6FW

$P_v = 2 \times 220 \times 4.2 \times 255 \times 10^{-3} = \underline{471 \text{ kN}} > 275 \text{ kN}$

$\therefore$ Weld OK

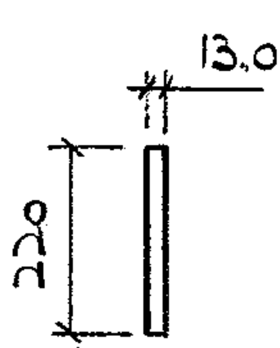
iv) Check Web behind End Plt. for Shear

$A_v = 0.9 \times 220 \times 13.0 = 2574 \text{ mm}^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2574 = \underline{548 \text{ kN}} > 275 \text{ kN}$

$\therefore$ Web OK

v) Check Beam for Notch



$Z = 13.0 \times 220^2/6 = 104\,867 \text{ mm}^3$

$M = 275 \times 108 \times 10^{-3} = \underline{29.7 \text{ kNm}}$

$P_b = 104\,867 \times 355 \times 10^{-6}$

$= \underline{37 \text{ kNm}} > 29.7 \text{ kNm}$

$\therefore$ Beam OK

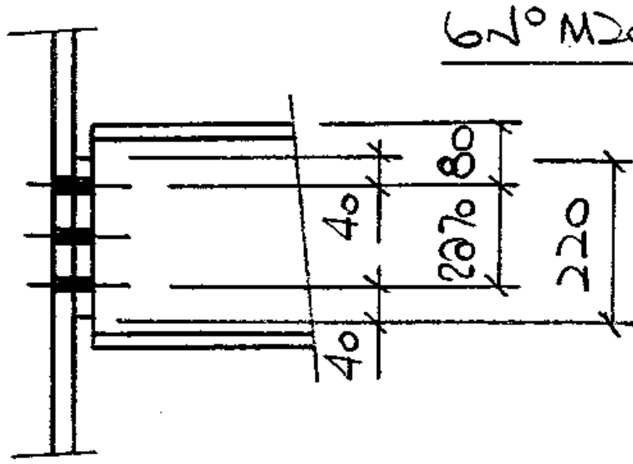
CONN R4

Cellular Beams B2·R

Beam to Column

150×8 End Plt - 6FW

6N° M20 Gr 8.8 Bolts



Cellular Beam % 203×203×86UC 275kN

Conn as CONN R2

Check bolt bearing on supporting web

Column C8 254×254×89UC t = 10.5

Load per bolt = $2\times275/6$ = 91kN

Allowable bearing = $10.5\times20\times550\times10^{-3}$

= 116kN > 91kN

∴ Bolts OK

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SHT. No. C81 REV.

BY KM DATE Sep 98

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Conn R5

Cellular Beam B3-R

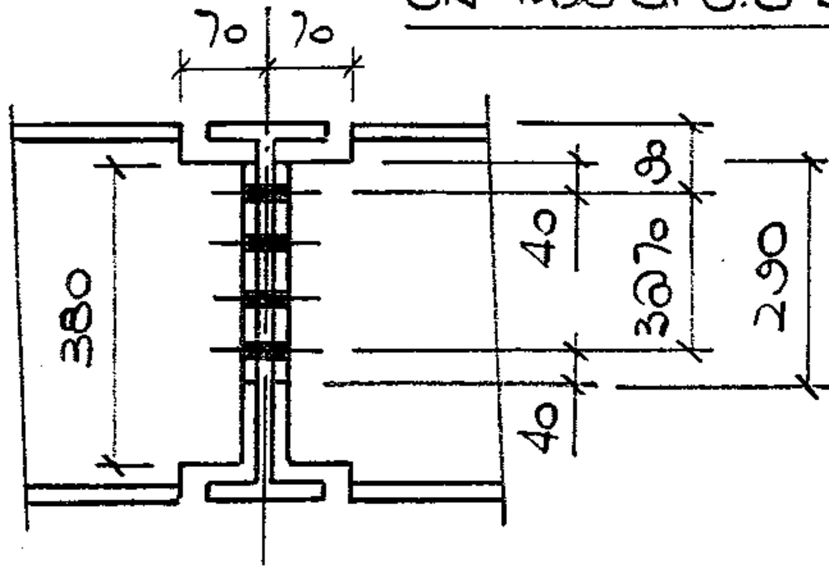
Beam to Beam

150×8 End Plt - 6FW

8N° M20 Gr8.8 Bolts

D = 479

t = 5.9



Cellular Beam B3-R % 356×127×33UB 200kN

i) Bolts

$F_v = 200/8 = \underline{25kN} < 92kN$

Allowable bearing on end plt. $= 8 \times 20 \times 550 \times 10^{-3}$

$= \underline{88kN} > 25kN$

Check supporting web t = 5.9

Load per bolt $= 2 \times 200/8 = \underline{50kN}$

Allowable bearing $= 5.9 \times 20 \times 550 \times 10^{-3}$

$= \underline{65kN} > 50kN$

∴ Bolts OK

CONN R5 cont.

ii/ End Plt

End Plt OK as CONN R1

iii/ End Plt Weld

End Plt Weld OK as CONN R1

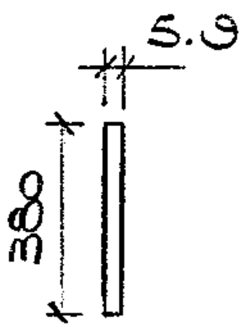
iv/ Check web behind End Plt. for Shear

$A_v = 0.9 \times 290 \times 5.9 = 1540\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1540 = \underline{328\ kN} > 200\ kN$

$\therefore$ Web OK

v/ Check Beam for Notch



$Z = 5.9 \times 380^2 / 6 = 141\ 993\ mm^3$

$M = 200 \times 70 \times 10^{-3} = \underline{14\ kNm}$

$P_b = 141\ 993 \times 355 \times 10^{-6}$

$= \underline{50\ kNm} > 14\ kNm$

$\therefore$ Beam OK

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SHT. No. C81A REV.

BY KM DATE Sep 98

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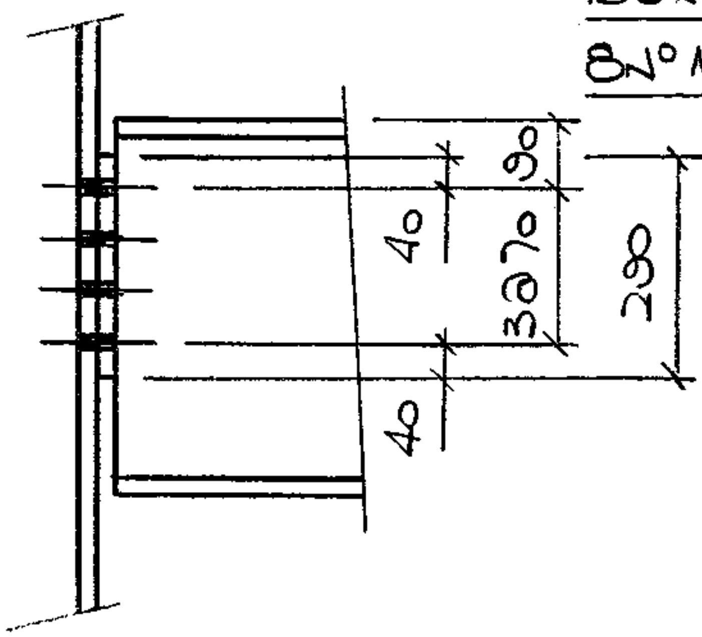
CONN R6

Cellular Beam B3-R

Beam to Column

150×8 End Plt - 6FW

8N° M20 Gr 8.8 Bolts



Cellular Beam B3-R % 356×127×33UB 200kN

Conn as CONN R5

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SHT. No. C82 REV.

BY KM DATE Sep 98

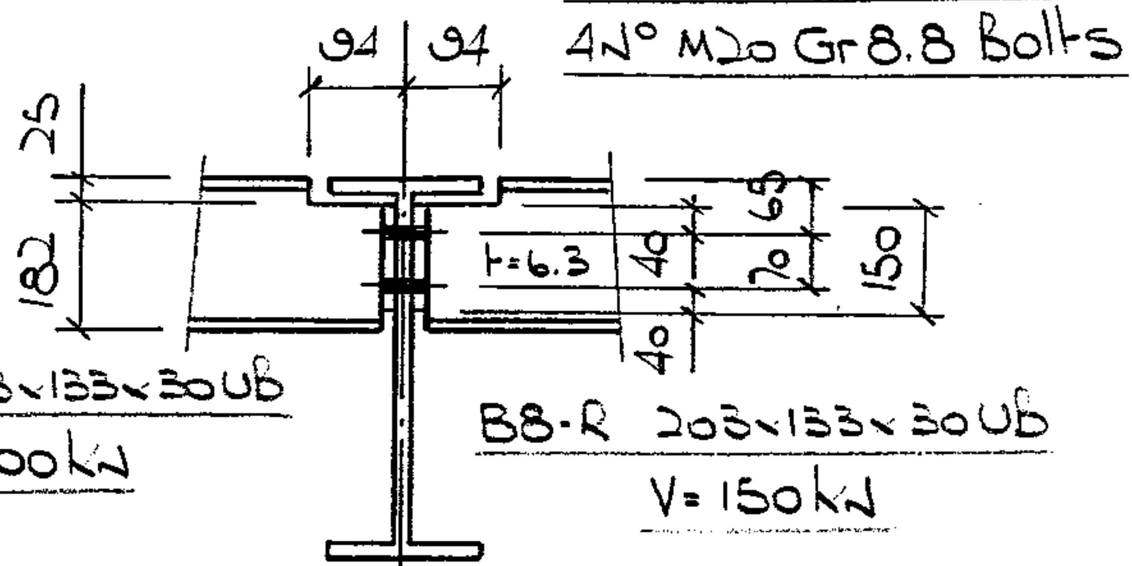
EX. DATE

CONN R7

B6·R to B9·R to B8·R

150×8 End Plt - 6FW

4N° M20 Gr 8.8 Bolts



B8·R End Conn (B6·R Similar)

i/ Bolts

$F_v = {}^{150}/_{4} = \underline{37.5\,kN} < 92\,kN$

Allowable bearing on end plt $= \underline{88\,kN} > 37.5\,kN$

Check supporting web $t = 8.0$

Load per bolt $= \dfrac{100+150}{4} = \underline{62.5\,kN}$

Allowable bearing $= \underline{88\,kN} > 62.5\,kN$

$\therefore$ Bolts ok

CONN R7 cont.

ii) End Plt.

$A_v = 0.9 \times 150 \times 8 = 1080\,mm^2$

$Av_{net} = 1080 - (2 \times 22 \times 8) = 728\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1080 = \underline{230\,kN} > 150/2\,kN$

$P_{V_{net}} = 0.5 \times 490 \times 10^{-3} \times 728 = \underline{178\,kN} > 150/2\,kN$

∴ End Plt OK

iii) End Plt Weld - 6FW

$P_v = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321\,kN} > 150\,kN$

∴ Weld OK

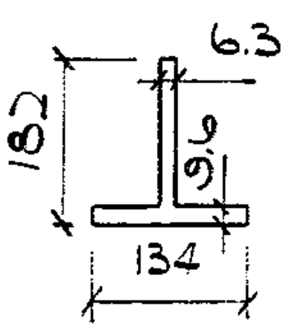
iv) Check web behind End Plt for Shear

$A_v = 0.9 \times 150 \times 6.3 = 850\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 850 = \underline{181\,kN} > 150\,kN$

∴ Web OK

v) Check Beam for Notch



$A = 2373\,mm^2$

$\bar{x} = 46\,mm$

$I_{NA} = 757\,cm^4$

$M = 150 \times 94 \times 10^{-3} = \underline{14\,kNm}$

$P_b = \frac{757 \times 10^4}{(182 - 46)} \times 355 \times 10^{-6}$

$= \underline{20\,kNm} > 14\,kNm$

∴ Beam OK

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SHT. No. C83 REV.

BY KM DATE Sep 98

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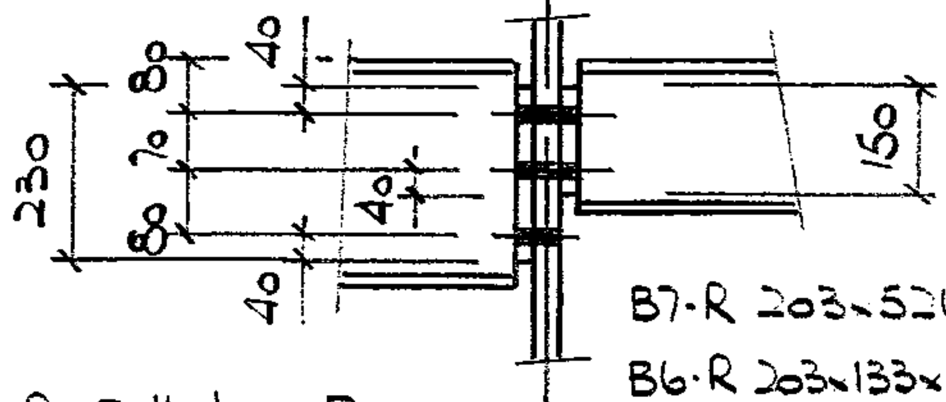
CONN R8

B2 · R to Col. C8 to B7 · R / B6 · R

150×8 End Plts 6FW

6N° M20 Gr8.8 Bolts

C8 254×89UC t=10.5



B7·R 203×52UC V=100kN

B6·R 203×133×30UB V=100kN

End Plt as CONN R7

B2·R Cellular Beam

% 203×86UC V=275kN

End Plt as CONN. R3

Check bolt bearing on supporting web

Load per bolt = $275/6 + 100/4 = \underline{71\,kN}$

Allowable bearing $= 10.5 \times 20 \times 550 \times 10^{-3}$

$= \underline{116\,kN} > 71\,kN$

$\therefore$ Bolts OK

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SHT. No. C84 REV.

BY KM DATE Sep 98

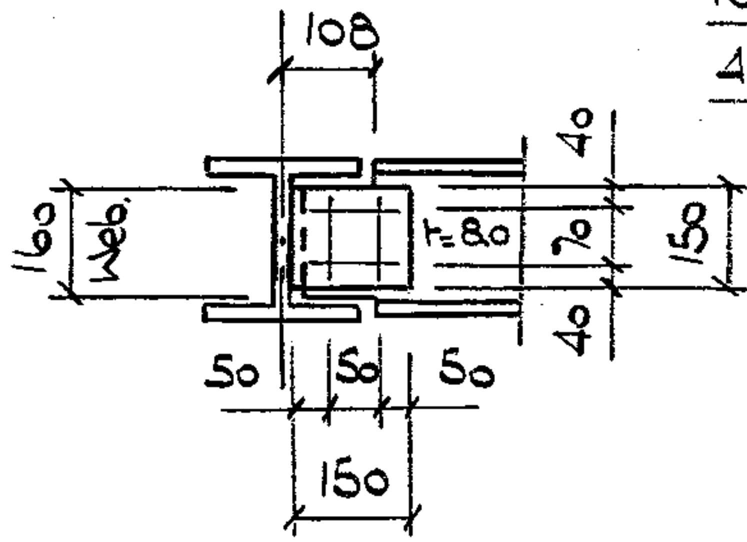
EX. DATE

CONN R9

B7-R

10 thk Fin Plt - 6FW

4 N° M20 Gr 8.8 Bolts



B7-R

203×203×52UC

V = 100 kN

i) Bolts

$$Z = \frac{4 \times (35^2)}{35} = 140\,mm$$

$$F_v = 100/4 = 25\,kN$$

$$F_h = \frac{100 \times 75}{140} = 54\,kN$$

$$F_r = \sqrt{(25^2 + 54^2)} = \underline{60\,kN} < 92\,kN$$

Allowable bearing on fin plt / beam web $= 8 \times 20 \times 550 \times 10^{-3}$

$= \underline{88\,kN} > 60\,kN$

∴ Bolts OK

Conn R9 cont.

iii/ Fin Plt.

a. Shear

$A_v = 0.9 \times 150 \times 10 = 1350\ mm^2$

$A_{v_{net}} = 1350 - (2 \times 22 \times 10) = 910\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1350 = \underline{288\ kN} > 100\ kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 910 = \underline{223\ kN} > 100\ kN$

b. Bending

$Z = 10 \times 150^2 / 6 = 37\,500\ mm^3$

$M = 100 \times 75 \times 10^{-3} = \underline{7.5\ kNm}$

$P_b = 37\,500 \times 355 \times 10^{-6} = \underline{13.3\ kNm} > 7.5\ kNm$

$\therefore$ Fin Plt OK

iii/ Fin Plt. Weld - 6FW

$P_v = 2 \times 150 \times 4.2 \times 255 \times 10^{-3} = \underline{321\ kN} > 100\ kN$

$\therefore$ Weld OK

iv/ Check Beam for Notch

a. Shear

$A_v = 0.9 \times 160 \times 8.0 = 1152\ mm^2$

$A_{v_{net}} = 1152 - (2 \times 22 \times 8) = 800\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1152 = \underline{245\ kN} > 100\ kN$

$P_{v_{net}} = 0.5 \times 355 \times 10^{-3} \times 800 = \underline{142\ kN} > 100\ kN$

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SHT. No. C84 REV.

BY KM DATE Sep 98

EX. DATE

CONN R9 cont.

Beam Notch cont.

b. Bending

$$Z = \frac{8 \times 160^2}{6} = 34133 \text{ mm}^3$$

$$M = 100 \times 108 \times 10^{-3} = \underline{10.8 \text{ kNm}}$$

$$P_b = 34133 \times 355 \times 10^{-6} = \underline{12.1 \text{ kNm}} > 10.8 \text{ kNm}$$

$\therefore$ Beam OK

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SHT. No. C84A REV.

BY KM DATE Sep 98

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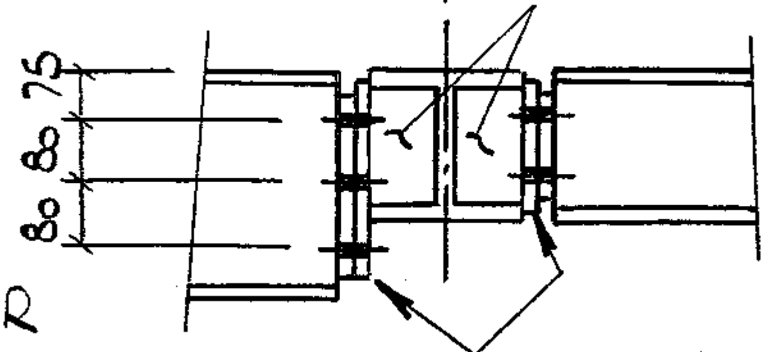
CONN R10

B2-R to B7-R to B6-R

B7-R 203×203×52UC

8thk stiffs



B6-R 203×133×30UB

V=100kN

150×8 End Plt-6FW

4N° M20 Gr8.8 Bolts

B2-R

Cellular Beam

150×10 Plt.

As CONN R7

% 203×203×86UC

V= 275kN

All weld 6FW

150×8 End Plt-6FW

6N° M20 Gr8.8 Bolts

As CONN R3

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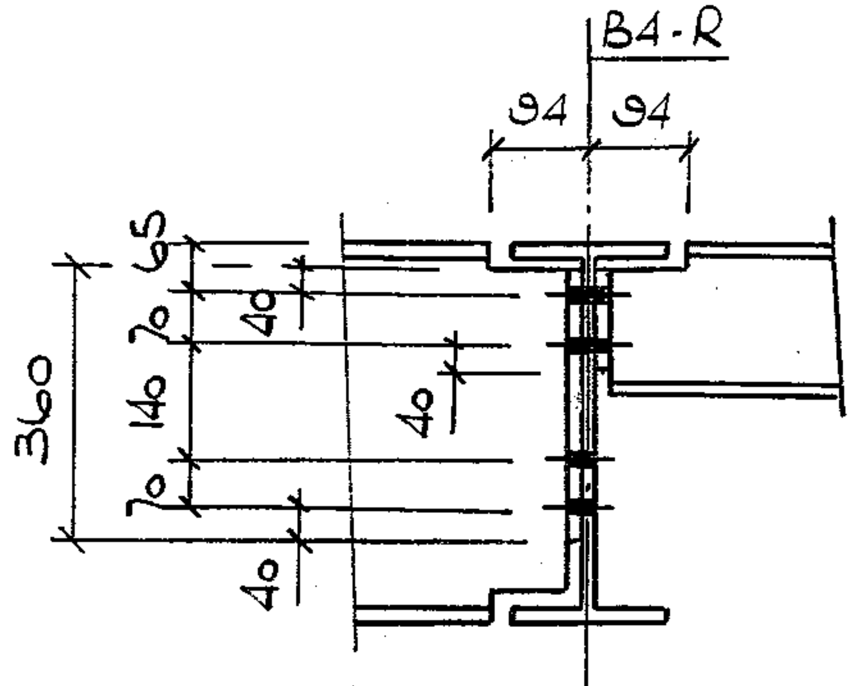
SHT. No. C85 REV.

BY KM DATE Sep 98

EX. DATE

CONN R11

B1-R to B4-R to B6-R



B6-R V=100kN

203×133×30UB

150×8 End Plt

6FW

As CONN R7

B1-R V=275kN

Cellular Beam

o/o 356×171×57UB

150×8 End Plt - 6FW

8N° M20 Gr8.8 Bolts

As CONN R1

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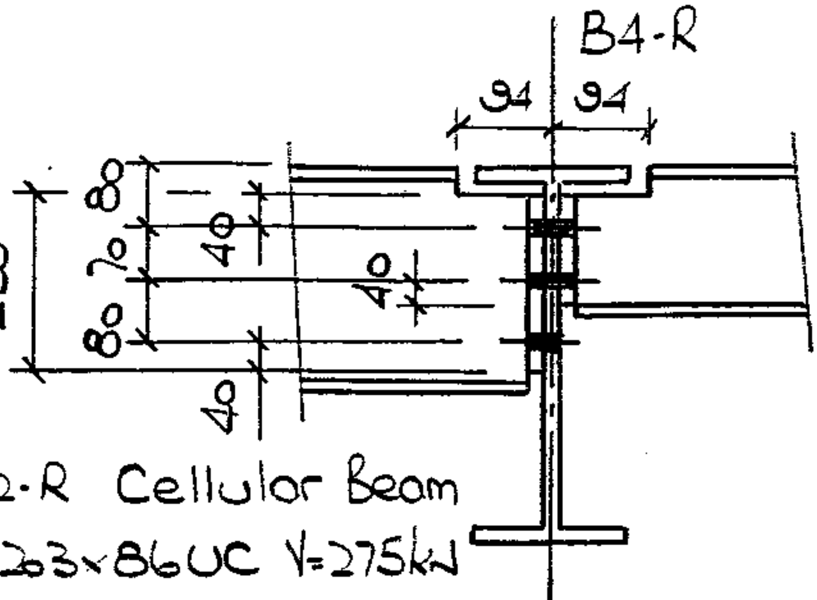
SHT. No. C85A REV.

BY KM DATE Sep 98

EX. DATE

CONN R12

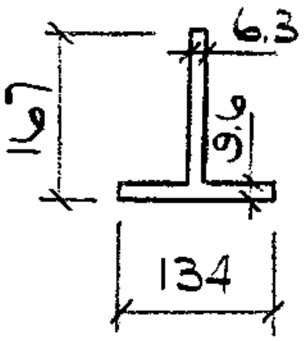
B2-R to B4-R to B6-R



B6-R V=100kN
203×133×30 UB
150×8 End Plt - 6FW
As CONN R7

B2-R Cellular Beam
% 203×86 UC V=275kN
150×8 End Plt - 6FW
6N° M20 Gr 8.8 Bolts
As CONN R3

203UB - Check Beam for Notch



$A = 2278\,mm^2$

$\bar{x} = 41\,mm$

$I_{NA} = 595\,cm^4$

$M = 100 \times 94 \times 10^{-3} = \underline{9.4\,kNm}$

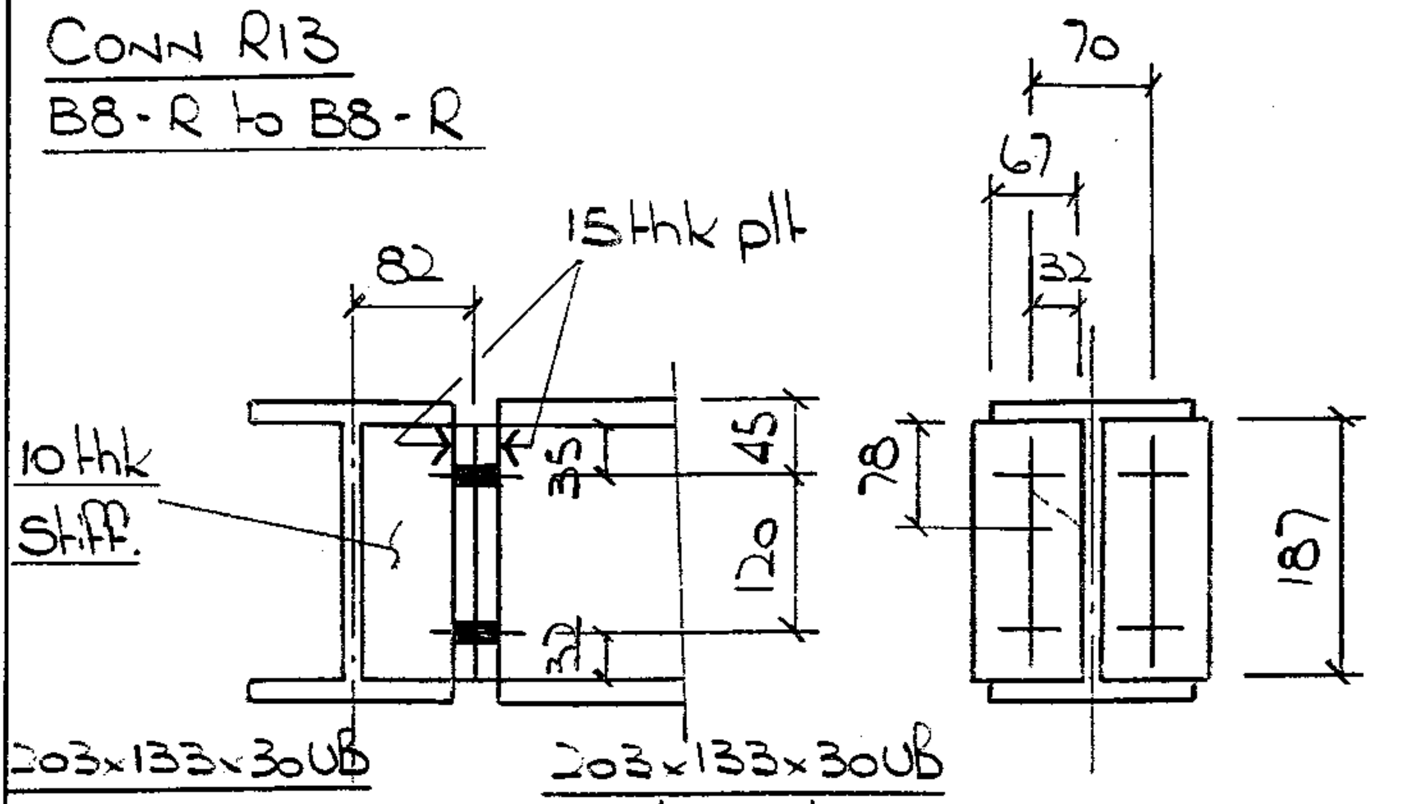
$P_b = \frac{595 \times 10^4}{(167 - 41)} \times 355 \times 10^{-6}$

$= \underline{16.8\,kNm} > 9.4\,kNm$

$\therefore$ Beam OK

Conn R13

B8-R to B8-R



V = 150 kN

All weld 6FW

4N° M20 Gr 8.8 Bolts

i) Bolts

Shear = $150/4 = \underline{37.5\text{kN}} < 92\text{kN}$

Tension = $\dfrac{82 \times 150}{(120+32)\times 2} = \underline{40.5\text{kN}} < 110\text{kN}$

Combined shear & tension

$37.5/92 + 40.5/110 = 0.78 < 1.4$

Check bearing on end plt / T. plt.

Allowable bearing $= 15 \times 20 \times 550 \times 10^{-3}$

$= \underline{165\text{kN}} > 37.5\text{kN}$

$\therefore$ Bolts ok

CONN R13 cont.

ii) End Plt.

a. Shear

$A_v = 0.9 \times 187 \times 15 = 2525\,mm^2$

$A_{v_{net}} = 2525 - (2 \times 22 \times 15) = 1865\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2525 = 538\,kN > 150/2\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 1865 = 457\,kN > 150/2\,kN$

b. Bending

$Z = 67 \times 15^2/6 = 2513\,mm^3$

$M = \dfrac{40.5 \times 35 \times 10^{-3}}{2} = \underline{0.71\,kNm}$

$P_b = 2513 \times 355 \times 10^{-6} = \underline{0.89\,kNm} > 0.71\,kNm$

∴ Plts OK

iii) Weld - 6FW

$F_v = \dfrac{150 \times 10^3}{2 \times 187 \times 4.2} = \underline{95\,N/mm^2}$

$F_t = \dfrac{40.5 \times 10^3}{(67+78) \times 4.2} = \underline{67\,N/mm^2}$

$F_r = \sqrt{(95^2 + 67^2)} = \underline{116\,N/mm^2} < 255\,N/mm^2$

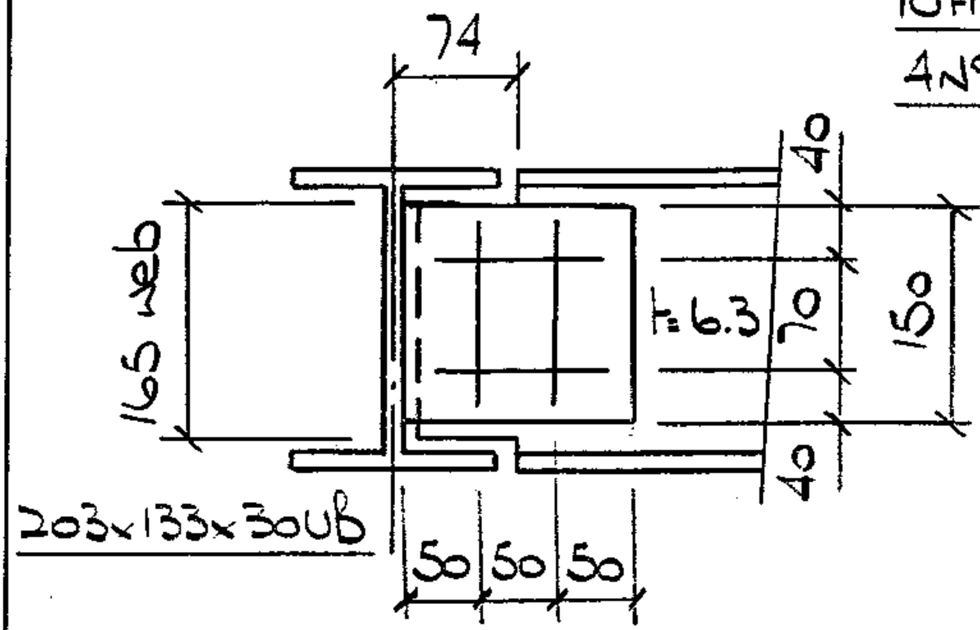
∴ Weld OK

CONN R14

B6-R to B6-R / B8-R

10thk Fin Plt - 6FW

4N° M20 Gr 8.8 Bolts



203x133x30 UB

V = 100 kN

i) Bolts

$$Z = \frac{4 \times (35^2)}{35} = 140\text{mm}$$

$$F_v = {}^{100}/_{4} = 25\text{kN}$$

$$F_h = {}^{100 \times 75}/_{140} = 54\text{kN}$$

$$F_r = \sqrt{(25^2 + 54^2)} = \underline{60\text{kN}} < 92\text{kN}$$

Allowable bearing on fin plt / beam web $= 6.3 \times 20 \times 550 \times 10^{-3}$

$= \underline{69\text{kN}} > 60\text{kN}$

$\therefore$ Bolts ok

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CONN R14 cont.

ii/ Fin Plt

Fin Plt OK as CONN R9

iii/ Fin Plt. Weld 6FW

Weld OK as CONN R9

iv/ Check Beam for Notch

a. Shear

$A_v = 0.9 \times 165 \times 6.3 = 936\,mm^2$

$Av_{net} = 936 - (2 \times 22 \times 6.3) = 659\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 936 = \underline{199\,kN} > 100\,kN$

$P_{Vnet} = 0.5 \times 490 \times 10^{-3} \times 659 = \underline{161\,kN} > 100\,kN$

b. Bending

$Z = \dfrac{6.3 \times 165^2}{6} = 28\,586\,mm^3$

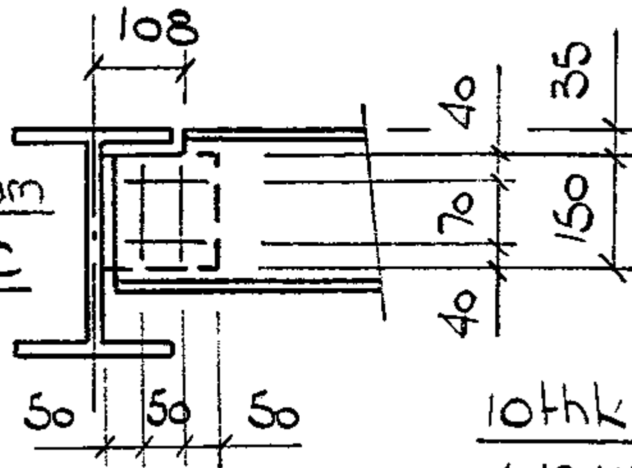
$M = 100 \times 74 \times 10^{-3} = 7.4\,kNm$

$P_b = 28\,586 \times 355 \times 10^{-6} = \underline{10.1\,kNm} > 7.4\,kNm$

$\therefore$ Beam OK

CONN R16

B6-R to B2-R



Cellular Beam

% 203x86 UC

203x133x30 UB

V=100 kN

10thk Fin Plt - 6FW

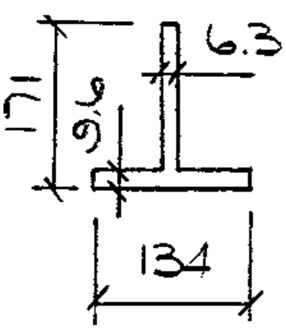
4N° M20 Gr8.8 Bolts

As CONN R14

Check Notch Beam

a. Shear OK As CONN R14

b. Bending



$A = 2303 mm^2$

$x = 43mm$

$I_{NA} = 636 cm^4$

$M = 100 \times 108 \times 10^{-3} = \underline{10.8 kNm}$

$P_b = \dfrac{636 \times 10^4}{(171-43)} \times 355 \times 10^{-6}$

$= \underline{17.6 kNm} > 10.8 kNm$

$\therefore$ Beam OK

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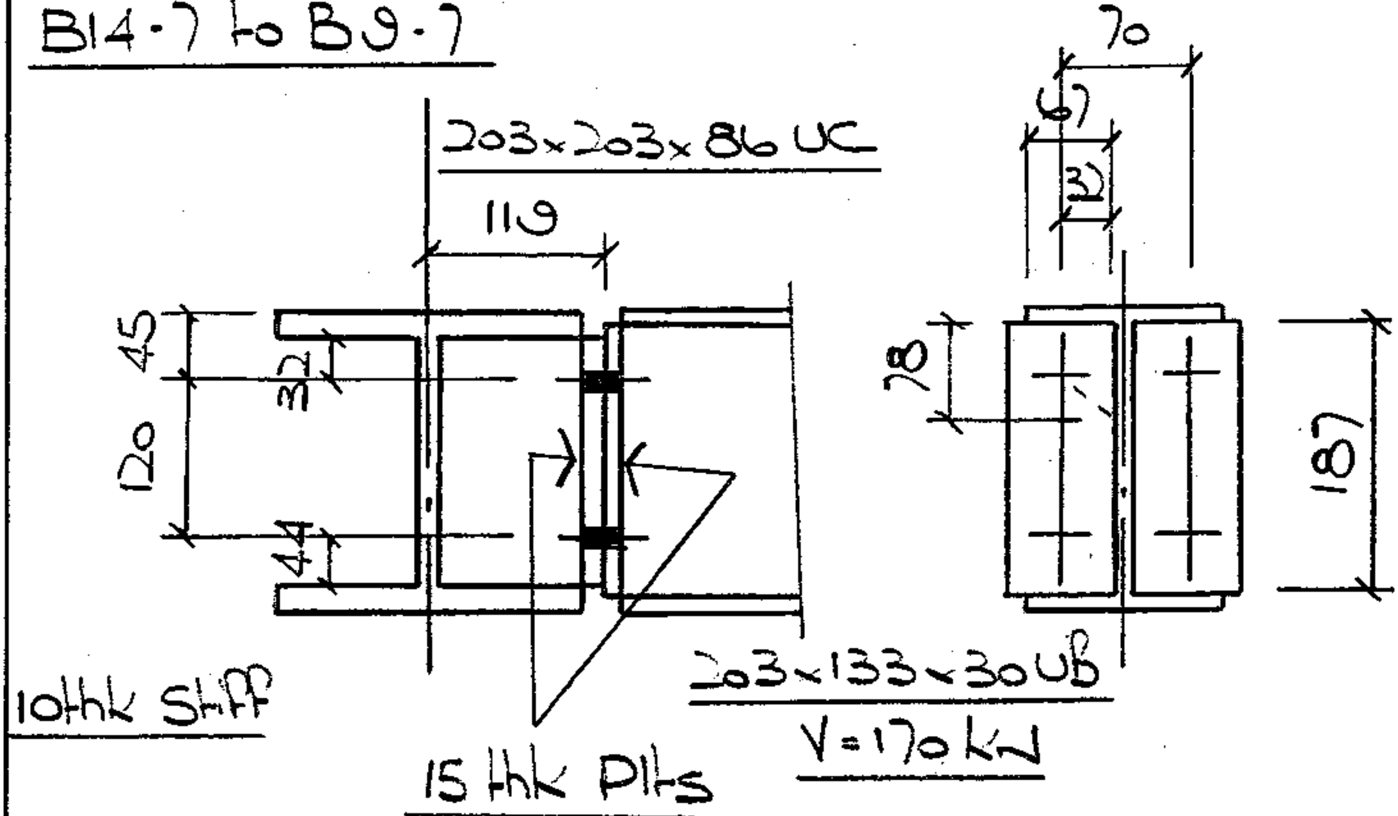
B14-7 to B9-7



All welds 6mm FW

4No M20 Gr 8.8 Bolts

i) Bolts

Shear = $170/4 = \underline{42.5\,kN} < 92\,kN$

Tension = $\dfrac{119 \times 170}{(120 + 44) \times 2} = \underline{62\,kN} < 110\,kN$

Combined shear & tension

$42.5/_{92} + 62/_{110} = \underline{1.0} < 1.4$

Check bearing on end plt

$= 15 \times 20 \times 550 \times 10^{-3} = \underline{165\,kN} > 42.5\,kN$

∴ bolts ok

B14-7 to B9-7 cont.

ii) End Plt.

a. Shear

$A_v = 0.9 \times 187 \times 15 = 2525\,mm^2$

$A_{v_{net}} = 2525 - (2 \times 22 \times 15) = 1865\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2525 = \underline{538\,kN} > {}^{170}/_{2}\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 1865 = \underline{457\,kN} > {}^{170}/_{2}\,kN$

b. Bending

$z = (67 + 78) \times 15^2/_6 = 5\,438\,mm^3$

$M = \dfrac{62 \times 32 \times 10^{-3}}{2} = \underline{1.0\,kNm}$

$P_b = 5\,438 \times 355 \times 10^{-6} = \underline{1.9\,kNm} > 1.0\,kNm$

$\underline{\therefore \text{Plt. OK}}$

iii) Welds - 6FW

$F_v = \dfrac{170 \times 10^3}{2 \times 187 \times 4.2} = 108\,N/mm^2$

$F_t = \dfrac{62 \times 10^3}{(67+78) \times 4.2} = 102\,N/mm^2$

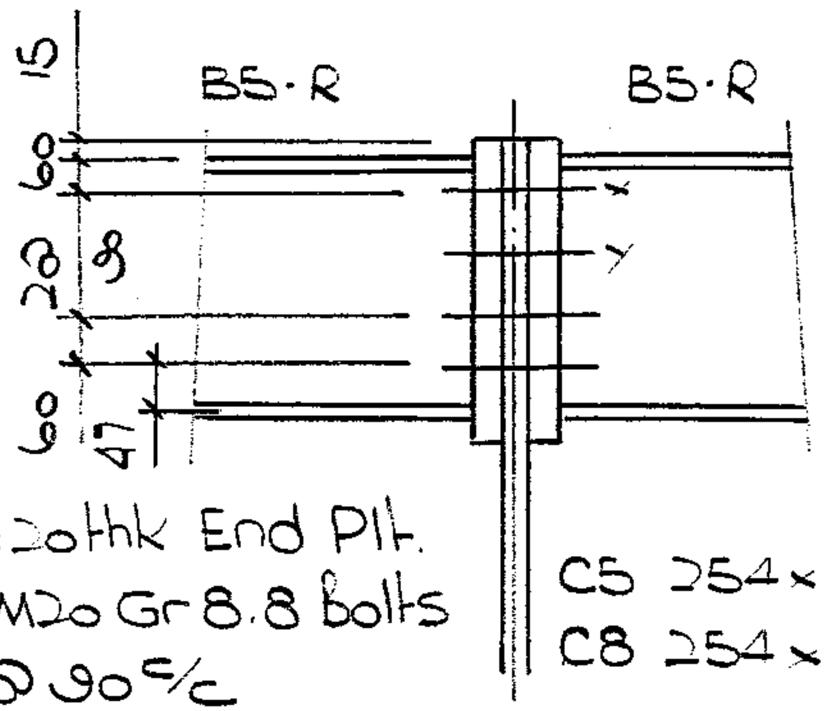
$F_r = \sqrt{(108^2 + 102^2)} = \underline{148\,N/mm^2} < 255\,N/mm^2$

$\underline{\therefore \text{Weld OK}}$

B5-R to Col. C5/C8 to B5R

CN17

Rev A - Redrawn



356×171×45UB
V=225kN
M=50kNm

200×20thk End Plt.
8N° M20 Gr 8.8 Bolts
@ 90 c/c

C5 254×167UC
C8 254×89UC t=10.5

End Plt - Beam 8FW

i/ Bolts

a Shear - Consider Btm 4 Bolts only

$225/4 = \underline{56\,kN} < 92\,kN$

Check Col web for Bearing

Load per bolt $= 2 \times 225/4 = \underline{112.5\,kN}$

Allowable bearing $= 10.5 \times 20 \times 550 \times 10^{-3}$
$= \underline{115.5\,kN} > 112.5\,kN$

$\therefore$ OK

B5-R to Col C5/C8 cont.

Bolts cont.

b. Tension - Consider Top 4 Bolts only

$M = 50\,kNm$

$$Z = \frac{2(28.7^2 + 19.7^2)}{28.7} = 84.4\,cm$$

Tension per bolt

$$\text{row } x = \frac{50 \times 10^2}{84.4} = \underline{59\,kN} < 110\,kN$$

$$\text{row } y = \frac{50 \times 10^2}{84.4} \times 19.7/28.7 = \underline{41\,kN}$$

∴ Bolts OK

ii) Check Comp Force in Btm. Flg

Comp force $= 2(59 + 41) = \underline{200\,kN}$

171 × 9.7 thk Flg

$P_b = 171 \times 9.7 \times 355 \times 10^{-3} = \underline{589\,kN} > 200\,kN$

∴ Flg OK

iii) End Plt

a. Bending @ Bolt row x

$M = 59 \times 41 \times 10^{-3} = \underline{2.4\,kNm}$

$Z = (45 + 60 + 171/2) \times 20^2/6 = 12\,700\,mm^3$

$P_b = 12\,700 \times 345 \times 10^{-6} = \underline{4.4\,kNm} > 2.4\,kNm$

B5-R to Col C5/C8 cont

iii/ End Plt. cont.

b. Bending @ Bolt row y

$M = 41 \times 41 \times 10^{-3} = \underline{1.7\ kNm}$

$Z = 90 \times 20^2/6 = 6000 mm^3$

$P_b = 6000 \times 345 \times 10^{-6} = \underline{2.1\ kNm} > 1.7 kN$

∴ End Plt OK

iv/ End Plt. Weld - 8FW

$f_v = \frac{225 \times 10^3}{2 \times 356 \times 5.6} = 56\ N/mm^2$

@ Bolt row y

$f_h = \frac{41 \times 10^3}{90 \times 5.6} = 81\ N/mm^2$

$f_r = \sqrt{(56^2 + 81^2)} = \underline{98\ N/mm^2} < 255\ N/mm^2$

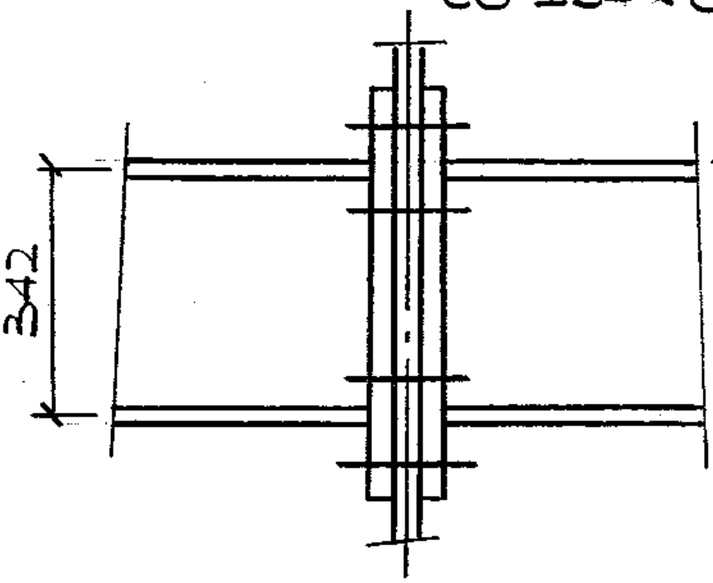
∴ Weld OK

CONN R15

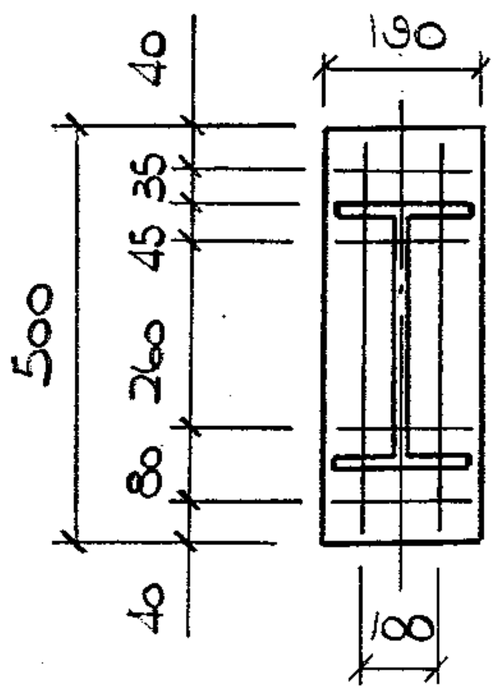
B5-R to Col. C5 / C8 to B5-R

C5 254 × 167 UC t = 19.2
C8 254 × 89 UC t = 10.5

B5-R



B5-R
356×171×45UB
V = 225 kN
M = 50 kNm



20 thk End Plt
6FW
8 N° M20 Gr 8.8 Bolts

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Conn R15 cont

i) Bolts

Load per flg due to mmt $= 50\times10^3/342 = 146\text{kN}$

$\therefore$ Tension per bolt $= 146/4 = \underline{36.5\text{kN}} < 110\text{kN}$

Shear per bolt $= 225/8 = \underline{28\text{kN}} < 92\text{kN}$

Combined shear & tension

$28/92 + 36.5/110 = 0.64 < 1.4$

Check bearing on Col. web

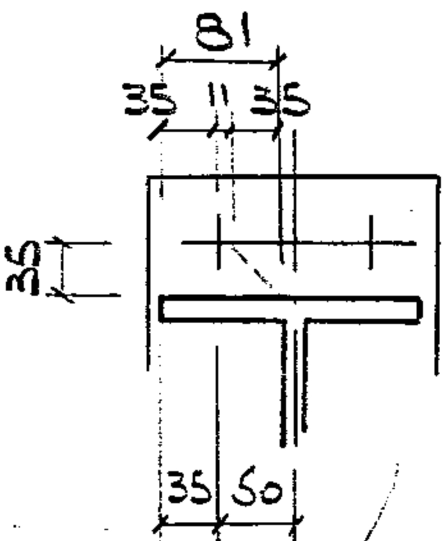
Load per bolt $= 2\times225/8 = \underline{56\text{kN}}$

Allowable bearing $= 10.5\times20\times550\times10^{-3}$

$= \underline{116\text{kN}} > 56\text{kN}$

$\therefore$ Bolts OK

ii) End Plate



a. Shear

$A_v = 0.9\times500\times20 = 9000\text{mm}^2$

$A_{v\,net} = 9000 - (4\times22\times20) = 7240\text{mm}^2$

$P_v = 0.6\times355\times10^{-3}\times9000 = \underline{1917\text{kN}} > 225/2\text{kN}$

$P_{v\,net} = 0.5\times490\times10^{-3}\times7240 = \underline{1774\text{kN}} > 225/2\text{kN}$

b. Bending

$Z = 81\times20^2/6 = 5400\text{mm}^3$

$M = 36.5\times35\times10^{-3} = \underline{1.28\text{kNm}}$

$P_b = 5400\times355\times10^{-6}$

$= \underline{1.9\text{kNm}} > 1.28\text{kNm}$

$\therefore$ Plt. OK

CONN R15 cont

iii/ Weld - 6FW

$$F_v = \frac{28 \times 10^3}{81 \times 4.2} = \underline{82\ N/mm^2}$$

$$F_t = \frac{36.5 \times 10^3}{81 \times 4.2} = \underline{107\ N/mm^2}$$

$$F_r = \sqrt{(82^2 + 107^2)} = \underline{135\ N/mm^2} < 255\ N/mm^2$$

$\therefore$ Weld OK

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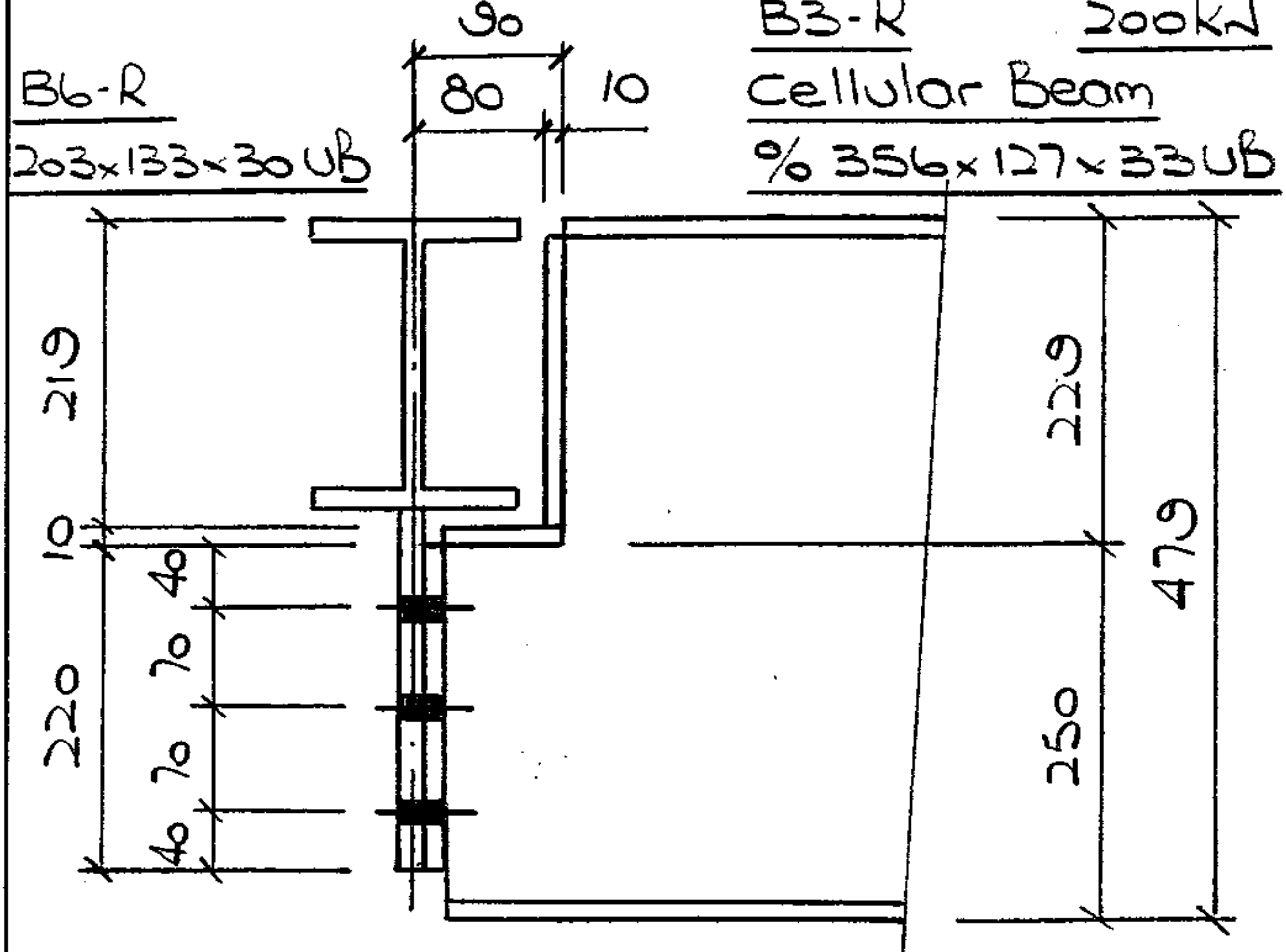
CONN R17

B3-R to B6-R



All plates 150 x 10 thk

All welds 6FW

6N° M20 Gr 8.8 Bolts (90 c/c)

CONN R17 cont.

i) Bolts

$F_v = 200/6 = \underline{33kN} < 92kN$

Allowable bearing on end plt. $= 10 \times 20 \times 550 \times 10^{-3}$
$= \underline{110kN} > 33kN$

$\therefore$ Bolts OK

ii) End Plt.

$A_v = 0.9 \times 220 \times 10 = 1980\,mm^2$

$A_{v_{net}} = 1980 - (3 \times 22 \times 10) = 1320\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1980 = \underline{422\,kN} > 200/2\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 1320 = \underline{323\,kN} > 200/2\,kN$

$\therefore$ End Plt OK

iii) End Plt Weld - 6FW

$P_v = 2 \times 220 \times 4.2 \times 255 \times 10^{-3} = \underline{471\,kN} > 200\,kN$

$\therefore$ Weld OK

iv) Check web behind End Plt for Shear

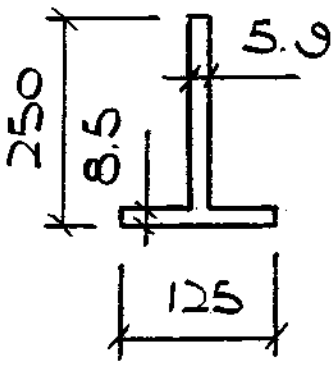
$A_v = 0.9 \times 220 \times 5.9 = 1168\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1168 = \underline{249\,kN} > 200\,kN$

$\therefore$ Web OK

CONN R17 cont.

V) Check Beam for Notch



$A = 2487 \text{ mm}^2$

$\bar{x} = 76 \text{ mm}$

$I_{NA} = 1644 \text{ cm}^4$

$M = 200 \times 90 \times 10^{-3} = \underline{18 \text{ kNm}}$

$P_b = \dfrac{1644 \times 10^4}{(250 - 76)} \times 355 \times 10^{-6}$

$= \underline{33.5 \text{ kNm}} > 18 \text{ kNm}$

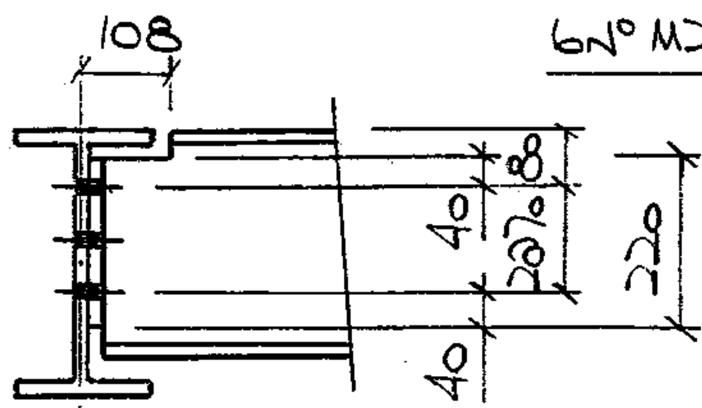
$\therefore$ Beam OK

CONN R18

B2-R to B5-R

150×8 End Plt. - 6FW

6N° M20 Gr 8.8 Bolts



356×171×45 UB

t = 6.9

Cellular Beam

o/o 203×203×86UC

275kN

Conn Design as Conn. R3

Check 356 UB web for Bolt bearing

Load per bolt = 275/6 = 46kN

Allowable bearing = $6.9 \times 20 \times 550 \times 10^{-3}$

= 76kN > 46kN

∴ Bolts ok

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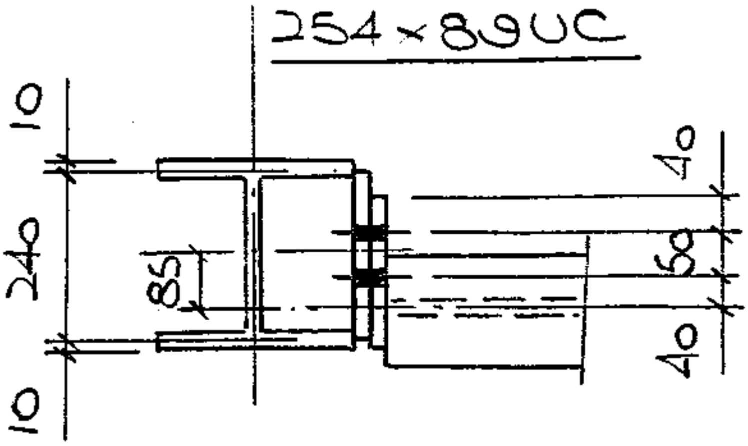
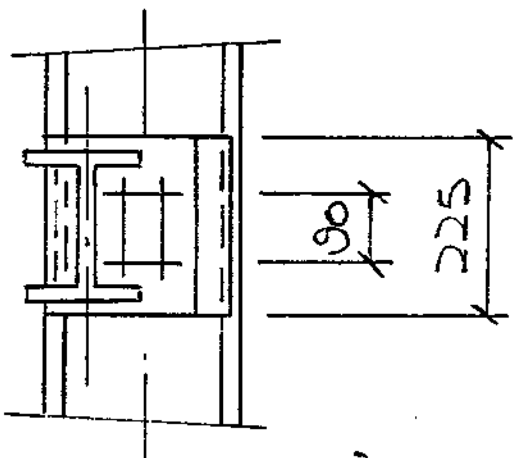
Conn R19

B8-R to Col. C8 (85 offset)



203 x 133 x 30 UB

150kN



10thk Bridge Plt.

10thk End Plt

4N° M20 Gr 8.8 Bolts

All welds 6FW

i) Bolts $Z = 4 \times 45^2/45 = 180\text{mm}$

$F_v = 150/4 = 37.5\text{kN}$

$F_h = 150 \times 65/180 = 54.2\text{kN}$

$F_r = \sqrt{(37.5^2 + 54.2^2)} = \underline{66\text{kN}} < 92\text{kN}$

Allowable bearing on bridge plt / End plt.

$= 10 \times 20 \times 550 \times 10^{-3}$

$= \underline{110\text{kN}} > 66\text{kN}$

$\therefore$ Bolts ok

CONN R19 cont

ii) End Plt / Bridge Plt

a. Shear

$A_v = 0.9 \times 225 \times 10 = 2025\ mm^2$

$A_{v_{net}} = 2025 - (2 \times 22 \times 10) = 1585\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2025 = \underline{431\ kN} > 150/2\ kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 1585 = \underline{388\ kN} > 150/2\ kN$

b. Bending

$Z = 10 \times 225^2/6 = 84\ 375\ mm^3$

$M = 150 \times 65 \times 10^{-3} = \underline{9.75\ kNm}$

$P_b = 84\ 375 \times 355 \times 10^{-6} = \underline{30\ kNm} > 9.75\ kNm$

$\therefore$ Plts OK

iii) Bridge Plt to Col Weld 6FW

$P_v = 225 \times 4.2 \times 255 \times 10^{-3} = \underline{240\ kN} > 150\ kN$

$\therefore$ Weld OK

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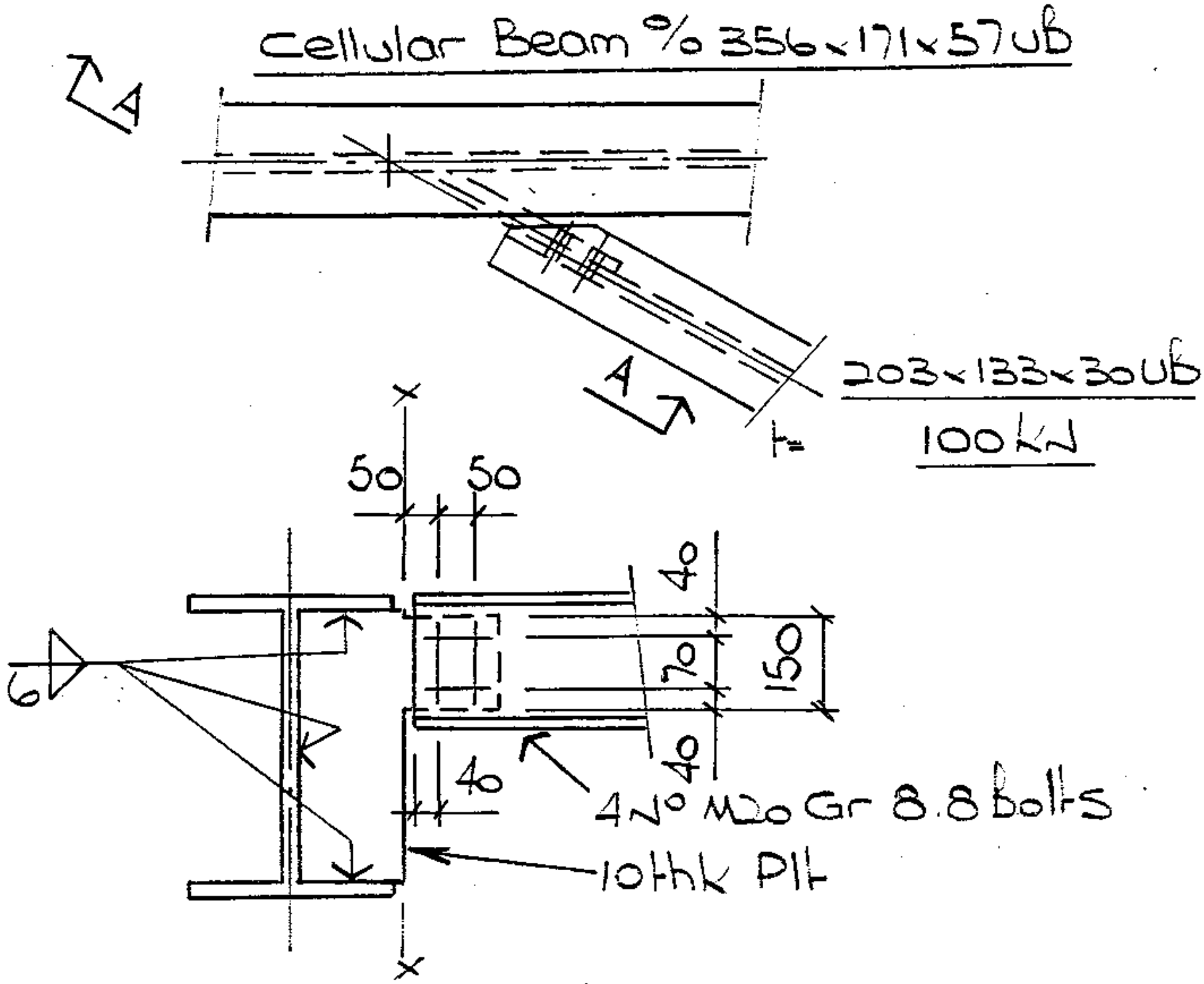
Conn R20

B6-R to B1-R



SECTION A-A

Bolt & Fin Plt (@ sect. x-x) check

as Conn R14

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BEAM B1-7 TO COLUMN C7

(CN ~ 17)

SERIES SHEET 1 OF 1

SHT. No. C92 REV.

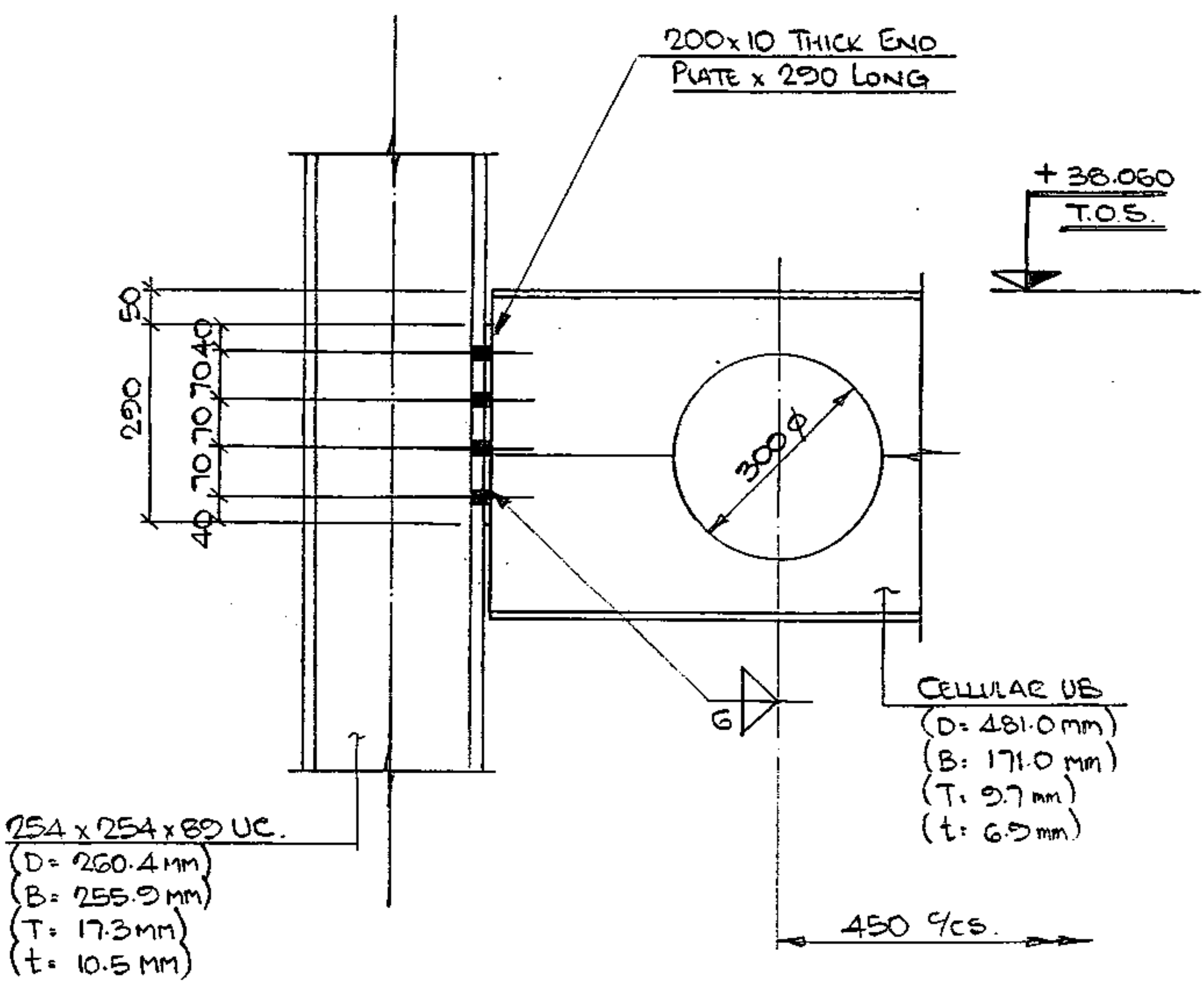
BY KB DATE 08/98

EX. DATE

BEAM B1-7 TO COLUMN C7

481 x 171 x 45 CELLULAR UB TO 254 x 254 x 89 UC

END REACTION = 200 kN



M20 BOLTS (GRADE 8.8) @ 140 C/CS

END PLATE DEPTH

MIN. DEPTH : 0.6 x 481.0 = 288.6 ⟶ SAY 290 MM

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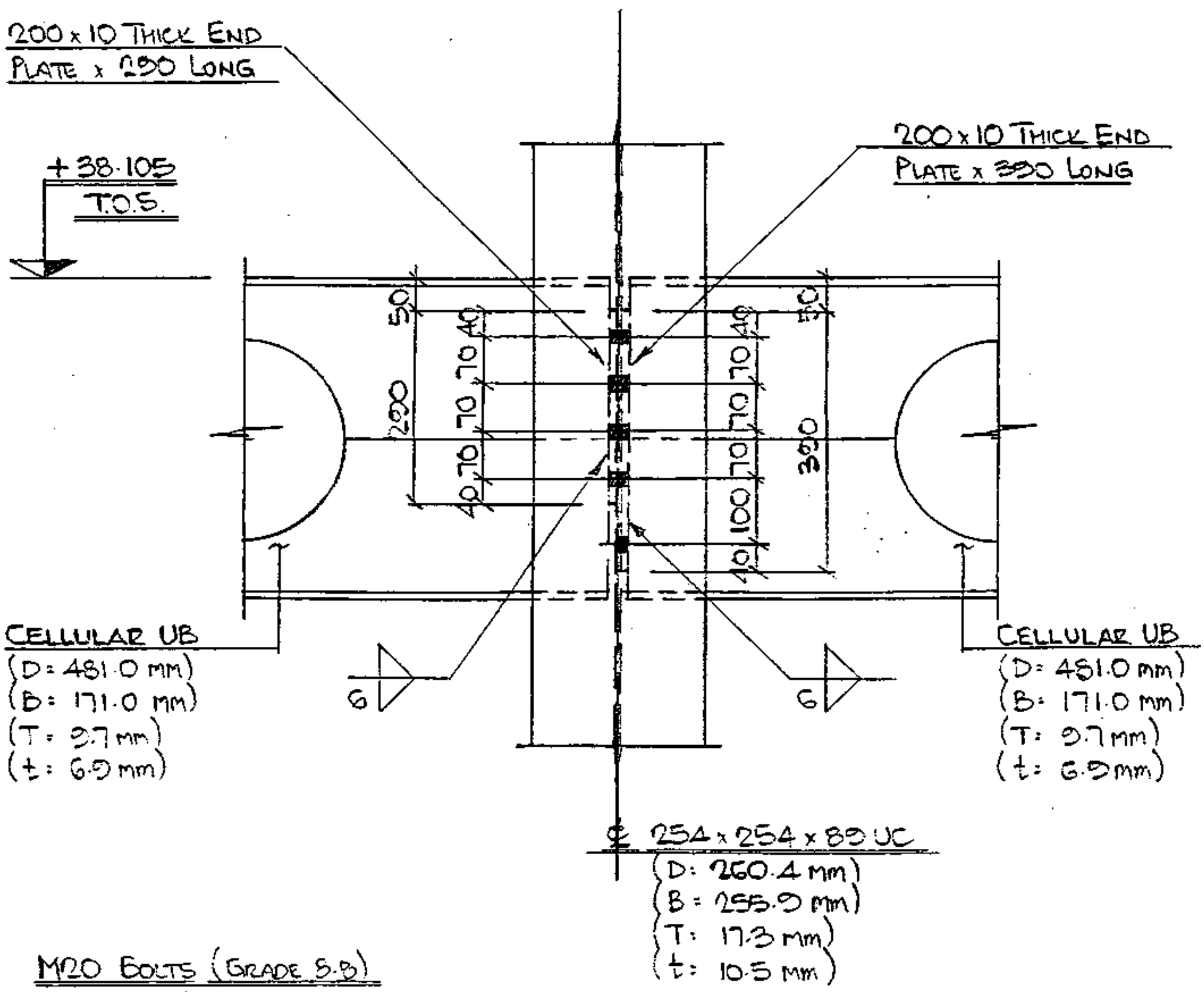
O/No. 325 JOB CARLTON GARDENS | SERIES SHEET 1 OF 1
SHT. No. C92A REV.
SUBJECT CONNECTION DESIGN | BY KB DATE 09/98
BEAMS B1-7 TO COLUMN C8 | EX. DATE
(CN ~17)

BEAMS B1-7 TO COLUMN C8

481 x 171 x 45 CELLULAR UB TO 254 x 254 x 89 UC

END REACTION = 200 KN



M20 BOLTS (GRADE 8.8) @ 140 c/cs.

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SUBJECT CONNECTION DESIGN

BEAMS B1-7 TO COLUMN C8

(CN~17)

SERIES SHEET 1 OF 1

SHT. No. C92B REV.

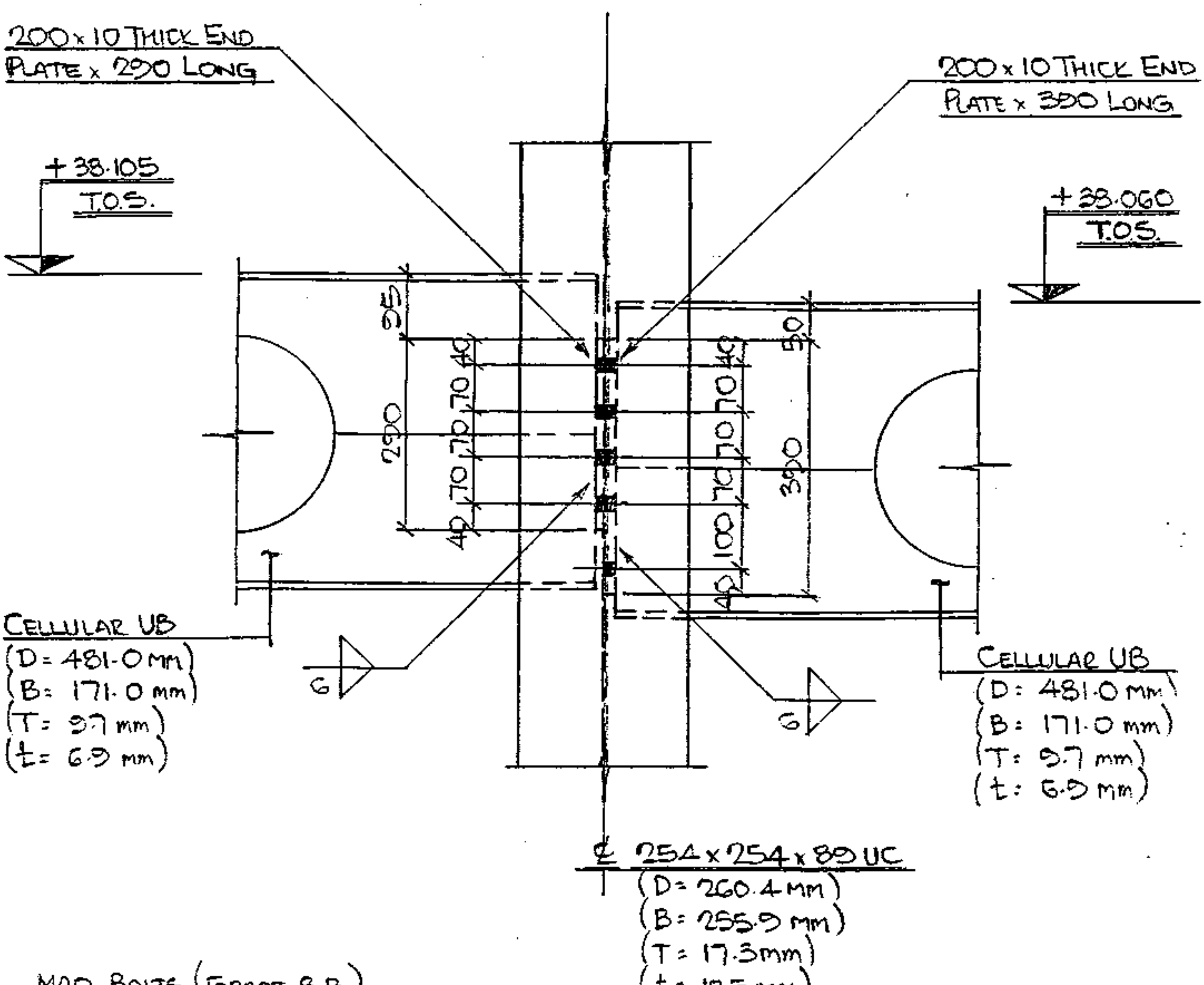
BY KB DATE 09/92

EX. DATE

BEAMS B1-7 TO COLUMN C8

481 x 171 x 45 CELLULAR UB TO 254 x 254 x 89 UC

END REACTION = 200 KN



M20 BOLTS (GRADE 8.8)
@ 140 c/cs.

Kvaerner Cleveland Bridge Ltd.

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SUBJECT Connection Design

SERIES SH. 1 OF 1

SHT. No. C92C REV.

BY KM DATE Oct 98

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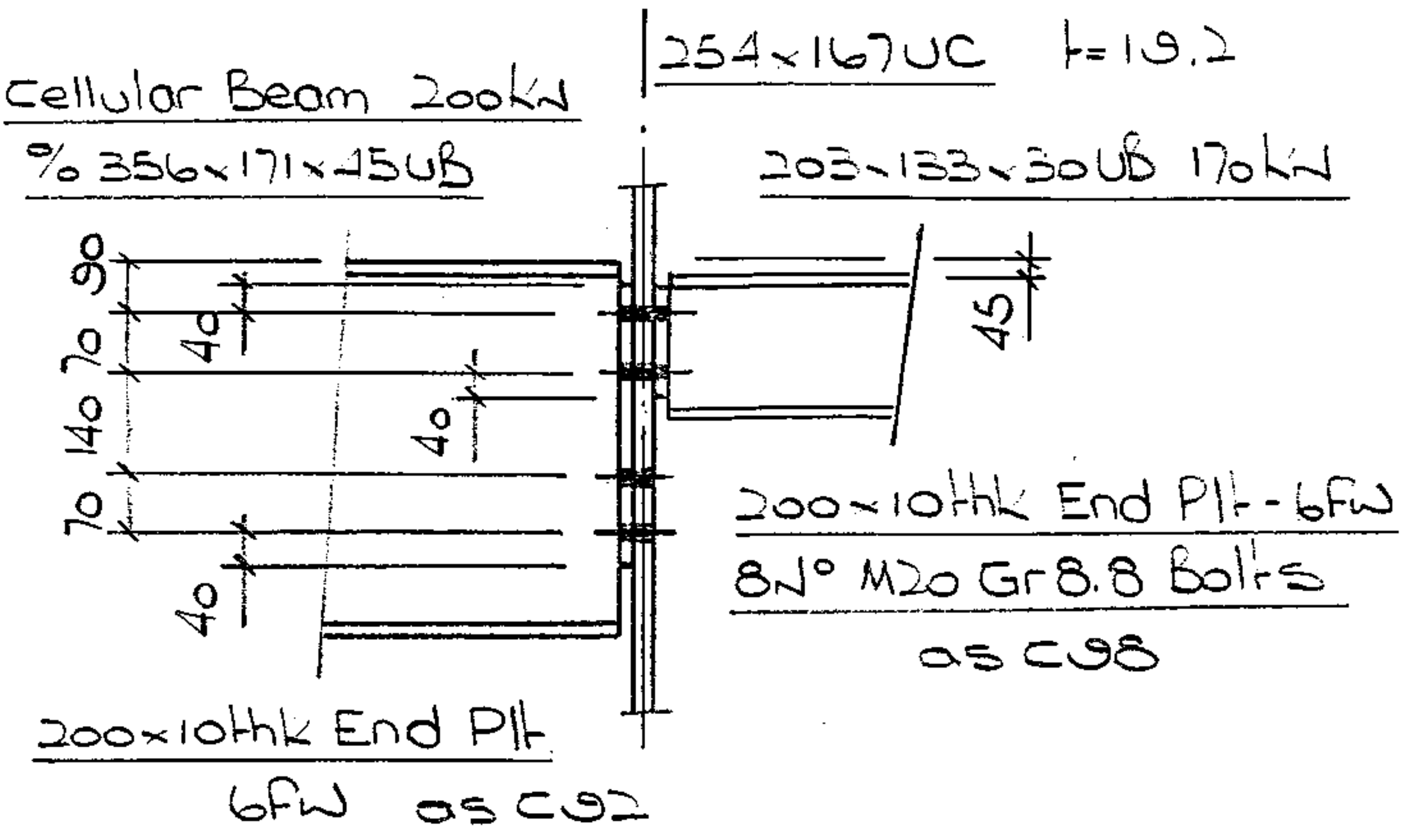
CN17

Beam B1·7 / B8·7 to Col. C5 (Col. C8 Similar)



Check bolt bearing on Col web

Load per bolt $= 200/8 + 170/4 = \underline{67.5\text{kN}}$

Allowable bearing $= 19.2 \times 20 \times 550 \times 10^{-3}$

$= \underline{211\text{ kN}} > 67.5\text{kN}$

$\therefore$ Bolts OK

C8. 254×89UC t = 10.5

Allowable bearing $= 10.5 \times 20 \times 550 \times 10^{-3}$

$= \underline{115.5\text{kN}} > 67.5\text{kN}$

$\therefore$ Bolts OK

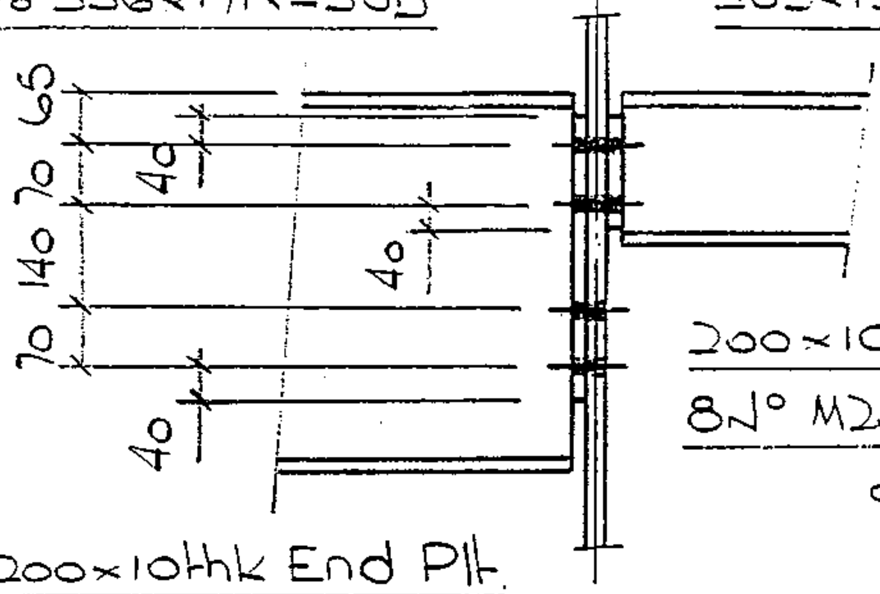
Beam B1·7 / B8·7 to Col. C5 (Col. C8 Similar)

Cellular Beam 200kN

% 356×171×45UB

254×167UC t=19.2

203×133×30UB 170kN



200×10thk End Plt - 6FW
8N° M20 Gr8.8 Bolts
as C98

200×10thk End Plt
6FW as C92

Check bolt bearing on Col web

Load per bolt = $200/8 + 170/4$ = 67.5kN
Allowable bearing = $19.2 \times 20 \times 550 \times 10^{-3}$
= 211 kN > 67.5kN
∴ Bolts OK

C8. 254×89UC t = 10.5

Allowable bearing = $10.5 \times 20 \times 550 \times 10^{-3}$
= 115.5kN > 67.5 kN
∴ Bolts OK

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O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

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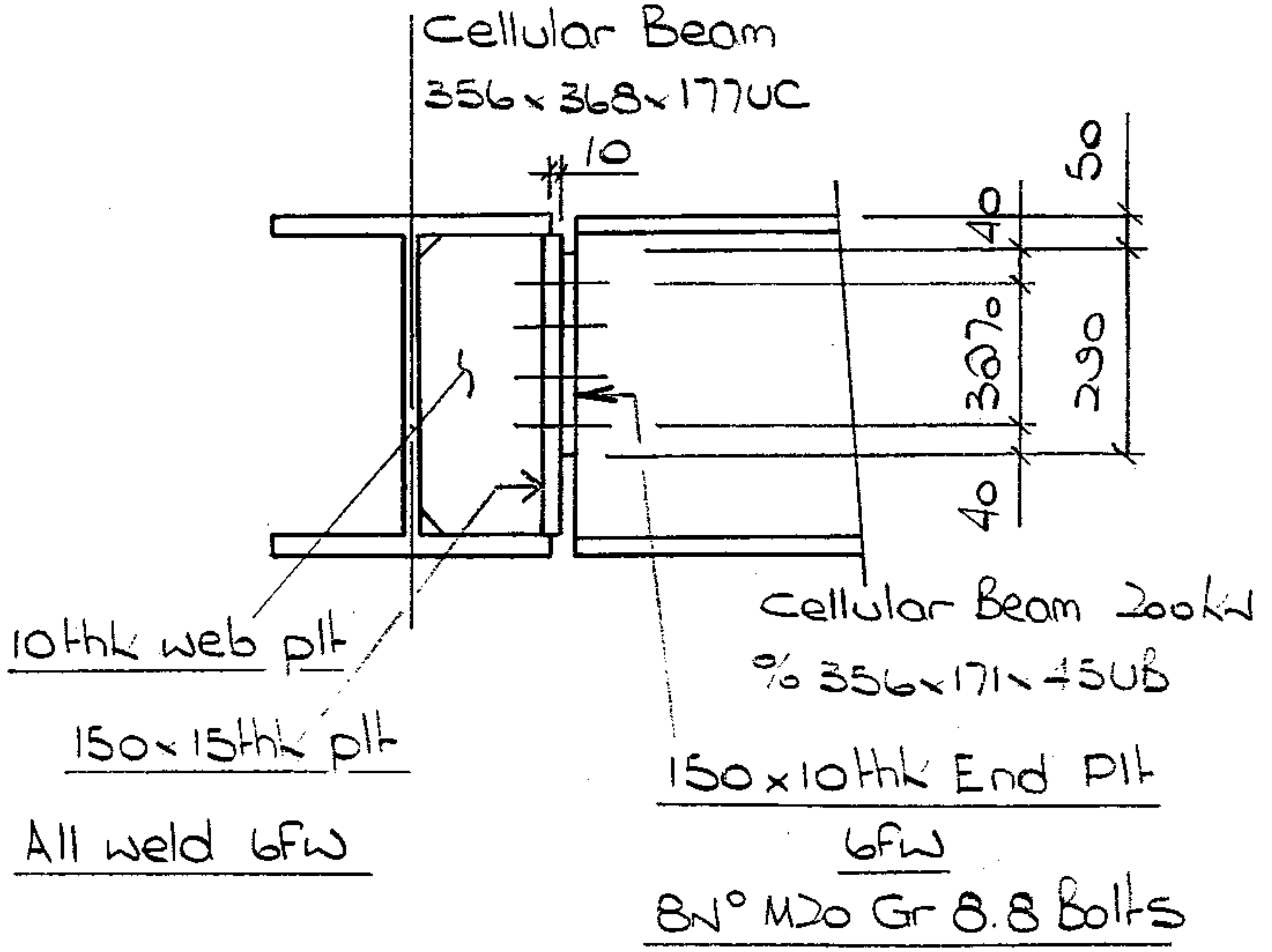
SHT. No. C92E REV.

BY KM DATE Oct 98

EX. DATE

CN17

Conn of Beam B1·7 to B5·7



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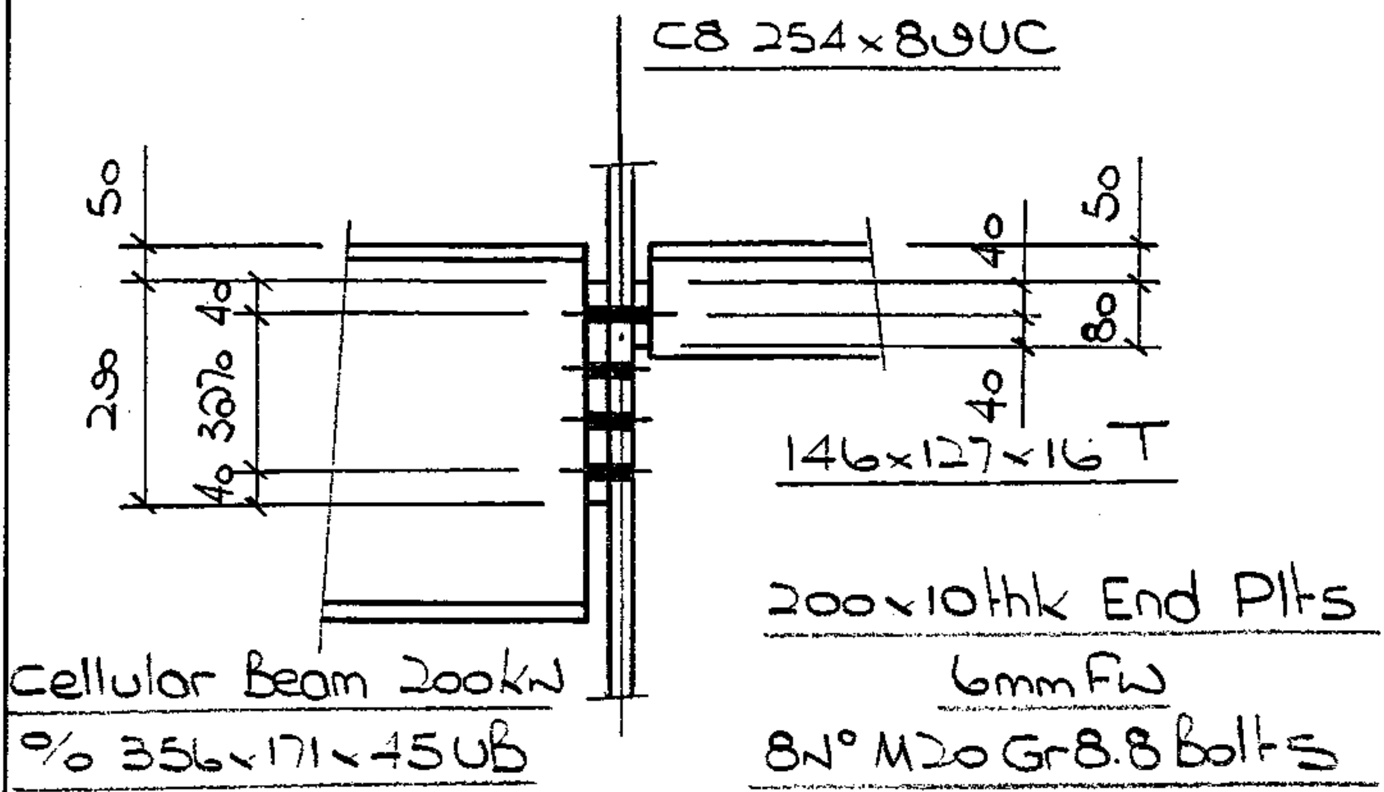
SHT. No. C92F REV.

BY KM DATE Oct. 98

EX. DATE

CN17

Conn of Beams B1·7 / B11·7 to Col. C8



BEAMS B1-7 ; B4-7 & B5-7 TO COLUMN C3

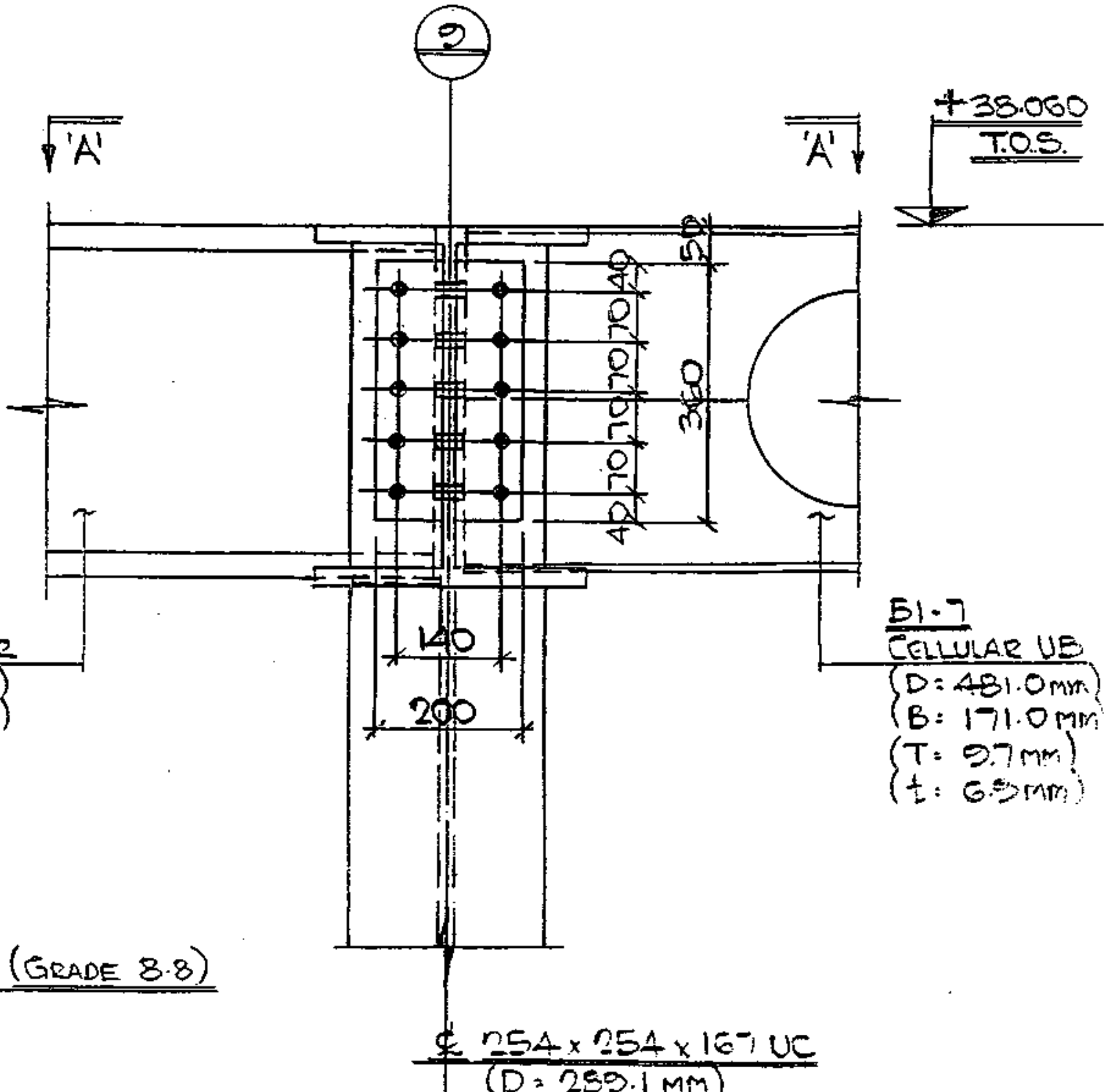
481 x 171 x 45 CELLULAR UB ; 400 x 200 x 35 x 25 PG & 498 x 368 x 177 CELL UB
TO 254 x 254 x 167 UC

END REACTIONS :—

BEAM B1-7 = 200 KN

BEAM B4-7 = 350 KN

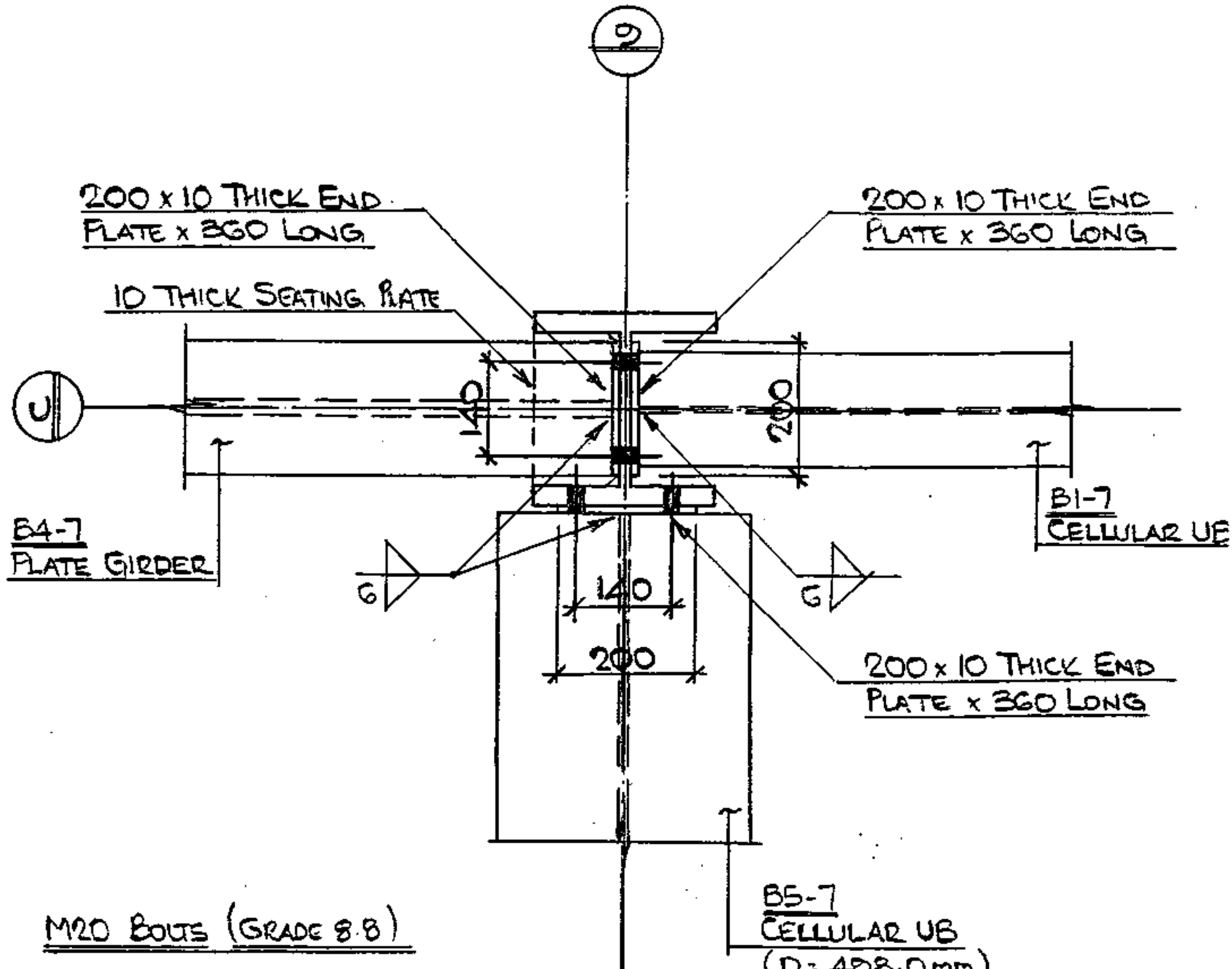
BEAM B7-7 = 300 KN.



M20 BOLTS (GRADE 8.8)

℄ 254 x 254 x 167 UC
(D = 289.1 MM)
(B = 264.5 MM)
(T = 31.7 MM)
(t = 19.2 MM)

(CN – 17)



M20 Bolts (Grade 8.8)

6 Leg Fillet Welds (U.N.O.)

View On 'A-A'

BEAMS B4-7 ; B5-7 & B7-7 TO COLUMN C3

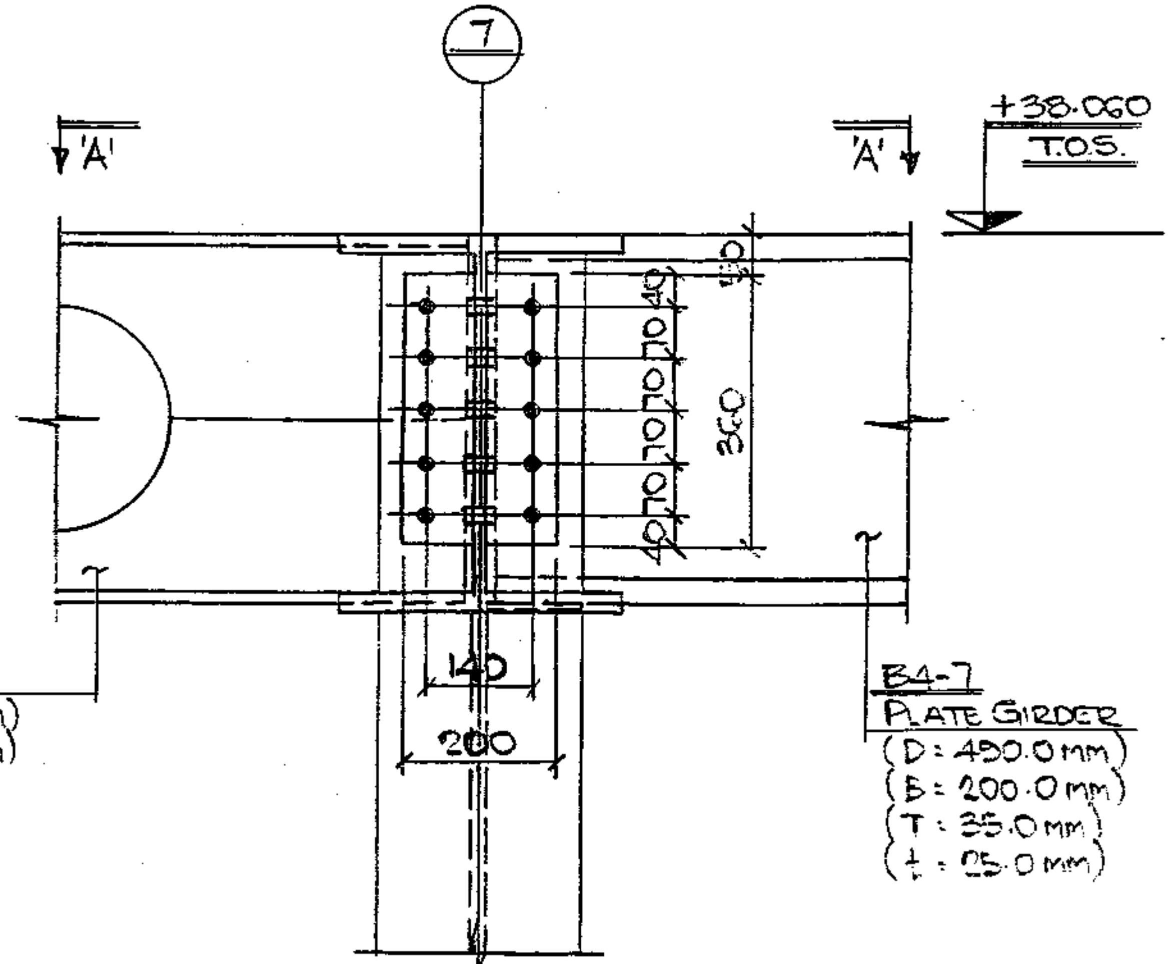
511 x 406 x 235 CELLULAR UC ; 450 x 200 x 35 x 25 P.G. & 453 x 171 x 67 CELLULAR UB TO 254 x 254 x 167 UC

END REACTIONS :—

BEAM B4-7 = 350 KN

BEAM B5-7 : 420 KN

BEAM B7-7 : 300 KN



M20 BOLTS (GRADE 8.8)

℄ 254 x 254 x 167 UC
(D: 289.1 mm)
(B: 264.5mm)
(T: 31.7 mm)
(t: 19.2 mm)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

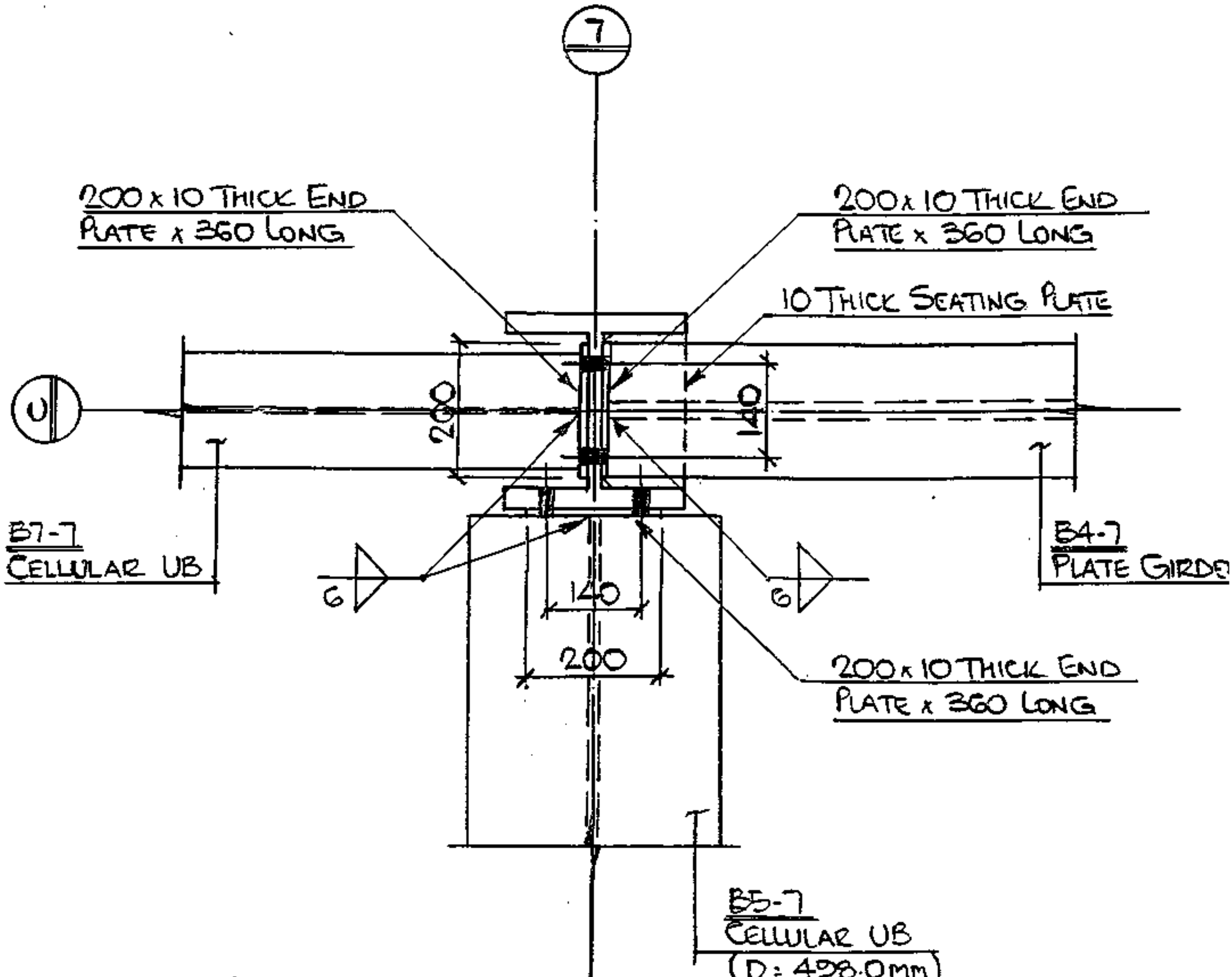
SERIES SHEET 2 OF 2

SHT. No. C93A REV.

BY KB DATE 05/98

EX. DATE

(CN~17)



M20 BOLTS (GRADE 8.8)

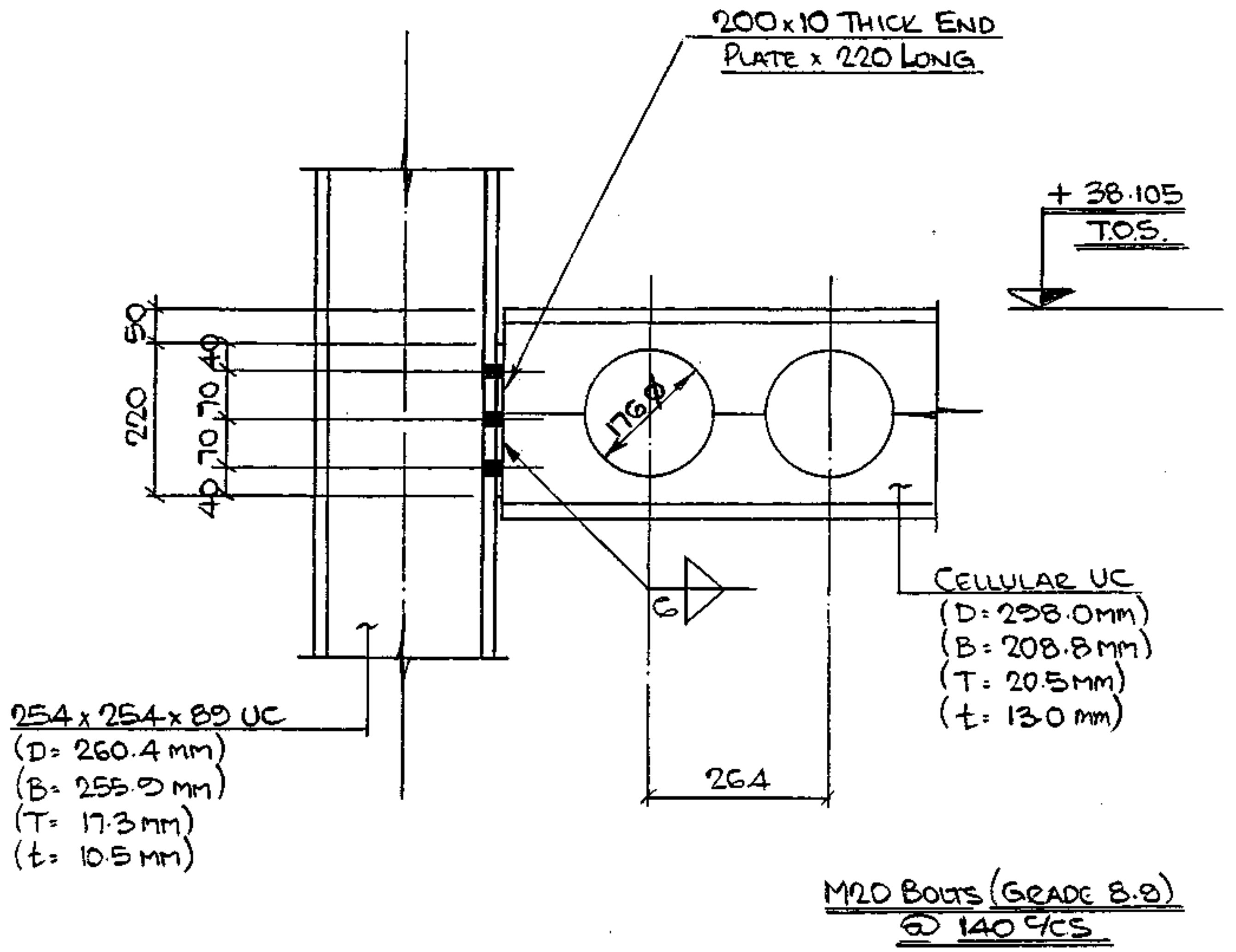
6 LEG FILLET WELDS (U.N.O.)

VIEW ON 'A-A'

BEAM B2-7 TO COLUMN C8

298 x 203 x 86 CELLULAR UC TO 254 x 254 x 89 UC

END REACTION = 260 KN



END PLATE DEPTH :—

MIN DEPTH = 0.6 x 298.0 = 178.8 ⟶ SAY 220 mm

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B2-7 TO BEAM B1-7

(CN~17)

SERIES SHEET 1 OF

SHT. No. C94A REV.

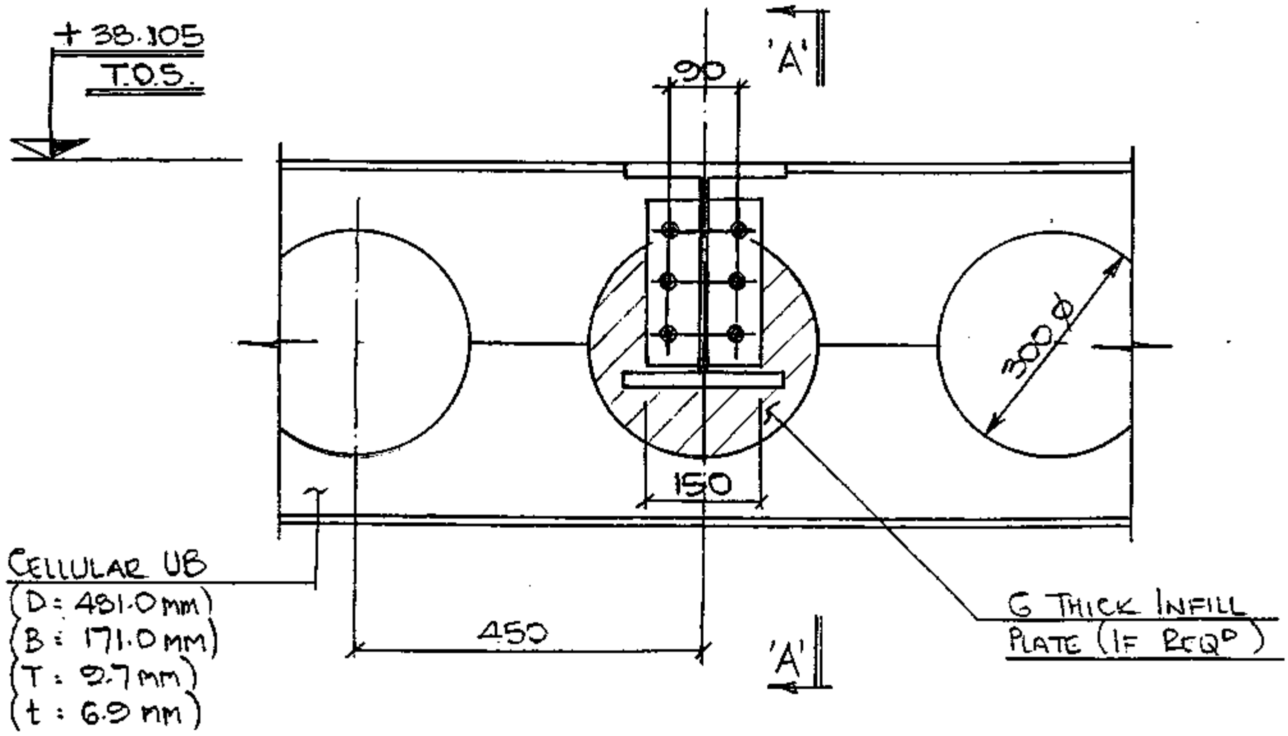
BY KB DATE 03/98

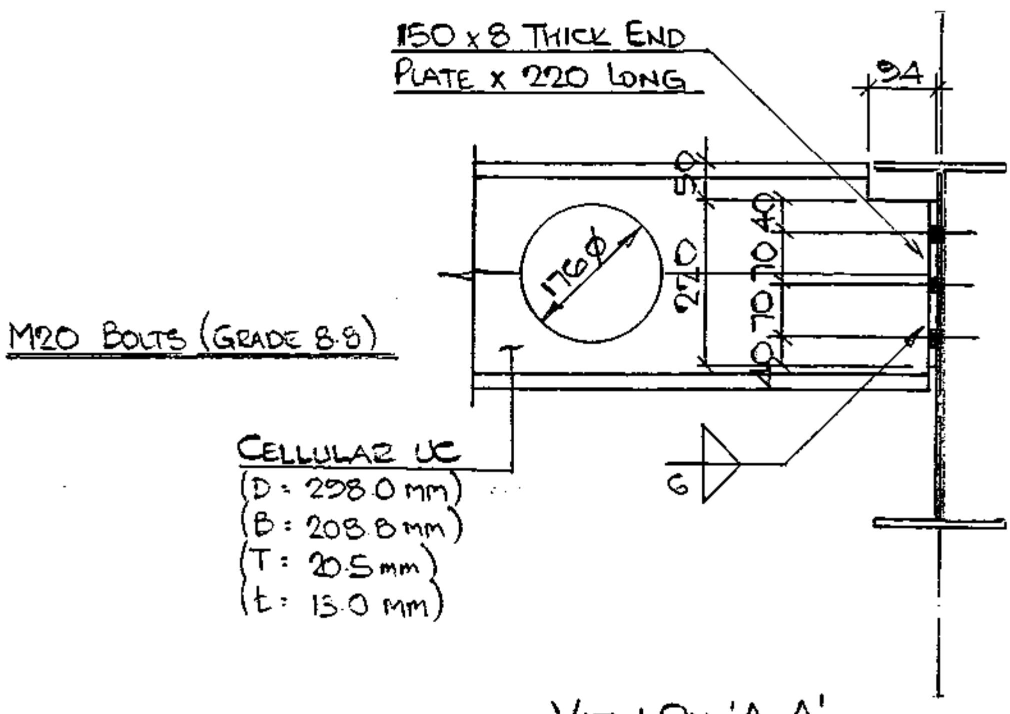
EX. DATE

BEAM B2-7 TO BEAM B1-7

298 x 203 x 86 CELLULAR UC TO 481 x 171 x 45 CELLULAR UB

END REACTION = 260 KN





VIEW ON 'A-A'

BEAM B3-7 TO BEAM B7-7

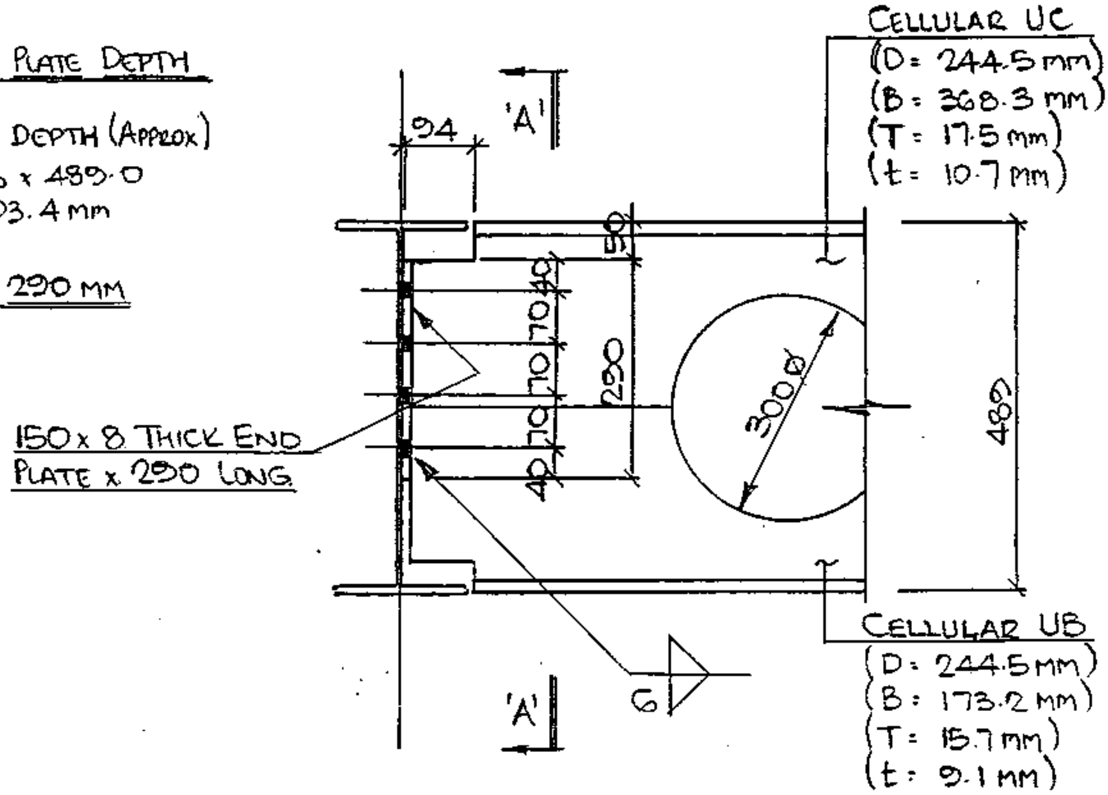
489 HYBRID CELLULAR BEAM TO 493 x 171 x 67 CELLULAR UB

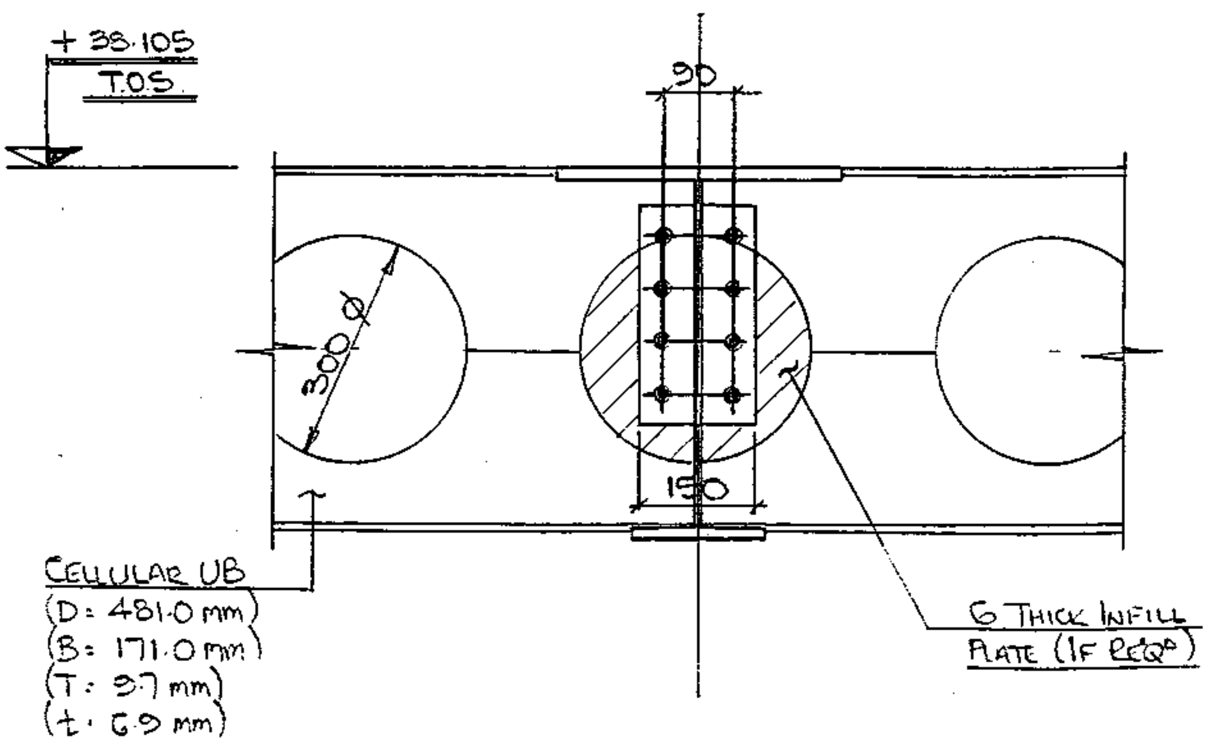
END REACTION = 500 KN

END PLATE DEPTH

MIN DEPTH (APPROX)
= 0.6 x 489.0
= 293.4 mm

SAY 290 MM





(CN ~ 17)

DESIGN OF BOLTS

TRY : — 8 No ~ M20 BOLTS (GRADE 8.8)

END REACTION = 500 KN

LOAD PER BOLT = 500 ÷ 8 = 62.5 KN

BOLT SHEAR, Ps = 245 × 375 ÷ 10^3 = 91.8 KN

BOLT BEARING ; Pbb = 1035 × 20 × 6 ÷ 10^3 = 124.2 KN

PLY BEARING ; Pbs

(a) d × t × pbs = 20 × 6 × 550 ÷ 10^3 = 66.0 KN

(b) ½ × e × t × pbs = ½ × 40 × 6 × 550 ÷ 10^3 = 66.0 KN

Pbs = 66.0 KN

BOLT CAPACITY = 66.0 KN > 62.5 KN OK

USE : — 8 No ~ M20 BOLTS (GRADE 8.8)

DESIGN OF END PLATE

TRY : — 8 THICK PLATE

END REACTION = 500 KN

SHEAR LOAD PER LINE OF BOLTS = 0.5 × 500 = 250 KN

Av = 0.9 × 8 × 290 = 2088 mm^2

Avnet = 2088 − (4 × 22 × 8) = 1384 mm^2

0.6 × Py × Av = 0.6 × 355 × 2088 ÷ 10^3 = 444 KN

0.5 × Us × Avnet = 0.5 × 490 × 1384 ÷ 10^3 = 339 KN

Pv = 339 KN > 250 KN OK

(CN ~ 17)

$A_{VI} = 250 \times 8 = 2000\ mm^2$

$A_{TEFF} = (30 - \{0.5 \times 22\}) \times 8 = 152\ mm^2$

$P_{VB} = (0.6 \times P_y \times A_{VI}) + (0.5 \times U_s \times A_{TEFF})$

$= (0.6 \times 355 \times 2000 \div 10^3) + (0.5 \times 490 \times 152 \div 10^3)$

$P_{VB} = \underline{463\ KN > 250\ KN}$ OK.

USE :– 150 x 8 THICK END PLATE x 290 LONG

DESIGN OF WELD FOR END PLATE TO BEAM WEB

LENGTH OF END PLATE : 290 mm

EFFECTIVE WELD LENGTH = $290 - (2 \times 10) = 270$ mm

LOAD PER mm RUN OF WELD = $500 \div (2 \times 270) = 0.93$ KN/mm

WELD STRENGTH (GRADE 50) = 255 N/mm²

WELD SIZE REQ^D = $\dfrac{0.93 \times 10^3}{0.70 \times 255}$ = 5.2 mm → 6 LEG F.W (MIN^M/)

USE :– 6 LEG FILLET WELD

CHECK BEAM WEB AT END PLATE

SHEAR LOAD = 500 KN

EFFECTIVE WEB DEPTH = 290 mm

$A_V = 0.9 \times 290 \times 9.1 = 2375\ mm^2$

$F_V = 0.6 \times 355 \times 2375 \div 10^3$

$= \underline{505\ KN > 500\ KN}$ OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

SERIES SHEET 4 OF 4

SHT. No. C95 REV.

BY KB DATE 09/98

EX. DATE

(CN ~ 17)

CHECK BEAM SECTION AT END OF NOTCH

SHEAR LOAD = 500 KN

LEVER ARM = 94 mm

BENDING MOMENT = 500 x 94 = 47000 KNmm

Z (CONSERVATIVE) = 9.1 x 389^2 ÷ 6 = 229503 mm^3

Mc = 345 x 229503 ÷ 10^3

= 79178 KNmm > 47000 KNmm OK

SECTION SATISFACTORY

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B3-7 TO BEAM B6-7

SERIES SHEET 1 OF 1

SHT. No. C95A REV.

BY KB DATE 09/98

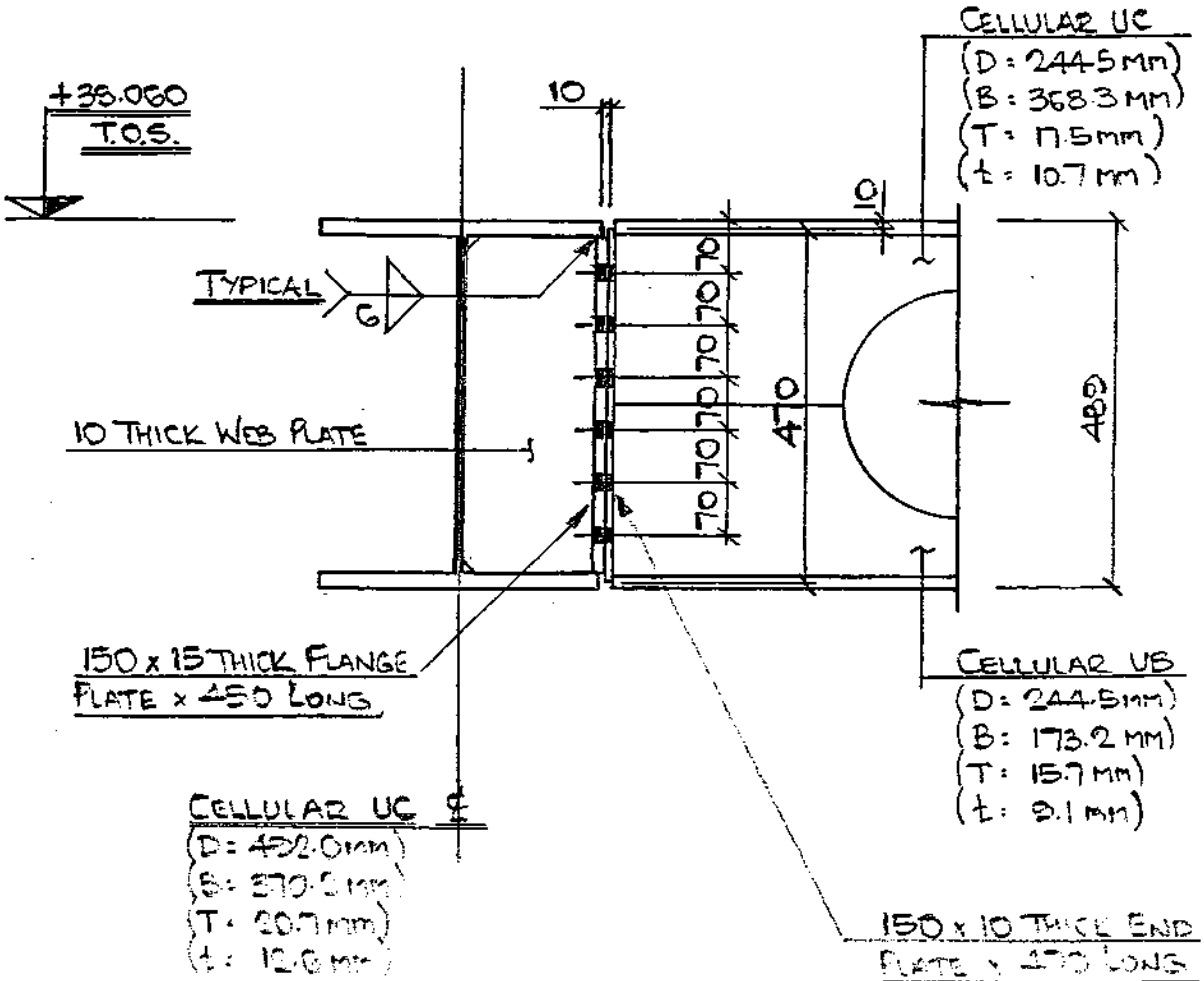
EX. DATE

(CN-17)

BEAM B3-7 TO BEAM B6-7

485 HYBRID CELLULAR BEAM TO 492 x 368 x 153 CELLULAR UC

END REACTION = 500 KN



M20 BOLTS (GRADE 8.8) @ 90 c/s

6 LEG FILLET WELDS (U.N.O.)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH. 1 OF 1

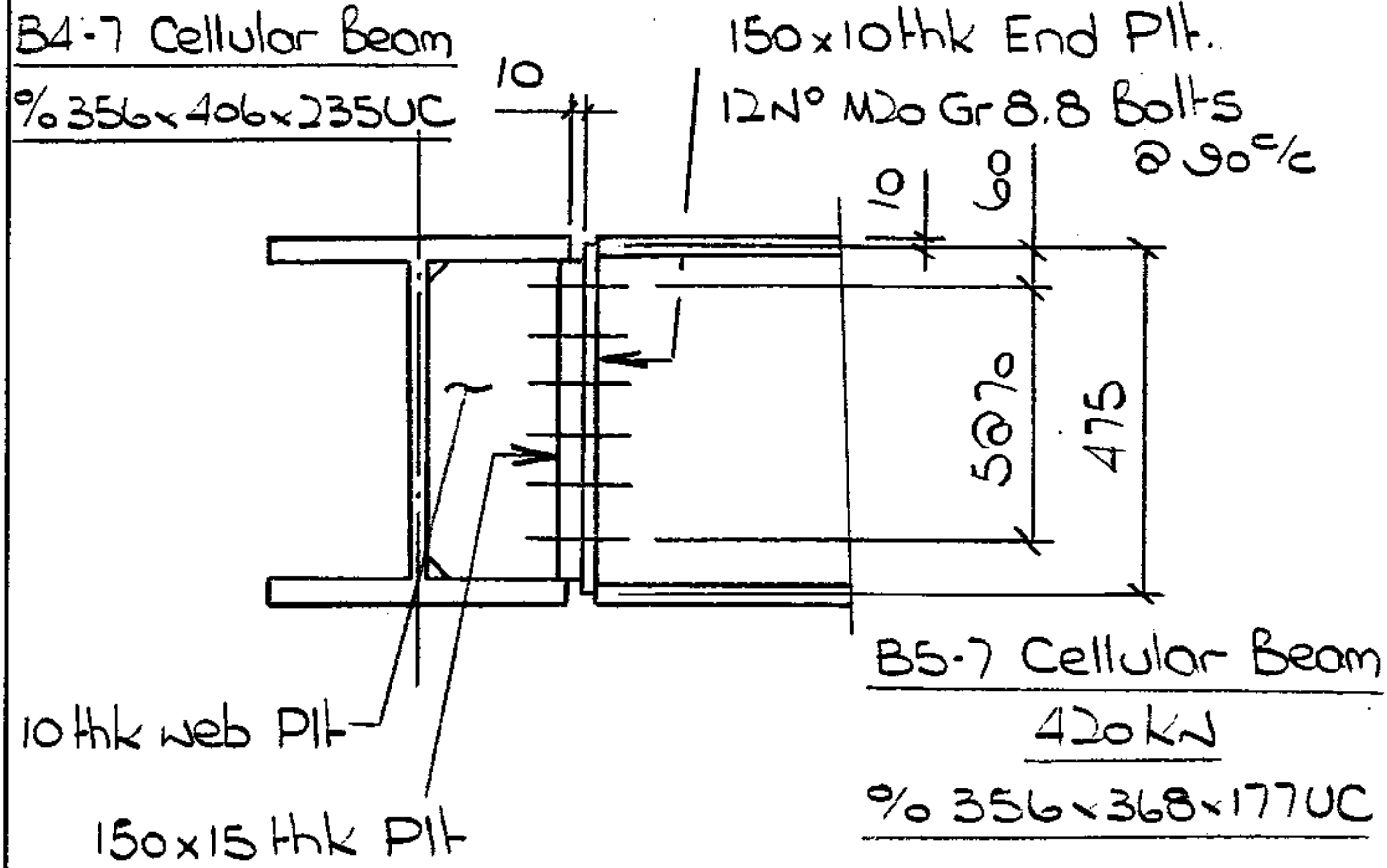
SHT. No. C95B REV.

BY KM DATE Oct 98

EX. DATE

CN17

Conn of Beam B5-7 to B4-7



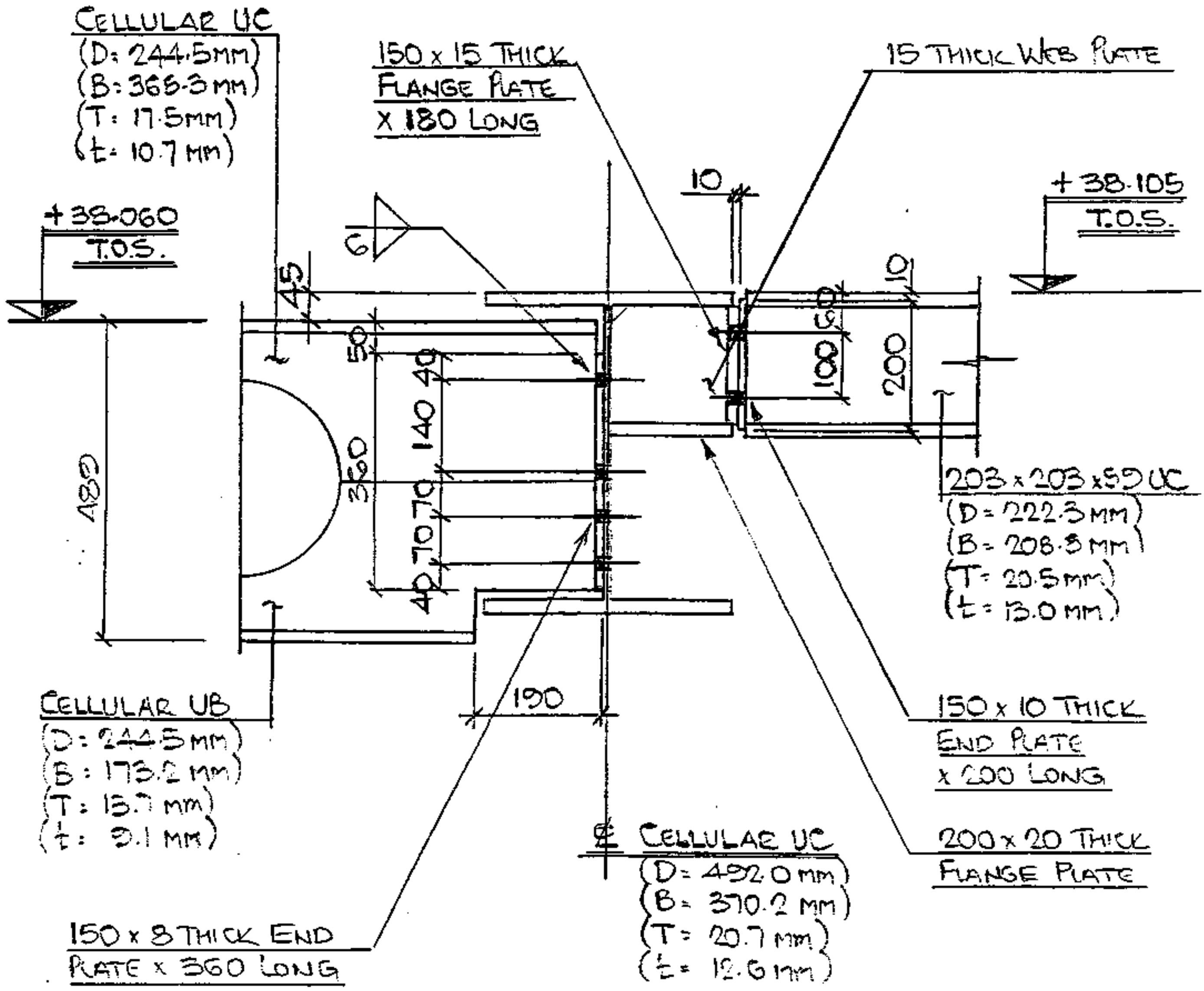
All welds 6mm FW

BEAM B3-7 & BEAM B9-7 TO BEAM B6-7

489 HYBRID CELLULAR BEAM & 203 x 203 x 56 UC TO 492 x 365 x 153 CELLULAR UC

END REACTIONS : —

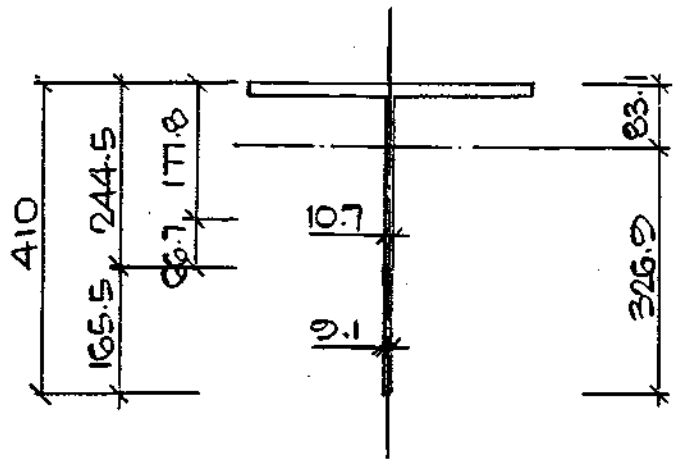
BEAM B3-7 = 500 KN ; BEAM B9-7 = 200 KN



M20 BOLTS (GRADE 8.8) @ 90 %cs

6 LEG FILLET WELD (U.N.O.)

CHECK HYBRID CELLULAR SECTION



SHEAR AREA

$AV = (0.9 \times 244.5 \times 10.7) + (0.9 \times 165.5 \times 9.1)$

$= 3710\ mm^2$

AREA OF SECTION $= (0.5 \times 164.9) + (6.67 \times 1.07) + (16.55 \times 0.91)$

$= 82.45 + 7.14 + 15.10 = \underline{104.69\ cm^2}$

TO FIND NEUTRAL AXIS ~ TAKE MOMENTS ABOUT TOP OF FLANGE

$104.69\ \bar{x} = (82.45 \times 2.73) + (7.14 \times 21.115) + (15.10 \times 32.725)$

$\bar{x} = 870 \div 104.69 = \underline{8.31\ cm}$

$I_{NA} = 14510000 + \left(\frac{10.7 \times 66.7^3}{12}\right) + \left(\frac{9.1 \times 165.5^3}{12}\right) + (8245 \times 55.8^2)$

$+ (714.0 \times 128.05^2) + (1510.0 \times 244.15^2) = 145601389\ mm^4$

$Z_{NA} = 145601389 \div 326.9 = 445400\ mm^3$

$P_y = 345\ N/mm^2 \quad (T > 16.0\ mm)$

a) SHEAR

$P_V = 0.6 \times A_V \times P_y$

$= 0.6 \times 3710 \times 345 \div 10^3$

$= \underline{767\ kN} > 500\ kN$ OK

b) MOMENT AT END OF NOTCH

BM = 500 x 190 = 95000 KN mm

$M_c = P_y \times Z$

$= 345 \times 445400 \div 10^3$

$= 153663$ KNmm > 95000 KNmm OK

SECTION SATISFACTORY

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B5-7 TO COLUMN C8

SERIES SHEET 1 OF 1

SHT. No. C97 REV.

BY KB DATE 08/98

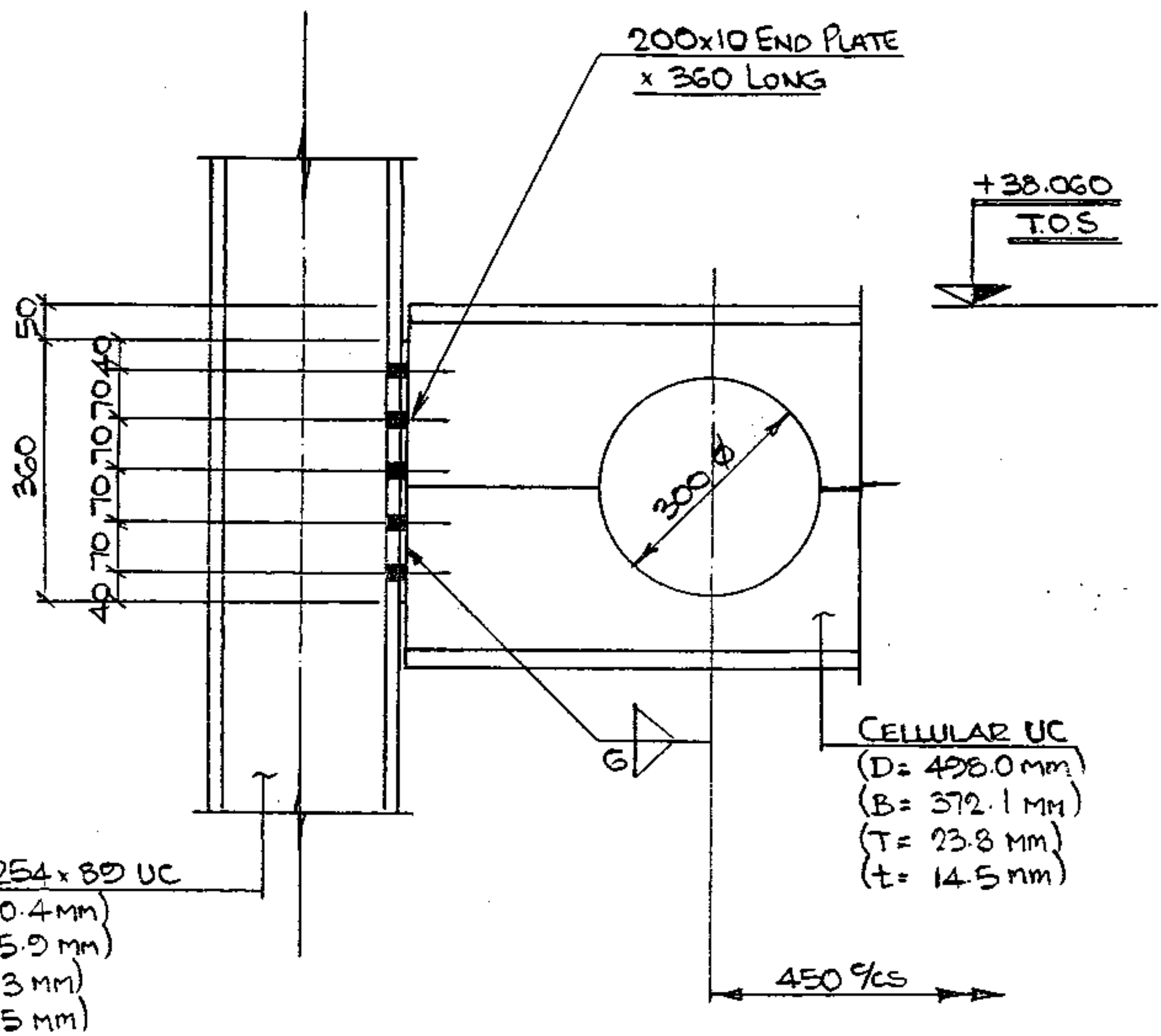
EX. DATE

(CN~17)

BEAM B5-7 TO COLUMN C8

498 x 368 x 177 CELLULAR UC TO 254 x 254 x 89 UC

END REACTION : 420 KN.



M20 BOLTS (GRADE 8.8)
@ 140 c/cs

END PLATE DEPTH

MIN DEPTH : 0.6 x 498.0 : 298.8 → SAY 360 mm

Conn of Beam B5-7 to B6-7

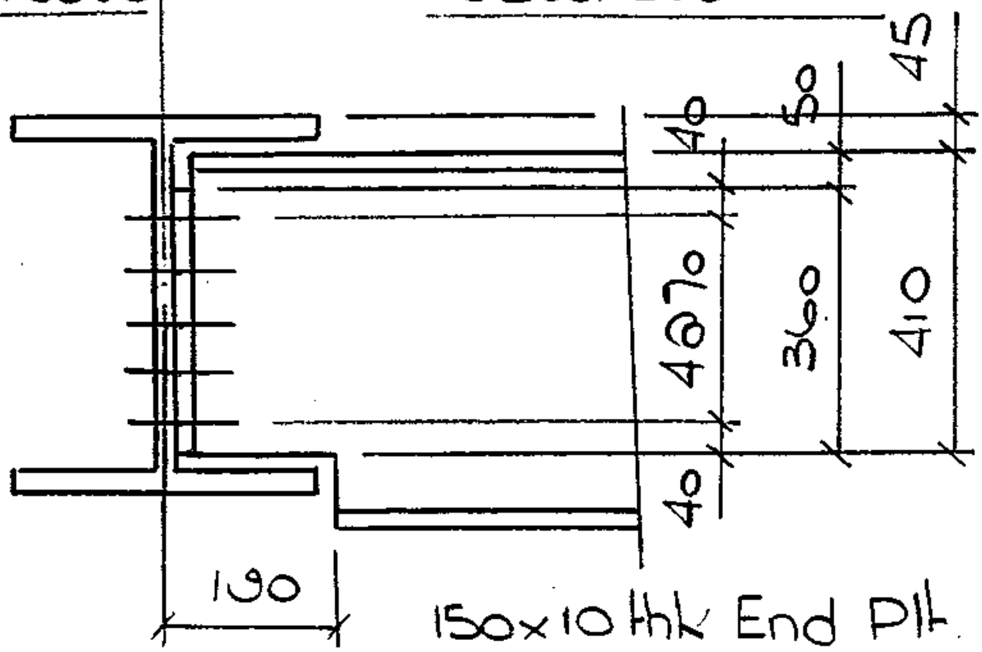
Cellular Beam

% 356×368×153UC

D=492

B5-7 Cellular Beam 420kN

% 356×368×177UC

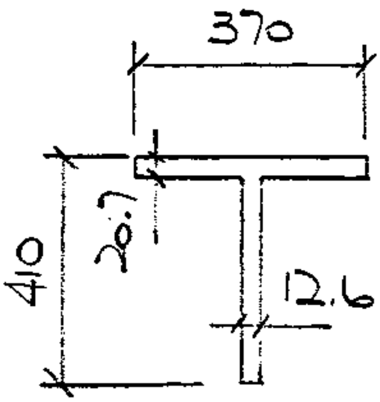


150×10 thk End Plt

6FW

10N° M20 Gr 8.8 Bolts

Check Beam for Notch



$A = 12\,564\,mm^2$

$\bar{x} = 90mm \qquad I = 18\,761\ cm^3$

i) Shear

$A_v = 0.9 \times 410 \times 12.6 = 4649\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 4649 = \underline{990kN} > 420kN$

ii) Bending

$M = 420 \times 190 \times 10^{-3} = \underline{79.8\ kNm}$

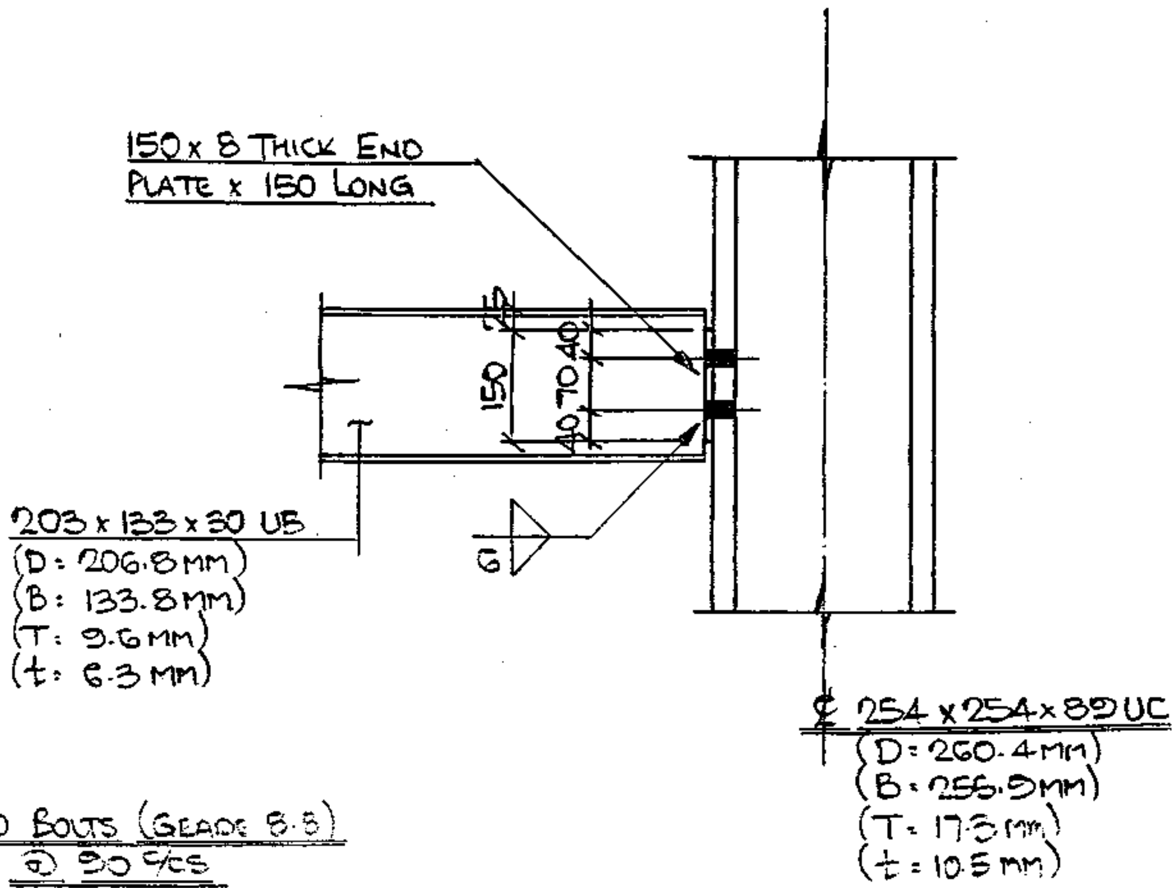
$P_b = \dfrac{18\,761 \times 10^4}{(410-90)} \times 355 \times 10^{-6}$

$= 208kNm > 79.8kN \quad \therefore$ Beam OK

BEAM B8-7 TO COLUMN C8

203 x 133 x 30 UB TO 254 x 254 x 89 UC

END REACTION = 170 KN.



M20 BOLTS (GRADE 8.8)
@ 90 c/cs

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES SHEET 1 OF 2

SUBJECT CONNECTION DESIGN — SHT. No. C98A REV.

BEAM B8-7 TO BEAM B1-7 — BY KB DATE 10/98

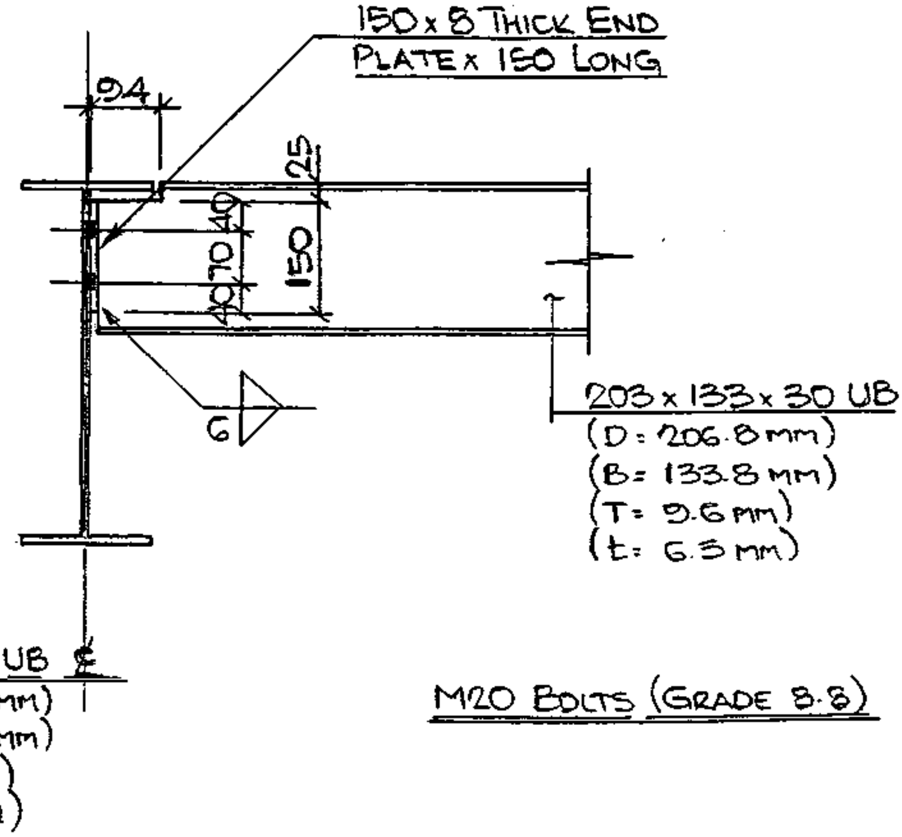
EX. DATE

(CN-17)

BEAM B8-7 TO BEAM B1-7

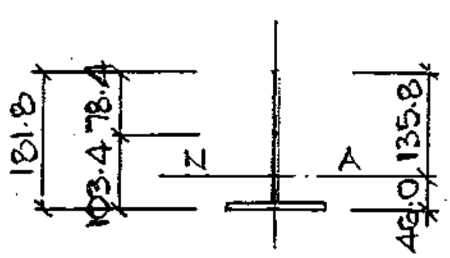
203 x 133 x 30 UB TO 481 x 171 x 45 CELLULAR UB

END REACTION = 170 KN.



M20 BOLTS (GRADE 8.8)

CHECK BEAM SECTION



AREA OF SECTION

$= (0.5 \times 38.0) + (7.84 \times 0.63)$

$= 19.00 + 4.94 = \underline{23.94\ cm^2}$

TO FIND NEUTRAL AXIS ~ TAKE MOMENTS ABOUT BOTTOM

$23.94\bar{x} = (19.00 \times 21.0) + (4.94 \times 142.6)$

$\bar{x} = 1103 \div 23.94 = \underline{46.0\ mm}$

$I_{xx} = 1526000 + \left(\frac{6.3 \times 78.4^3}{12}\right) + (1900 \times 25.0^2) + (494 \times 96.6^2)$

$= 7576283 \text{ mm}^4$

$Z_{xx} = 7576283 \div 135.8 = 55790 \text{ mm}^2$

(a) <u>SHEAR</u>

$A_V = 0.9 \times 181.8 \times 6.3 = 1030 \text{ mm}^2$

$P_V = 0.6 \times 355 \times 1030 \div 10^3$

$= \underline{219 \text{ KN} > 170 \text{ KN}}$ OK

(b) <u>MOMENT</u>

$M_c = 355 \times 55790 \div 10^3 = 19805 \text{ KNmm}$

$BM = 170 \times 94$

$= \underline{15980 \text{ KNmm} < 19805 \text{ KNmm}}$ OK

<u>SECTION SATISFACTORY</u>

Kvaerner Cleveland Bridge Ltd.

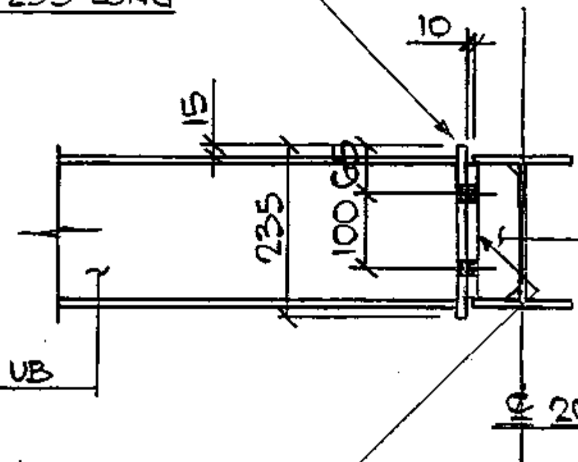
O/No. 325 JOB CARLTON GARDENS | SERIES SHEET 1 OF 1
SUBJECT CONNECTION DESIGN. | SHT. No. C98B REV.
BEAM B8-7 TO BEAM B8-7 | BY KB DATE 09/98
(CN~17) | EX. DATE

BEAM B8-7 TO BEAM B8-7

203 x 133 x 30 UB TO 203 x 133 x 30 UB

END REACTION = 170 KN

150 x 10 THICK END PLATE x 235 LONG



6 THICK WEB PLATE

203 x 133 x 30 UB
(D = 206.8 mm)
(B = 133.8 mm)
(T = 9.6 mm)
(t = 6.3 mm)

℄ 203 x 133 x 30 UB

150 x 15 THICK END PLATE, x 185 LONG

M20 BOLTS (GRADE 8.8) @ 90 c/cs

6 LEG FILLET WELD (U.N.O.)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 1

SHT. No. C98C REV.

BY KM DATE Oct 98

EX. DATE

CN17

Beam B8·7 to B4·7 (B8·7 to B6·7 Similar)

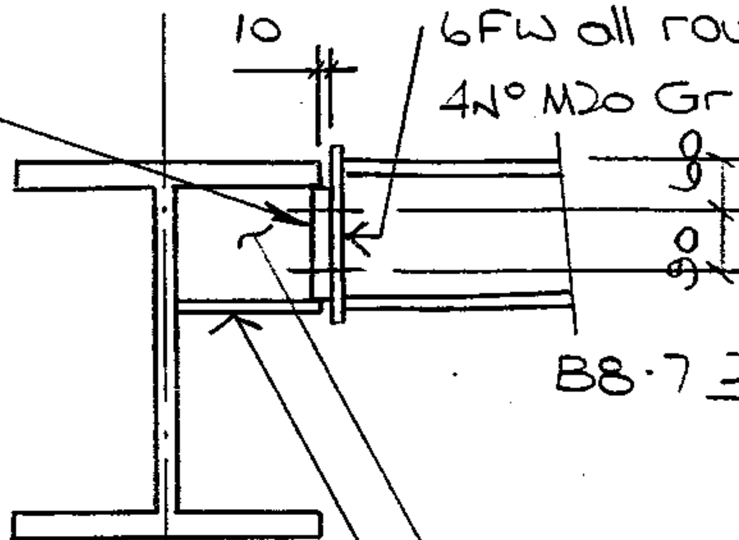
150x15thk Plt.

150x10 thk End Plt

10

6FW all round

4N° M20 Gr 8.8 Bolts @ 90 c/c



B8·7 203x133x30 UB

170kN

D=207

B=134

T= 9.6

t= 6.3

Cellular Beam

c/o 356x406x235UC

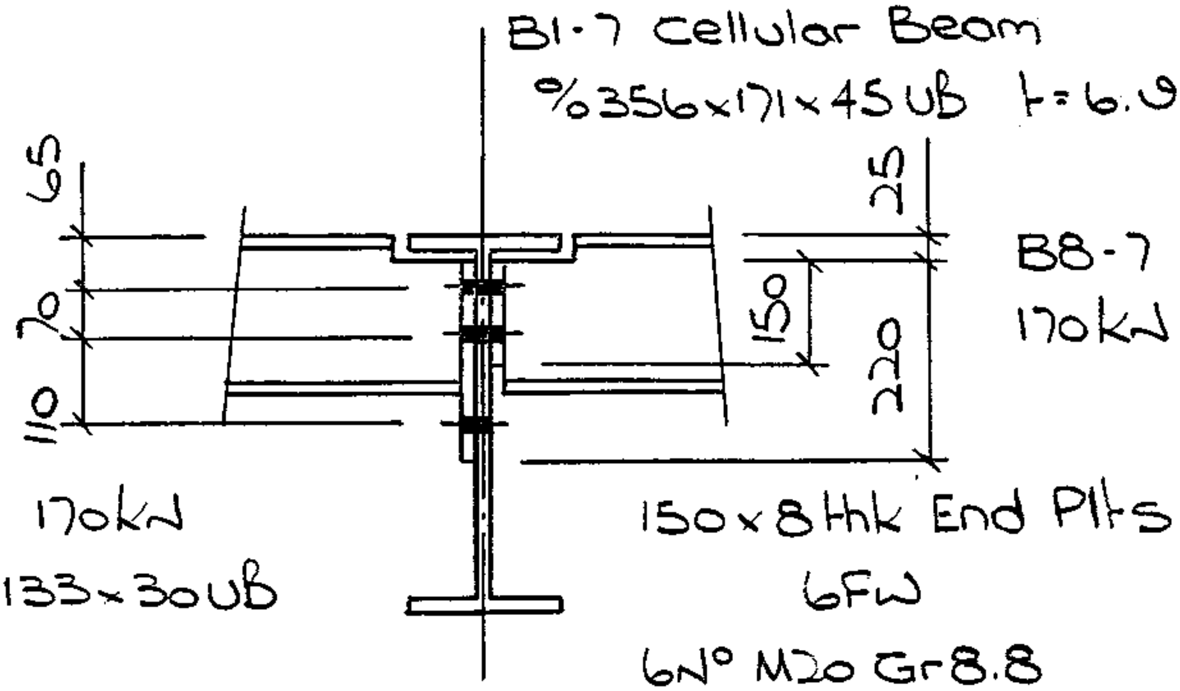
T= 30.2

10 thk web plt

150x10thk flg plt

All weld 6FW

Conn of Beams B8-7 | B8-7 to B1-7



CONNECTION SIMILAR TO C98A

Check Bolt bearing on Beam B1-7 web

Load per bolt = $170/6 + 170/4 = \underline{71\text{ kN}}$

Allowable bearing $= 6.9 \times 20 \times 550 \times 10^{-3}$

$= \underline{76\text{ kN}} > 71\text{ kN}$

$\therefore$ Bolts ok

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAMS B8-7 TO BEAM B1-7

(CN-17)

SERIES SHEET 1 OF 1

SHT. No. C98E REV.

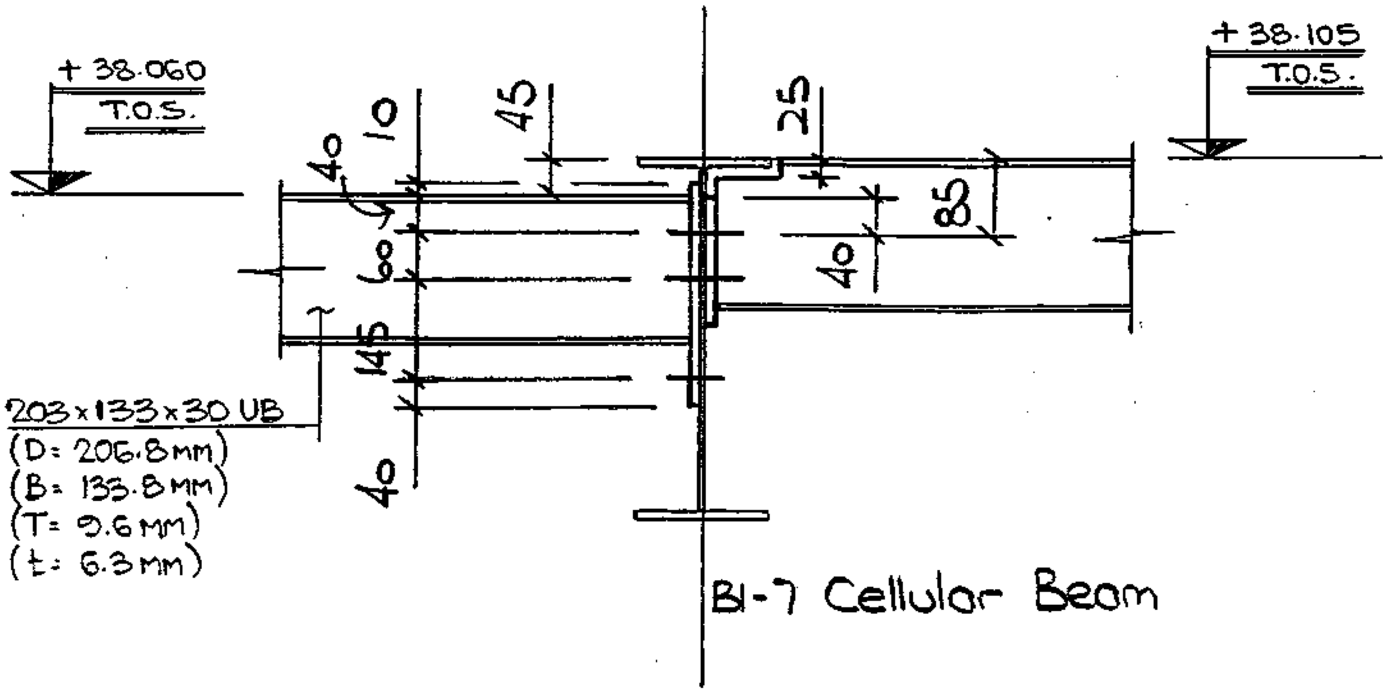
BY KB DATE 09/98

EX. DATE

BEAMS B8-7 TO BEAM B1-7

203 x 133 x 30 UB TO 481 x 171 x 45 CELLULAR UB

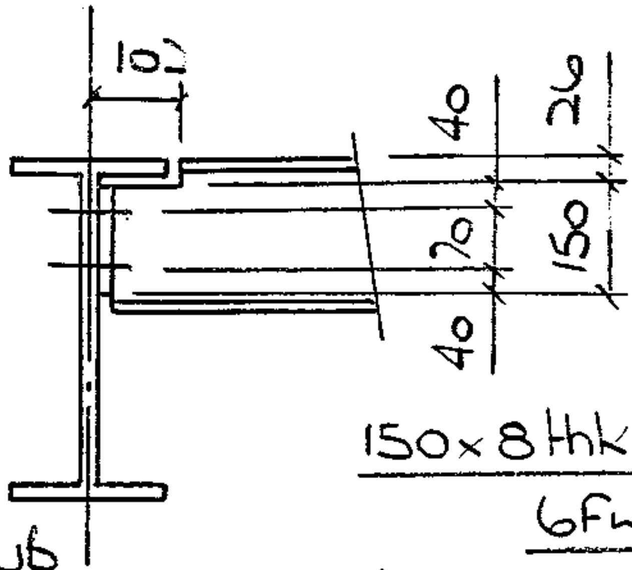
END REACTION = 170 KN.



150x8thk End Plts - 6FW

6N° M20 Gr. 8.8 Bolts @ 90 c/c

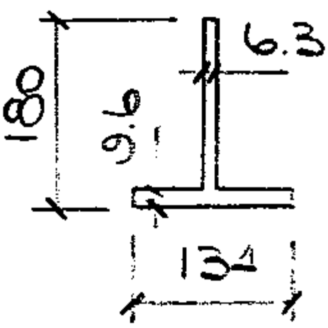
Conn of Beam B8-7 to B10-7



203x133x30 UB
170kN

150x8 thk End Plt
6FW
4No M20 Gr 8.8 Bolts

Check Beam for Notch



$A = 2360\,mm^2$

$\bar{x} = 46\,mm \qquad I = 734\,cm^4$

i) Shear

$A_v = 0.9 \times 180 \times 6.3 = 1021\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1021 = \underline{217\,kN} > 170\,kN$

ii) Bending

$M = 170 \times 102 \times 10^{-3} = \underline{17.3\,kNm}$

$P_b = \frac{734 \times 10^4}{(180-46)} \times 355 \times 10^{-6} = \underline{19.4\,kNm} > 17.3\,kNm$

∴ Beam OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

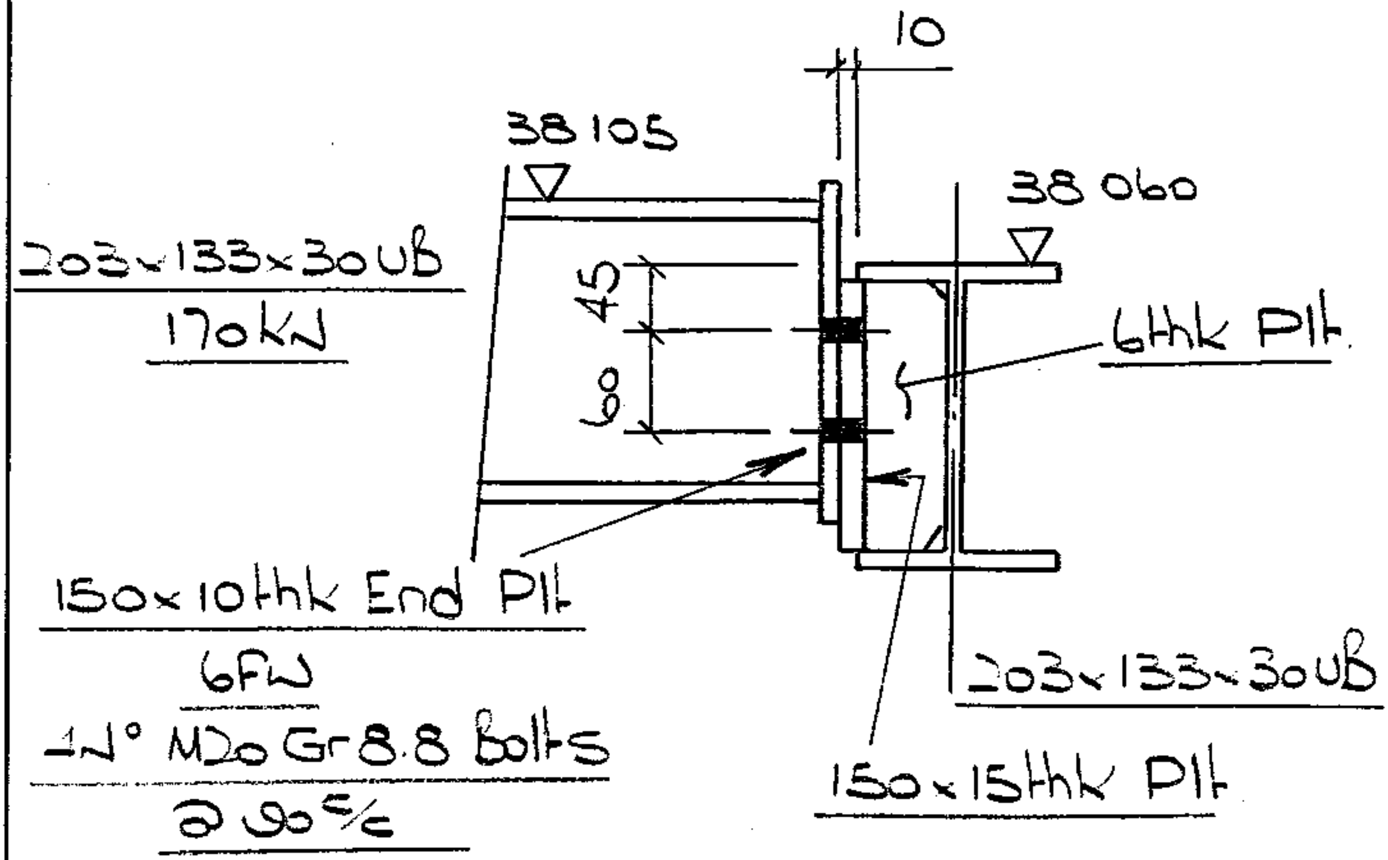
SERIES

SHT. No. C98G REV.

BY K M DATE Oct 98

EX. DATE

Conn of B8.7 to B8.7



All welds 6mm FW

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 2

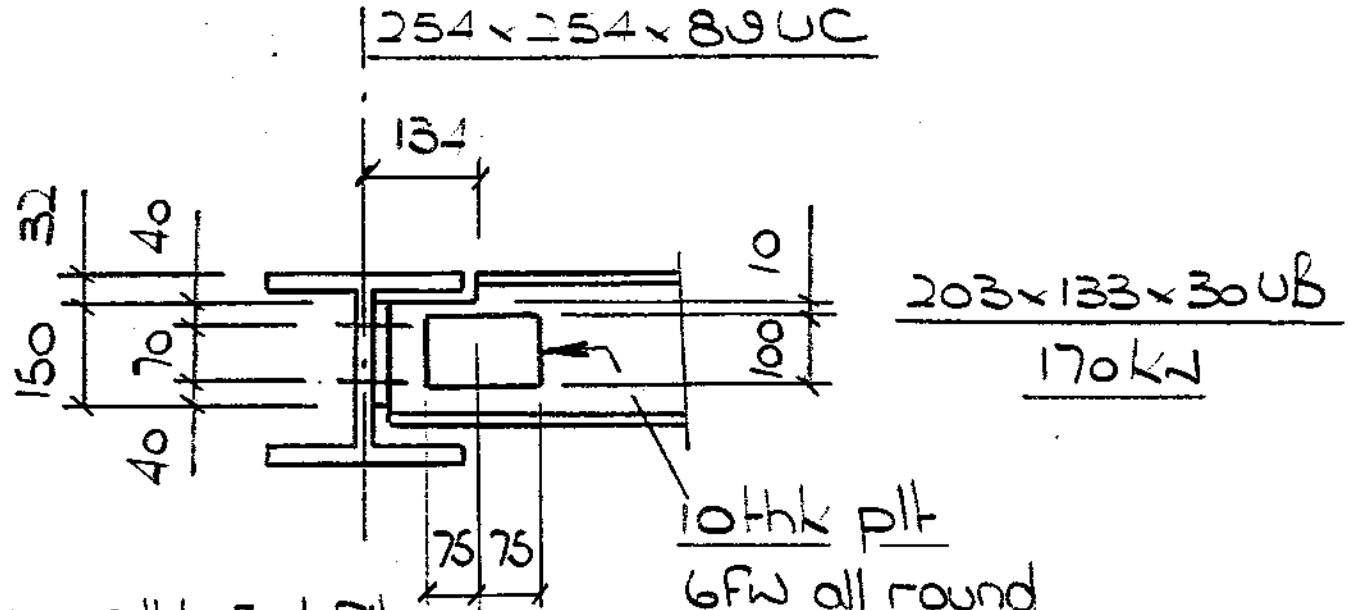
SHT. No. C98H REV.

BY KM DATE Nov 98

EX. DATE

CN17

Conn of B8·7 to 254 UC

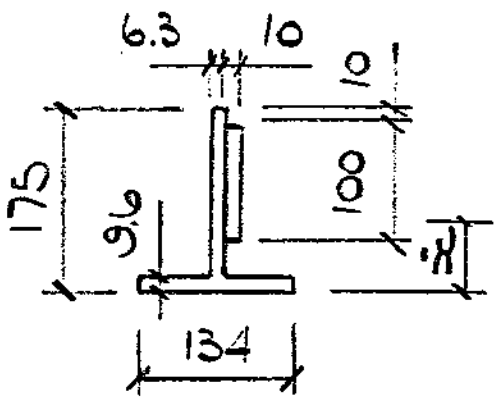


150×8thk End Plt.

6FW

4N° M20 Gr8.8 Bolts

Check Beam for Notch



$A = 165.4 \times 6.3 + 134 \times 9.6 + 100 \times 10$

$= 1042 + 1286 + 1000 = 3328 mm^2$

$\bar{x} = \dfrac{1042 \times 92.3 + 1286 \times 4.8 + 1000 \times 115}{3328}$

$= \underline{65 mm}$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 2 OF 2

SHT. No. C98H REV.

BY KM DATE Nov 98

EX. DATE

CN17

B8·7 to 254UC cont

Check Beam for Notch cont

$$
\begin{aligned}
I_{NA} &= 6.3\times175^3/12 &&= 2\,813\,672\\
&+ 10\times100^3/12 &&= 833\,333\\
&+ 1042\times27.3^2 &&= 776\,592\\
&+ 1286\times60.2^2 &&= 4\,660.515\\
&+ 1000\times50^2 &&= 2\,500\,000\\
& && \underline{11\,584\,112 \text{ mm}^4}
\end{aligned}
$$

$M = 170\times134\times10^{-3} = \underline{22.8 \text{ kNm}}$

$P_b = 11\,584\,112/(175-65) \times 355\times10^{-6}$

$= \underline{37.4 \text{ kNm}} > 22.8 \text{ kNmm}$

$\therefore$ Beam OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

BEAM B9-7 TO COLUMN C3 & C8

(CN-17)

SERIES SHEET 1 OF 4

SHT. No. C99 REV.

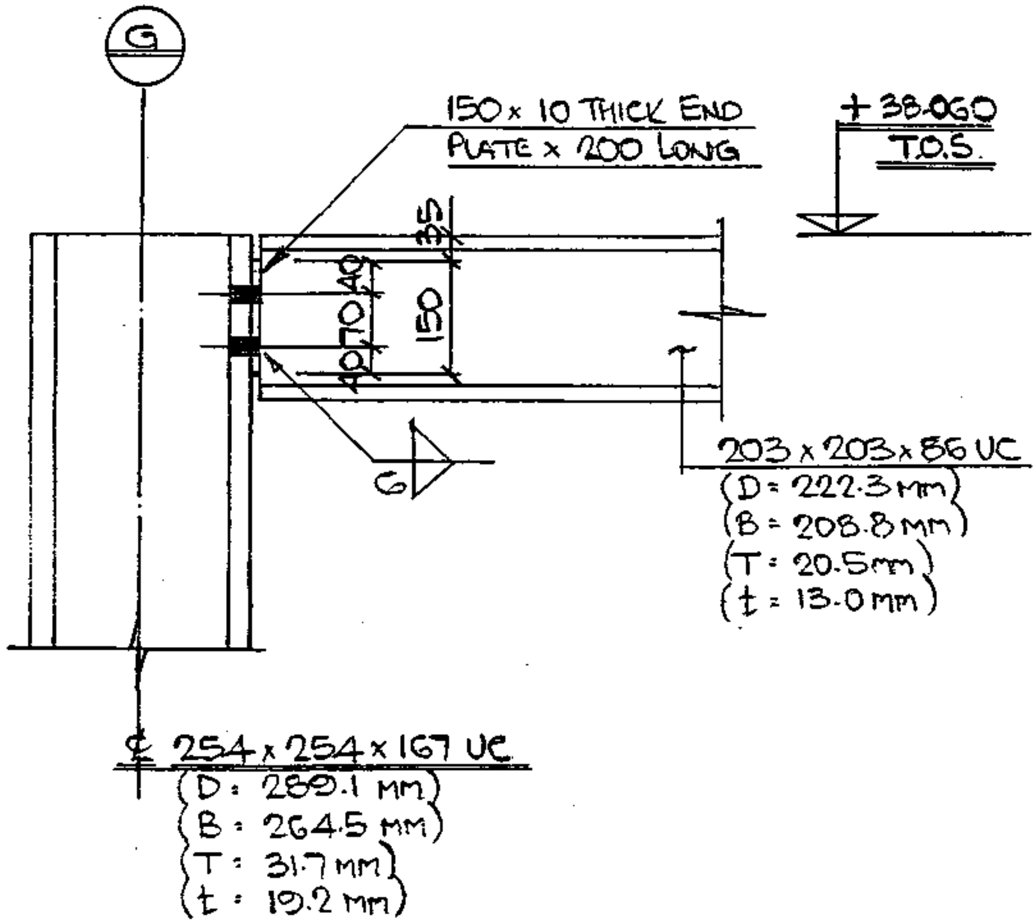
BY KB DATE 03/98

EX. DATE

<u>BEAM B9-7 TO COLUMN C3</u> <u>(BEAM B9-7 TO COLUMN C8 ~ SIMILAR)</u>

<u>203 x 203 x 86 UC TO 254 x 254 x 167 UC</u>

END REACTION = 200 KN



<u>M20 BOLTS (GRADE 8.8)</u>
<u>@ 140 C/CS</u>

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

SERIES SHEET 2 OF 4

SHT. No. C99 REV.

BY KB DATE 03/98

EX. DATE

(CN-17)

DESIGN OF BOLTS

TRY :— 4 No ~ M20 BOLTS (GRADE 8.8)

LOAD PER BOLT : 200 ÷ 4 = 50.0 KN

BOLT SHEAR ; Ps : 245 x 375 ÷ 10³ = 91.8 KN

BOLT BEARING ; Pbb = 1035 x 20 x 10 ÷ 10³ = 207.0 KN

PLY BEARING ; Pbs

(a) d x t x pbs : 20 x 10 x 550 ÷ 10³ = 110.0 KN

(b) ½ x e x t x pbs : ½ x 40 x 10 x 550 ÷ 10³ = 110.0 KN

Pbs = 110.0 KN

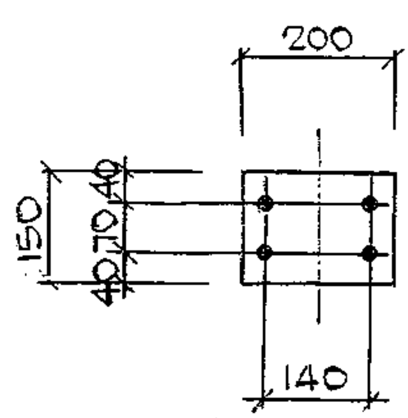
BOLT CAPACITY : 91.8 KN > 50.0 KN OK

USE :— 4 No ~ M20 BOLTS (GRADE 8.8)

DESIGN OF END PLATE

TRY :— 10 THICK PLATE

SHEAR LOAD PER LINE OF BOLTS : 0.5 x 200 = 100 KN



A_V = 0.9 x 150 x 10 = 1350 mm²

A_{VNET} = 1350 − (2 x 10 x 22) = 910 mm²

A_{VI} = 10 x 110 = 1100 mm²

A_{TEFF} = (30 − {0.5 x 22}) x 10 = 190 mm²

(CN~17)

$0.6 \times P_y \times A_V = 0.6 \times 355 \times 1350 \div 10^3 = 257$ KN

$0.5 \times U_s \times A_{V1} = 0.5 \times 490 \times 910 \div 10^3 = 222$ KN

$P_V = 222$ KN $\gg 100$ KN OK

$P_{VB} = (0.6 \times P_y \times A_{V1}) + (0.5 \times U_s \times A_{TEFF})$

$= (0.6 \times 355 \times 1100 \div 10^3) + (0.5 \times 490 \times 150 \div 10^3)$

$P_{VB} = 280$ KN $\gg 100$ KN OK

USE :— 200 x 10 THICK END PLATE x 150 LONG

DESIGN OF WELD

LENGTH OF END PLATE : 150 mm

EFFECTIVE WELD LENGTH : $150 - (2 \times 10) = 130$ mm

LOAD PER mm RUN OF WELD : $200 \div (2 \times 150) = 0.67$ KN/mm

WELD STRENGTH (GRADE 50) : 255 N/mm²

WELD SIZE REQD $= \dfrac{0.67 \times 10^3}{0.7 \times 255} = 3.8$ mm $\longrightarrow$ 6 LEG F.W. (MIN^m/)

USE : — 6 LEG FILLET WELD

CHECK BEAM WEB AT END PLATE

SHEAR LOAD = 200 KN

EFFECTIVE WEB DEPTH = 150 mm

$A_v = 0.9 \times 150 \times 13.0 = 1755 \text{ mm}^2$

$P_v = 0.6 \times 345 \times 1755 \div 10^3$

$= \underline{\underline{363 \text{ KN} > 200 \text{ KN}}}$ OK

BEAM WEB SATISFACTORY

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 1

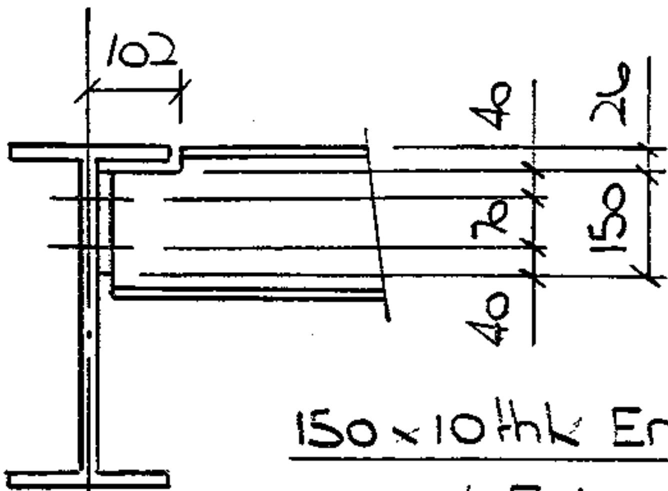
SHT. No. C99A REV.

BY KM DATE Oct. 98

EX. DATE

CN17

Conn of Beam B9.7 to B10.7



457×191×74 UB

203×86 UC

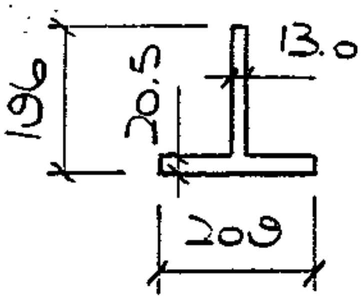
200kN

150×10thk End Plt.

6FW.

4N° M20 Gr 8.8 Bolts

Check Beam for Notch



$A = 6566\ mm^2$

$\bar{x} = 44\ mm \qquad I = 2015\ cm^4$

i) Shear

$A_v = 0.9 \times 196 \times 13 = 2293\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 2293$

$= \underline{488\ kN} > 200\ kN$

ii) Bending

$M = 200 \times 102 \times 10^{-3} = 20.4\ kNm$

$P_b = \dfrac{2015 \times 10^4}{(196-44)} \times 355 \times 10^{-6} = \underline{47\ kNm} > 20.4\ kNm$

∴ Beam OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN
BEAM B10-7 TO COLUMN C8

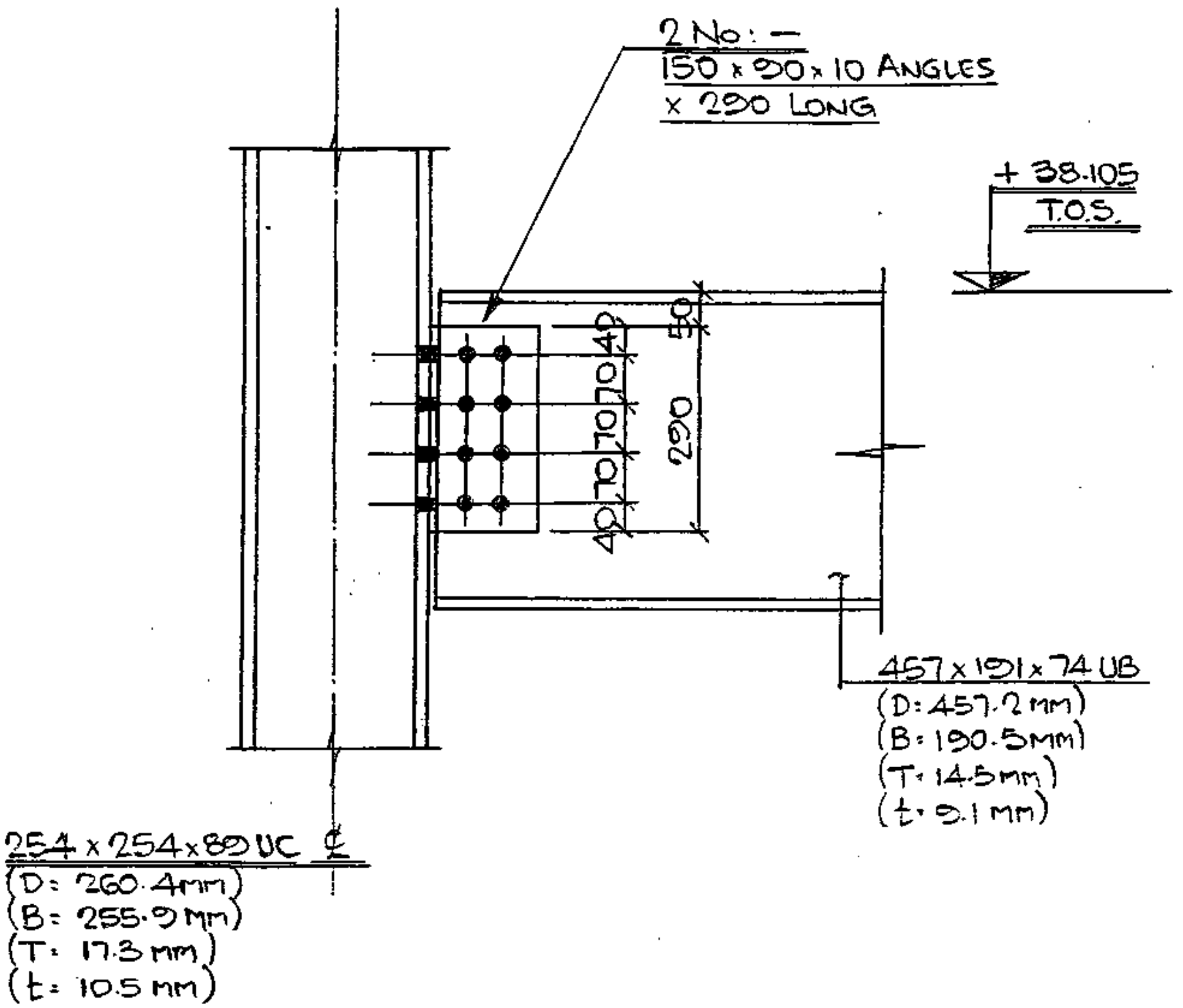
(CN~17)

SERIES SHEET 1 OF
SHT. No. C100 REV.
BY KB DATE 09/98
EX. DATE

BEAM B10-7 TO COLUMN C8

457 x 191 x 74 UB TO 254 x 254 x 89 UC

END REACTION : 230 KN



M20 BOLTS (GRADE 8.8)

CONNECTION REFERENCE : —

EX BCSA/SCI PUBLICATION : —
JOINTS IN SIMPLE CONSTRUCTION
VOLUME 2 : PRACTICAL APPLICATIONS

CAPACITY : 463 KN

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 4

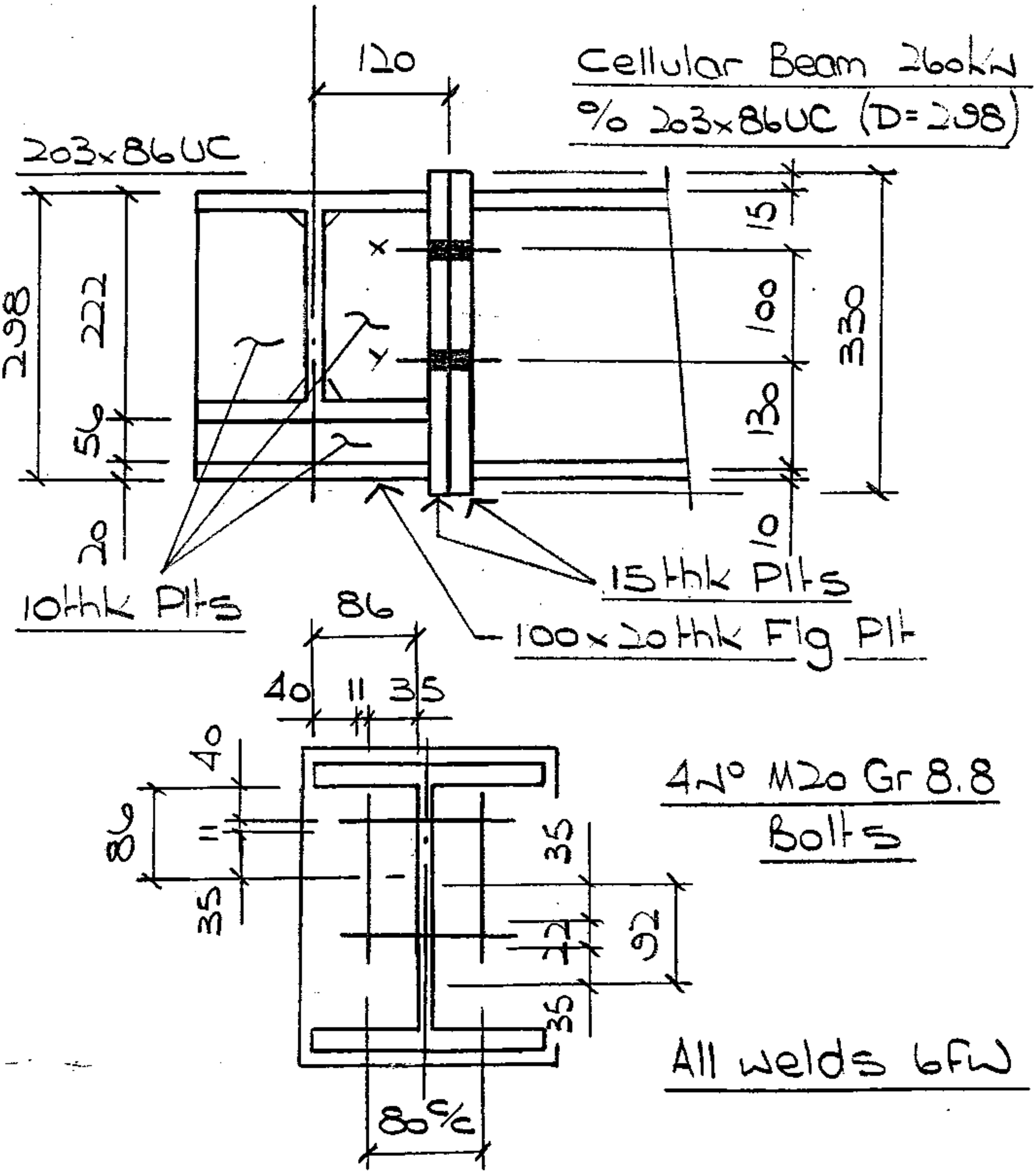
SHT. No. C101 REV.

BY KM DATE Oct 98

EX. DATE

CN17

Beam B2-7 to B9-7



B2.7 to B9.7 cont.

i/ Bolts

a. Shear $260/4 = 65\text{kN} < 92\text{kN}$

b. Tension

$M = 260 \times 120 \times 10^{-3} = 31.2\text{kNm}$

$Z = \dfrac{2(13^2 + 23^2)}{23} = 60.7\text{cm}^3$

Tension per bolt

row x $= \dfrac{31.2 \times 10^2}{60.7} = 51\text{kN} < 110\text{kN}$

row y $= \dfrac{31.2 \times 10^2}{60.7} \times 13/23 = 29\text{kN}$

Combined Tension + Shear

$65/92 + 51/110 = 1.17 < 1.4$

∴ Bolts OK

ii/ Check comp. force in Tee / Btm. flanges

Comp force $= 2(51 + 29) = 160\text{kN}$

100 × 20thk Tee Flg.

$P_b = 100 \times 20 \times 345 \times 10^{-3} = 690\text{kN} > 160\text{kN}$

∴ OK

B2.7 to B9.7 cont.

iii) End Plt.

a. Shear

$A_v = 0.9 \times 330 \times 15 = 4455\ mm^2$

$Av_{net} = 4455 - (2 \times 22 \times 15) = 3795\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 4455 = 949\ kN > 160/2\ kN$

$P_{vnet} = 0.5 \times 490 \times 10^{-3} \times 3795 = 930\ kN > 160/2\ kN$

b. Bending @ Bolt row x

$M = 51 \times 35 \times 10^{-3} = \underline{1.8\ kNm}$

$z = (86 + 86) \times 15^2/6 = 6450\ mm^3$

$P_b = 6450 \times 355 \times 10^{-6} = \underline{2.3\ kNm} > 1.8\ kNm$

c. Bending @ Bolt row y

$M = 29 \times 35 \times 10^{-3} = \underline{1.0\ kNm}$

$z = 92 \times 15^2/6 = 3450\ mm^2$

$P_b = 3450 \times 355 \times 10^{-6} = \underline{1.2\ kNm} > 1.0\ kNm$

$\therefore$ End Plt OK

B2.7 to B9.7 cont.

iv) End Plt. Weld 6FW

$$f_v = \frac{260 \times 10^3}{2 \times 298 \times 4.2} = \underline{104 \text{ N/mm}^2}$$

@ Bolt row x

$$f_h = \frac{51 \times 10^3}{2 \times 86 \times 4.2} = \underline{71 \text{ N/mm}^2}$$

$$f_r = \sqrt{(104^2 + 71^2)} = \underline{126 \text{ N/mm}^2} < 255 \text{ N/mm}^2$$

@ Bolt row y

$$f_h = \frac{29 \times 10^3}{92 \times 4.2} = 75 \text{ N/mm}^2$$

$$f_r = \sqrt{(104^2 + 75^2)} = \underline{128 \text{ N/mm}^2} < 255 \text{ N/mm}^2$$

∴ Weld OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 3

SHT. No. C102 REV.

BY KM DATE Oct. 98

EX. DATE

CN17

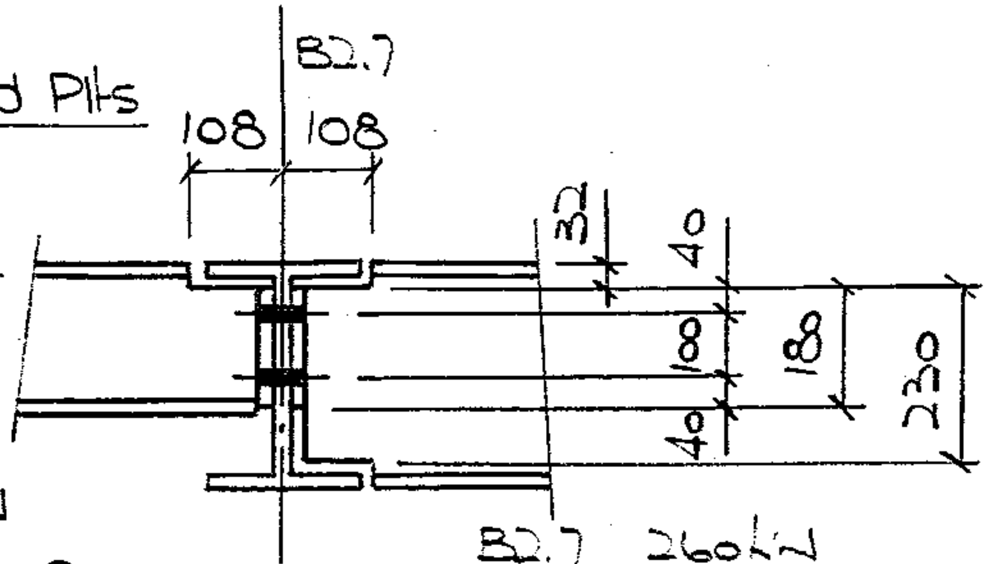
Beams B9-7 / B2-7 to B2-7

200×10 thk End Plts

6FW

4N° M20 Gr 8.8

Bolts



B9.7 200kN

203×203×86UC

B2.7 260kN

Cellular Beam

% 203×203×86UC

D = 298 , t = 13.0

A. B2.7 End Conn.

i) Bolts

$F_v = 260/4 = \underline{65\text{ kN}} < 92\text{ kN}$

Allowable bearing on end Plt. $= 10 \times 20 \times 550 \times 10^{-3} = \underline{110\text{ kN}} > 65\text{ kN}$

Check bearing on Supporting Beam web

Load per bolt $= 200/4 + 260/4 = \underline{115\text{ kN}}$

Allowable bearing $= 13 \times 20 \times 550 \times 10^{-3}$

$= \underline{143\text{ kN}} > 115\text{ kN}$

$\therefore$ Bolts OK

B9.7 / B2.7 to B2.7 cont.

B2.7 End Conn. cont

ii) End Plt.

$A_V = 0.9 \times 180 \times 10 = 1620\,mm^2$

$A_{V_{net}} = 1620 - (2 \times 22 \times 10) = 1180\,mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 1620 = \underline{345\,kN} > 260/2\,kN$

$P_{V_{net}} = 0.5 \times 490 \times 10^{-3} \times 1180 = \underline{289\,kN} > 260/2\,kN$

$\therefore$ End Plt. OK

iii) End Plt. Weld - 6FW

$P_V = 2 \times 180 \times 4.2 \times 255 \times 10^{-3} = \underline{386\,kN} > 260\,kN$

$\therefore$ Weld OK

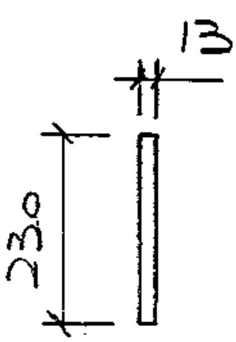
iv) Check web behind End Plt for Shear

$A_V = 0.9 \times 180 \times 13 = 2106\,mm^2$

$P_V = 0.6 \times 355 \times 10^{-3} \times 2106 = \underline{449\,kN} > 260\,kN$

$\therefore$ Web OK

v) Check Beam for Notch



$Z = 13 \times 230^2 / 6 = 114\,617\,mm^3$

$M = 260 \times 108 \times 10^{-3} = \underline{28.1\,kNm}$

$P_b = 114\,617 \times 355 \times 10^{-6}$

$= \underline{40.7\,kNm} > 28.1\,kNm$

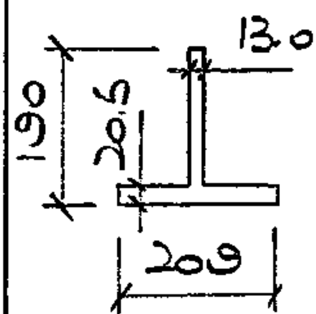
$\therefore$ Beam OK

B9·7 / B2·7 to B2·7 cont

B. B9·7 End Conn

End Plt & Bolts as B2.7 Conn.

Check Beam for Notch



$\bar{x} = 43\text{mm}$

$I = 1841\text{cm}^4$

$M = 200 \times 108 \times 10^{-3} = \underline{21.6\text{kNm}}$

$P_b = \dfrac{1841 \times 10^4}{(190 - 43)} \times 355 \times 10^{-6}$

$= \underline{44.5\text{kNm}} > 21.6\text{kNm}$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 1

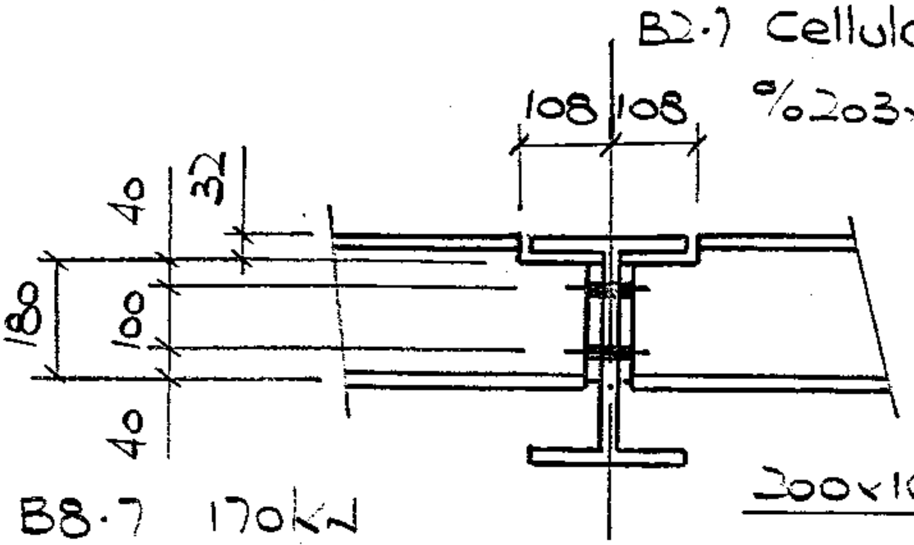
SHT. No. C102A REV.

BY KM DATE Oct 98

EX. DATE

CN17

Beams B8·7 / B9·7 to B2·7



B9.7 200kN

203×203×86UC

200×10thk End Plts

6FW

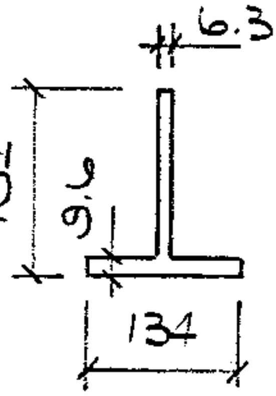
4N° M20 Gr 8.8 Bolts

B8·7 170kN

203×133×30 UB

CONN. SIMILAR TO C102

Check B8·7 for Notch



$\bar{x} = 46mm$

$I = 757\,cm^4$

Shear

$A_v = 0.9 \times 182 \times 6.3 = 1032\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1032 = \underline{220\,kN} > 170\,kN$

Bending

$M = 170 \times 108 \times 10^{-3} = \underline{18.4\,kNm}$

$P_b = \dfrac{757 \times 10^4}{(182 - 46)} \times 355 \times 10^{-6}$

$= \underline{19.8\,kNm} > 18.4\,kNm$

$\therefore$ Beam ok

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 1 OF 3

SHT. No. C103 REV.

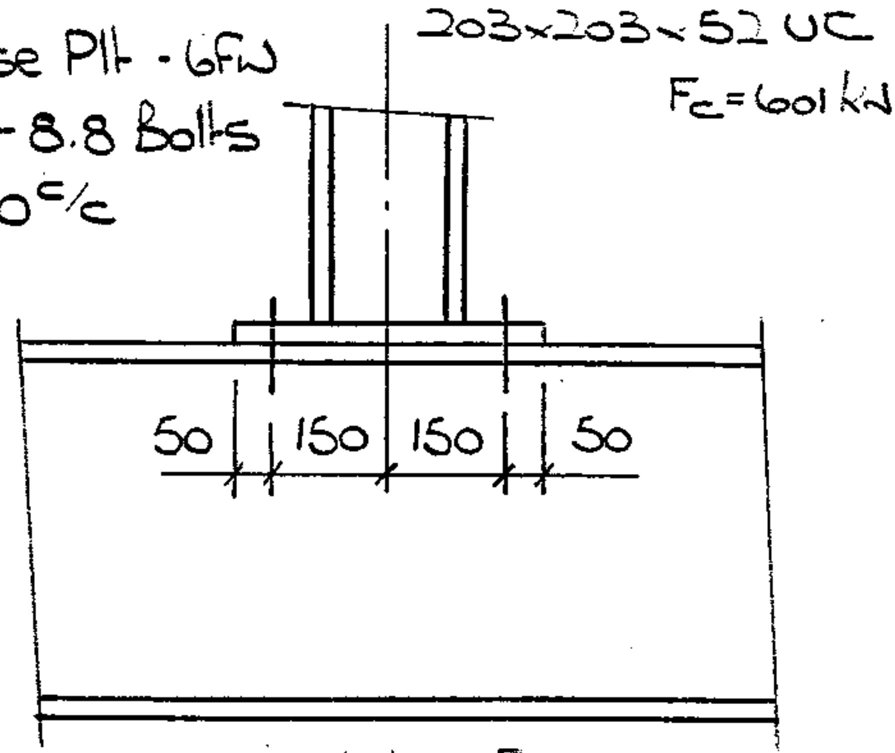
BY KM DATE Oct 98

EX. DATE

CN17

Conn. of Col C9 to Beam B5-7

15thk Base Plt - 6FW
4N° M20 Gr 8.8 Bolts
@ 140 c/c



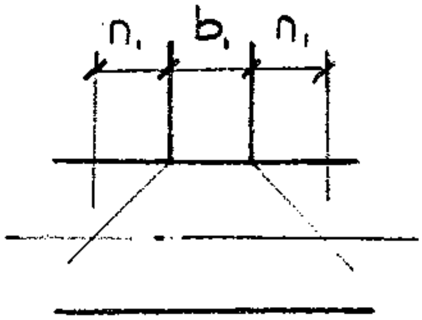
B5-7 Cellular Beam
o/o 356×368×177 UC

D = 498, d = 420, T = 23.8, t = 14.5, r = 15.2

Col C9 to B5·7 cont

Check beam for web buckling & bearing due to F_c (601 kN)

Web Buckling



$P_w = (b_1 + n_1)\, t .\, p_c$

$b_1 = 206$

$n_1 = 498/2 = 249$, $t = 14.5$

Slenderness of beam web

$\lambda = 2.5\, d/t = 2.5 \times 420/14.5 = 72$

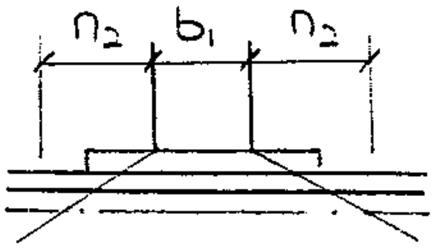
$p_y = 355\ \mathrm{N/mm^2}$

From Table 27(c) $p_c = 210\ \mathrm{N/mm^2}$

$\therefore P_w = (206 + 2 \times 249) \times 14.5 \times 210 \times 10^{-3}$

$= \underline{2144\ \mathrm{kN}} > 601\ \mathrm{kN}$

Web Bearing



15
23.8
15.2
54.0

$P_B = (b_1 + n_2)\, t\ p_{yw}$

$b_1 = 206$

$n_2 = 2.5 \times 54.0 = 135$

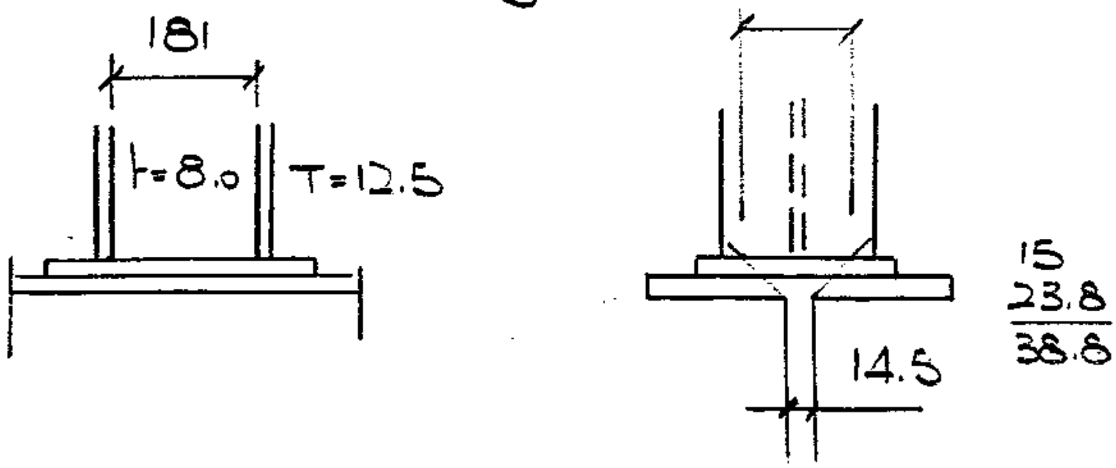
$t = 14.5$, $p_{yw} = 355\ \mathrm{N/mm^2}$

$\therefore P_B = (206 + 2 \times 135) \times 14.5 \times 355 \times 10^{-3}$

$= \underline{2450\ \mathrm{kN}} > 601\ \mathrm{kN}$

Col C9 to B5·7 cont

Check Col for Bearing



Bearing Area = 181 × 8.0 + 2 × 92.1 × 12.5

= 3750 mm^2

P_B = 3750 × 355 × 10^{-3} = 1331 kN > 601 kN

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDEN

SUBJECT Connection Design

SERIES SH 1 OF 1

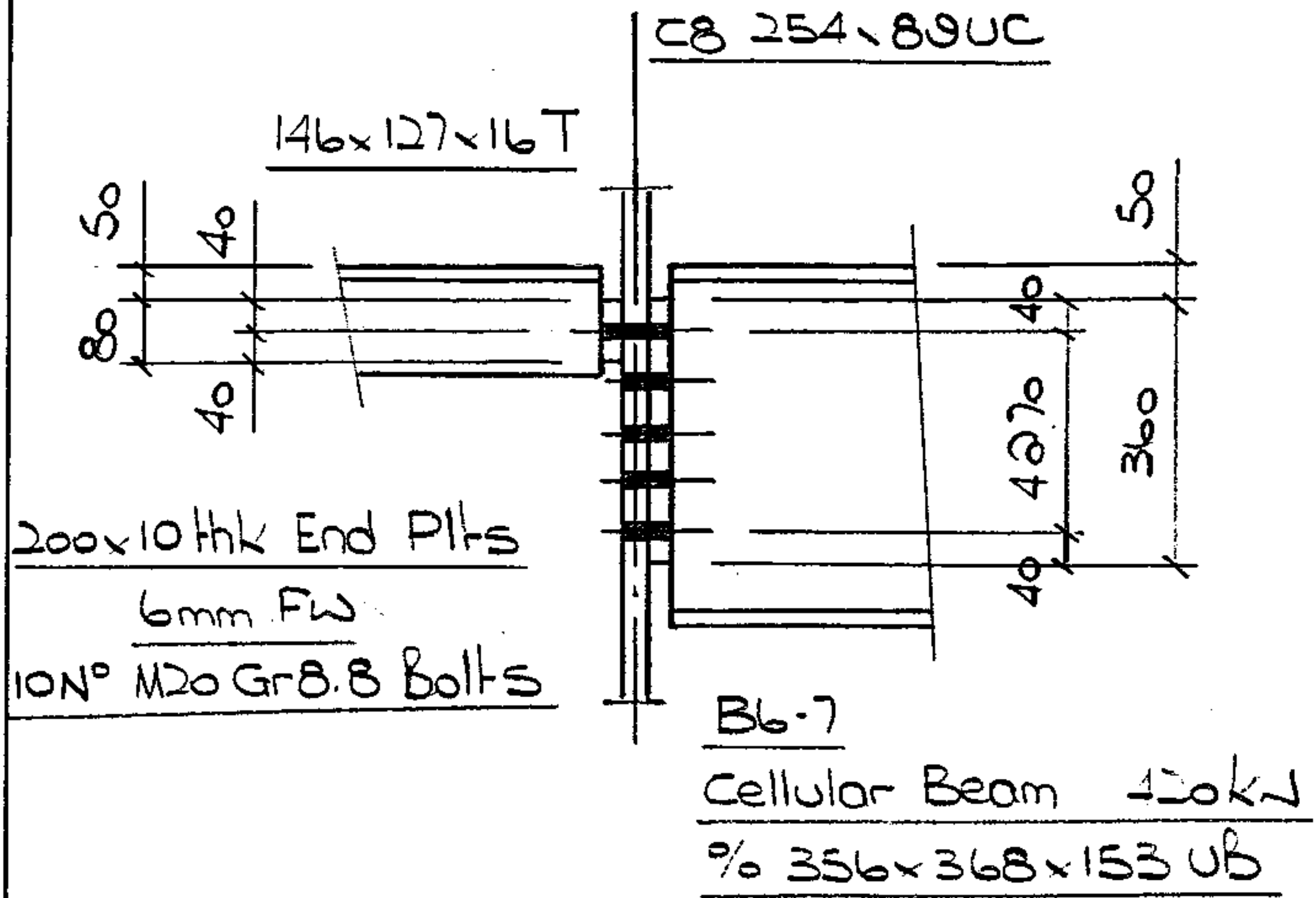
SHT. No. C104 REV.

BY KM DATE Oct.98

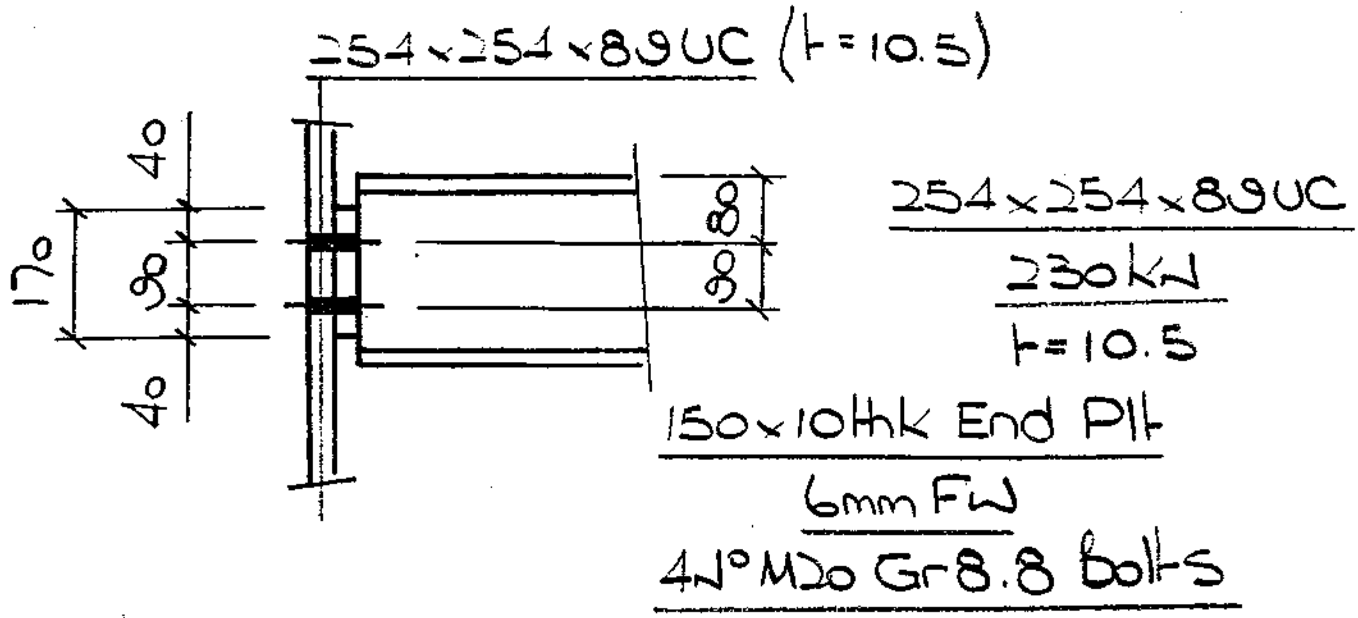
EX. DATE

CN17

Conn of Beams B6.7 / B11.7 to Col. C8



Conn of 254UC to Col C8



i) Bolts

$F_v = 230/4 = \underline{57.5 kN} < 92 kN$

Allowable bearing on end plt / Col. web $= 10 \times 20 \times 550 \times 10^{-3}$
$= \underline{110 kN} > 57.5 kN$

∴ Bolts ok

ii) End Plt.

$A_v = 0.9 \times 170 \times 10 = 1530 mm^2$

$Av_{net} = 1530 - (2 \times 22 \times 10) = 1090 mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1530 = \underline{326 kN} > 230/2 kN$

$Pv_{net} = 0.5 \times 490 \times 10^{-3} \times 1090 = \underline{267 kN} > 230/2 kN$

∴ End Plt. ok

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES SH 2 OF 2

SHT. No. C105 REV.

BY KM DATE Nov 98

EX. DATE

CN17

254 UC to Col C8 cont.

iii) End Plt Weld - 6FW

$P_v = 2 \times 170 \times 4.2 \times 255 \times 10^{-3} = \underline{364\ kN} > 230\ kN$

$\therefore$ Weld OK

iv) Check web behind End Plt. for Shear

$A_v = 0.9 \times 170 \times 10.5 = 1607\ mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 1607 = \underline{342\ kN} > 230\ kN$

$\therefore$ Web OK

3

Column Splices

The connection designs are based upon the following forces taken from Ove Arup drawing S1343.

With regard to shear forces, a nominal value equivalent to the corresponding moment divided by floor height has been used.

Force Summary between Ground/First Floor;

Column	Ref.	Compression	Tension	Moment x-x	Moment y-y
C1	2	3150	560	12	5
C2	5	6668	560	49	11
C3	9	4894	560	10	14
C4	13	3150	560	12	5
C5	16	4948	560	18	16
C6	20	3150	560	12	5

Force Summary between Second/Third Floor;

Column	Ref.	Compression	Tension	Moment x-x	Moment y-y
C1	3	1901	560	12	5
C2	6	5140	560	49	11
C3	10	4299	560	10	14
C4	14	1901	560	12	5
C5	17	4318	560	18	5

Force Summary between Fourth/Fifth Floor;

Column	Ref.	Compression	Tension	Moment x-x	Moment y-y
C2	7	4693	560	78	14
C3	11	3031	560	14	74

Force Summary between Fifth /Sixth Floor;

Column	Ref.	Compression	Tension	Moment x-x	Moment y-y
C5	18	2484	560	34	66

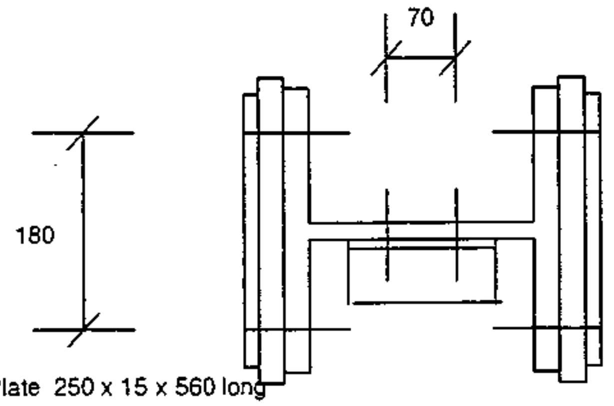
Calculation Sheet

James Lupton Consultants - Inhouse Design Software

Details of Splice Reference - Column Splice 2,13,20 at C1,C4,C6

Upper Section 254 x 254 x 132 x UC Grade 50

Lower Section 305 x 305 x 158 x UC Grade 50



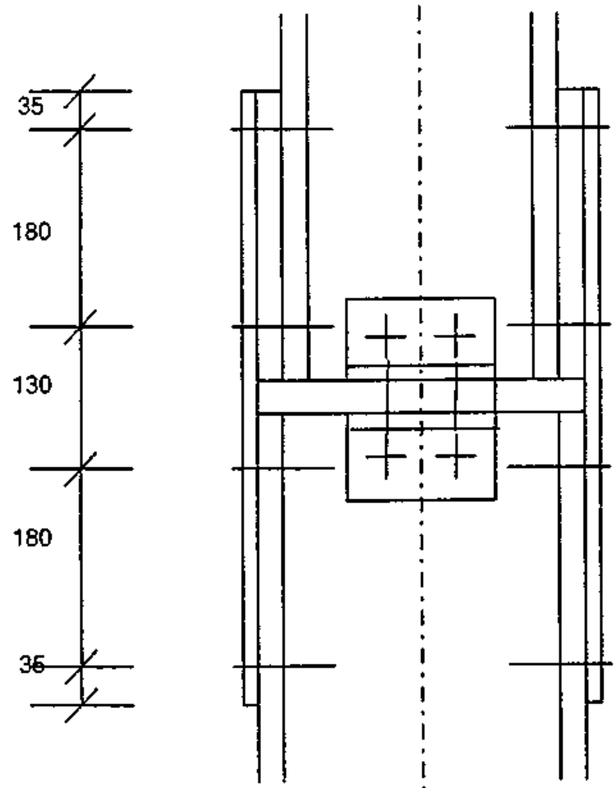
Use External Flange Plate 250 x 15 x 560 long

Division plate Required 30 mm thk Column ends to be prepared for bearing

Flange packer plates required 25 mm thk x 280 mm wide

Use 4 x 4 M24 Grade 8.8 Flange Bolts in 2 rows of 2 - Capacity 425.23 kN (x2)

Flange Bolt centres 180 , Bolt pitch 180 , Edge distance 50



Use a single web plate 150 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 70 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants - **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. **Column Splice 2,13,20 at C1,C4,C6**

Connection Type	Compressive Loads Carried in Bearing
Flange Plates	External Flange Plates only
Web Plates	Plates to one side of web

Section Details

Upper Stack	254 x 254 x 132 x UC Grade 50
Lower Stack	305 x 305 x 158 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**3150.00**	**12.00**	**6.00**	**5.00**	**2.50**
Tension	**560.00**	**12.00**	**6.00**	**5.00**	**2.50**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	3313.79	723.79

Flange Plate Details

	Outer	Inner		
				Minimum Bolts / flange
Grade of Flange Bolts		8.8		4
Size of Flange Bolts		24		Minimum Bolt Spacing
No. Bolts per row		2		**180** mm
				Bolt Spacing
Bolt Centres	50	**180**	**Bolts too wide**	180
Additional hole size for countersunk bolts		**0**		Section Width
Preffered Outer flange plate width		**250**	**mm**	261
Preffered Inner flange plate width		**80**	**No Inner Flange plate,**	110
Grade for flange/web plates		43		

Area of largest flange 7765 mm^2

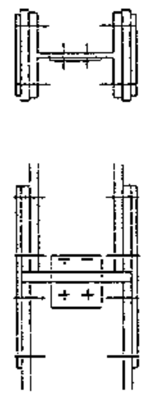
Use External Flange Plate 250 x 15 x 560 long

Area of Flange Plates 3750 mm^2		Ae of flange plates	3888
Force through Flange Bolts 361.89 kN		Tensile Stress	186.2
Bolt Force reduction factor	0.81	Ae of remaining section	14268.8
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	50.7

Single Shear 106.31 kN Bearing 222.28 kN

Use 4 x 4 M24 Grade 8.8 Flange Bolts in 2 rows of 2 - Capacity 425.23 kN (x2)
Flange Bolt centres 180 , Bolt pitch 180 , Edge distance 50

Web Plate Details



Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**70**
Prefered plate width	**150**

Area of largest web 4570 mm^2
Use a single web plate 150 x 8 x 160 long
Area of Web Plates 1200 mm^2
No web pack plate required
Web Pack Reduction factor 1.00
Capacity of Web 15.43 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 143.52 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 70 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 25 mm
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	835 kN
Depth of flange plate	560 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	12 **mm fillet**
Capacity of weld	1.81 kN/mm
Length of pack plate required	231 mm

All checks above assume packer plate is bolted to main stanchion

Division plate Required 30 mm thk

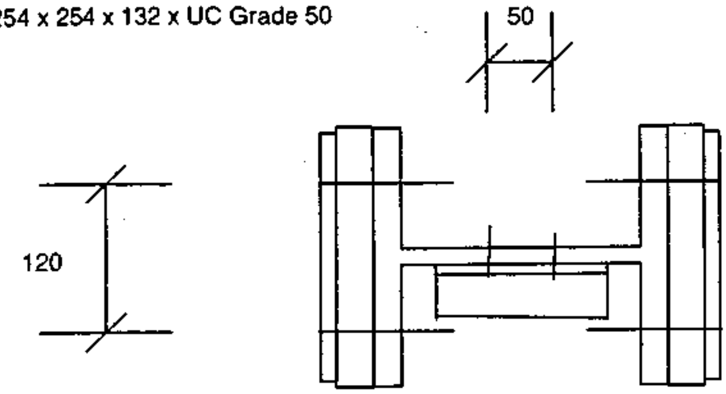
Kvaerner Cleveland Bridge Ltd

O/No. : 151 Job : Carlton Gardens
Subject : Connection design – Column Splices

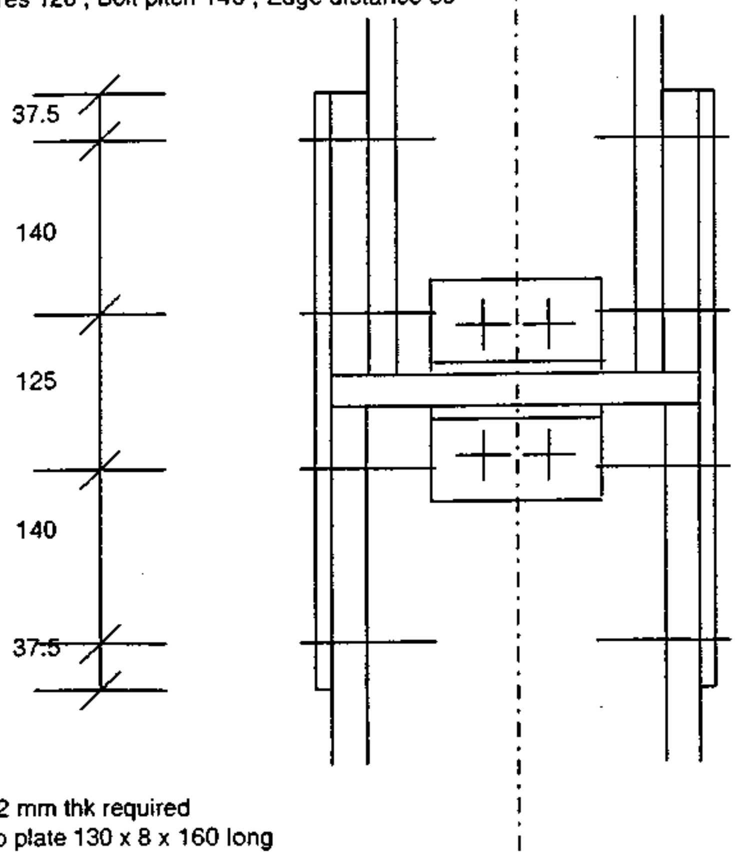
Series :
Sht. No. : CS3/1 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

James Lupton Consultants - Inhouse Design Software
Details of Splice Reference - Column Splice 3,14 at C1, C4
Upper Section 203 x 203 x 86 x UC Grade 50
Lower Section 254 x 254 x 132 x UC Grade 50



Use External Flange Plate 200 x 12 x 480 long
Division plate Required 25 mm thk Column ends to be prepared for bearing
Flange packer plates required 30 mm thk x 210 mm wide
Use 4 x 4 M20 Grade 8.8 Flange Bolts in 2 rows of 2 - Capacity 417.53 kN (x2)
Flange Bolt centres 120 , Bolt pitch 140 , Edge distance 50



Web pack plate 2 mm thk required
Use a single web plate 130 x 8 x 160 long
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 50 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants - **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. Column Splice 3,14 at C1,C4

Connection Type Compressive Loads Carried in Bearing
Flange Plates External Flange Plates only
Web Plates Plates to one side of web

Section Details
Upper Stack 203 x 203 x 86 x UC Grade 50
Lower Stack 254 x 254 x 132 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**1901.00**	**12.00**	**6.00**	**5.00**	**2.50**
Tension	**560.00**	**12.00**	**6.00**	**5.00**	**2.50**

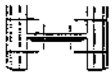
Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	2136.13	795.13

Flange Plate Details

Minimum Bolts / flange
Grade of Flange Bolts 8.8 6
Size of Flange Bolts 20 Minimum Bolt Spacing
No. Bolts per row 2 **140** mm
Outer Inner Bolt Spacing
Bolt Centres **50** **100 mm** 140
Additional hole size for countersunk bolts **0** Section Width
Preffered Outer flange plate width **200 mm** 208.8
Preffered Inner flange plate width **80 No inner Flange plate,** 87.7
Grade for flange/web plates 43
Area of largest flange 6603 mm^2
Use External Flange Plate 200 x 12 x 720 long
Area of Flange Plates 2400 mm^2 Ae of flange plates 3340.8
Force through Flange Bolts 397.56 kN Tensile Stress 238.0
Bolt Force reduction factor 0.75 Ae of remaining section 9196
Capacity of Flange Bolts - M20 Grade 8.8 Tensile Stress 86.5
Single Shear 68.60 kN Bearing 140.78 kN
Use 4 x 6 M20 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 411.58 kN (x2)
Flange Bolt centres 100 , Bolt pitch 140 , Edge distance 40

Web Plate Details



Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**80**
Prefered plate width	**160**

Area of largest web 3693 mm^2
Use a single web plate 160 x 8 x 160 long
Area of Web Plates 1280 mm^2
Web pack plate 2 mm thk required
Web Pack Reduction factor 1.00
Capacity of Web 13.50 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 119.60 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 80 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section

Thickness required for the packer plate	30 mm
Packer plate loose bolted	
Part compression - Full tension carried	

Design force on plate(s) on one flange	608 kN
Depth of flange plate	720 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	25 **mm fillet**
Capacity of weld	3.76 kN/mm
Length of pack plate required	81 mm

All checks above assume packer plate is bolted to main stanchion

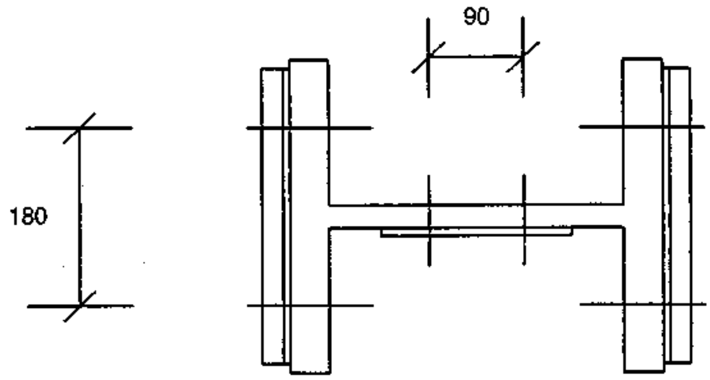
Division plate Required 25 mm thk

James Lupton Consultants - Inhouse Design Software

Details of Splice Reference - Column Splice 5 at C2

Upper Section 305 x 305 x 240 x UC Grade 50

Lower Section 305 x 305 x 283 x UC Grade 50



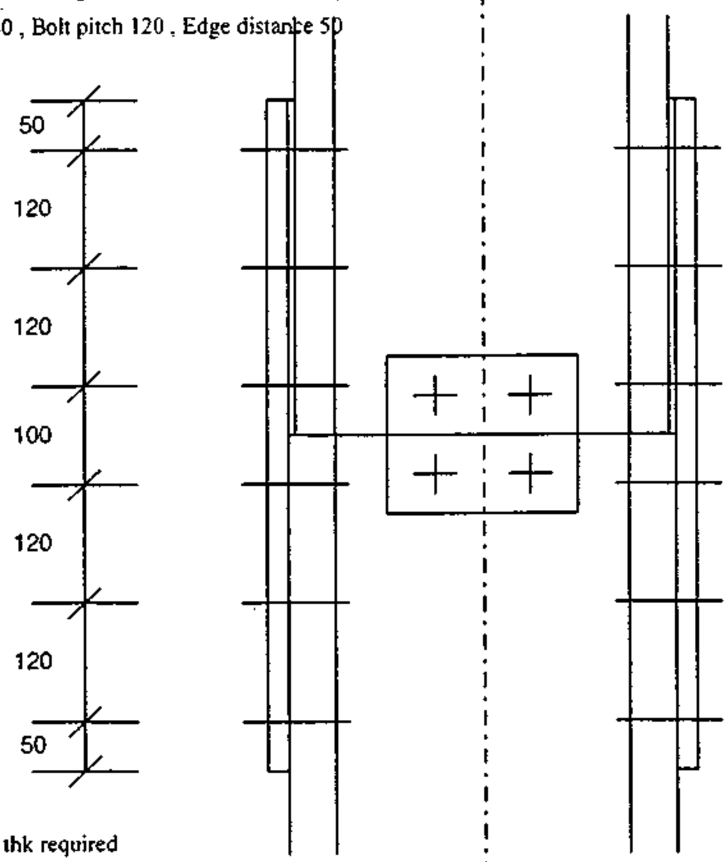
Use External Flange Plate 300 x 20 x 680 long

No division plate Required Column ends to be prepared for bearing

Flange packer plates required 6 mm thk x 300 mm wide

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)

Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50



Web pack plate 2 mm thk required

Use a single web plate 180 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

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Splice Design to BS 5950

Splice Ref. Column Splice 5 at C2

Connection Type	Compressive Loads Carried in Bearing
Flange Plates	External Flange Plates only
Web Plates	Plates to one side of web

Section Details

Upper Stack	305 x 305 x 240 x UC Grade 50
Lower Stack	305 x 305 x 283 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**6668.00**	**49.00**	**24.50**	**11.00**	**5.50**
Tension	**560.00**	**49.00**	**24.50**	**11.00**	**5.50**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	7147.25	1039.25

Flange Plate Details

Minimum Bolts / flange 6

Minimum Bolt Spacing **120** mm

Grade of Flange Bolts 8.8
Size of Flange Bolts 24
No. Bolts per row 2

	Outer	Inner	
Bolt Centres	**50**	**180 mm**	

Bolt Spacing 120

Additional hole size for countersunk bolts **0**

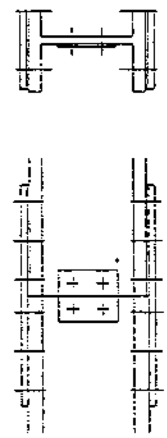
Section Width 317.9 132.25

Preffered Outer flange plate width **300 mm**
Preffered Inner flange plate width **80 No Inner Flange plate,**
Grade for flange/web plates 43
Area of largest flange 14191 mm^2
Use External Flange Plate 300 x 20 x 680 long

Area of Flange Plates 6000 mm^2		Ae of flange plates	5184
Force through Flange Bolts 519.62 kN		Tensile Stress	200.5
Bolt Force reduction factor	1.00	Ae of remaining section	26679.2
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	39.0

Single Shear 132.00 kN Bearing 416.21 kN
Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)
Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50

Web Plate Details



Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**90**
Prefered plate width	**180**

Area of largest web 7617 mm^2
Use a single web plate 180 x 8 x 160 long
Area of Web Plates 1440 mm^2
Web pack plate 2 mm thk required
Web Pack Reduction factor 1.00
Capacity of Web 49.00 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 211.60 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 6 mm
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	520 kN
Depth of flange plate	680 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	**6 mm fillet**
Capacity of weld	0.90 kN/mm
Length of pack plate required	288 mm

All checks above assume packer plate is bolted to main stanchion

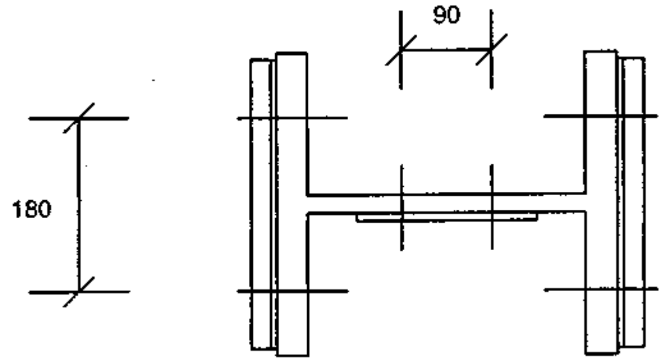
No division plate Required column ends to be prepared for bearing

James Lupton Consultants - Inhouse Design Software

Details of Splice Reference - Column Splice 6 at C2

Upper Section 305 x 305 x 198 x UC Grade 50

Lower Section 305 x 305 x 240 x UC Grade 50



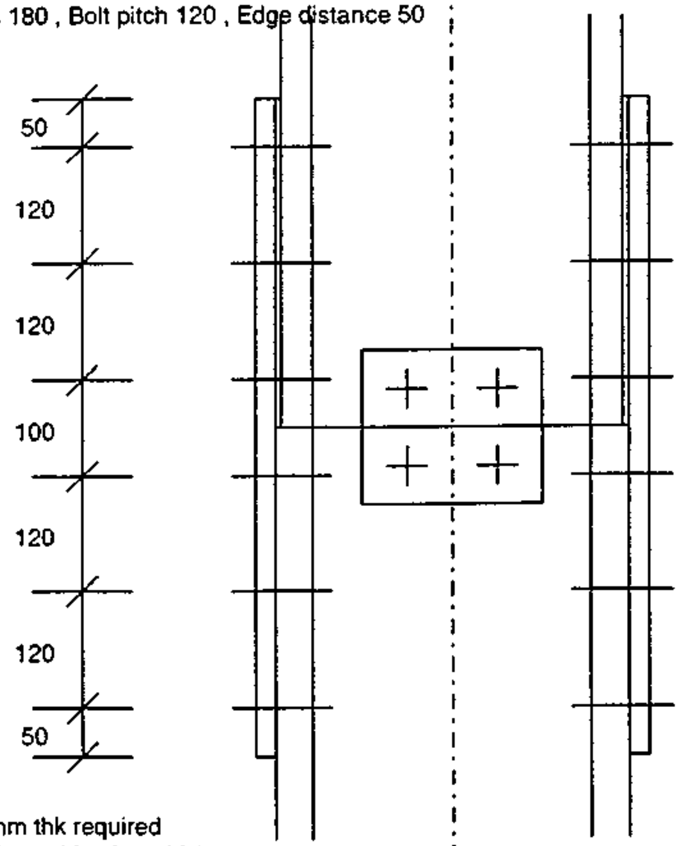
Use External Flange Plate 300 x 20 x 680 long

No division plate Required Column ends to be prepared for bearing

Flange packer plates required 6 mm thk x 300 mm wide

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)

Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50



Web pack plate 2 mm thk required

Use a single web plate 180 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants - **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. **Column Splice 6 at C2**

Connection Type Compressive Loads Carried in Bearing
Flange Plates External Flange Plates only
Web Plates Plates to one side of web

Section Details
Upper Stack 305 x 305 x 198 x UC Grade 50
Lower Stack 305 x 305 x 240 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**5140.00**	**49.00**	**24.50**	**11.00**	**5.50**
Tension	**560.00**	**49.00**	**24.50**	**11.00**	**5.50**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	5619.25	1039.25

Flange Plate Details

			Minimum Bolts / flange	
Grade of Flange Bolts		8.8	6	
Size of Flange Bolts		24	Minimum Bolt Spacing	
No. Bolts per row		2	**120** mm	
	Outer	Inner		Bolt Spacing
Bolt Centres	**50**	**180 mm**		120
Additional hole size for countersunk bolts		**0**		Section Width
Preffered Outer flange plate width		**300 mm**		314.1
Preffered Inner flange plate width		**80 No Inner Flange plate,**		132.25
Grade for flange/web plates		43		

Area of largest flange 11985 mm^2
Use External Flange Plate 300 x 20 x 680 long

Area of Flange Plates 6000 mm^2		Ae of flange plates	5184
Force through Flange Bolts	519.62 kN	Tensile Stress	200.5
Bolt Force reduction factor	1.00	Ae of remaining section	21934.4
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	47.4

Single Shear 132.00 kN Bearing 346.66 kN
Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)
Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50

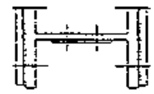
Kvaerner Cleveland Bridge Ltd

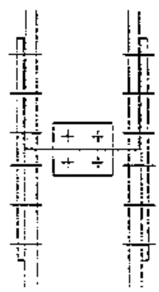
O/No. : 151 Job : Carlton Gardens
Subject : Connection design – Column Splices

Series :
Sht. No. : CS6/3 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Web Plate Details





Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**90**
Prefered plate width	**180**

Area of largest web 6630 mm^2
Use a single web plate 180 x 8 x 160 long
Area of Web Plates 1440 mm^2
Web pack plate 2 mm thk required
Web Pack Reduction factor 1.00
Capacity of Web 49.00 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 176.64 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 6 mm
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	520 kN
Depth of flange plate	680 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	6 **mm fillet**
Capacity of weld	0.90 kN/mm
Length of pack plate required	288 mm

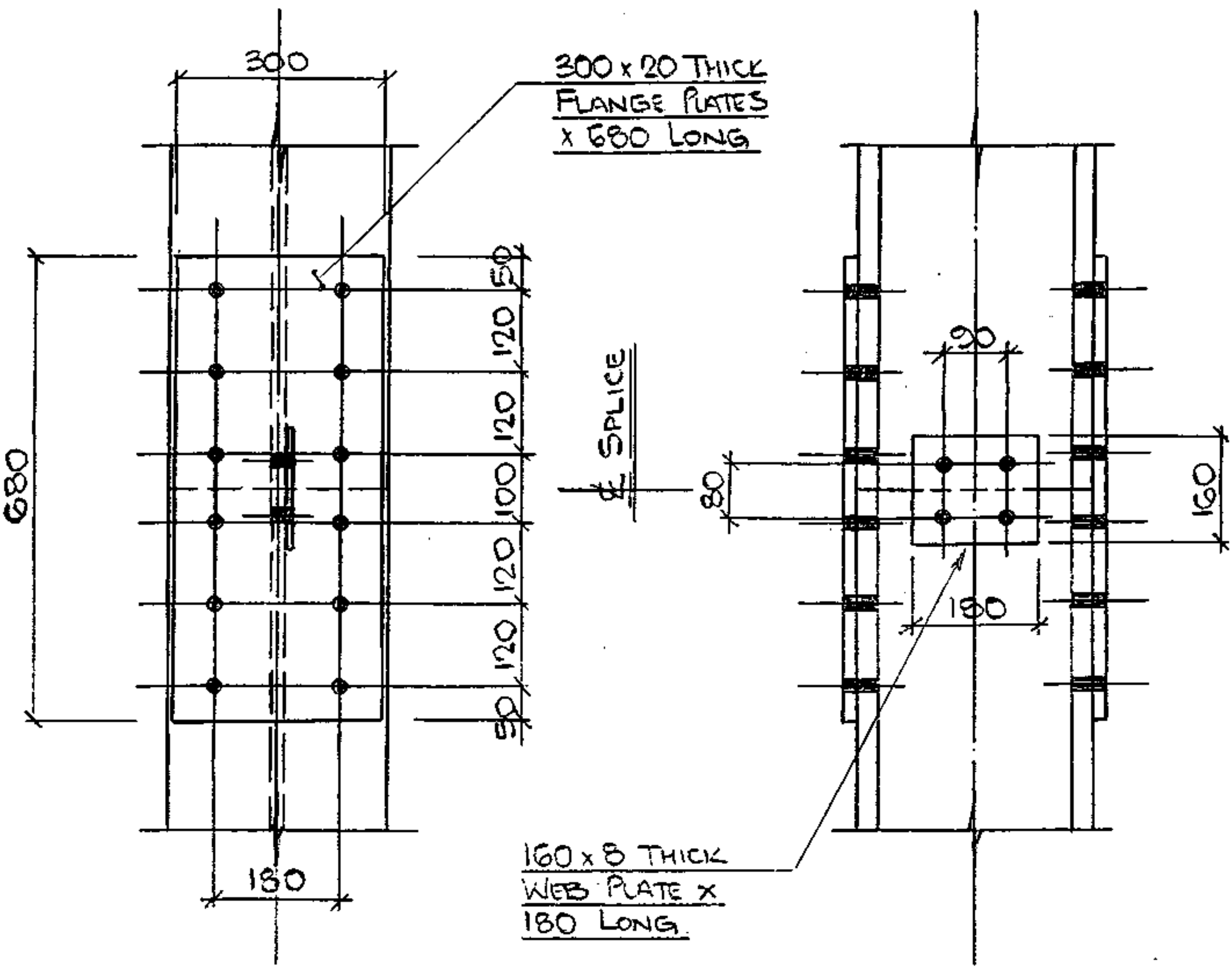
All checks above assume packer plate is bolted to main stanchion

No division plate Required column ends to be prepared for bearing

COLUMN SPLICE (7)/C2 ~ ABOVE FLOOR LEVEL 4

305 x 305 x 198 UC TO 305 x 305 x 198 UC

MATERIAL : — GRADE 50



NOTE! : — ENDS OF COLUMN SHAFTS TO BE PREPARED FOR CONTACT BEARING

BOLTS : —

a) COLUMN FLANGES ~ M24 (GRADE 8.8)

b) COLUMN WEB ~ M20 (GRADE 8.8)

REFERENCE : — SHEET NOS CS9/1 – 3 (INCLUSIVE)

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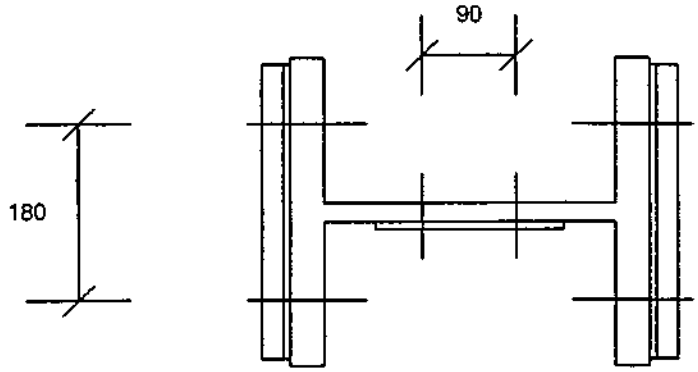
Details of Splice Reference - Column Splice 9 at C3

Upper Section 305 x 305 x 198 x UC Grade 50

Lower Section 305 x 305 x 240 x UC Grade 50



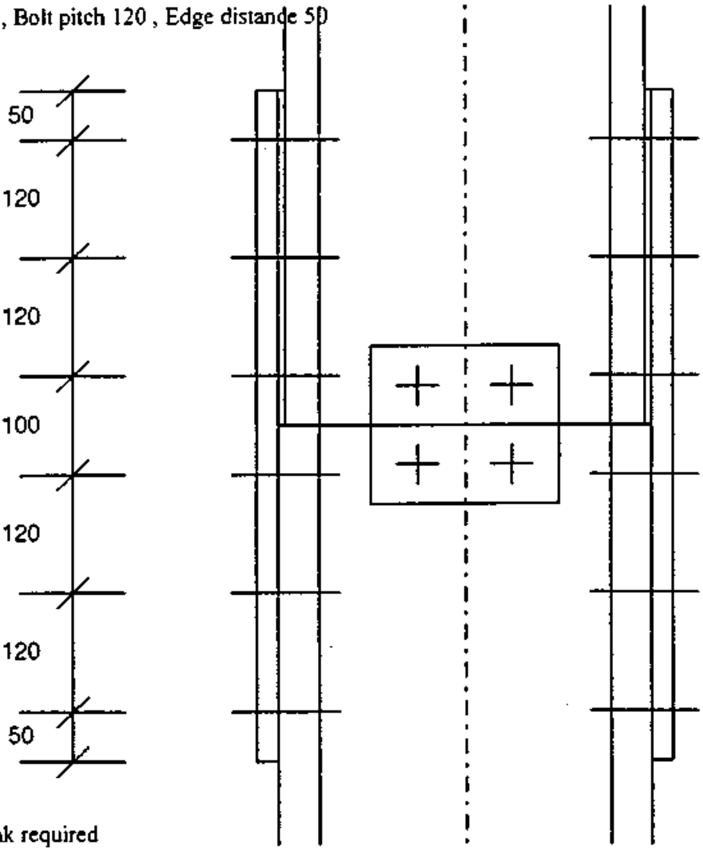
Use External Flange Plate 300 x 20 x 680 long

No division plate Required Column ends to be prepared for bearing

Flange packer plates required 6 mm thk x 300 mm wide

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)

Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50



Web pack plate 2 mm thk required

Use a single web plate 180 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

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Splice Design to BS 5950

Splice Ref. **Column Splice 9 at C3**

Connection Type Compressive Loads Carried in Bearing
Flange Plates External Flange Plates only
Web Plates Plates to one side of web

Section Details
Upper Stack 305 x 305 x 198 x UC Grade 50
Lower Stack 305 x 305 x 240 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**4894.00**	**10.00**	**5.00**	**14.00**	**7.00**
Tension	**560.00**	**10.00**	**5.00**	**14.00**	**7.00**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	5126.01	792.01

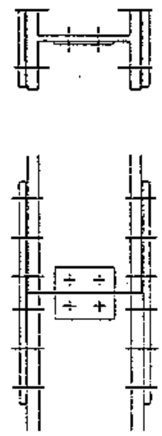
Flange Plate Details

			Minimum Bolts / flange	
Grade of Flange Bolts		8.8	6	
Size of Flange Bolts		24	Minimum Bolt Spacing	
No. Bolts per row		2	**120** mm	
	Outer	Inner		Bolt Spacing
Bolt Centres	**50**	**180 mm**		120
Additional hole size for countersunk bolts		**0**		Section Width
Preffered Outer flange plate width		**300 mm**		314.1
Preffered Inner flange plate width		**80**	**No Inner Flange plate,**	132.25
Grade for flange/web plates		43		

Area of largest flange 11985 mm^2
Use External Flange Plate 300 x 20 x 680 long

Area of Flange Plates 6000 mm^2		Ae of flange plates	5184
Force through Flange Bolts	396.00 kN	Tensile Stress	152.8
Bolt Force reduction factor	1.00	Ae of remaining section	21934.4
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	36.1

Single Shear 132.00 kN Bearing 346.66 kN
Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)
Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50



Web Plate Details

Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**90**
Prefered plate width	**180**

Area of largest web 6630 mm^2
Use a single web plate 180 x 8 x 160 long
Area of Web Plates 1440 mm^2
Web pack plate 2 mm thk required
Web Pack Reduction factor 1.00
Capacity of Web 10.00 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 176.64 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section

Thickness required for the packer plate	6 mm
Packer plate loose bolted	
Part compression - Full tension carried	
Design force on plate(s) on one flange	431 kN
Depth of flange plate	680 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	**6 mm fillet**
Capacity of weld	0.90 kN/mm
Length of pack plate required	239 mm

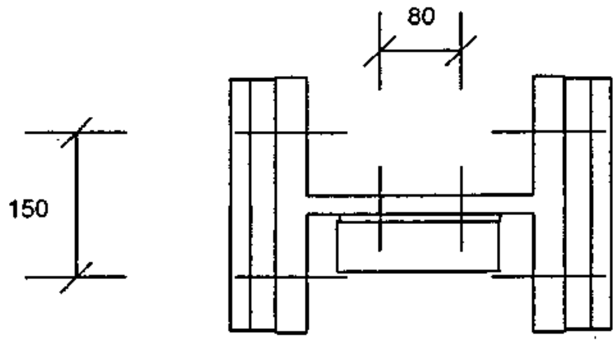
All checks above assume packer plate is bolted to main stanchion

No division plate Required column ends to be prepared for bearing

James Lupton Consultants - Inhouse Design Software
Details of Splice Reference - Column Splice 10 at C3

Upper Section 254 x 254 x 167 x UC Grade 50

Lower Section 305 x 305 x 198 x UC Grade 50



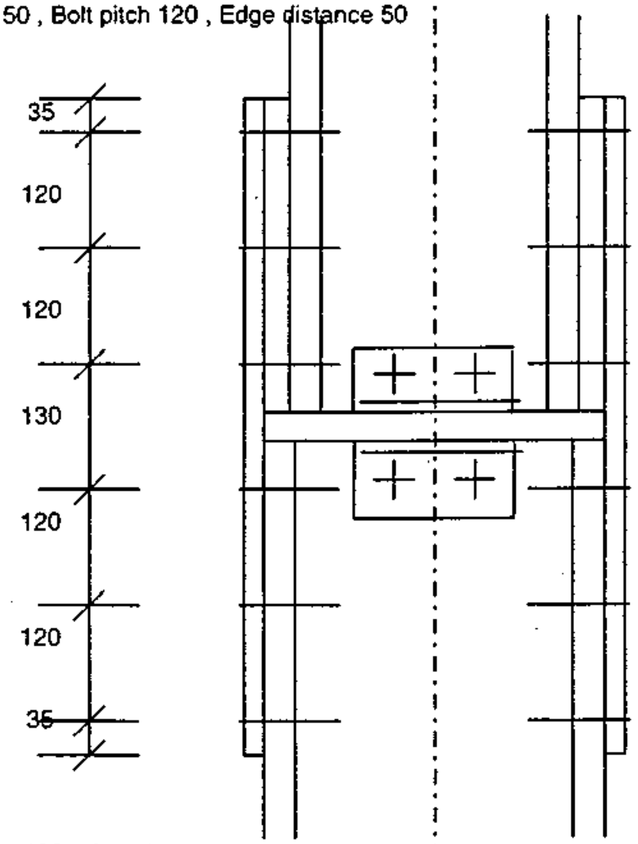
Use External Flange Plate 260 x 20 x 680 long

Division plate Required 30 mm thk Column ends to be prepared for bearing

Flange packer plates required 25 mm thk x 260 mm wide

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 637.85 kN (x2)

Flange Bolt centres 150 , Bolt pitch 120 , Edge distance 50



Use a single web plate 160 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 80 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants · **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. **Column Splice 10 at C3**

Connection Type Compressive Loads Carried in Bearing
Flange Plates External Flange Plates only
Web Plates Plates to one side of web

Section Details

Upper Stack 254 x 254 x 167 x UC Grade 50
Lower Stack 305 x 305 x 198 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**4299.00**	**10.00**	**5.00**	**14.00**	**7.00**
Tension	**560.00**	**10.00**	**5.00**	**14.00**	**7.00**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	4579.41	840.41

Flange Plate Details

	Outer	Inner		
Grade of Flange Bolts		8.8		Minimum Bolts / flange 6
Size of Flange Bolts		24		Minimum Bolt Spacing
No. Bolts per row		2		**120** mm
Bolt Centres	**50**	**150**	**mm**	Bolt Spacing 120
Additional hole size for countersunk bolts		**0**		Section Width
Preffered Outer flange plate width		**260**	**mm**	264.5
Preffered Inner flange plate width		**80**	**No Inner Flange plate,**	109.95
Grade for flange/web plates		43		

Area of largest flange 9863 mm^2
Use External Flange Plate 260 x 20 x 680 long

Area of Flange Plates 5200 mm^2		Ae of flange plates	5184
Force through Flange Bolts	420.21 kN	Tensile Stress	162.1
Bolt Force reduction factor	0.81	Ae of remaining section	17903.2
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	46.9

Single Shear 106.31 kN Bearing 279.19 kN
Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 637.85 kN (x2)
Flange Bolt centres 150 , Bolt pitch 120 , Edge distance 50

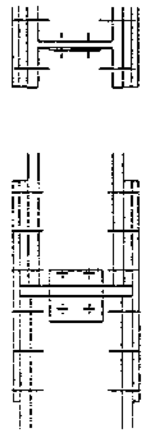
Kvaerner Cleveland Bridge Ltd

O/No. : 151 Job : Carlton Gardens
Subject : Connection design – Column Splices

Series :
Sht. No. : CS10/3 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Web Plate Details



Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**80**
Prefered plate width	**160**

Area of largest web 5475 mm^2
Use a single web plate 160 x 8 x 160 long
Area of Web Plates 1280 mm^2
No web pack plate required
Web Pack Reduction factor 1.00
Capacity of Web 11.25 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 176.64 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 80 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 25 mm
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	1024 kN
Depth of flange plate	680 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	25 **mm fillet**
Capacity of weld	3.76 kN/mm
Length of pack plate required	136 mm

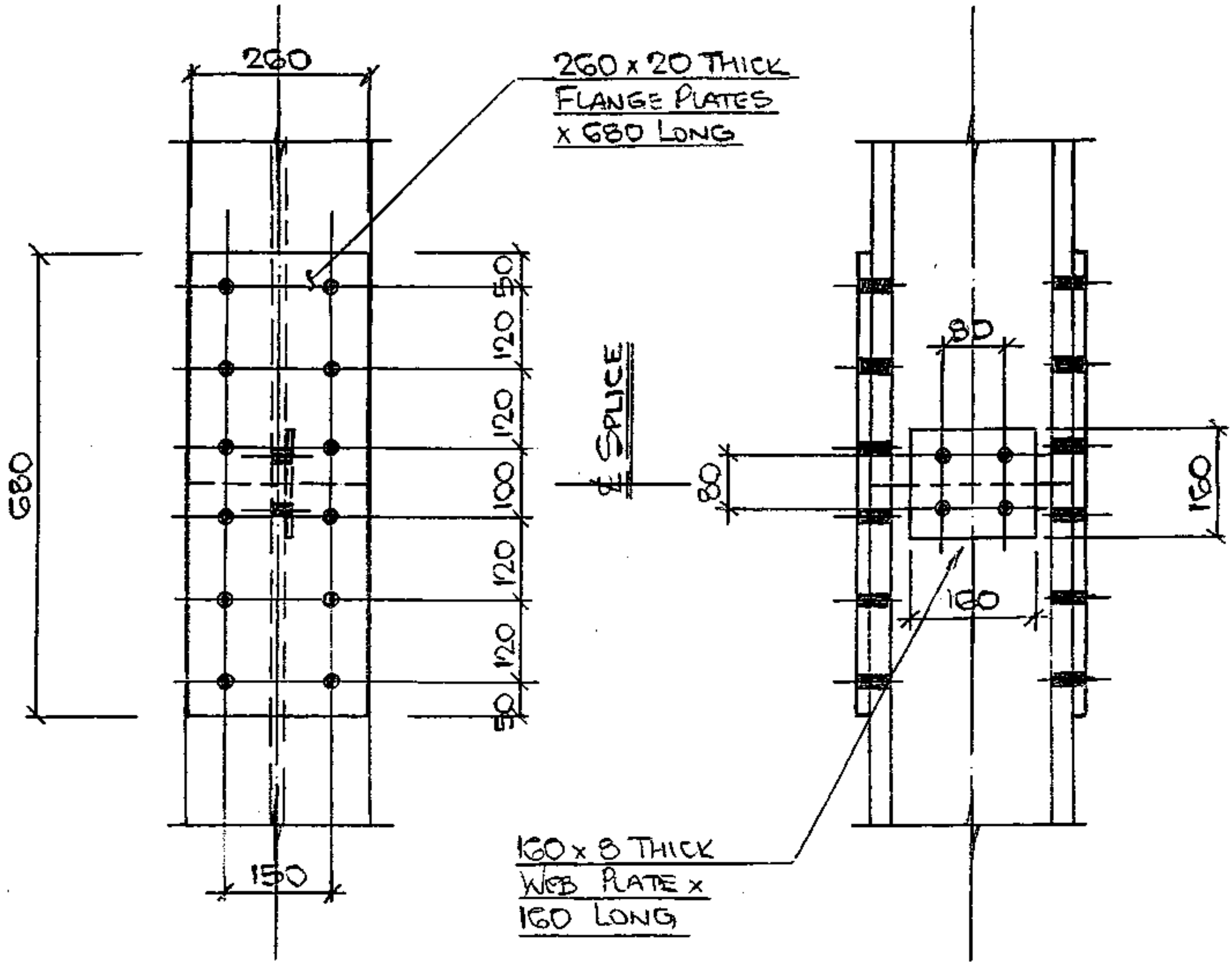
All checks above assume packer plate is bolted to main stanchion

Division plate Required 30 mm thk

COLUMN SPLICE (11)/C3 ~ ABOVE FLOOR LEVEL 5

254 x 254 x 167 UC TO 254 x 254 x 167 UC

MATERIAL :— GRADE 50



NOTE! :— ENDS OF COLUMN SHAFTS TO BE PREPARED FOR CONTACT BEARING

BOLTS :—

a) COLUMN FLANGES ~ M24 (GRADE 8.8)

b) COLUMN WEB ~ M20 (GRADE 8.8)

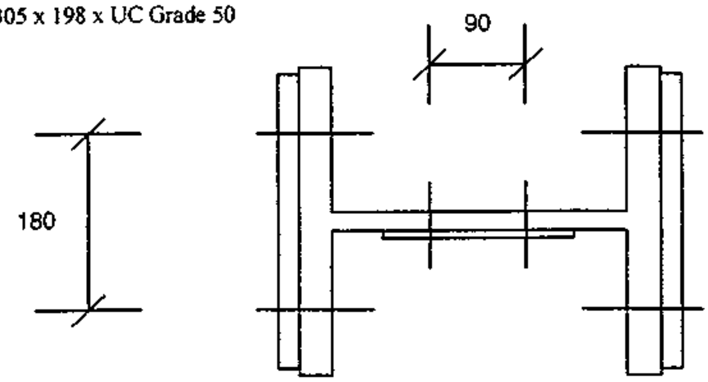
REFERENCE :— SHEET NOS CS10/1-3 (INCLUSIVE)

James Lupton Consultants - Inhouse Design Software

Details of Splice Reference - Column Splice 16 at C5

Upper Section 305 x 305 x 198 x UC Grade 50

Lower Section 305 x 305 x 198 x UC Grade 50

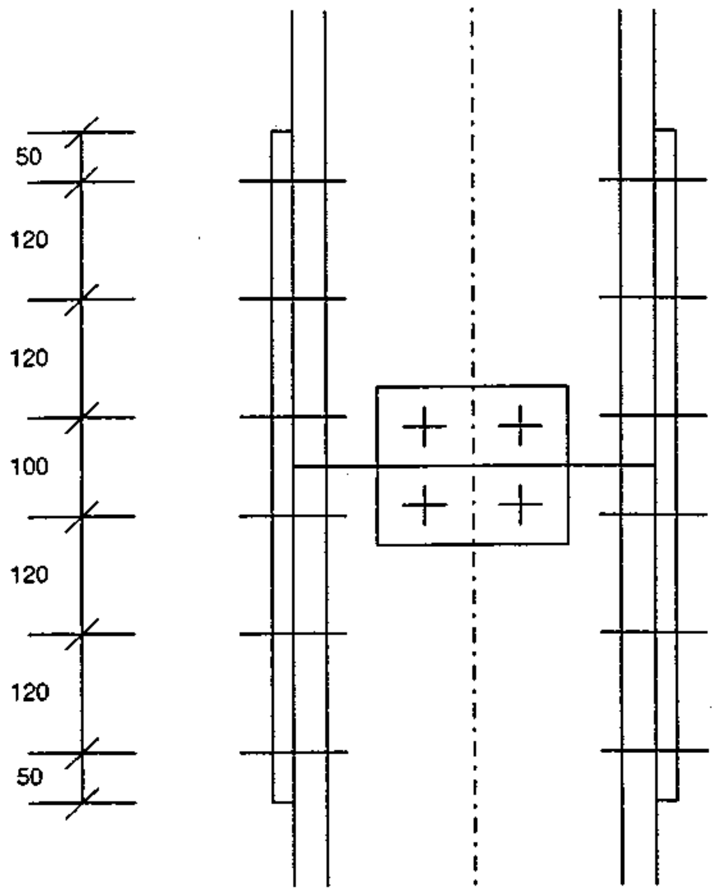


Use External Flange Plate 300 x 20 x 680 long

No division plate Required Column ends to be prepared for bearing

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)

Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50



Use a single web plate 180 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants - **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. **Column Splice 16 at C5**

Connection Type	Compressive Loads Carried in Bearing
Flange Plates	External Flange Plates only
Web Plates	Plates to one side of web

Section Details

Upper Stack	305 x 305 x 198 x UC Grade 50
Lower Stack	305 x 305 x 198 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**4948.00**	**18.00**	**9.00**	**16.00**	**8.00**
Tension	**560.00**	**18.00**	**9.00**	**16.00**	**8.00**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	5260.58	872.58

Flange Plate Details

			Minimum Bolts / flange
Grade of Flange Bolts		8.8	6
Size of Flange Bolts		24	Minimum Bolt Spacing
No. Bolts per row		2	**120** mm
	Outer	Inner	Bolt Spacing
Bolt Centres	**50**	**180 mm**	120
Additional hole size for countersunk bolts		**0**	Section Width
Preffered Outer flange plate width		**300 mm**	314.1
Preffered Inner flange plate width		**80 No Inner Flange plate,**	132.25
Grade for flange/web plates		43	

Area of largest flange 9863 mm^2

Use External Flange Plate 300 x 20 x 680 long

Area of Flange Plates 6000 mm^2		Ae of flange plates	5184
Force through Flange Bolts 436.29 kN		Tensile Stress	168.3
Bolt Force reduction factor	1.00	Ae of remaining section	21934.4
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	39.8

Single Shear 132.00 kN Bearing 346.66 kN

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 792.00 kN (x2)

Flange Bolt centres 180 , Bolt pitch 120 , Edge distance 50

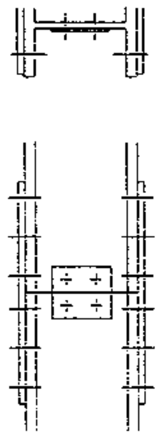
Kvaerner Cleveland Bridge Ltd

O/No. : 151 Job : Carlton Gardens
Subject : Connection design – Column Splices

Series :
Sht. No. : CS16/3 Rev. :
By : AJC Date : May 98
EX : JL Date : May 98

Calculation Sheet

Web Plate Details



Grade of Web Bolts 8.8
Size of Web Bolts 20
No. Bolts per row 2
Bolt Centres **90**
Prefered plate width **180**

Area of largest web 5475 mm^2
Use a single web plate 180 x 8 x 160 long
Area of Web Plates 1440 mm^2
No web pack plate required
Web Pack Reduction factor 1.00
Capacity of Web 18.00 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 176.64 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 90 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 0 mm - No packer plate required
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	436	kN
Depth of flange plate	680	mm
Number of welds / flange	2	(i.e. 1 plate per flange)
Weld size	4	**Weld too big**
Capacity of weld	0.60	kN/mm
Length of pack plate required	362	mm

All checks above assume packer plate is bolted to main stanchion

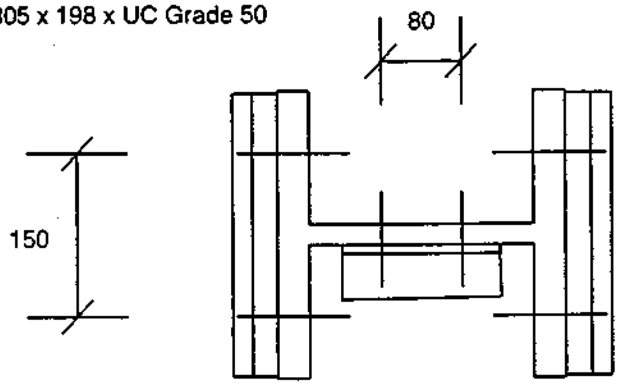
No division plate Required column ends to be prepared for bearing

James Lupton Consultants - Inhouse Design Software

Details of Splice Reference - Column Splice 17 at C5

Upper Section 254 x 254 x 167 x UC Grade 50

Lower Section 305 x 305 x 198 x UC Grade 50



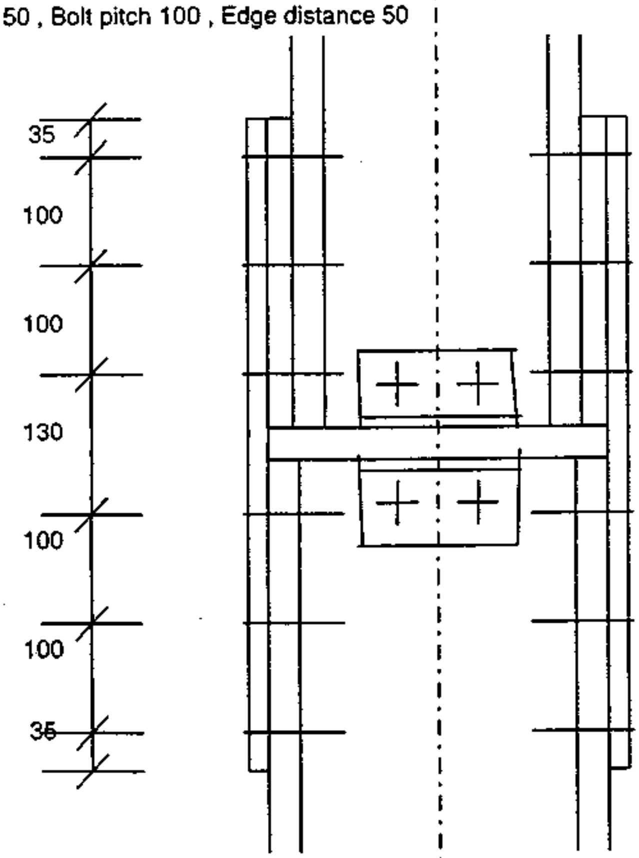
Use External Flange Plate 260 x 20 x 600 long

Division plate Required 30 mm thk Column ends to be prepared for bearing

Flange packer plates required 25 mm thk x 260 mm wide

Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 637.85 kN (x2)

Flange Bolt centres 150 , Bolt pitch 100 , Edge distance 50



Use a single web plate 160 x 8 x 160 long

Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN

Web Bolt centres 80 , Bolt pitch 50 , Edge distance 40

James Lupton Consultants - **Inhouse Design Software**

Splice Design to BS 5950

Splice Ref. Column Splice 17 at C5

Connection Type Compressive Loads Carried in Bearing
Flange Plates External Flange Plates only
Web Plates Plates to one side of web

Section Details
Upper Stack 254 x 254 x 167 x UC Grade 50
Lower Stack 305 x 305 x 198 x UC Grade 50

Applied Loadings

Ultimate Loads	Axial kN	BM xx kNm	Shear xx kN	BM yy kNm	Shear yy kN
Compression	**4318.00**	**18.00**	**9.00**	**5.00**	**2.50**
Tension	**560.00**	**18.00**	**9.00**	**5.00**	**2.50**

Bending effects can be modelled as an equivalent Axial load

	Compressio	Tension
Total Equivalant Axial load	4545.84	787.84

Flange Plate Details

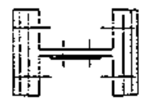
			Minimum Bolts / flange
Grade of Flange Bolts		8.8	6
Size of Flange Bolts		24	Minimum Bolt Spacing
No. Bolts per row		2	**100** mm
	Outer	Inner	Bolt Spacing
Bolt Centres	50	**150 mm**	100
Additional hole size for countersunk bolts		**0**	Section Width
Preffered Outer flange plate width		**260 mm**	264.5
Preffered Inner flange plate width		**80 No Inner Flange plate,**	109.95
Grade for flange/web plates		43	

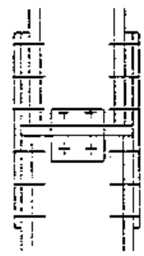
Area of largest flange 9863 mm^2
Use External Flange Plate 260 x 20 x 600 long

Area of Flange Plates 5200 mm^2		Ae of flange plates	5184
Force through Flange Bolts	393.92 kN	Tensile Stress	152.0
Bolt Force reduction factor	0.81	Ae of remaining section	17903.2
Capacity of Flange Bolts - M24 Grade 8.8		Tensile Stress	44.0

Single Shear 106.31 kN Bearing 279.19 kN
Use 4 x 6 M24 Grade 8.8 Flange Bolts in 3 rows of 2 - Capacity 637.85 kN (x2)
Flange Bolt centres 150 , Bolt pitch 100 , Edge distance 50

Web Plate Details





Grade of Web Bolts	8.8
Size of Web Bolts	20
No. Bolts per row	2
Bolt Centres	**80**
Prefered plate width	**160**

Area of largest web 5475 mm^2
Use a single web plate 160 x 8 x 160 long
Area of Web Plates 1280 mm^2
No web pack plate required
Web Pack Reduction factor 1.00
Capacity of Web 20.25 kN
Capacity of Web Bolts - M20 Grade 8.8
Single Shear 91.90 kN Bearing 176.64 kN
Use 2 M20 Grade 8.8 Web Bolts in 1 rows of 2 - Capacity 183.80 kN
Web Bolt centres 80 , Bolt pitch 50 , Edge distance 40

Flange Packer plate
Design packer plate to transfer maximum compression or tension into flange of lower section
Thickness required for the packer plate 25 mm
Packer plate loose bolted
Part compression - Full tension carried

Design force on plate(s) on one flange	1016 kN
Depth of flange plate	600 mm
Number of welds / flange	2 (i.e. 1 plate per flange)
Weld size	25 **mm fillet**
Capacity of weld	3.76 kN/mm
Length of pack plate required	135 mm

All checks above assume packer plate is bolted to main stanchion

Division plate Required 30 mm thk

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION DESIGN

COLUMN SPLICE (18)/C5 ~ ABOVE FLOOR 5

SERIES

SHT. No. CS18/1 REV.

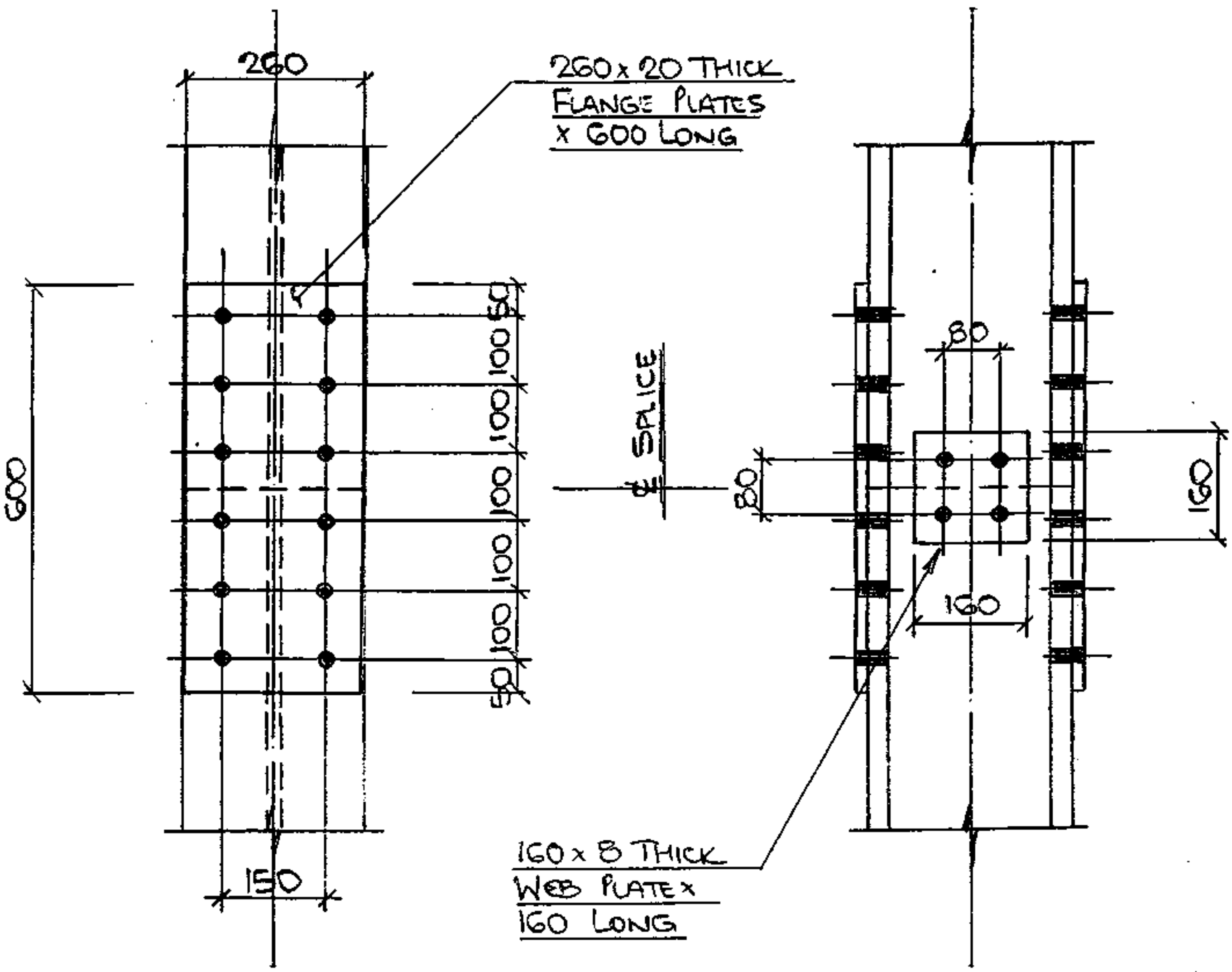
BY KB DATE 08/98

EX. DATE

COLUMN SPLICE (18)/C5 ~ ABOVE FLOOR LEVEL 5

254 x 254 x 167 UC TO 254 x 254 x 167 UC

MATERIAL :— GRADE 50



NOTE ! :— ENDS OF COLUMN SHAFTS TO BE PREPARED FOR CONTACT BEARING.

BOLTS :—

a) COLUMN FLANGES ~ M24 (GRADE 8.8)

b) COLUMN WEB ~ M20 (GRADE 8.8)

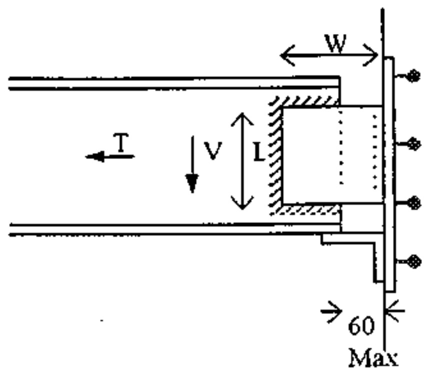
REFERENCE :— SHEET NOS CS17/1-3 (INCLUSIVE)

4

Design Calculations
for
Steelwork Connections to Core Walls
at
5-7 Carlton Gardens
for

April 1998

Connections to Concrete Core Wall



Seating cleat for erection only and not considered for load transfer.
100 % of moment taken in connections to the beam web therefore only shear force and tension apply at weld of fin plate to embedded plate.

Beam B1;

Section 254 x 146 x 37 UB

Shear Force V = 200 kN
Tension Force T = 75 kN

Try 200mm x 210 mm fin plate

ie. S = 150 mm

$$BM = \frac{200\ kN \times (150/2 + 60)}{1000} = 27.0\ kNm$$

By inspection with following sheet adopt a 200 mm x 210 mm x 10 finplate.

Weld size required = 6 mm

However, as weld is to be formed on site adopt next size up;

∴ *Adopt 8 mm full wrap round fillet weld.*

SUPERSEDED
SEE SHEET Nos:
CW/22 - CW/24

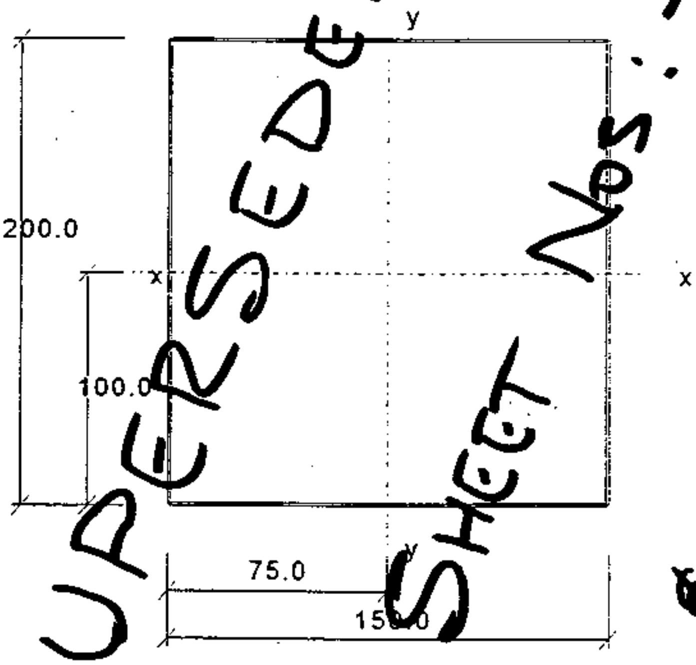
Calculation Sheet

James Lupton Consultants - Inhouse Design Software
General Properties Calculations **Scale 1/ 2**
Typical weld group (up to 8 parts)
Applied Forces

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN	kN	kN	kNm	kNm	kNm
0	**200**	**75**	**27**	**0**	**0**

Weld leg Coordinates Weld Grade 43

Leg	X start	Y Start	X end	Yend	alpha
1	0	0	0	200	90
2	0	200	150	200	-3.8197E-09
3	150	200	150	0	-90
4	150	0	0	0	-180
5	0	0	0	0	-90
6	0	0	0	0	-90
7	0	0	0	0	-90
8	0	0	0	0	-90



SUPERSEDED SEE SHEET Nos: CW/23 & CW/24

Length of weld	70[illegible].00	mm
Ixx of Weld	43333[illegible].33	mm^3
Iyy of weld	2812533.33	mm^3
Min Zxx of weld	4[illegible]117.99	mm^2
Min Zyy of weld	[illegible]7252.10	mm^2
Ixy of Weld	7[illegible]45891.67	mm^3
Max polar radius	1[illegible].00	mm
Z polar of weld	57167.13	mm^2

Weld Stresses due to each force

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN/mm	kN/mm	kN/mm	kN/mm	kN/mm	kN/mm
0.00	0.29	0.11	0.47	0.00	0.00

All weld stresses taken as directly additive
Total weld Load 0.87 kN/mm
Weld Size Required 6 mm fillet Grade 43

Check weld to embedded plate;

For simplicity – add Tension and Shear together.

Total Force = 200 + 75 = 275 kN

Weld Length = 200 x 2 = 400 mm

Weld strength required = 275 ÷ 400 = 0.69 kN/mm

Adopt 8 mm fillet weld for compatibility with beam welds.

Check Fin plate for Shear;

Shear force = 200 kN

Py = 275 N/mm^2

Shear area = 0.9 x 200 x 10 = 1800 mm^2

Pv = 0.6 x 1800 x 275 = 297 kN

200 kN < 297 kN ∴ ok ✓

Fv > 0.6 Pv
∴ *High Shear Condition*

Check Moment Capacity;

$$Z = \frac{200^2 \times 10}{6} = 6.67 \times 10^4 \text{ mm}^3$$

Mcx = 1.2pyZ

Mcx = $1.2 \times 275 \times 6.67 \times 10^{-2}$ = 22 kNm

M = 200 x 0.060 = 12 kNm

For a plate Sx = Sv

Sx = $10 \times 200^2 / 4$ = 100000 mm^3

Mcx = 0.275 x 100 x (1 – (2.5 x 200 / 297 – 1.5))
= 22.45

M < Mcx and Fv > 0.6 Pv;

However, High Shear Condition will not reduce moment capacity to less than that required. ∴ ok ✓

Calculation Sheet

Check Shear Capacity of Studs;

19 mm x 100 Long standard studs.

Shear capacity of studs = 100 kN

Capacity of 6No. studs = 600 kN

Reduction due to Tension;

$Pv' = Pv - T/\sqrt{3} = 600 - 75/\sqrt{3} = 557$ kN

557 kN > 200 kN ∴ ok ✓

Check Tensile Capacity of Studs;

Assume 90 mm effect embedment.

Assume 45° cone and no reinforcement.

Perimeter of cone base = $2\pi r$ = $2 \times \pi \times 90$ = 565 mm

Pull out capacity = $0.35 \times 565 \times 90 \div 1000$ = 18 kN

Tension per stud = $75 \div 6$ = 12.5 kN

12.5 kN < 18 kN ∴ ok ✓

Beam B4;

Section 203 x 133 x 30 UB

Shear Force V = 123 kN
Tension Force T = 75 kN

Try 150mm x 160mm fin plate;

ie. S = 100 mm

$$BM = \frac{123\ kN \times (100/2 + 65)}{1000} = 13.53\ kNm$$

By inspection with following sheet adopt a 150 mm x 160 mm x 10 finplate.

Weld size required = 6 mm

However, as weld is to be formed on site adopt next size up;

∴ Adopt 8 mm full wrap round fillet weld

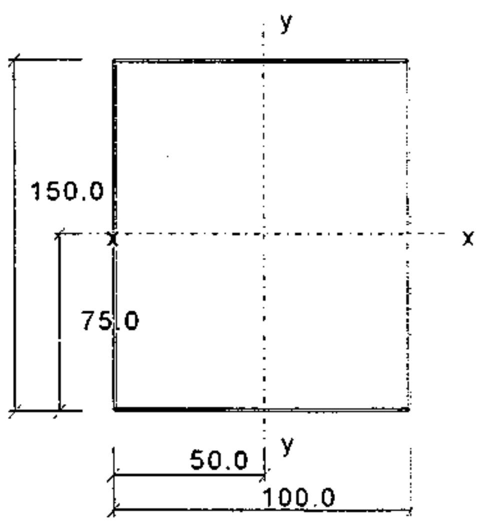
Calculation Sheet

James Lupton Consultants - Inhouse Design Software
General Properties Calculations **Scale 1/ 2**
Typical weld group (up to 8 parts)
Applied Forces

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN	kN	kN	kNm	kNm	kNm
0	**123**	**75**	**13.53**	**0**	**0**

Weld leg Coordinates Weld Grade 43

Leg	X start	Y Start	X end	Yend	alpha
1	0	0	0	150	90
2	0	150	100	150	-5.7296E-09
3	100	150	100	0	-90
4	100	0	0	0	-180
5	0	0	0	0	-90
6	0	0	0	0	-90
7	0	0	0	0	-90
8	0	0	0	0	-90



Length of weld	500.00	mm
Ixx of Weld	1687516.67	mm^3
Iyy of weld	916691.67	mm^3
Min Zxx of weld	22351.21	mm^2
Min Zyy of weld	18152.31	mm^2
Ixy of Weld	2604208.33	mm^3
Max polar radius	90.14	mm
Z polar of weld	28891.10	mm^2

Weld Stresses due to each force

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN/mm	kN/mm	kN/mm	kN/mm	kN/mm	kN/mm
0.00	0.25	0.15	0.47	0.00	0.00

All weld stresses taken as directly additive
Total weld Load 0.86 kN/mm
Weld Size Required 6 mm fillet Grade 43

Check weld to embedded plate;

For simplicity – add Tension and Shear together.

Total Force	=	123 + 75	=	198 kN		
Weld Length	=	150 x 2	=	300 mm		
Weld strength required			=	198 ÷ 300	=	0.66 kN/mm

Adopt 8 mm fillet weld for compatibility with beam welds.

Check Fin plate for Shear;

Shear force	=	123 kN		
Py	=	275 N/mm^2		
Shear area	=	0.9 x 150 x 10	=	1350 mm^2
Pv	=	0.6 x 1350 x 0.275	=	223 kN
123 kN	<	223 kN		∴ ok ✓

Fv < 0.6 Pv

∴ Low Shear Condition

Check Moment Capacity

$$Z = \frac{150^2 \times 10}{6} = 3.75 \times 10^4 \text{ mm}^3$$

Mcx = 1.2pyZ

Mcx = $1.2 \times 275 \times 3.75 \times 10^{-2}$ = 12.4 kNm

M = 123 x 0.060 = 7.4 kNm

As M < Mcx and Fv < 0.6 Pv;

Adopt a 150 mm x 160 mm x 10 finplate.

∴ ok ✓

Check Shear Capacity of Studs;

19 mm x 100 Long standard studs.

Shear capacity of studs = 100 kN

Capacity of 6No. studs = 600 kN
Reduction due to Tension;

$Pv' = Pv - T/\sqrt{3} = 600 - 75/\sqrt{3} = 556$

556 kN > 123 kN ∴ ok ✓

Check Tensile Capacity of Studs;

Assume 90 mm effect embedment.

Assume 45° cone and no reinforcement.

Perimeter of cone base = $2\pi r$ = $2 \times \pi \times 90$ = 565 mm

Pull out capacity = $0.35 \times 565 \times 90 \div 1000$ = 18 kN

Tension per stud = $75 \div 6$ = 12.5 kN

12.5 kN < 18 kN ∴ ok ✓

Beam B5; Ground

Section 305 x 102 x 33 UB

Shear Force V = 177 kN
Tension Force T = 75 kN

Try 225mm x 135mm fin plate;

ie. S = 75 mm

$$BM = \frac{177\ kN \times (75/2 + 65)}{1000} = 18.14\ kNm$$

By inspection with following sheet adopt a 225 mm x 135 mm x 10 finplate.

Weld size required 6 mm

However, as weld is to be formed on site adopt next size up;

∴ *Adopt 8 mm full wrap round fillet weld.*

SUPERSEDED

SEE SHEET NOS

CW/37 TO CW/40

Kvaerner Cleveland Bridge Ltd

O/No. :M347 Job : Carlton Gardens
Subject : Design of Connections to Core Wall

Series :
Sht. No. : CW/11 Rev. : A
By : AJC Date : April 98
EX : JL Date : May 98

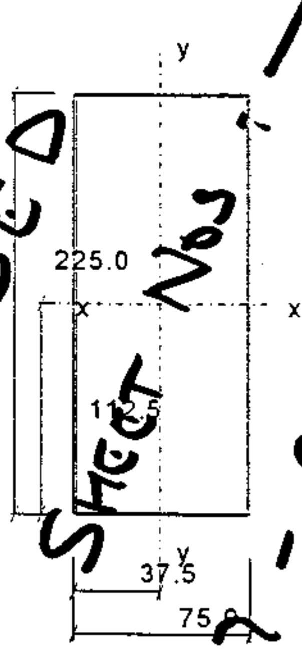
Calculation Sheet

James Lupton Consultants - Inhouse Design Software
General Properties Calculations **Scale 1/ 2.5**
Typical weld group (up to 8 parts)
Applied Forces

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN	kN	kN	kNm	kNm	kNm
0	**177**	**75**	**18.14**	**0**	**0**

Weld leg Coordinates Weld Grade 43

Leg	X start	Y Start	X end	Yend	alpha
1	0	0	0	225	90
2	0	225	75	225	-7.6394E-09
3	75	225	75	0	-90
4	75	0	0	0	-180
5	0	0	0	0	-90
6	0	0	0	0	-90
7	0	0	0	0	-90
8	0	0	0	0	-90



SUPERSEDED

SEE SHEET Nos CW/37 - CW/40

Length of weld	600.00	mm
Ixx of Weld	3796887.50	mm^3
Iyy of weld	70[illegible]16[illegible].50	mm^3
Min Zxx of weld	[illegible]360[illegible].77	mm^2
Min Zyy of weld	18504.28	mm^2
Ixy of Weld	4500050.00	mm^3
Max polar radius	118.59	mm
Z polar of weld	37947.75	mm^2

Weld Stresses due to each force

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN/mm	kN/mm	kN/mm	kN/mm	kN/mm	kN/mm
0.00	0.30	0.13	0.48	0.00	0.00

All weld stresses taken as directly additive
Total weld Load 0.90 kN/mm
Weld Size Required 6 mm fillet Grade 43

Check weld to embedded plate;

For simplicity – add Tension and Shear together.

Total Force = 177 + 75 = 252 kN

Weld Length = 225 x 2 = 450 mm

Weld strength required = 252 ÷ 450 = 0.56 kN/mm

Adopt 8 mm fillet weld for compatibility with beam welds.

Check Fin plate for Shear;

Shear force = 177 kN

Py = 275 N/mm^2

Shear area = 0.9 x 225 x 10 = 2025 mm^2

Pv = 0.6 x 2025 x 0.275 = 334 kN

177 kN < 334 kN ∴ ok ✓

Fv < 0.6 Pv

∴ *Low Shear Condition*

Check Moment Capacity

$Z = \dfrac{225^2 \times 10}{6} = 8.44 \times 10^4$ mm^3

Mcx = 1.2pyZ

Mcx = 1.2 x 275 x 8.44 x 10^{-3} = 27.85 kNm

M = 177 x 0.060 = 10.62 kNm

As M < Mcx and Fv < 0.6 Pv;

Adopt a 225 mm x 135 mm x 10 finplate.

∴ ok ✓

SUPERSEDED SHEET NOS 1 TO CW/40

(SEE CW/37)

Check Shear Capacity of Studs;

19 mm x 100 Long standard studs.

Shear capacity of studs = 100 kN

Capacity of 6No. studs = 600 kN
Reduction due to Tension;

$Pv' = Pv - T/\sqrt{3} = 600 - 75/\sqrt{3} = 556$

556 kN > 177 kN ∴ ok ✓

Check Tensile Capacity of Studs;

Assume 90 mm effect embedment.

Assume 45° cone and no reinforcement.

Perimeter of cone base = $2\pi r$ = $2 \times \pi \times 90$ = 565 mm

Pull out capacity = $0.35 \times 565 \times 90 \div 1000$ = 18 kN

Tension per stud = $75 \div 6$ = 12.5 kN

12.5 kN < 18 kN ∴ ok ✓

Kvaerner Cleveland Bridge Ltd

O/No. :M347 Job : Carlton Gardens
Subject : Design of Connections to Core Wall

Series :
Sht. No. : CW/15 Rev. : A
By : AJC Date : April 98
EX : JL Date : May 98

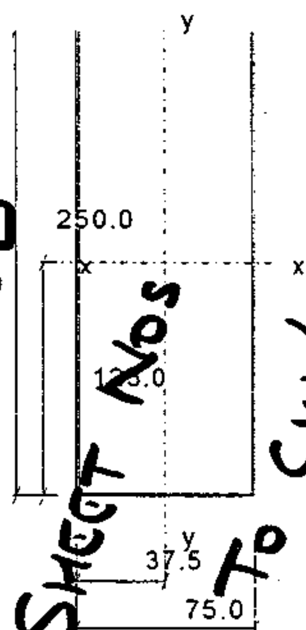
Calculation Sheet

James Lupton Consultants - Inhouse Design Software
General Properties Calculations **Scale 1/ 2.5**
Typical weld group (up to 8 parts)
Applied Forces

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN	kN	kN	kNm	kNm	kNm
0	200	75	20.5	0	0

Weld leg Coordinates Weld Grade 43

Leg	X start	Y Start	X end	Yend	alpha
1	0	0	0	250	90
2	0	250	75	250	-7.6394E-09
3	75	250	75	0	-90
4	75	0	0	0	-180
5	0	0	0	0	-90
6	0	0	0	0	-90
7	0	0	0	0	-90
8	0	0	0	0	-90



SUPERSEDED SEE SHEET NOS CW/29 TO CW/36

Length of weld	650.00	mm
Ixx of Weld	49479[illegible].17	mm^3
Iyy of weld	772[illegible]9.17	mm^3
Min Zxx of weld	[illegible]9425.73	mm^2
Min Zyy of weld	203[illegible]4.71	mm^2
Ixy of Weld	5721408.33	mm^3
Max polar radius	130.50	mm
Z polar of weld	43840.92	mm^2

Weld Stresses due to each force

Axial Z	Shear X	Shear Y	Moment Mzz	Moment Mxx	Moment Myy
kN/mm	kN/mm	kN/mm	kN/mm	kN/mm	kN/mm
0.00	0.31	0.12	0.47	0.00	0.00

All weld stresses taken as directly additive
Total weld Load 0.89 kN/mm
Weld Size Required 6 mm fillet Grade 43

Beam B5; Ground (LOWER)

Section 356 x 171 x 51 UB

Shear Force V	=	200 kN
Tension Force T	=	75 kN

Try 250mm x 135mm fin plate;

ie. S = 75 mm

$$BM = \frac{200\text{ kN} \times (75/2 + 65)}{1000} = 20.5\text{ kNm}$$

By inspection with following sheet adopt a 250 mm x 135 mm x 10 finplate.

Weld size required = 6 mm

However, as weld is to be formed on site adopt next size up;

∴ Adopt 8 mm full wrap round fillet weld

SUPERSEDED
SEE SHEET NOS
CW/29 TO CW/36

Kvaerner Cleveland Bridge Ltd

O/No. :M347 Job : Carlton Gardens
Subject : Design of Connections to Core Wall

Series :
Sht. No. : CW/16 Rev. : A
By : AJC Date : April 98
EX : JL Date : May 98

Calculation Sheet

Check weld to embedded plate;

For simplicity – add Tension and Shear together.

Total Force = 200 + 75 = 275 kN

Weld Length = 250 x 2 = 500 mm

Weld strength required = 275 ÷ 500 = 0.55 kN/mm

Adopt 8 mm fillet weld for compatibility with beam welds.

Check Fin plate for Shear;

Shear force = 200 kN

Py = 275 N/mm^2

Shear area = 0.9 x 250 x 10 = 2250 mm^2

Pv = 0.6 x 2250 x 0.275 = 371 kN

200 kN < 371 kN ∴ ok ✓

Fv < 0.6 Pv

∴ *Low Shear Condition*

SUPERSEDED SEE SHEET Nos CW/29 TO CW/36

Check Moment Capacity

$$Z = \frac{250^2 \times 10}{6} = 1.04 \times 10^5 \text{ mm}^3$$

Mcx = 1.2pyZ

Mcx = 1.2 x 275 x 1.04 / 10 = 34.32 kNm

M = 200 x 0.060 = 12.0 kNm

As M < Mcx and Fv < 0.6 Pv;

Adopt a 250 mm x 135 mm x 10 finplate.

∴ ok ✓

Check Shear Capacity of Studs;

19 mm x 100 Long standard studs.

Shear capacity of studs = 100 kN

Capacity of 6No. studs = 600 kN
Reduction due to Tension;

$Pv' = Pv - T/\sqrt{3} = 600 - 75/\sqrt{3} = 556$

556 kN > 200 kN $\therefore$ ok ✓

Check Tensile Capacity of Studs;

Assume 90 mm effect embedment.

Assume 45° cone and no reinforcement.

Perimeter of cone base $= 2\pi r = 2 \times \pi \times 90 =$ 565 mm

Pull out capacity $= 0.35 \times 565 \times 90 \div 1000 =$ 18 kN

Tension per stud $= 75 \div 6 =$ 12.5 kN

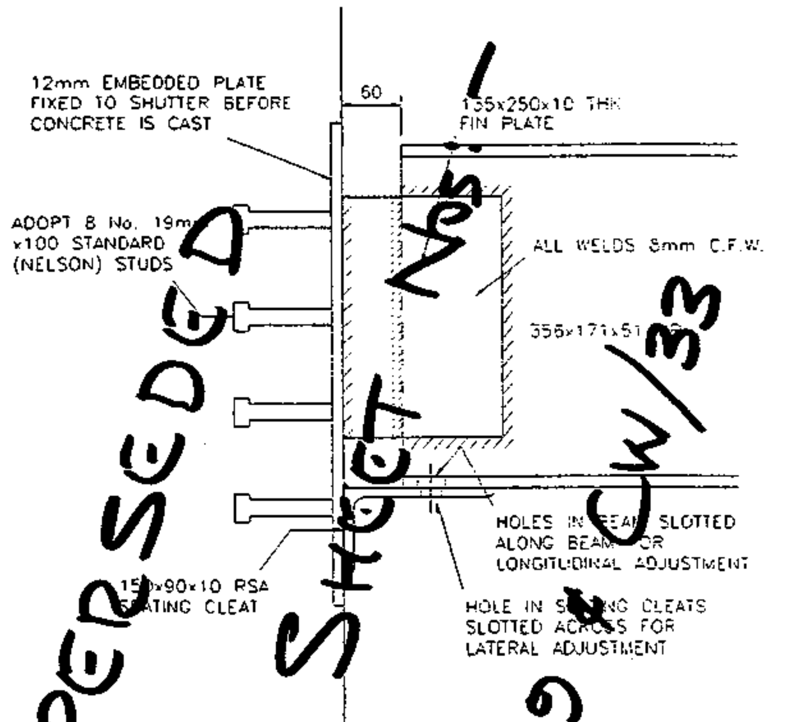
12.5 kN < 18 kN $\therefore$ ok ✓

Kvaerner Cleveland Bridge Ltd

O/No. :M347 Job : Carlton Gardens
Subject : Design of Connections to Core Wall

Series :
Sht. No. : CW/18 Rev. : A
By : AJC Date : April 98
EX : JL Date : May 98

Calculation Sheet



CONCRETE CORE WALL CONNECTION
TO BEAM B5 - LOWER GROUND FLOOR

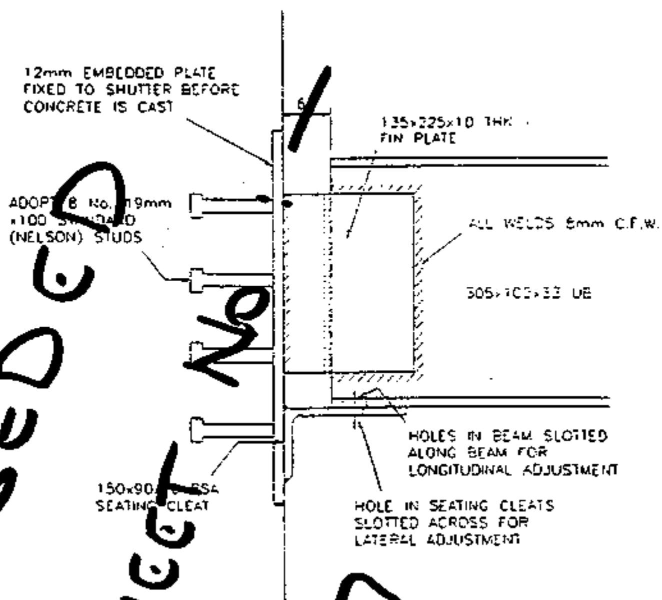
SUPERSEDED
SEE SHEET Nos CW/33 & CW/29

Kvaerner Cleveland Bridge Ltd

O/No. :M347 Job : Carlton Gardens
Subject : Design of Connections to Core Wall

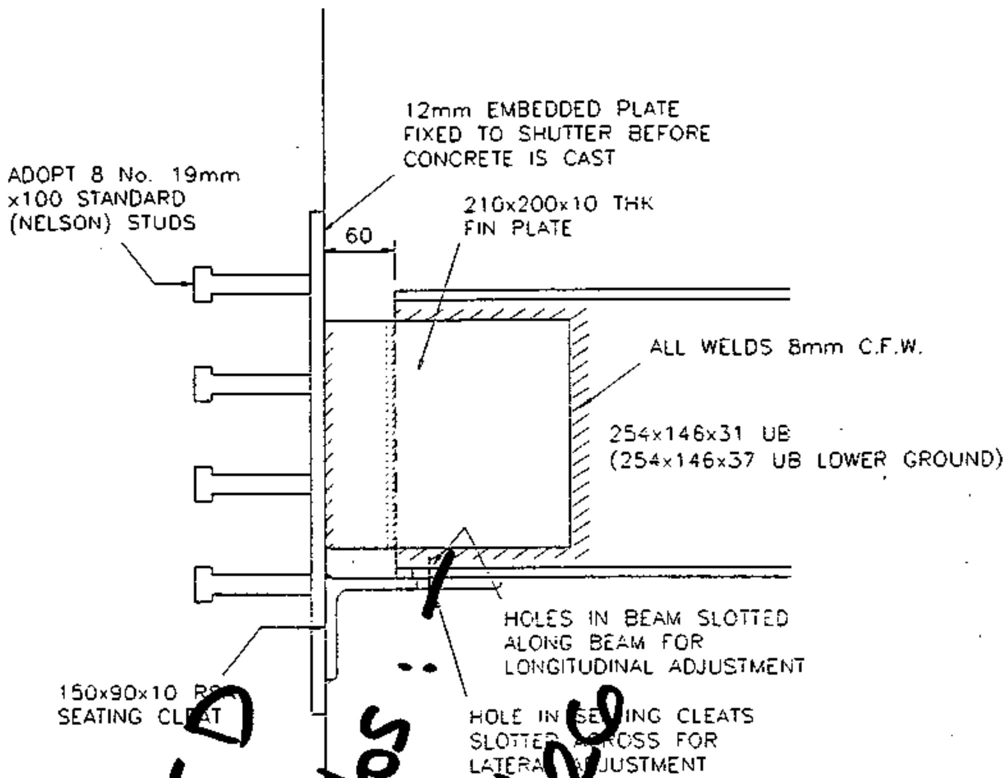
Series :
Sht. No. : CW/19 Rev. : A
By : AJC Date : April 98
EX : JL Date : May 98

Calculation Sheet



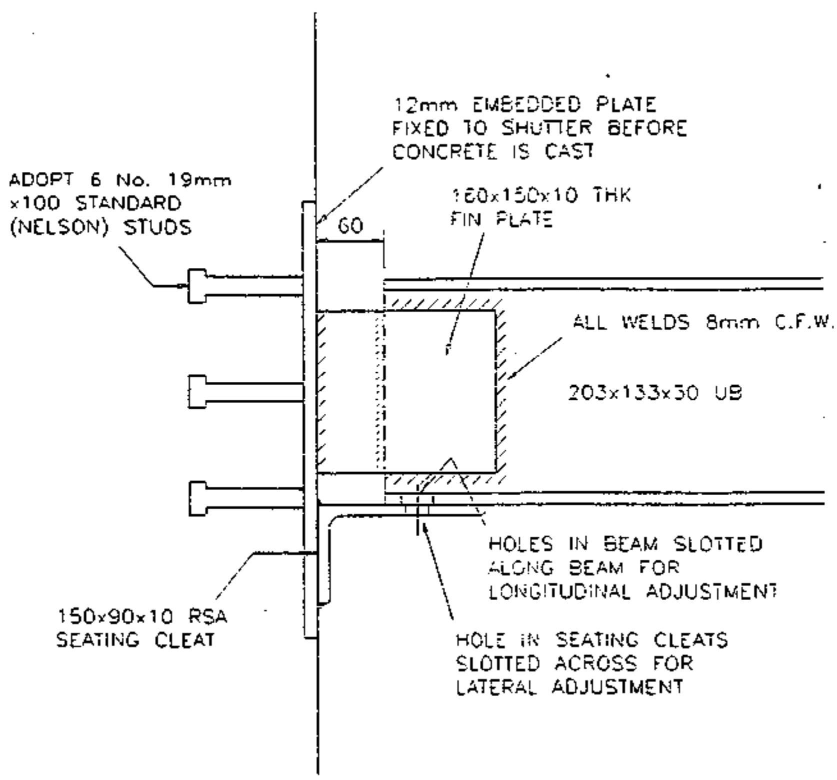
CONCRETE CORE WALL CONNECTION
TO BEAM B5 - GROUND FLOOR

SUPERSEDED
SEE SHEET No CW/37



CONCRETE CORE WALL CONNECTION
TO BEAM B1 – GROUND FLOOR

SUPERSEDED
SEE SHEET Nos: CW/22 & CW/26



CONCRETE CORE WALL CONNECTION
TO BEAM B4 – FIRST FLOOR

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES .... SHT. No. CW/22 REV. ....

SUBJECT CONNECTIONS TO CONCRETE CORE WALL — BY KB DATE 05/98

BEAM MARK B1-LG & B1-G — EX. .... DATE ....

(B7-G ~ SIMILAR)

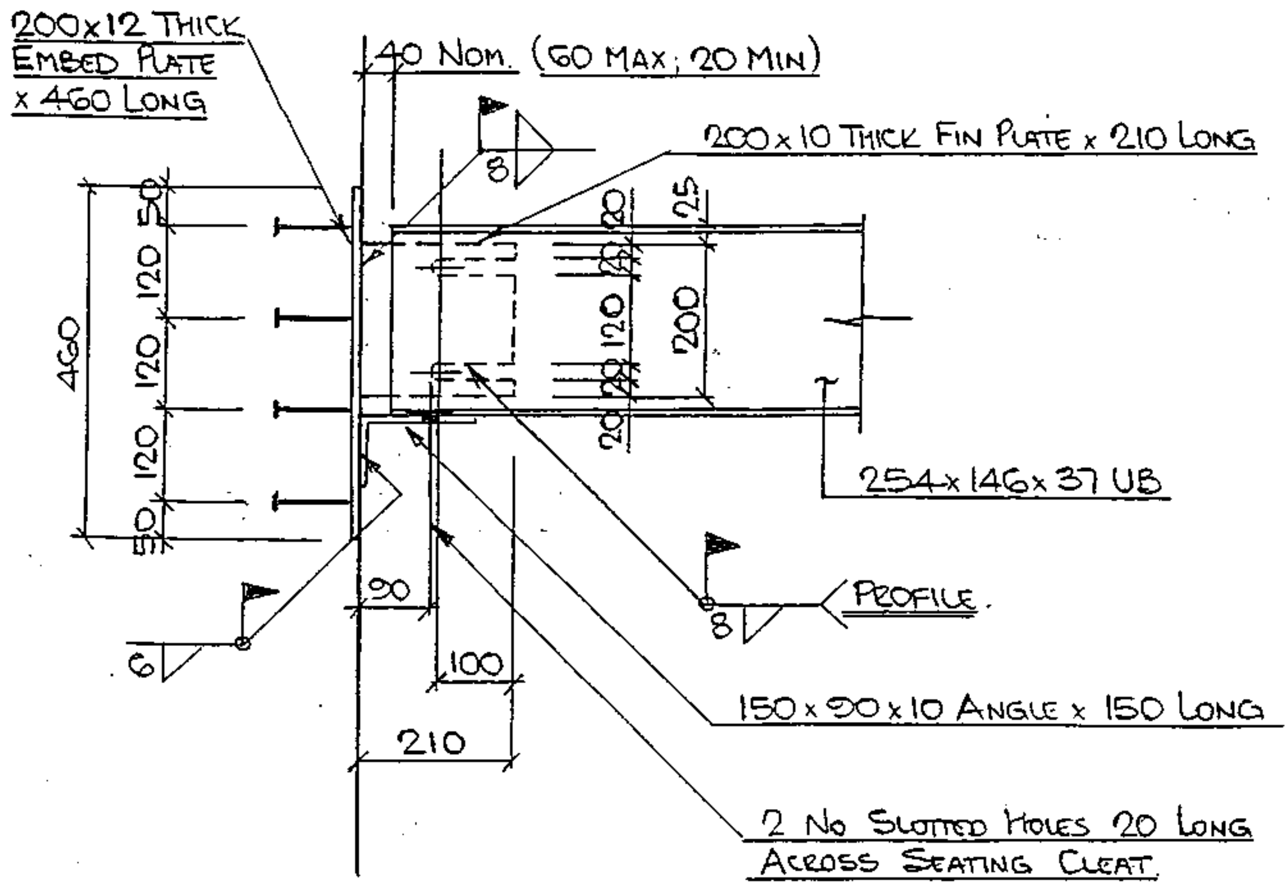
BEAM MARK :— B1-LG

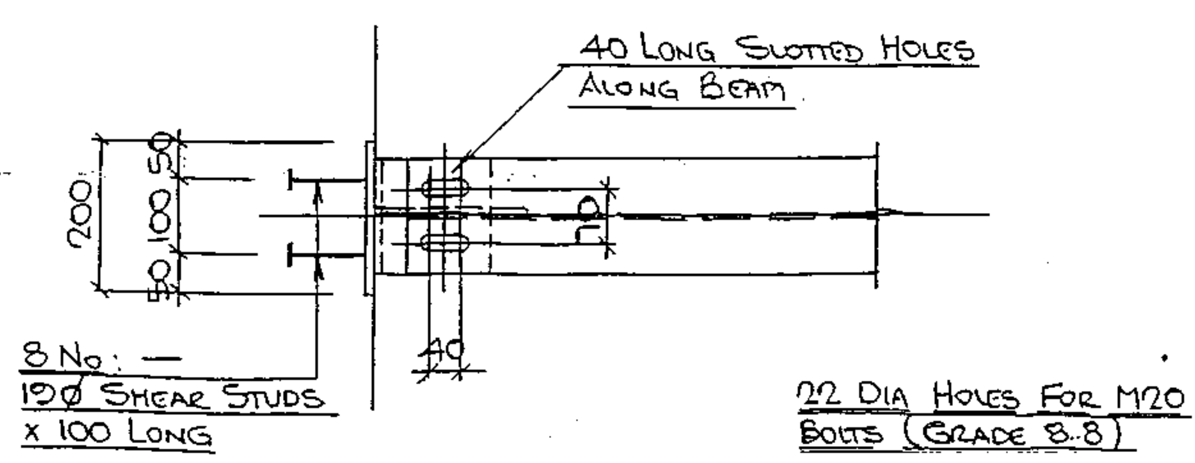
SECTION : — 254 x 146 x 37 UB

SHEAR LOAD : 200 KN

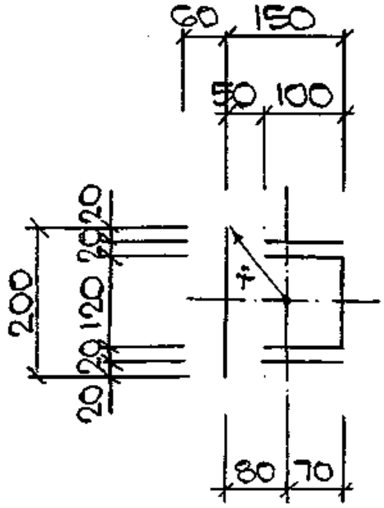
AXIAL LOAD = 75 KN (TENSION)

REFERENCE : — SHEET NOS CW/2 TO CW/5 (INCLUSIVE)





DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA = $200 + (4 \times 100) + 120 = 720\ mm^2$

TO FIND NEUTRAL AXIS ~

TAKE MOMENTS ABOUT L.H. SIDE

$720\bar{x} = (400 \times 100) + (120 \times 150)$

$\bar{x} = 58000 \div 720 = \underline{80mm}$

$I_{xx} = \left(\frac{200^3}{12}\right) + \left(\frac{4 \times 100}{12}\right) + \left(\frac{120^3}{12}\right) + (2 \times 100 \times \{60^2 + 80^2\}) = 2810700\ mm^4$

$I_{yy} = \left(\frac{200}{12}\right) + \left(\frac{4 \times 100^3}{12}\right) + \left(\frac{120}{12}\right) + (200 \times 80^2) + (4 \times 100 \times 20^2) + (120 \times 70^2)$

$= 2361360\ mm^4$

$I_p = 2810700 + 2361360 = 5172060\ mm^4$

$\bar{r} = \sqrt{100^2 + 80^2} = 128.0\ mm$

$Z_p = 5172060 \div 128.0 = 40406\ mm^3$

$f_V = 200 \div 720 = 0.28\ KN/mm^2$

$f_H = 75 \div 720 = 0.11\ KN/mm^2$

$f_m = 200 \times (80 + 60) \div 40406 = 0.70\ KN/mm^2$

$f_m(V_c) = 0.70 \times 80 \div 128.0 = 0.44\ KN/mm^2$

$f_m(H_c) = 0.70 \times 100 \div 128.0 = 0.54\ KN/mm^2$

$f_{RES} = \sqrt{(0.28 + 0.44)^2 + (0.11 + 0.54)^2} = 0.97\ KN/mm^2$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B1-LG & B1-G

(B7-G ~ SIMILAR)

SERIES

SHT. No. CW/24 REV.

BY KB DATE 05/98

EX. DATE

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D $= \dfrac{0.97 \times 10^3}{0.7 \times 215} = 6.5$ mm $\longrightarrow$ 8 LEG F.W.

USE :— 8 LEG FILLET WELD

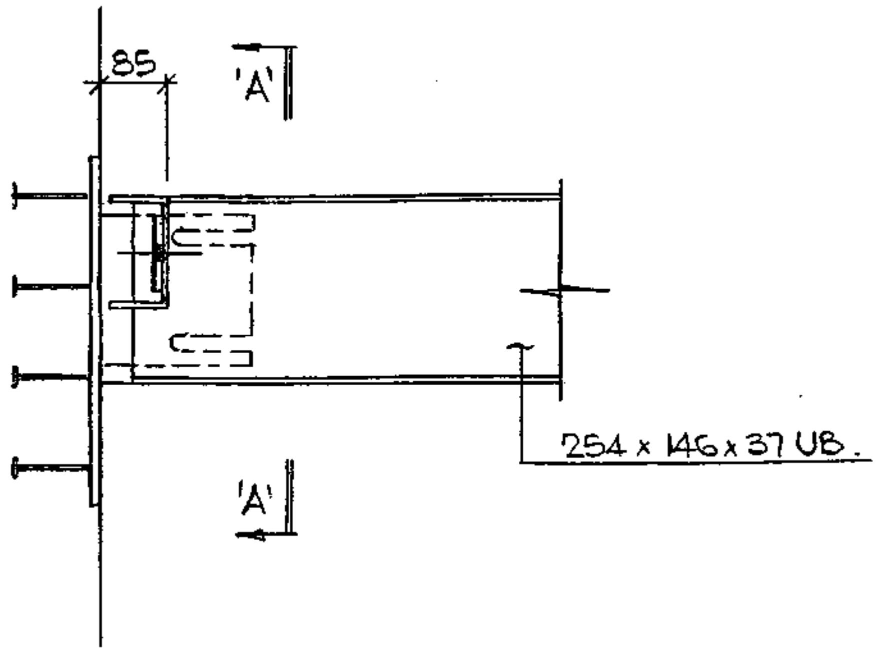
Kvaerner Cleveland Bridge Ltd.

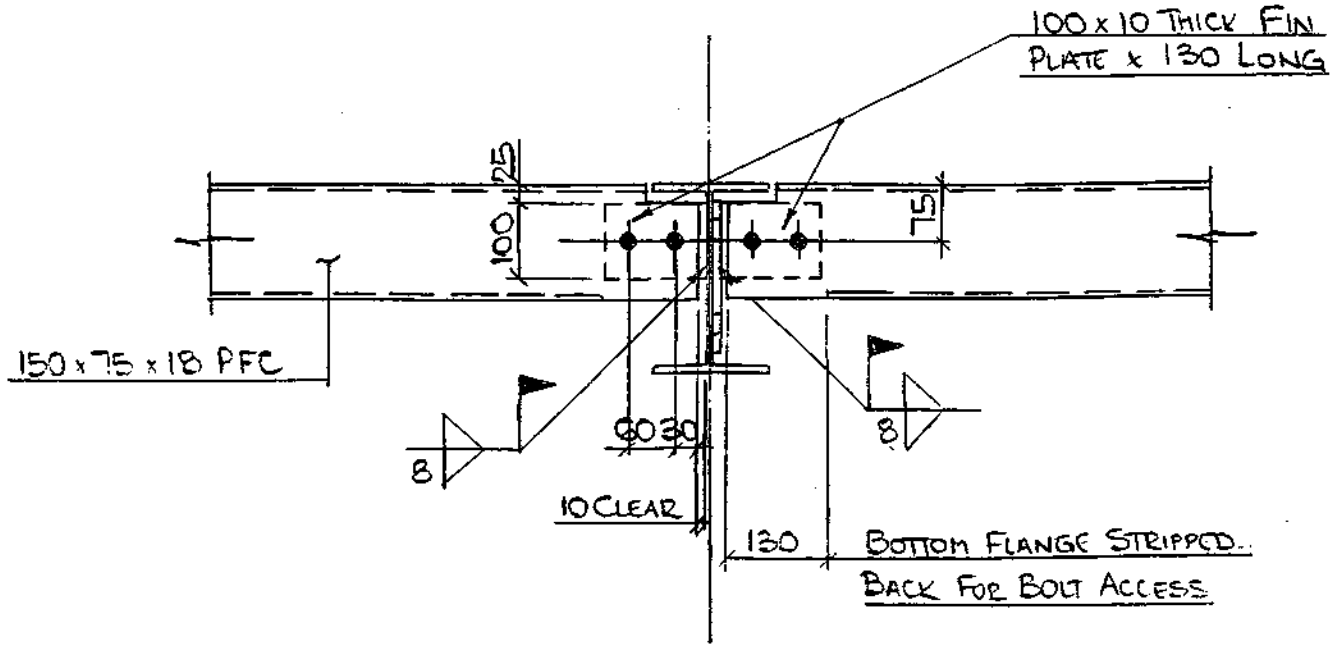
O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL
B12-LG & B12-G (REPLACEMENT) TO B1-LG & B1-G

SERIES
SHT. No. CW/25 REV.
BY KB DATE 05/98
EX. DATE

150 x 75 x 18 PFC TO 254 x 146 x 37 UB





VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B1-LG# & B1-LG#

SERIES

SHT. No. CW/26 REV.

BY KB DATE 05/98

EX. DATE

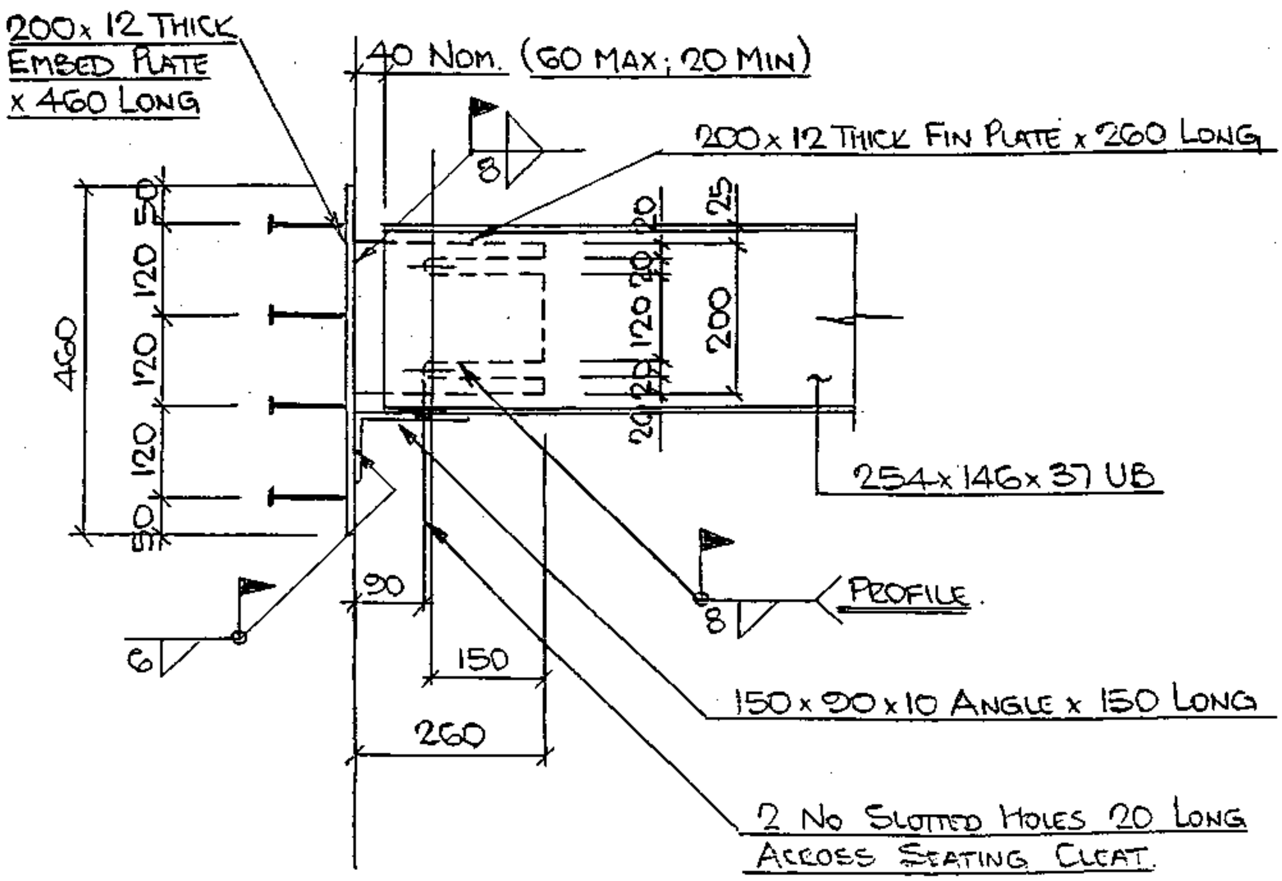
BEAM MARK :— B1-LG#

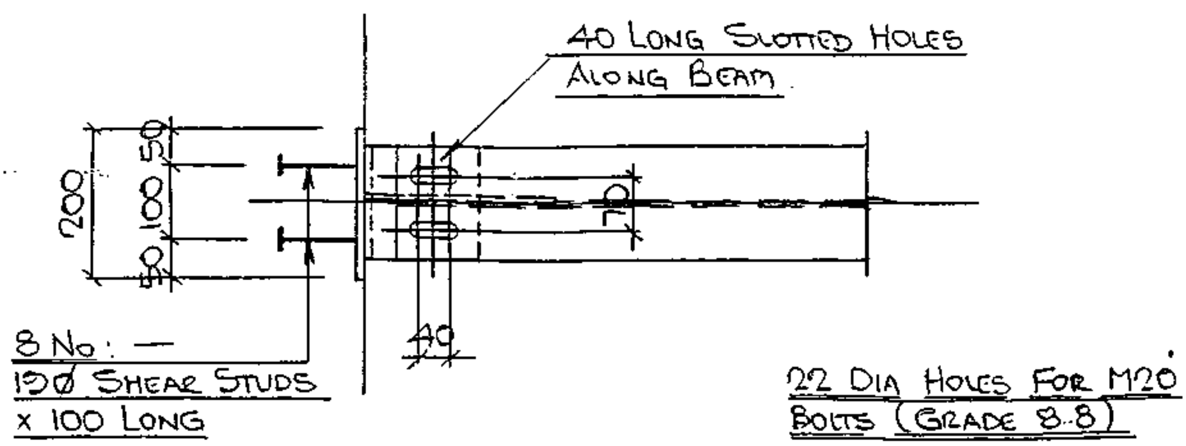
SECTION : — 254 x 146 x 37 UB

SHEAR LOAD : 300 KN

AXIAL LOAD : 75 KN (TENSION)

REFERENCE : — SHEET NOS CW/2 TO CW/5 (INCLUSIVE)





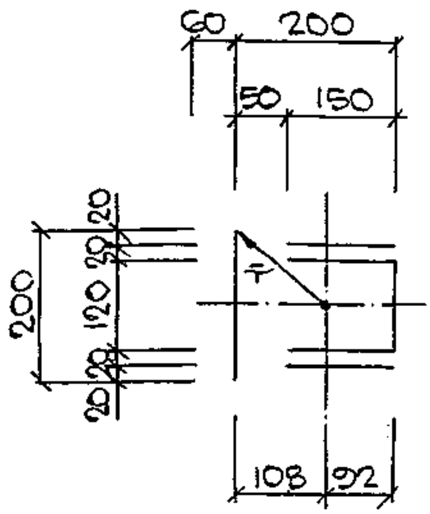
Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES .... SHT. No. CW/27 REV. ....

SUBJECT CONNECTIONS TO CONCRETE CORE WALL — BY KB DATE 05/98

BEAM MARK B1-LG# & B1-G# — EX. .... DATE ....

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

$$\text{AREA} : 200 + (4 \times 150) + 120 = 920 \text{ mm}^2$$

TO FIND NEUTRAL AXIS

TAKE MOMENTS ABOUT L.H. SIDE

$$920\bar{x} = (600 \times 125) + (120 \times 200)$$

$$\bar{x} = 99000 \div 920 = \underline{108 \text{ mm}}$$

$$I_{xx} = \left(\frac{200^3}{12}\right) + \left(\frac{4 \times 150}{12}\right) + \left(\frac{120^3}{12}\right) + \left(2 \times 150 \times \{60^2 + 80^2\}\right) = 3810716 \text{ mm}^4.$$

$$I_{yy} = \left(\frac{200}{12}\right) + \left(\frac{4 \times 150^3}{12}\right) + \left(\frac{120}{12}\right) + (200 \times 108^2) + (600 \times 17^2) + (120 \times 92^2)$$

$$= 4646906 \text{ mm}^4$$

$$I_p = 3810716 + 4646906 = 8457622 \text{ mm}^4.$$

$$r = \sqrt{100^2 + 108^2} = 147.2 \text{ mm}.$$

$$Z_p = 8457622 \div 147.2 = 57456 \text{ mm}^3$$

$$f_V = 300 \div 920 = 0.33 \text{ KN/mm}^2$$

$$f_H = 75 \div 920 = 0.08 \text{ KN/mm}^2$$

$$f_M = 300 \times (108 + 60) \div 57456 = 0.88 \text{ KN/mm}^2$$

$$f_M(V_C) = 0.88 \times 108 \div 147.2 = 0.65 \text{ KN/mm}^2$$

$$f_M(H_C) = 0.88 \times 100 \div 147.2 = 0.60 \text{ KN/mm}^2$$

$$F_{RES} = \sqrt{(0.33 + 0.65)^2 + (0.08 + 0.60)^2} = 1.20 \text{ KN/mm}^2.$$

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D = $\frac{1.20 \times 10^3}{0.7 \times 215}$ = 7.9 mm ⟶ 8 LEG F.W. (MIN^{M}/)

USE :— 8 LEG FILLET WELD

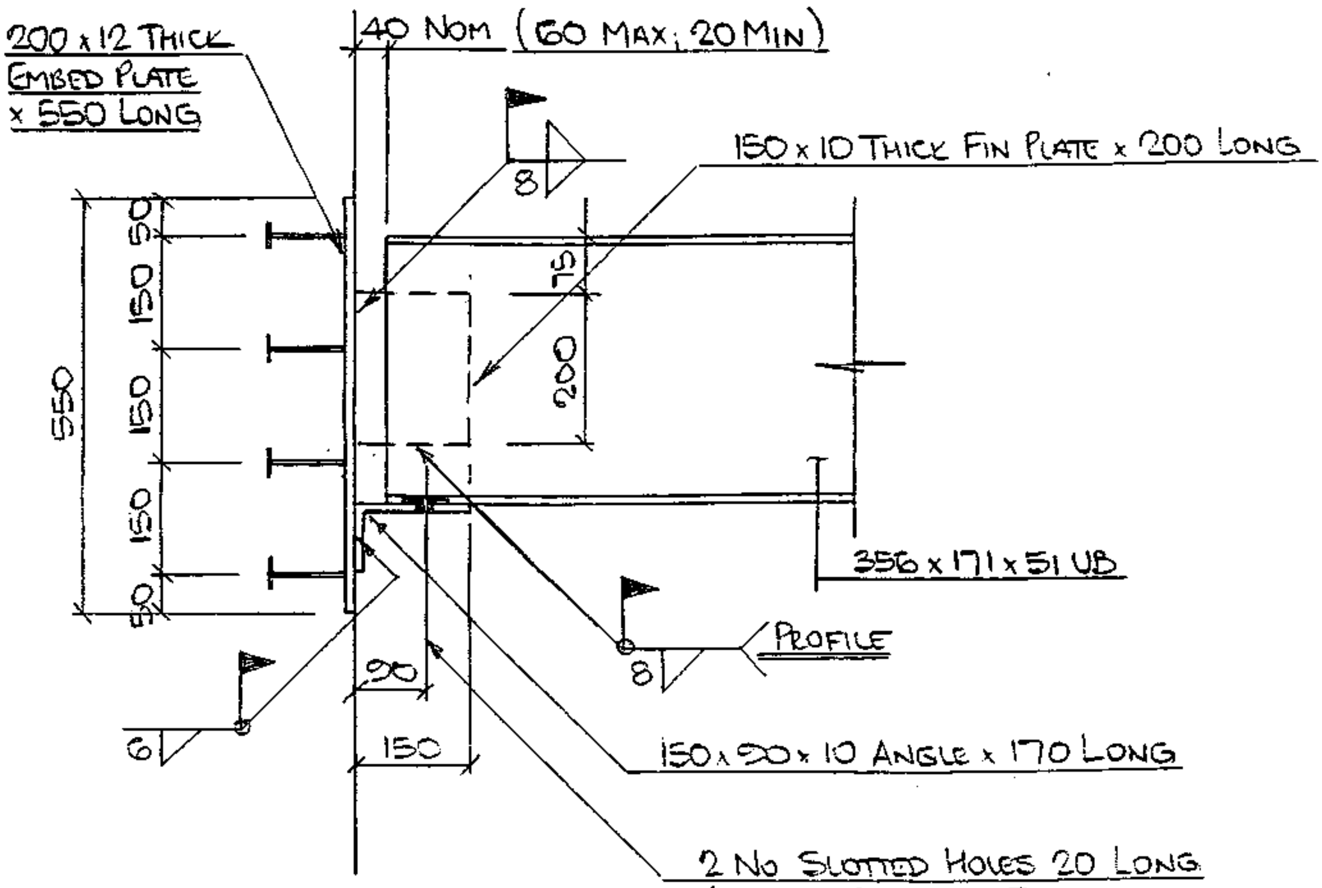
BEAM MARK : — B5 - LG

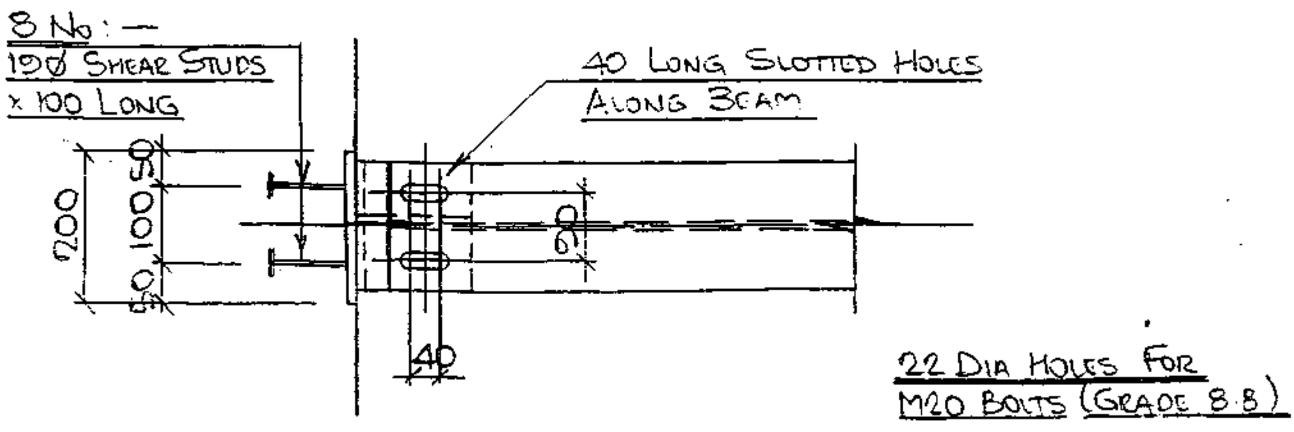
SECTION : — 356 x 171 x 51 UB

SHEAR LOAD = 200 KN

AXIAL LOAD = 75 KN (TENSION)

REFERENCE : — SHEET Nos CW/14 TO CW/17

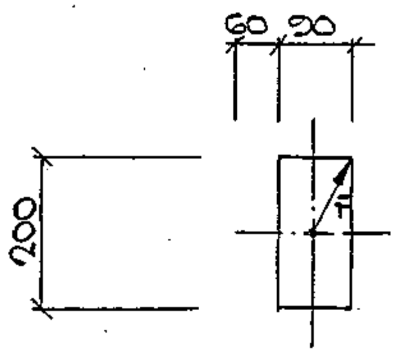




Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS	SERIES
SUBJECT CONNECTIONS TO CONCRETE CORE WALL	SHT. No. CW/30 REV.
BEAM MARK B5 - LG	BY KB DATE 05/98
	EX. DATE

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA = $2 \times (200 + 90) = 580\ mm^2$

$r = \sqrt{45^2 + 100^2} = 109.7\ mm$

$I_{xx} = \left(\frac{2 \times 200^3}{12}\right) + \left(\frac{2 \times 90}{12}\right) + (2 \times 90 \times 100^2) = 3133348\ mm^4$

$I_{yy} = \left(\frac{2 \times 200}{12}\right) + \left(\frac{2 \times 90^3}{12}\right) + (2 \times 200 \times 45^2) = 931533\ mm^4$

$I_p = 3133348 + 931533 = 4064881\ mm^4$

$Z_p = 4064881 \div 109.7 = 37054\ mm^3$

$f_V = 200 \div 580 = 0.35\ KN/mm^2$

$f_H = 75 \div 580 = 0.13\ KN/mm^2$

$f_M = 200 \times 105 \div 37054 = 0.57\ KN/mm^2$

$f_M (Vc) = 0.57 \times 45 \div 109.7 = 0.24\ KN/mm^2$

$f_M (Hc) = 0.57 \times 100 \div 109.7 = 0.52\ KN/mm^2$

$F_{RES} = \sqrt{(0.35 + 0.24)^2 + (0.13 + 0.52)^2} = 0.85\ KN/mm^2$

WELD STRENGTH (GRADE 43) = $215\ N/mm^2$

WELD SIZE REQ^D = $\frac{0.85 \times 10^3}{0.70 \times 215} = 5.7\ mm$ ⟶ 6 LEG F.W (MIN^M)

USE :— 15 LEG FILLET WELD (DETAIL STANDARDIZATION)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS | SERIES .... | SHT. No. CW/31 REV. ....

SUBJECT CONNECTIONS TO CONCRETE CORE WALL | BY KB DATE 05/98

BEAM MARK BS-LG | EX. .... DATE ....

<u>CHECK WELD TO EMBEDMENT PLATE</u>

LENGTH OF FIN PLATE : 200 mm

EFFECTIVE WELD LENGTH : $200 - (2 \times 8) = 184$ mm

$F_V = 200 \div (2 \times 184) = 0.55$ KN/mm²

$F_H = 75 \div (2 \times 184) = 0.21$ KN/mm²

$F_{RES} = \sqrt{0.55^2 + 0.21^2} = 0.59$ KN/mm²

WELD STRENGTH (GRADE 43) = 215 N/MM²

WELD SIZE REQ^D $= \dfrac{0.59 \times 10^3}{0.70 \times 215} = 4.0$ mm $\longrightarrow$ 6 LEG F.W. (MIN^M)

<u>USE : — 8 LEG FILLET WELD</u> <u>(DETAIL STANDARDIZATION)</u>

<u>CHECK FIN PLATE ~ 10 THICK</u>

<u>a) SHEAR</u>

SHEAR FORCE = 200 KN

$P_y = 275$ N/mm²

$A_V = 0.9 \times 10 \times 200 = 1800$ mm²

$P_V = 0.6 \times 275 \times 1800 \div 10^3$

$= \underline{297 \text{ KN} > 200 \text{ KN}}$ OK ($F_V > 0.6 P_V$)

<u>(b) MOMENT</u>

BENDING MOMENT = $200 \times 60 = 12000$ KNMM

$Z = 10 \times 200^2 \div 6 = 66666$ mm³

$M_{cx} = 1.2 \times P_y \times Z$

$= 1.2 \times 275 \times 66666 \div 1000 = 22000$ KNMM $>$ 12000 KNMM OK

FOR A PLATE Sx = Sv

Sx = $10 \times 200^2 \div 4 = 100000 \text{ mm}^3$

Mcx = $275 \times 100000 \times (1 - (25 \times 200 \div 297 - 1.5)) \div 1000$

= 22453 KNmm > 12000 KNmm OK

USE :— 10 THICK FIN PLATE

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B5-LG#

(B10-G ~ SIMILAR)

SERIES

SHT. No. CW/33 REV.

BY KB DATE 05/98

EX. DATE

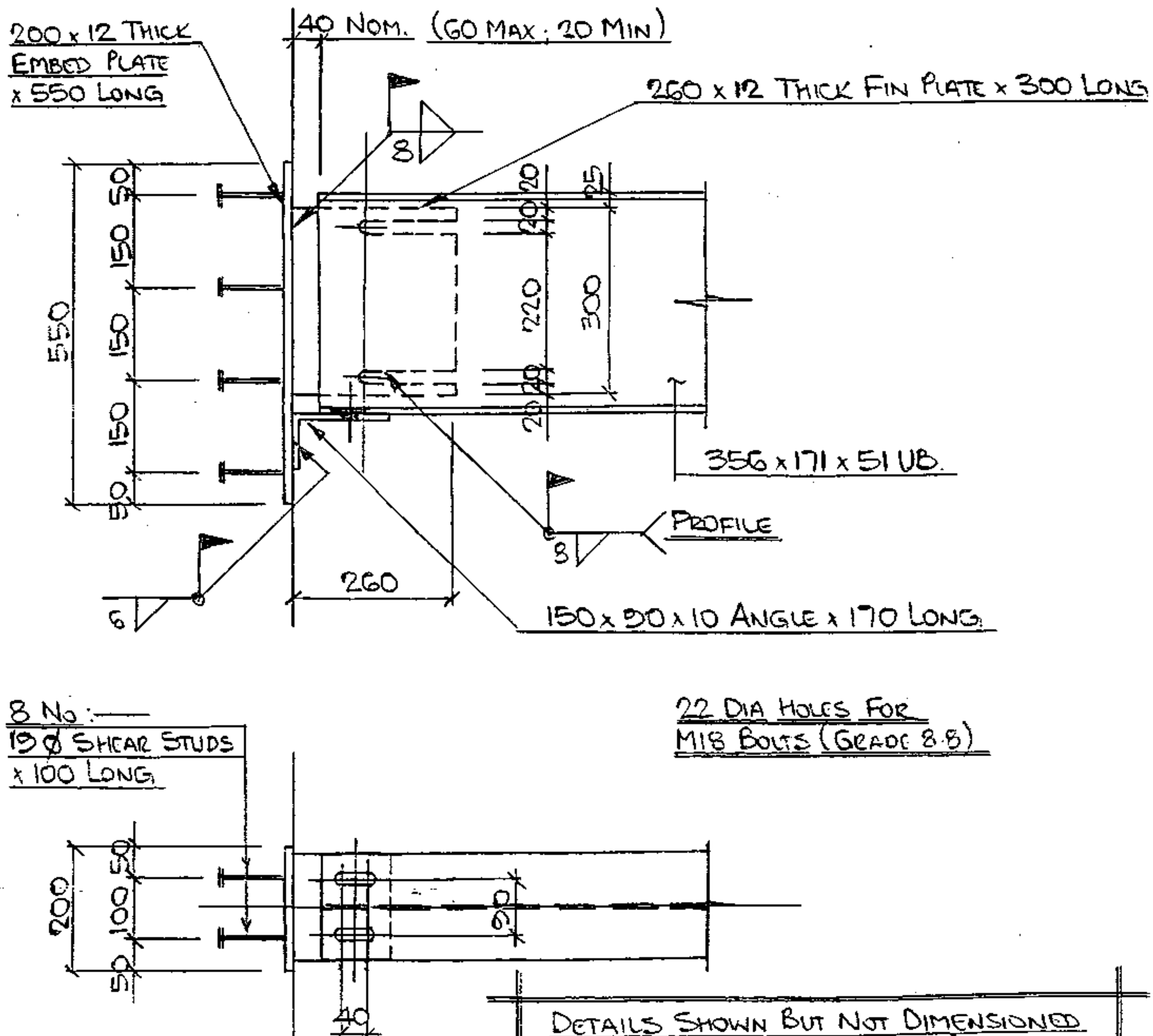
BEAM MARK : — B5-LG#

SECTION : — 356x171x51 UB

SHEAR LOAD : 400 KN.

AXIAL LOAD = 75 KN (TENSION)

REFERENCE : — SHEET Nos CW/14 TO CW/17



DETAILS SHOWN BUT NOT DIMENSIONED
SIMILAR TO BEAM MARK B5-LG

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B5-LG#

(B10-G ~ SIMILAR)

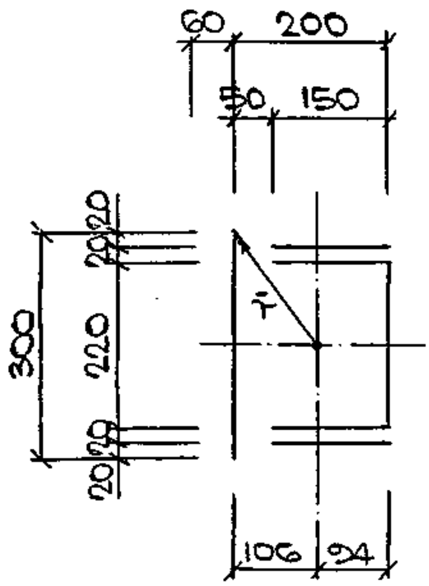
SERIES

SHT. No. CW/34 REV.

BY KB DATE 05/98

EX. DATE

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $300 + (4 \times 150) + 220 = 1120 \text{ mm}^2$

TO FIND NEUTRAL AXIS ~

TAKE MOMENTS ABOUT L.H SIDE

$1120\bar{x} = (600 \times 125) + (220 \times 200)$

$\bar{x} = 119000 \div 1120 = \underline{106 \text{ mm}}$

$I_{xx} = \left(\frac{300^3}{12}\right) + \left(\frac{4 \times 150}{12}\right) + \left(\frac{220^3}{12}\right) + \left(2 \times 150 \times \{110^2 + 130^2\}\right) = 11837383 \text{ mm}^4$

$I_{yy} = \left(\frac{300}{12}\right) + \left(\frac{4 \times 150^3}{12}\right) + \left(\frac{220}{12}\right) + (300 \times 106^2) + (600 \times 19^2) + (220 \times 94^2)$

$= 6656363 \text{ mm}^4$

$I_p = 11837383 + 6656363 = 18493746 \text{ mm}^4$

$\bar{r} = \sqrt{150^2 + 106^2} = 183.7 \text{ mm}$

$Z_p = 18493746 \div 183.7 = 100673 \text{ mm}^3$

$f_V = 400 \div 1120 = 0.36 \text{ KN/mm}^2$

$f_H = 75 \div 1120 = 0.07 \text{ KN/mm}^2$

$f_m = 400 \times (60 + 106) \div 100673 = 0.66 \text{ KN/mm}^2$

$f_m (V_c) = 0.66 \times 106 \div 183.7 = 0.38 \text{ KN/mm}^2$

$f_m (H_c) = 0.66 \times 150 \div 183.7 = 0.54 \text{ KN/mm}^2$

$F_{RES} = \sqrt{(0.36+0.38)^2 + (0.07+0.54)^2} = 0.96 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 43) = 215 N/mm²

WELD SIZE REQ^D = $\frac{0.96 \times 10^3}{0.7 \times 215} = 6.4\text{mm} \longrightarrow$ 8 LEG F.W. (MIN^M)

USE :— 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE = 300 mm

EFFECTIVE WELD LENGTH = 300 − (2 × 8) = 284 mm

$F_V = 400 \div (284 \times 2) = 0.71 \text{ KN/mm}^2$

$F_H \quad 75 \div (284 \times 2) = 0.14 \text{ KN/mm}^2$

$F_{RES} = \sqrt{0.71^2 + 0.14^2} = 0.73 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 43) = 215 N/mm²

WELD SIZE REQ^D = $\frac{0.73 \times 10^3}{0.7 \times 215} = 4.9 \text{ mm} \longrightarrow$ 6 LEG F.W. (MIN^M)

USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

CHECK FIN PLATE ~ 12 THICK.

a) SHEAR

SHEAR FORCE = 400 KN

$Py = 275 \text{ N/mm}^2$

$AV = 300 \times 12 \times 0.9 = 3240 \text{ mm}^2$

$Pv = 0.6 \times 3240 \times 275 \div 1000$

$= 534 \text{ KN} > 400 \text{ KN}$ OK ($Fv > 0.6Pv$)

(b) MOMENT

BENDING MOMENT = $400 \times 60 = 24000$ KNmm

$Z = 12 \times 300^2 \div 6 = 180000 \text{ mm}^3$

$Mcx = 1.2 \times Py \times Z$

$= 1.2 \times 275 \times 180000 \div 10^3 = 59400 \text{ KNmm} > 24000 \text{ KNmm}$ OK

FOR A PLATE Sx = Sv

$Sx = 12 \times 300^2 \div 4 = 270000 \text{ mm}^3$

$Mcx = 275 \times 270000 \times (1 - (2.5 \times 400 \div 534 - 1.5)) \div 10^3$

$= 46580 \text{ KNmm} > 24000 \text{ KNmm}$ OK

USE :— 12 THICK FIN PLATE.

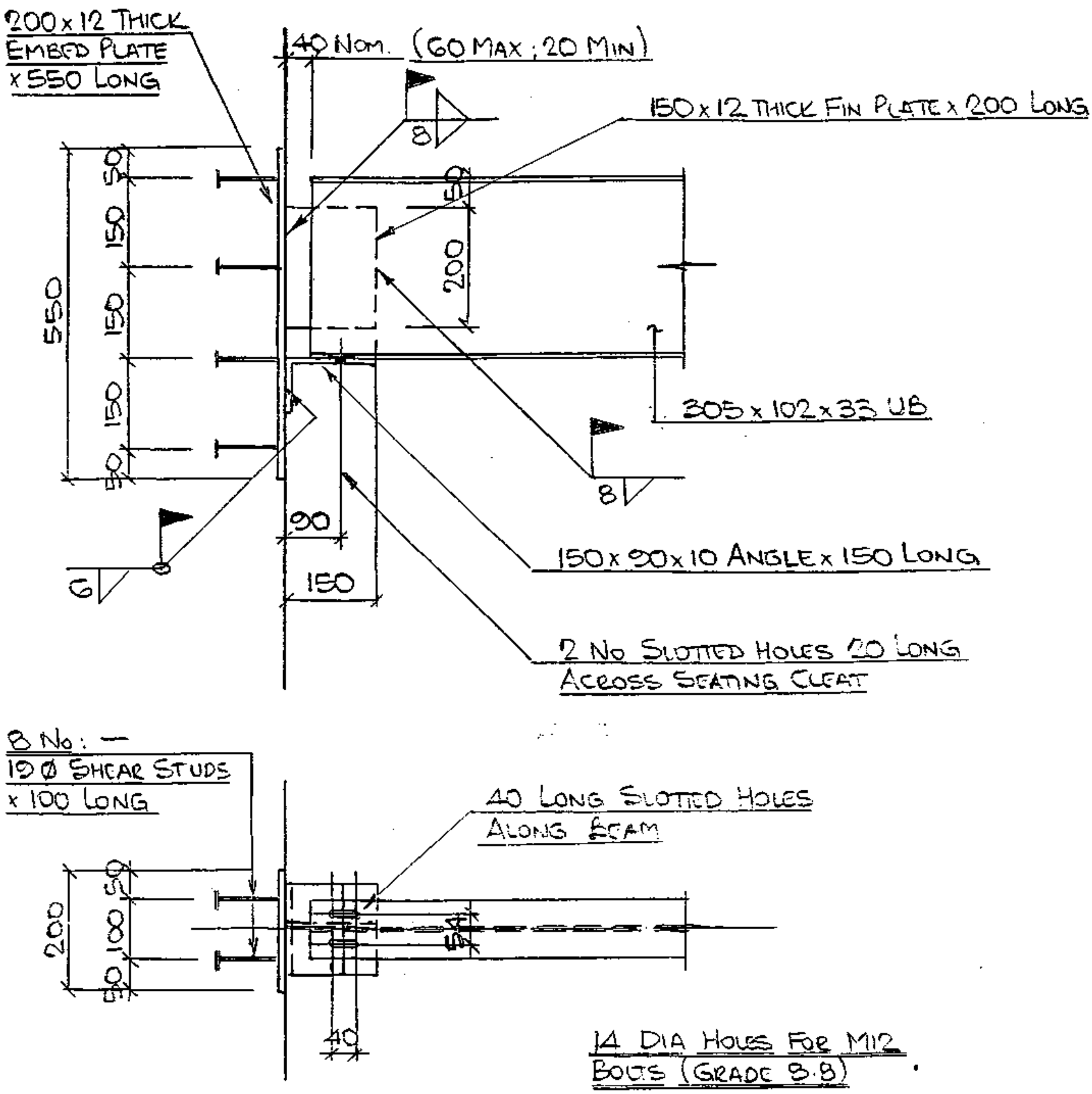
BEAM MARK :— B5-G# & B5-G

SECTION :— 305 x 102 x 33 UB

SHEAR LOAD = 246 KN.

AXIAL LOAD : 75 KN (TENSION)

REFERENCE :— SHEET NOS CW/10 TO CW/13



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B5-G# & B5-G

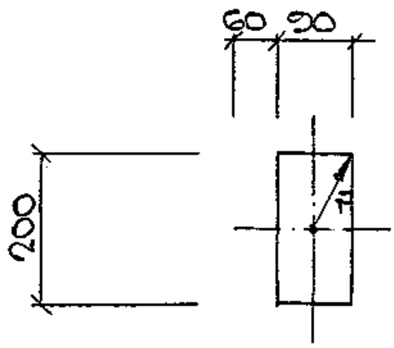
SERIES

SHT. No. CW/38 REV.

BY KB DATE 05/98

EX. DATE

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $2 \times (200 + 90) = 580\ mm^2$

$\bar{r} : \sqrt{45^2 + 100^2} : 109.7\ mm$

$I_{xx} : \left(\frac{2 \times 200^3}{12}\right) + \left(\frac{2 \times 90}{12}\right) + (2 \times 90 \times 100^2) : 3133348\ mm^4$

$I_{YY} : \left(\frac{2 \times 200}{12}\right) + \left(\frac{2 \times 90^3}{12}\right) + (2 \times 200 \times 45^2) : 931533\ mm^4.$

$I_p : 3133348 + 931533 : 4064881\ mm^4$

$Z_p : 4064881 \div 109.7 : 37054\ mm^3$

$f_V : 246 \div 580 : 0.43\ KN/mm^2$

$f_H : 75 \div 580 : 0.13\ KN/mm^2$

$f_M : 246 \times 105 \div 37054 = 0.70\ KN/mm^2$

$f_M\ (Vc) = 0.70 \times 45 \div 109.7 = 0.29\ KN/mm^2$

$f_M\ (Hc) = 0.70 \times 100 \div 109.7 = 0.64\ KN/mm^2$

$F_{RES} : \sqrt{(0.43 + 0.29)^2 + (0.13 + 0.64)^2} = 1.06\ KN/mm^2$

WELD STRENGTH (GRADE 43) : $215\ N/mm^2$

WELD SIZE REQ^D $= \frac{1.06 \times 10^3}{0.70 \times 215} = 7.0\ mm \longrightarrow 8$ LEG F.W

USE :— 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE : 200 mm

EFFECTIVE WELD LENGTH : $200 - (2 \times 8) = 184$ mm

$F_V = 246 \div (2 \times 184) = 0.67$ KN/mm^2

$F_H = 75 \div (2 \times 184) = 0.21$ KN/mm^2

$F_{RES} = \sqrt{0.67^2 + 0.21^2} = 0.71$ KN/mm^2

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D = $\dfrac{0.71 \times 10^3}{0.70 \times 215} = 4.7$ mm $\longrightarrow$ 6 LEG F.W. (MIN^M)

USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

CHECK FIN PLATE ~ 12 THICK

a) SHEAR

SHEAR FORCE = 246 KN

$P_y = 275$ N/mm^2

$A_V = 0.9 \times 12 \times 200 = 2160$ mm^2

$P_V = 0.6 \times 275 \times 2160 \div 10^3$

$= 356$ KN $>$ 246 KN OK ($F_V > 0.6 P_V$)

(b) MOMENT

BENDING MOMENT = $246 \times 60 = 14760$ KNmm

$Z = 12 \times 200^2 \div 6 = 80000$ mm^3

$M_{CX} = 1.2 \times P_y \times Z$

$= 1.2 \times 275 \times 80000 \div 1000 = 26400$ KNMM $>$ 14760 KNmm OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B5-LG

SERIES

SHT. No. CW/40 REV.

BY KB DATE 05/98

EX. DATE

FOR A PLATE Sx = Sv

$S_x = 12 \times 200^2 \div 4 = 120000 \text{ mm}^3$

$M_{cx} = 275 \times 120000 \times (1 - (2.5 \times 276 \div 356 - 1.5)) \div 1000$

$= \underline{18539 \text{ KNmm} > 14760 \text{ KNmm}}$ OK

<u>USE :— 12 THICK FIN PLATE</u>

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL
BEAM MARK BG-LG & BG-G

SERIES

SHT. No. CW/41 REV.

BY KB DATE 05/98

EX. DATE

(CN ~ 03)

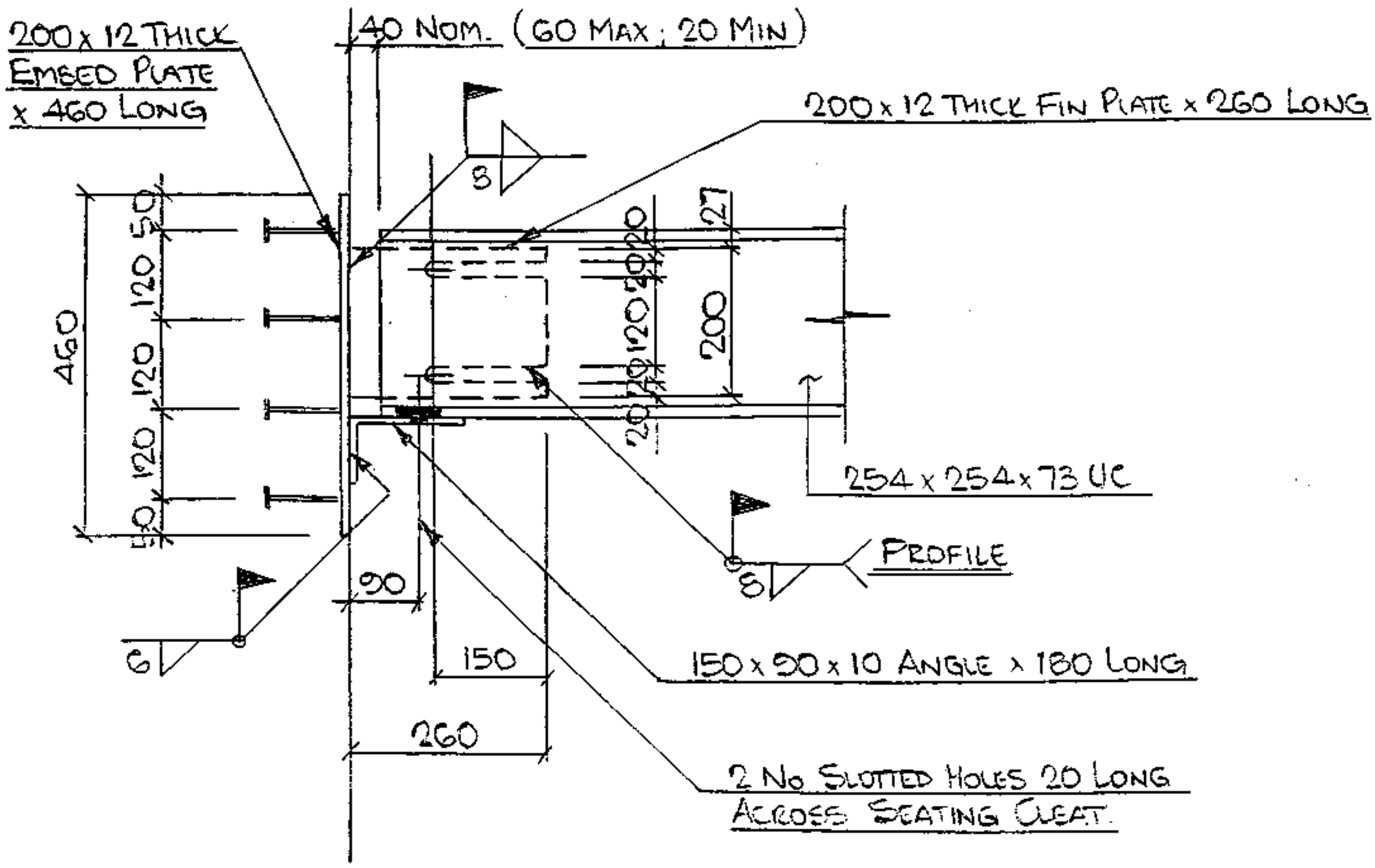
BEAM MARK :— BG-LG

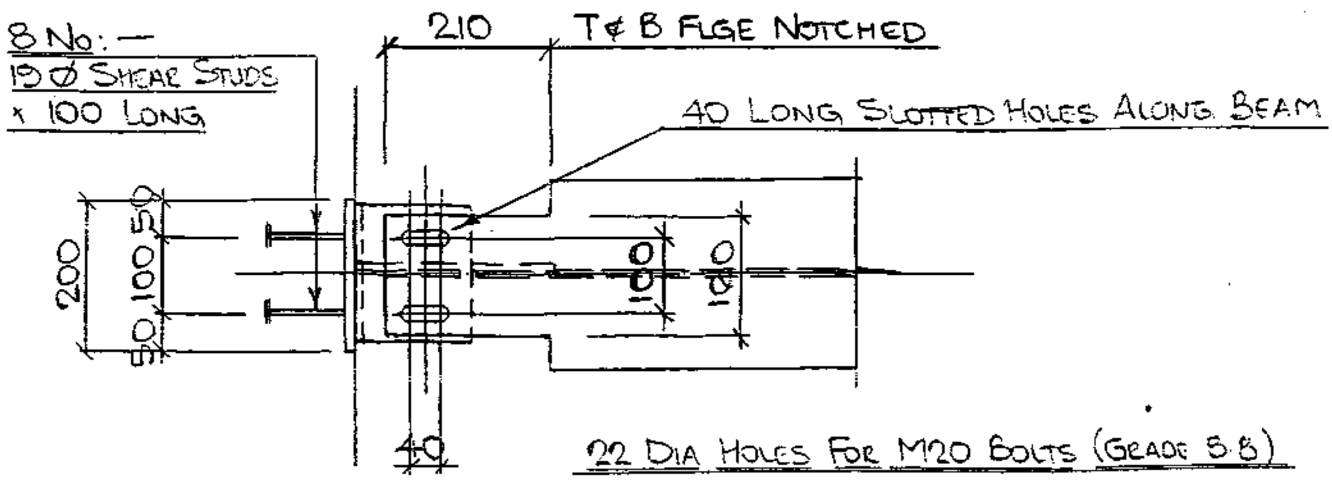
SECTION :— 254 x 254 x 73 UC

SHEAR LOAD = 260 KN

AXIAL LOAD = 75 KN (TENSION)

REFERENCE :— SHEET NOS CW/26 TO CW/28 (INCLUSIVE)





Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B3-LG (B10-7 ~ SIMILAR) A

SERIES

SHT. No. CW/42 REV. A

BY KB DATE 05/95

EX. DATE

(CN ~ 03)

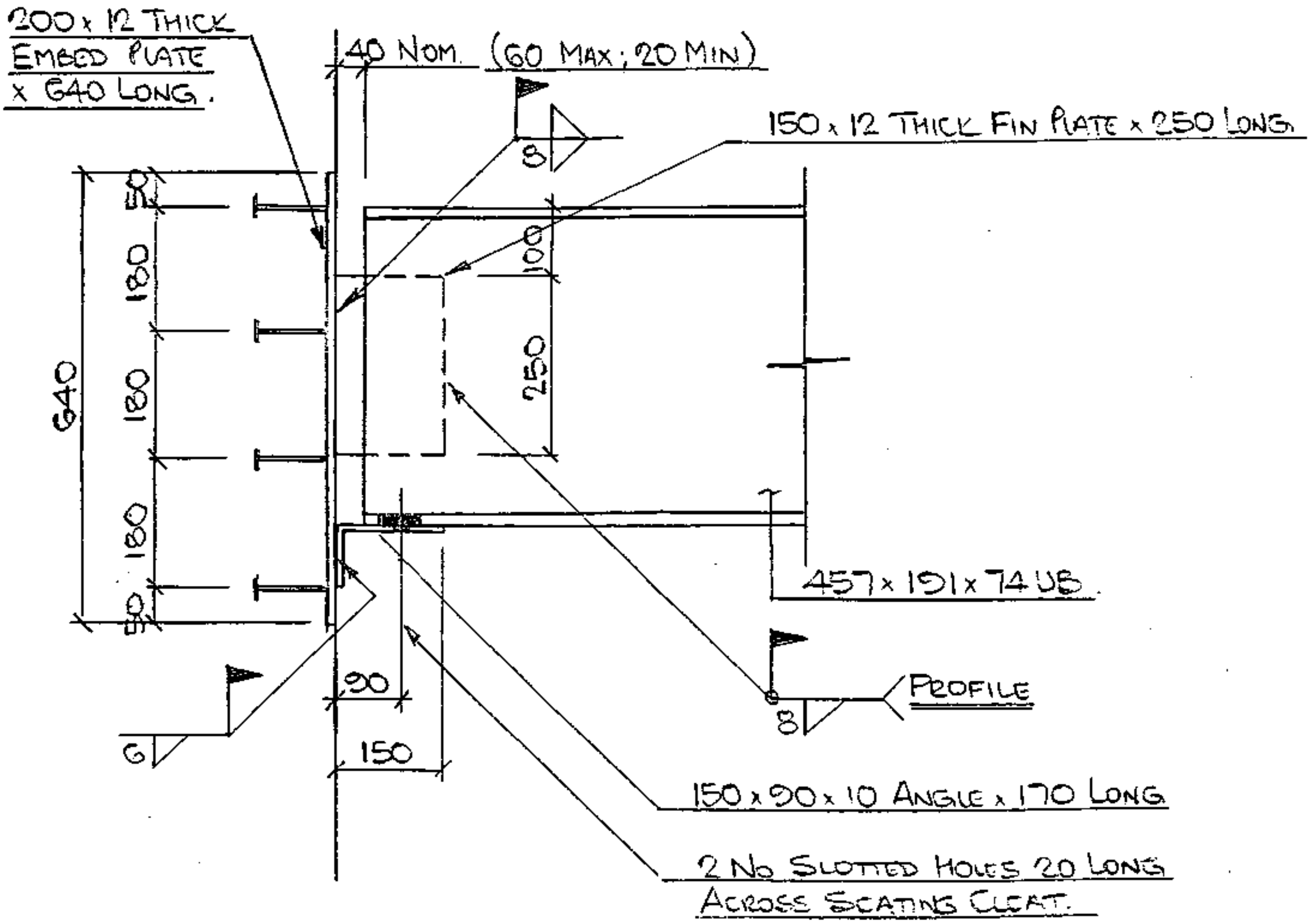
REV A | 28/09/98 | BEAM MARK B10-7 ADDED | CN~17 | KB

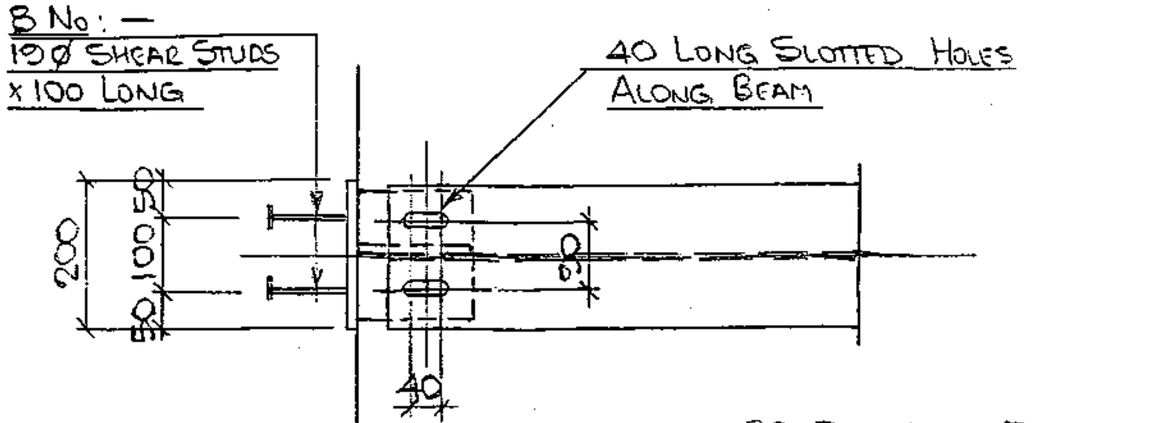
A BEAM MARK :— B3-LG (B10-7 ~ SIMILAR)

SECTION :— 457 x 191 x 74 UB

A SHEAR LOAD : 345 KN ~ B3-LG (230 KN ~ B10-7)

AXIAL LOAD : 75 KN (TENSION)





22 DIA HOLES FOR
M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B3-LG (B10-7 ~ SIMILAR) A

SERIES

SHT. No. CW/43 REV. A

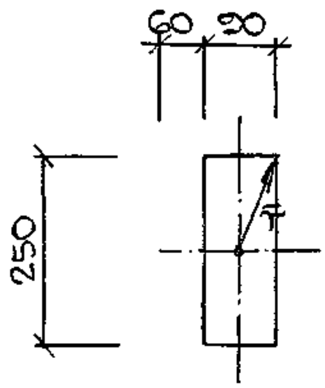
BY KB DATE 05/98

EX. DATE

(CN ~ 03)

REV A | 28/08/98 | BEAM MARK B10-7 ADDED | CN~17 | KB

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $2 \times (90 + 250) = 680 \text{ mm}^2$

$r = \sqrt{45^2 + 125^2} = 132.9 \text{ mm}$

$I_{xx} : \left(\frac{2 \times 250^3}{12}\right) + \left(\frac{2 \times 90}{12}\right) + (2 \times 90 \times 125^2) = 5416681 \text{ mm}^4$

$I_{yy} : \left(\frac{2 \times 250}{12}\right) + \left(\frac{2 \times 90^3}{12}\right) + (2 \times 250 \times 45^2) = 1134041 \text{ mm}^4$

$I_p = 5416681 + 1134041 = 6550722 \text{ mm}^4$

$Z_p : 6550722 \div 132.9 = 49290 \text{ mm}^3$

$f_v : 345 \div 680 = 0.51 \text{ kN/mm}^2$

$f_H : 75 \div 680 : 0.11 \text{ kN/mm}^2$

$f_m : 345 \times (60 + 45) \div 49290 = 0.74 \text{ kN/mm}^2$

$f_m (V_c) : 0.74 \times 45 \div 132.9 = 0.25 \text{ kN/mm}^2$

$f_m (H_c) : 0.74 \times 125 \div 132.9 = 0.70 \text{ kN/mm}^2$

$F_{RES} : \sqrt{(0.51 + 0.25)^2 + (0.11 + 0.70)^2} = 1.11 \text{ kN/mm}^2$

WELD STRENGTH (GRADE 43) = 215 N/mm²

WELD SIZE REQ^D = $\frac{1.11 \times 10^3}{0.70 \times 215} = 7.4 \text{ mm} \longrightarrow$ 8 LEG F.W (MIN^M)

USE: — 8 LEG FILLET WELD

REV A | 28/09/98 | BEAM MARK B10-7 ADDED | CN~17 | KB

CHECK WELD TO EMBEDMENT PLATE.

LENGTH OF FIN PLATE : 250 mm

EFFECTIVE WELD LENGTH : $250 - (2 \times 8) : 234$ mm

$F_V = 345 \div (2 \times 234) : 0.74$ KN/mm^2

$F_H : 75 \div (2 \times 234) : 0.16$ KN/mm^2

$F_{RES} : \sqrt{0.74^2 + 0.16^2} : 0.76$ KN/mm^2

WELD STRENGTH (GRADE 43) : 215 N/mm^2

WELD SIZE REQ^D $= \dfrac{0.76 \times 1000}{0.70 \times 215} = 5.1$ mm $\longrightarrow$ 6 LEG F.W (MIN^M)

USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

CHECK FIN PLATE ~ 12 THICK

a) SHEAR

SHEAR FORCE : 345 KN

$P_y : 275$ N/mm^2

$A_V : 0.9 \times 12 \times 250 = 2700$ mm^2

$P_V : 0.6 \times 275 \times 2700 \div 10^3$

$: \underline{445 \text{ KN} > 345 \text{ KN}}$ OK ($F_V > 0.6 P_V$)

b) MOMENT

BENDING MOMENT : $345 \times 60 : 20700$ KNmm

$Z : 12 \times 250^2 \div 6 = 125000$ mm^3

$M_{CX} : 1.2 \times P_y \times Z$

$: 1.2 \times 275 \times 125000 \div 10^3 : 41250$ KNmm $>$ 20700 KNmm OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALLS

BEAM MARK B3-LG (B10-7 ~ SIMILAR) A

SERIES

SHT. No. CW/45 REV. A

BY KB DATE 05/98

EX. DATE

(CN~03)

FOR A PLATE $S_x : S_v$

$S_x = 12 \times 250^2 \div 4 = 187500 \text{ mm}^3$

$M_{cx} = 275 \times 187500 \times (1 - (2.5 \times 345 \div 445 - 1.5)) \div 1000$

$= \underline{28967 \text{ KNmm} > 20700 \text{ KNmm}}$ OK

USE: — 12 THICK FIN PLATE

REV A | 28/09/98 | BEAM MARK B10-7 ADDED | CN~17 | KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS
SUBJECT CONNECTIONS TO CONCRETE CORE WALL
BEAM MARK B8-G

SERIES
SHT. No. CW/46 REV.
BY KB DATE 05/98
EX. DATE
(CN~03)

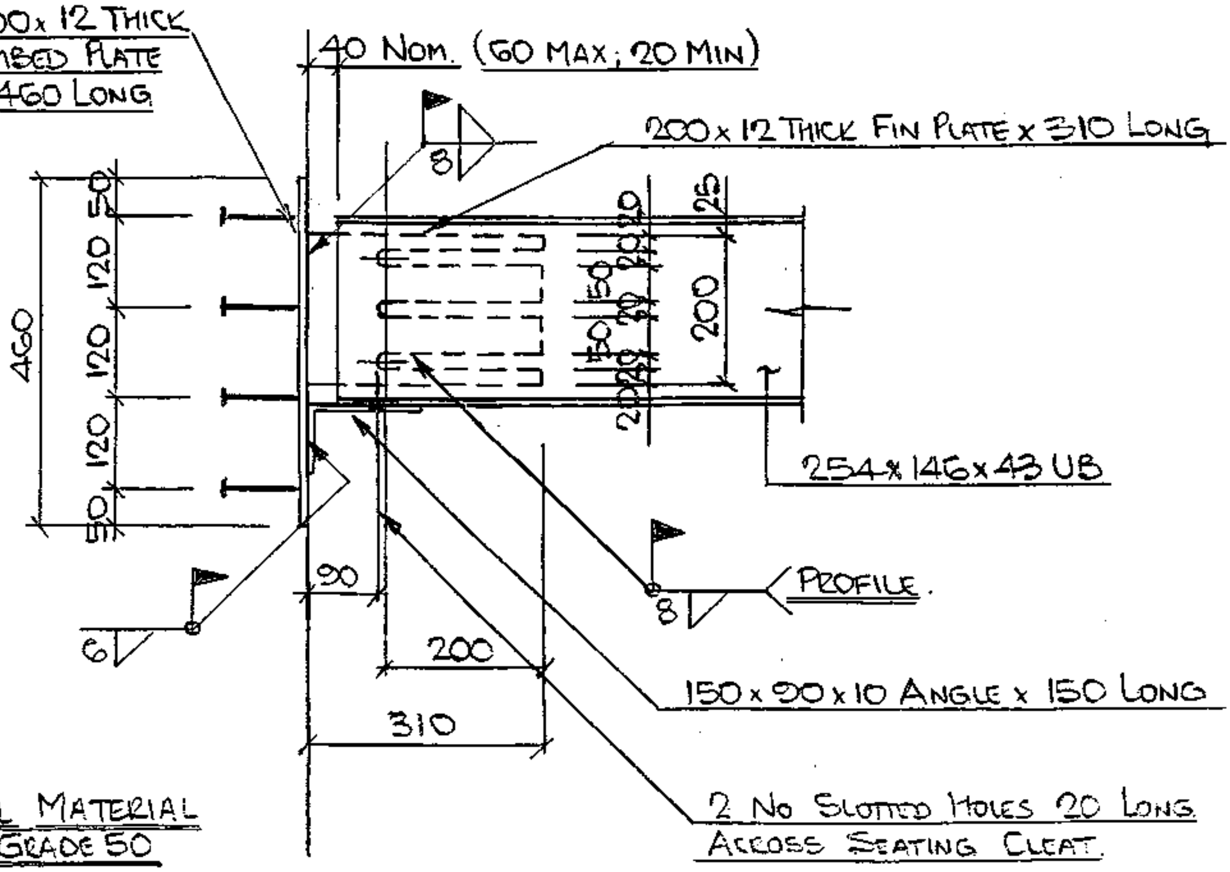
BEAM MARK :— B8·G.

SECTION : — 254 x 146 x 43 UB

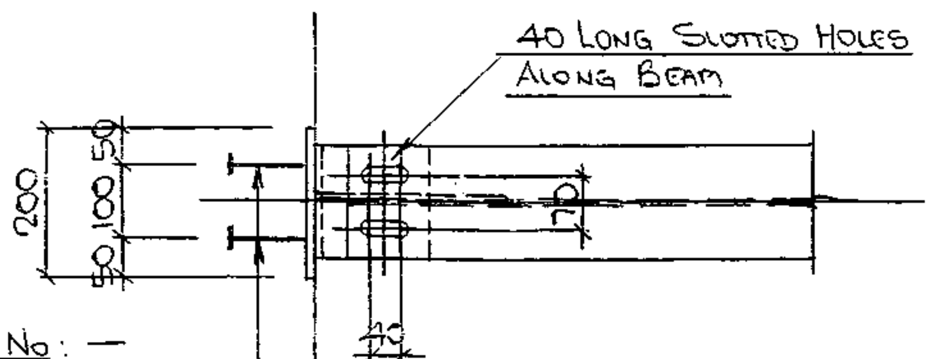
SHEAR LOAD : 400 KN

AXIAL LOAD : 75 KN (TENSION)

REFERENCE : — SHEET NOS CW/2 TO CW/5 (INCLUSIVE)



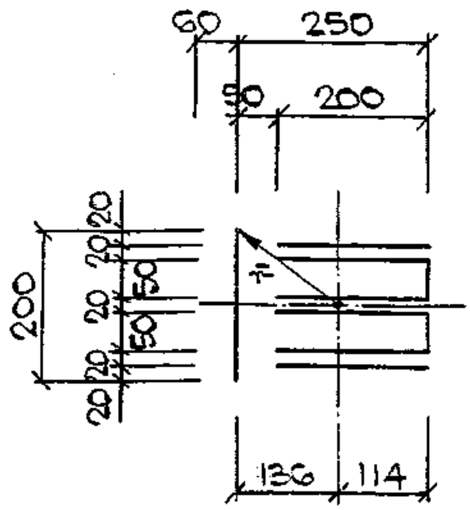
ALL MATERIAL GRADE 50



8 NO : — 19Ø SHEAR STUDS x 100 LONG

22 DIA HOLES FOR M20 BOLTS (GRADE 8.8)

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH.

AREA = 200 + (6 x 200) + (2 x 50) = 1500 mm^2

TO FIND NEUTRAL AXIS

TAKE MOMENTS ABOUT L.H. SIDE.

$1500\bar{x} = (1200 \times 150) + (100 \times 250)$

$\bar{x} = 205000 \div 1500 = \underline{136\text{ mm}}$

$$I_{XX} = \left(\frac{200^3}{12}\right) + \left(\frac{6 \times 200}{12}\right) + \left(\frac{2 \times 50^3}{12}\right) + \left(2 \times 200 \times \{10^2 + 60^2 + 90^2\}\right) + \left(2 \times 50 \times 35^2\right).$$

$$= 4850100\text{ mm}^4$$

$$I_{YY} = \left(\frac{200}{12}\right) + \left(\frac{6 \times 200^3}{12}\right) + \left(\frac{2 \times 50}{12}\right) + \left(200 \times 136^2\right) + \left(1200 \times 14^2\right) + \left(100 \times 114^2\right)$$

$$= 9234025\text{ mm}^4$$

$$I_p = 4850100 + 9234025 = 14084125\text{ mm}^4$$

$$\bar{r} = \sqrt{136^2 + 100^2} = 168.8\text{ mm}$$

$$Z_p = 14084125 \div 168.8 = 83437\text{ mm.}$$

$$f_V = 400 \div 1500 = 0.27\text{ kN/mm}^2$$

$$f_h = 75 \div 1500 = 0.05\text{ kN/mm}^2$$

$$f_m = 400 \times (60 + 136) \div 83437 = 0.94\text{ kN/mm}^2$$

$$f_m\,(V_c) = 0.94 \times 136 \div 168.8 = 0.76\text{ kN/mm}^2$$

$$f_m\,(H_c) = 0.94 \times 100 \div 168.8 = 0.56\text{ kN/mm}^2$$

$$F_{RES} = \sqrt{(0.27+0.76)^2 + (0.05+0.56)^2} = 1.20 \text{ kN/mm}^2$$

WELD STRENGTH (GRADE 50) : 255 N/MM^2

$$\text{WELD SIZE REQ}^D = \frac{1.20 \times 10^3}{0.70 \times 255} = 6.8 \longrightarrow 8 \text{ LEG F.W. (MIN}^M\text{)}$$

<u>USE :— 8 LEG FILLET WELD</u>

<u>CHECK WELD TO EMBEDMENT PLATE</u>

LENGTH OF FIN PLATE = 200 mm

EFFECTIVE WELD LENGTH : $200 - (2 \times 8) = 184$ mm

F_V : $400 \div (2 \times 184) = 1.09$ kN/mm^2

F_H : $75 \div (2 \times 184) = 0.21$ kN/mm^2

$F_{RES} = \sqrt{1.09^2 + 0.21^2} = 1.11$ kN/mm^2

WELD STRENGTH (GRADE 50) : 255 N/mm^2

$$\text{WELD SIZE REQ}^D = \frac{1.11 \times 10^3}{0.70 \times 255} = 6.3 \text{ mm} \longrightarrow 8 \text{ LEG F.W. (MIN}^M\text{)}$$

<u>USE :— 8 LEG FILLET WELD</u>

CHECK FIN PLATE ~ 15 THICK

a) SHEAR

SHEAR FORCE = 400 KN

$P_y = 355\ N/mm^2$

$A_v = 0.9 \times 15 \times 200 = 2700\ mm^2$

$P_v = 0.6 \times 355 \times 2700 \div 10^3$

$= \underline{575\ KN > 400\ KN}$ OK (FV > 0.6 PV)

b) MOMENT

BENDING MOMENT = $400 \times 60 = 24000$ KNmm

$Z = 15 \times 200^2 \div 6 = 100000\ mm^3$

$M_{cx} = 1.2 \times P_y \times Z$

$= 1.2 \times 355 \times 100000 \div 1000 = \underline{42600\ KNmm > 24000\ KNmm}$ OK

FOR A PLATE SX = SY

$S_x = 15 \times 200^2 \div 4 = 150000\ mm^3$

$M_{cx} = 355 \times 150000 \times (1 - (2.5 \times 400 \div 575 - 1.5)) \div 1000$

$= \underline{40516\ KNMM > 24000\ KNMM}$ OK

USE :— 15 THICK FIN PLATE (GRADE 50)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALLS

BEAM MARK B13 – G & BEAM MARK B5-5

SERIES ....

SHT. No. CW/50 REV. A

BY KB DATE 05/98

EX. .... DATE ....

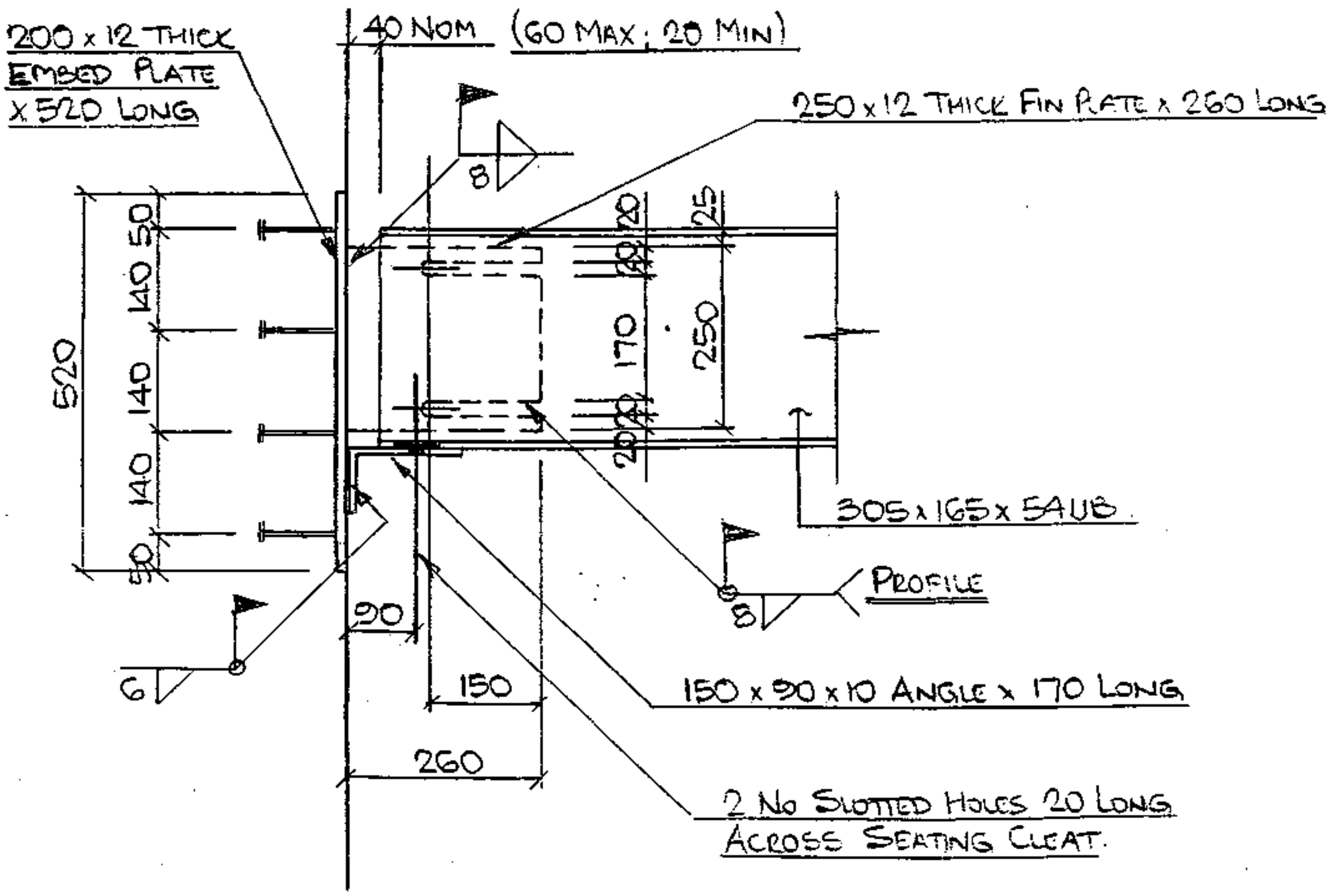
(CN ~ 03)

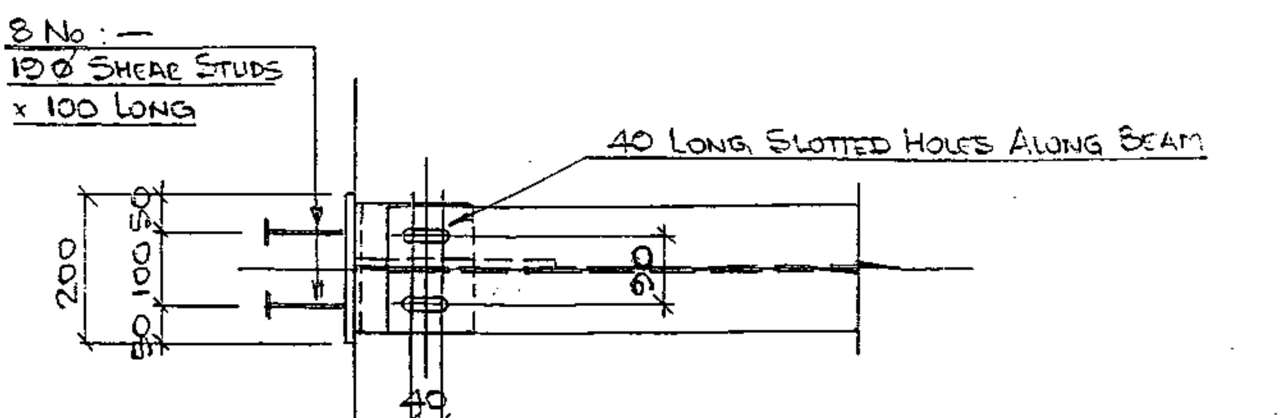
BEAM MARK :— B13-G (BEAM MARK :— B5-5 ~ SIMILAR) A

SECTION :— 305 x 165 x 54 UB

SHEAR LOAD = 350 KN (BEAM B5-5 = 234 KN) A

AXIAL LOAD = 75 KN (TENSION)

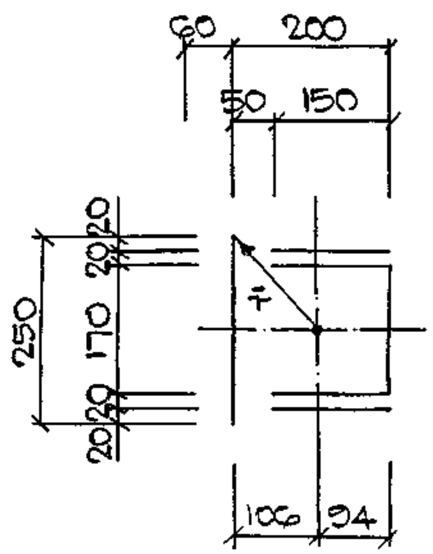




22 DIA HOLES FOR M20 BOLTS (GRADE 8.8)

REV 'A' 25/09/98 REFERENCES TO BEAM B5-5 ADDED KB

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA: $250 + (4 \times 150) + 170 = 1020 \text{ mm}^2$

TO FIND NEUTRAL AXIS

TAKE MOMENTS ABOUT L.H. SIDE

$1020\bar{x} = (600 \times 125) + (170 \times 200)$

$\bar{x} = 109000 \div 1020 = \underline{106 \text{ mm}}$

$I_{xx} = \left(\frac{250^3}{12}\right) + \left(\frac{4 \times 150}{12}\right) + \left(\frac{170^3}{12}\right) + (2 \times 150 \times \{85^2 + 105^2\}) = 7186550 \text{ mm}^4$

$I_{yy} = \left(\frac{250}{12}\right) + \left(\frac{4 \times 150^3}{12}\right) + \left(\frac{170}{12}\right) + (250 \times 106^2) + (600 \times 19^2) + (170 \times 94^2)$

$= 5652755 \text{ mm}^4$

$I_p = 7186550 + 5652755 = 12839305 \text{ mm}^4$

$r = \sqrt{106^2 + 125^2} = 163.9 \text{ mm}$

$Z_p = 12839305 \div 163.9 = 78336 \text{ mm}^3$

$f_V = 350 \div 1020 = 0.35 \text{ KN/mm}^2$

$f_H = 75 \div 1020 = 0.08 \text{ KN/mm}^2$

$f_m = 350 \times (60 + 106) \div 78336 = 0.74 \text{ KN/mm}^2$

$f_m (V_c) = 0.74 \times 106 \div 163.9 = 0.48 \text{ KN/mm}^2$

$f_m (H_c) = 0.74 \times 125 \div 163.9 = 0.57 \text{ KN/mm}^2$

$f_{RES} = \sqrt{(0.35 + 0.48)^2 + (0.08 + 0.57)^2} = 1.06 \text{ KN/mm}^2$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B13-G

SERIES

SHT. No. CW/52 REV.

BY KB DATE 05/98

EX. DATE

(CN~03)

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D = $\frac{1.06 \times 10^3}{0.7 \times 215}$ = 7.0 mm ⟶ 8 LEG F.W. (MIN^M/)

USE :— 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE = 250 mm

EFFECTIVE WELD LENGTH = 250 - (2 x 8) = 234 mm

F_V = 350 ÷ (2 x 234) = 0.75 KN/mm^2

F_H = 75 ÷ (2 x 234) = 0.16 KN/mm^2

F_{RES} = $\sqrt{0.75^2 + 0.16^2}$ = 0.77 KN/mm^2

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D = $\frac{0.77 \times 10^3}{0.70 \times 215}$ = 5.2 mm ⟶ 6 LEG F.W. (MIN^M/)

USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS
SUBJECT CONNECTIONS TO CONCRETE CORE WALLS
BEAM MARK B13-G

SERIES
SHT. No. CW/53 REV.
BY KB DATE 05/98
EX. DATE
(CN~03)

CHECK FIN PLATE ~ 12 THICK

a) SHEAR

SHEAR FORCE : 350 KN

P_y : 275 N/mm^2

A_V : $0.9 \times 12 \times 250$: 2700 mm^2

P_V : $0.6 \times 2700 \times 275 \div 10^3$

: 445 KN > 350 KN OK (F_V > $0.6 P_V$)

b) MOMENT

BENDING MOMENT : 350×60 : 21000 KNmm

$Z = 12 \times 200^2 \div 6$ = 80000 mm^3

M_{CX} = $1.2 \times P_y \times Z$

: $1.2 \times 275 \times 80000 \div 1000$ = 26400 KNmm > 21000 KNmm OK

FOR A PLATE S_x : S_v

S_x = $12 \times 250^2 \div 4$: 187500 mm^3

M_{cx} : $275 \times 187500 \times (1 - (2.5 \times 350 \div 445 - 1.5)) \div 1000$

= 27519 KNmm > 21000 KNmm OK

USE : — 12 THICK FIN PLATE.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE WALL
BEAM MARK B5-LG+ AND B5-LG#+

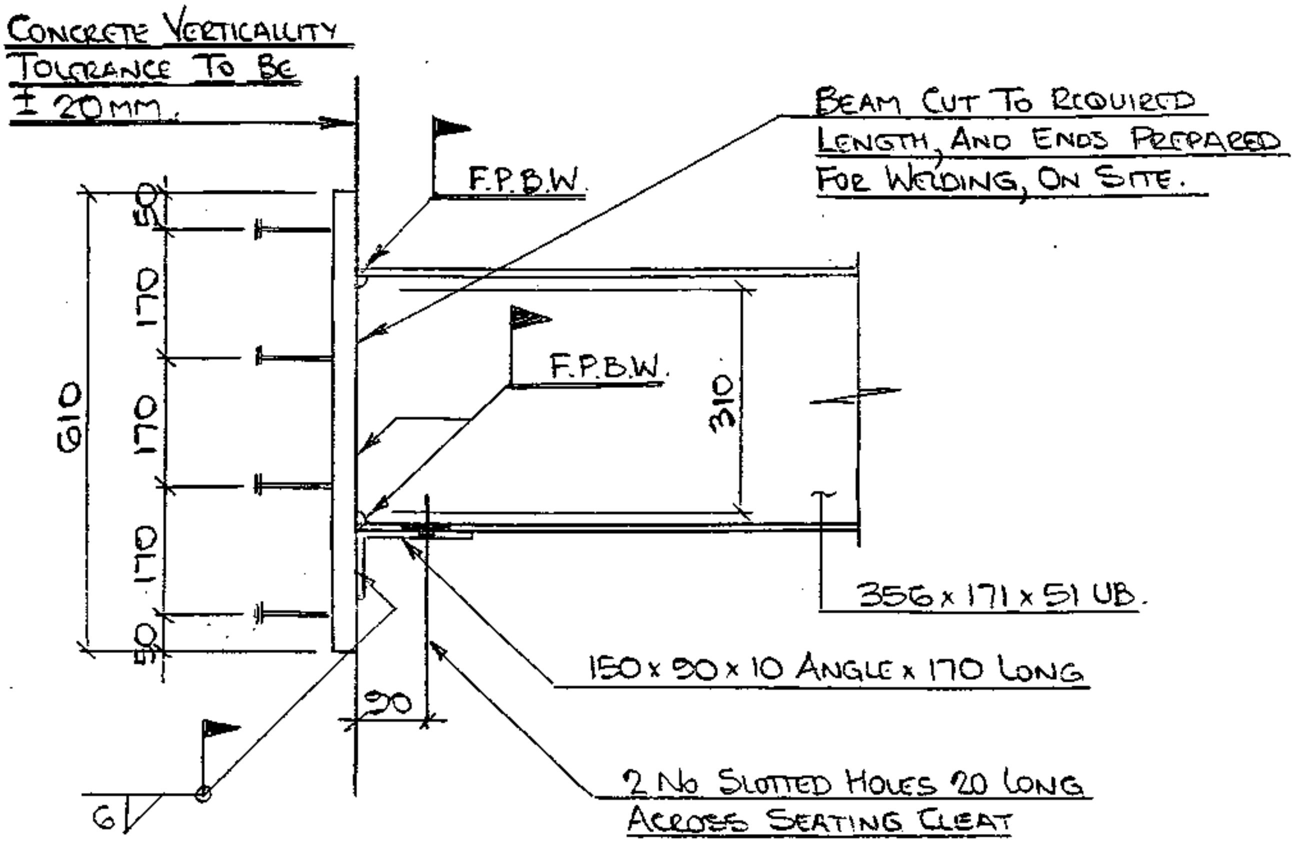
SERIES
SHT. No. CW/54 REV.
BY KB DATE 05/98
EX. DATE
(CN~03)

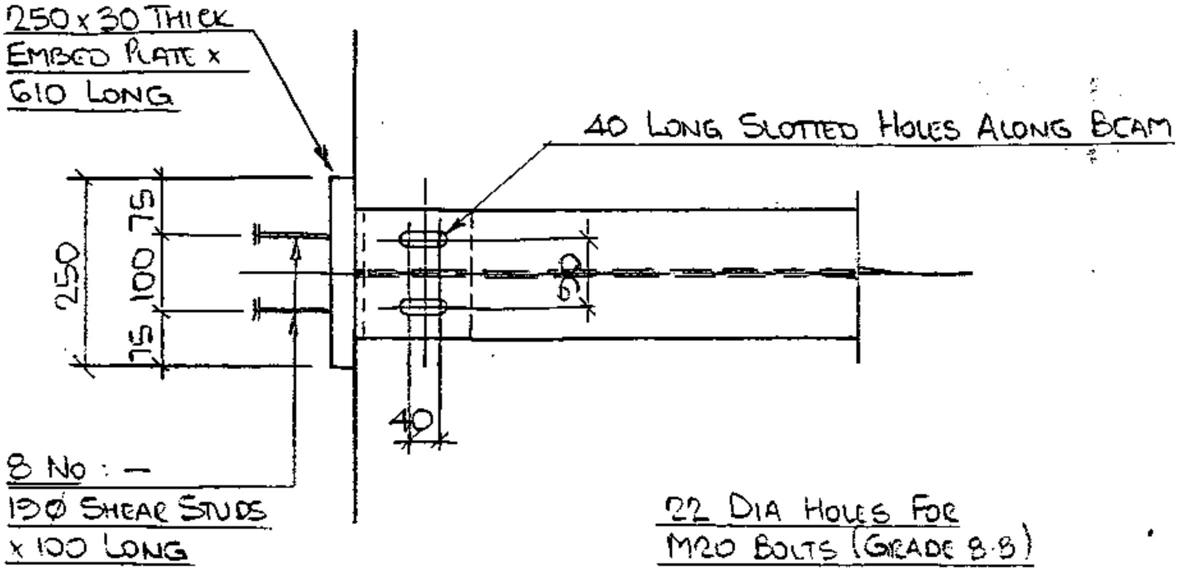
BEAM MARK :— B5-LG#+

SECTION :— 356x171x51UB

SHEAR LOAD : 400 KN.

AXIAL LOAD : 2000 KN (COMPRESSION)





DESIGN OF EMBEDMENT PLATE ~ EFFECTIVE AREA METHOD.

COMPRESSION LOAD = 2000 KN.

ALLOWABLE BEARING PRESSURE ON CONCRETE = $0.6 \times f_{cu}$

WHERE :- $f_{cu} = 35 \ N/mm^2$

ALLOWABLE CONCRETE BEARING PRESSURE = $21.0 \ N/mm^2$

EMBED PLATE AREA REQ^D = $2000 \times 10^3 \div 21.0 = 95238 \ mm^2$

$$95238 = [2 \times (2K + 11.5) \times (2K + 171.5)] + [(2K + 7.3) \times (332.6 - 2K)]$$

$$= [8K^2 + 732K + 3944.5] + [651.0K - 4K^2 + 2427.98]$$

$$0 = 4K^2 + 1383K - 88865.52$$

$$K = \frac{-1383.0 \pm \sqrt{1383.0^2 - (4 \times 4 \times -88865.52)}}{2 \times 4}$$

$$= +55.4 \text{ OR } -401.2$$

$$K = 55.4 \text{ mm}$$

$$t_p = 55.4 \times \left[\frac{3 \times 0.6 \times f_{cu}}{P_{yp}}\right]^{1/2} = 55.4 \times \left[\frac{3 \times 21.0}{270}\right]^{1/2}$$

$t_p = 26.8$ mm ⟶ SAY 30 MM.

FOR $t_p = 30.0$ THEN $K = \dfrac{30.0}{\left[\frac{3 \times 21.0}{270}\right]^{1/2}} = 62.1$ MM

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE WALL

BEAM MARK BS-LG+ AND BS-LG#+

SERIES

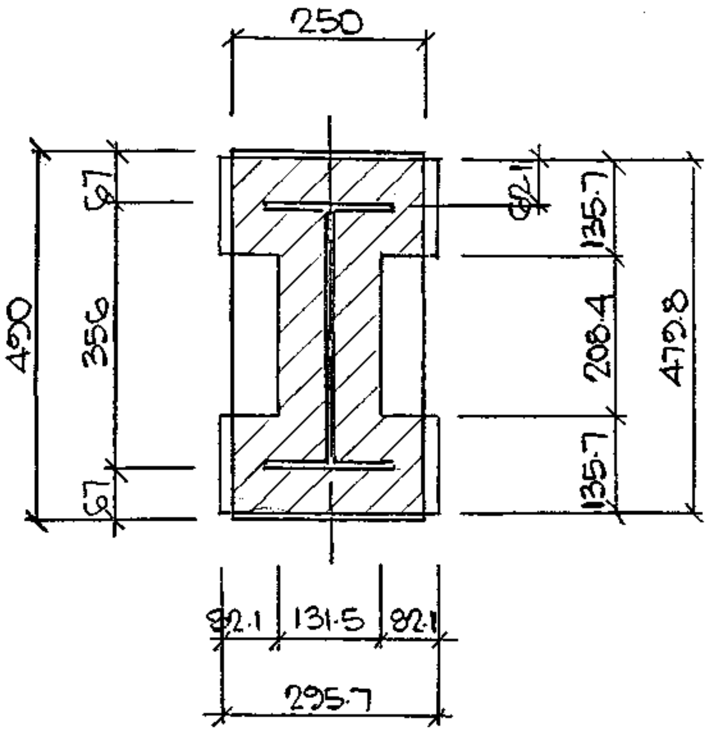
SHT. No. CW/56 REV.

BY KB DATE 05/98

EX. DATE

(CN ~ 03)

TRY : — 250 x 30 THICK END PLATE x 490 LONG (MINIMUM)



ACTUAL BEARING AREA = $(2 \times 250 \times 135.7) + (208.4 \times 131.5)$

$= 95254\ mm^2 > 95238\ mm^2$ OK

USE : — 250 x 30 THICK EMBED PLATE x 610 LONG (A/C DETAILS)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS
SUBJECT CONNECTIONS TO CONCRETE WALL
BEAM MARK BS-LG+ AND BS-LG#+

SERIES
SHT. No. CW/57 REV.
BY KB DATE 05/98
EX. DATE
(CN ~ 03)

LOAD DISTRIBUTION

SECTION AREA : 64.6 cm^2.

FLANGE LOAD : $2000 \times (17.15 \times 1.15) \div 64.6$: 610.6 KN

WEB LOAD : $2000 - (2 \times 610.6)$ = 778.8 KN

a) FLANGE WELD.

FLANGE WIDTH = 170 mm

WELD LENGTH : 2×170 : 340 mm

LOAD PER mm RUN = $610.6 \div 340$: 1.80 KN/mm

WELD STRENGTH (GRADE 50) : 255 N/mm^2

WELD SIZE REQ^D : $\dfrac{1.80 \times 10^3}{0.7 \times 255}$: 10.0 mm ⟶ 10 LEG F.W. (MIN^M/) NOT PRACTICAL

USE : — FULL PENETRATION BUTT WELD. (WELD CAPACITY : 700 KN)

b) WEB WELD.

WELD LENGTH : 2×310 = 620 mm

FV : $400 \div 620$: 0.65 KN/mm^2

FH : $778.8 \div 620$: 1.26 KN/mm^2

FRES : $\sqrt{0.65^2 + 1.26^2}$: 1.42 N/mm^2

WELD SIZE REQD/ : $\dfrac{1.42 \times 10^3}{0.70 \times 255}$: 7.9 mm ⟶ 8 LEG F.W (MIN^M/) NOT PRACTICAL

USE : — FULL PENETRATION BUTT WELD (WELD CAPACITIES : —
COMPRESSION : 803 KN &
SHEAR : 433 KN)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE WALL

BEAM MARK B3-LG+

SERIES

SHT. No. CW/58 REV.

BY KB DATE 05/98

EX. DATE

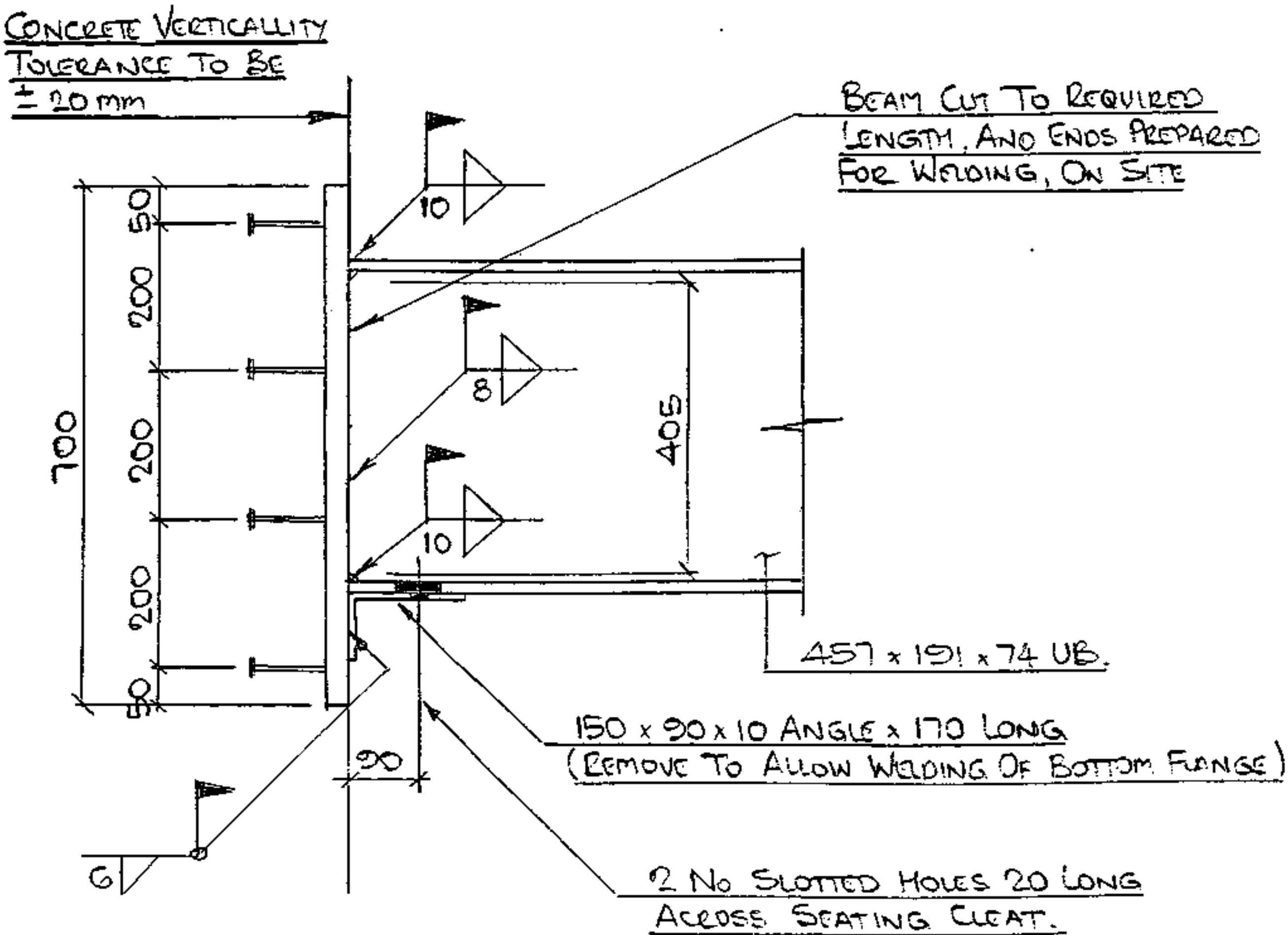
(CN ~ 03)

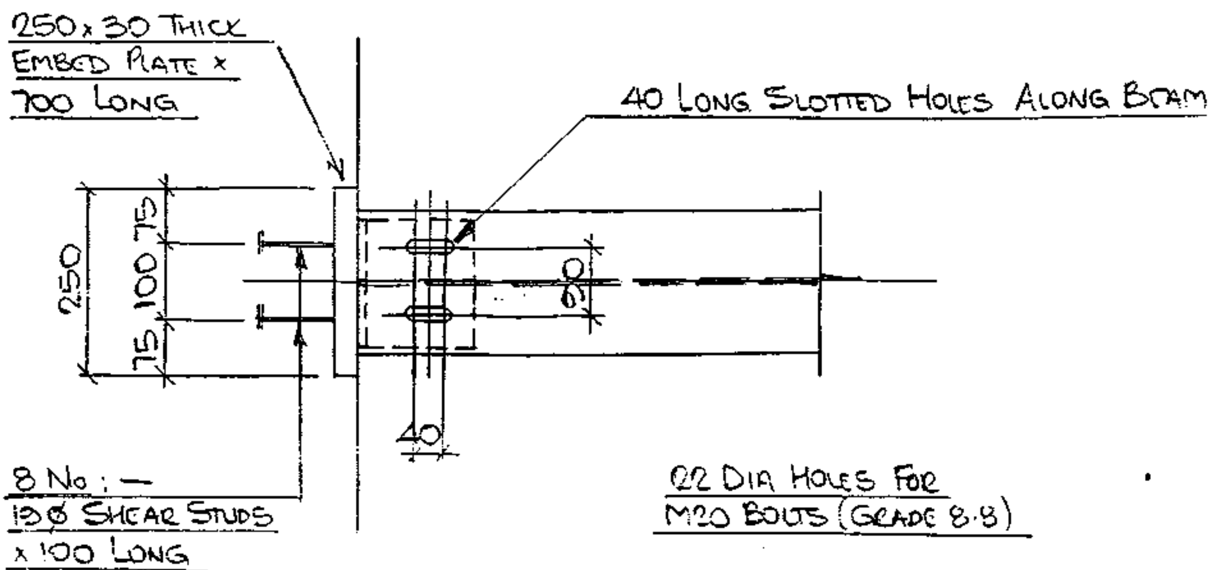
BEAM MARK : — B3-LG +

SECTION : — 457 x 191 x 74 UB

SHEAR LOAD : 345 KN

AXIAL LOAD = 2000 KN (COMPRESSION)





Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE WALL
BEAM MARK B3-LG+

SERIES

SHT. No. CW/59 REV.

BY KB DATE 05/98

EX. DATE

(CN-03)

DESIGN OF EMBEDMENT PLATE ~ EFFECTIVE AREA METHOD

COMPRESSION LOAD = 2000 KN

ALLOWABLE BEARING PRESSURE ON CONCRETE = $0.6 \times f_{cu}$

WHERE: — $f_{cu} = 35\ N/mm^2$

ALLOWABLE CONCRETE BEARING PRESSURE = $21.0\ N/mm^2$

EMBED PLATE AREA REQ^D = $2000 \times 10^3 \div 21.0 = 95238\ mm^2$

$$95238 = [2 \times (2K + 14.5) \times (2K + 190.5)] + [(2K + 9.1) \times (428.2 - 2K)]$$

$$= [8K^2 + 820K + 5524.5] + [838.2K - 4K^2 + 3896.62]$$

$$0 = 4K^2 + 1658.2K - 85816.88$$

$$K = \frac{-1658.2 \pm \sqrt{1658^2 - (4 \times 4 \times -85816.88)}}{2 \times 4}$$

$$= +46.5 \text{ OR } -461.0$$

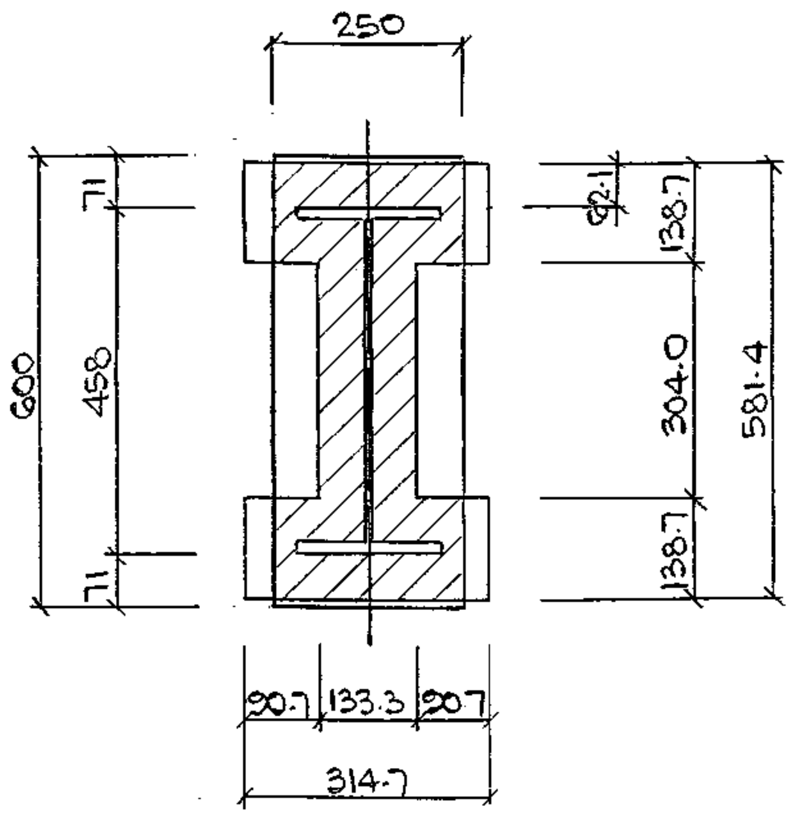
$$K = 46.5\ mm.$$

$$t_p = 46.5 \times \left[\frac{3 \times 0.6 \times f_{cu}}{P_{yp}}\right]^{1/2} = 46.5 \times \left[\frac{3 \times 21.0}{270}\right]^{1/2}$$

$t_p = 22.5\ mm$ ⟶ SAY 30 mm (4/C STANDARDIZATION)

FOR $t_p = 30.0$ THEN $K = \dfrac{30.0}{\left[\frac{3 \times 21.0}{270}\right]^{1/2}} = 62.1\ mm$

TRY : — 250 x 30 THICK END PLATE x 600 LONG (MINIMUM)



ACTUAL BEARING AREA = (2 x 250.0 x 138.7) + (304.0 x 133.3)

= 109873 mm² ≫ 95238 mm² OK

USE: — 250 x 30 THICK EMBED PLATE x 700 LONG

LOAD DISTRIBUTION

SECTION AREA = 95.0 cm^2

FLANGE LOAD = 2000 x (19.05 x 1.45) ÷ 95.0 = 581.5 KN

WEB LOAD = 2000 − (2 x 581.5) = 837 KN

a) FLANGE WELD

FLANGE WIDTH = 190 mm

WELD LENGTH = 2 x 190 = 380 mm

LOAD PER mm RUN = 581.5 ÷ 380 = 1.53 KN/mm

WELD STRENGTH (GRADE 50) = 255 N/mm^2

WELD SIZE REQ'D = $\frac{1.53 \times 10^3}{0.70 \times 255}$ = 8.6 mm ⟶ 10 LEG F.W. (MIN^m/)

USE :— 10 LEG FILLET WELD.

b) WEB WELD

WELD LENGTH = 2 x 405 = 810 mm

FV = 345 ÷ 810 = 0.43 KN/mm^2

FH = 837 ÷ 810 = 1.04 KN/mm^2

FRES = $\sqrt{0.43^2 + 1.04^2}$ = 1.12 KN/mm^2

WELD SIZE REQ'D = $\frac{1.12 \times 10^3}{0.70 \times 255}$ = 6.3 mm ⟶ 8 LEG F.W. (MIN^m/)

USE :— 8 LEG FILLET WELD.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B1-LG TO B5-LG#

SERIES

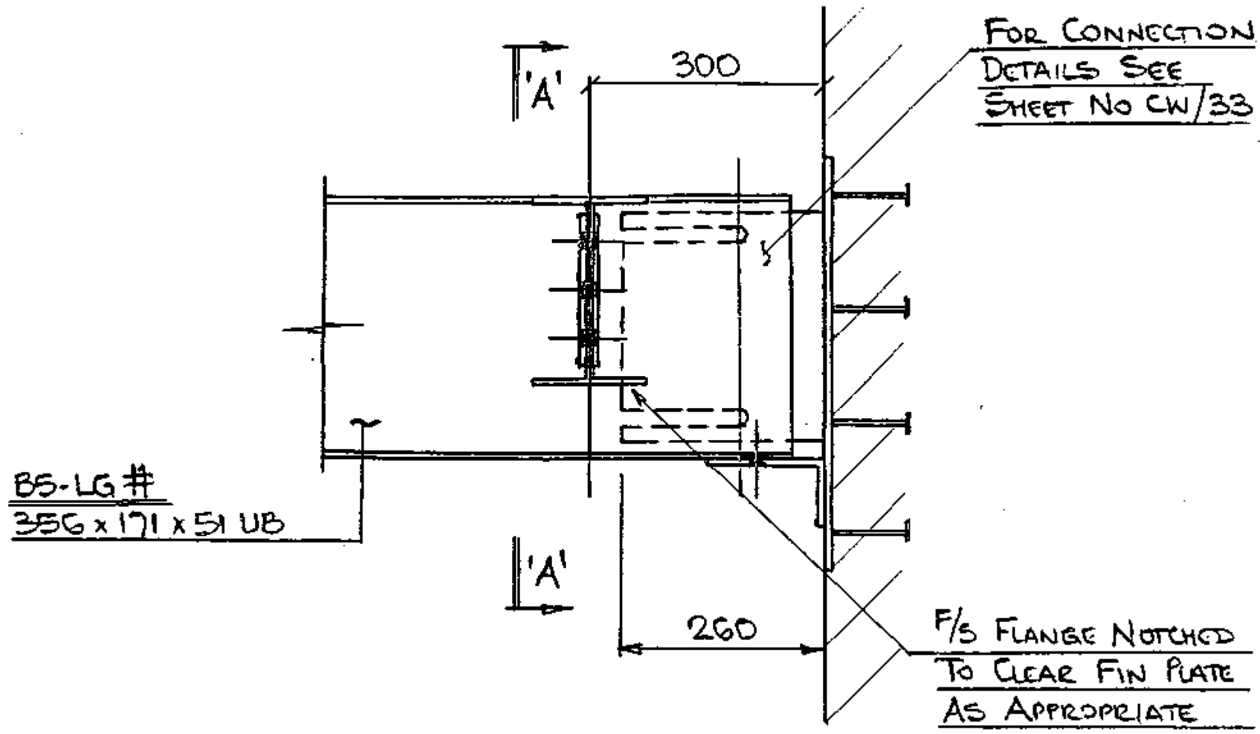
SHT. No. CW/62 REV.

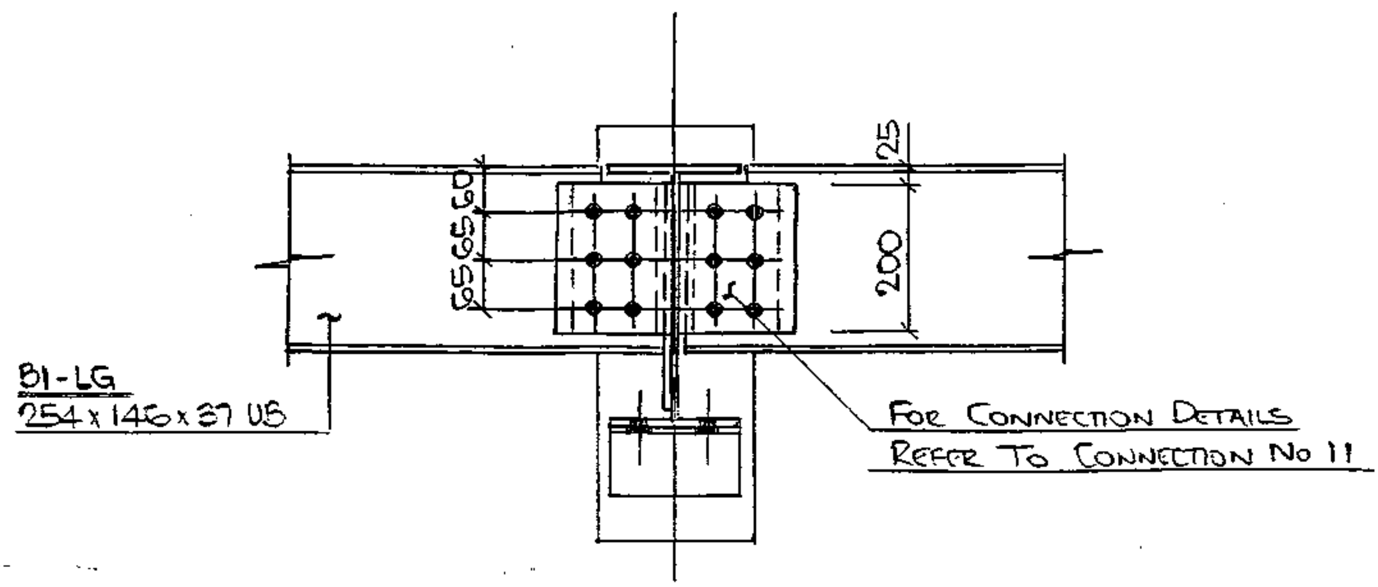
BY KB DATE 05/98

EX. DATE

(CN~03)

BEAM MARK B1-LG TO B5-LG#





Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

MIS-ALIGNMENT OF EMBEDMENT PLATE

(CN-28)

SERIES

SHT. No. CW/63 REV.

BY KB DATE 08/98

EX. DATE

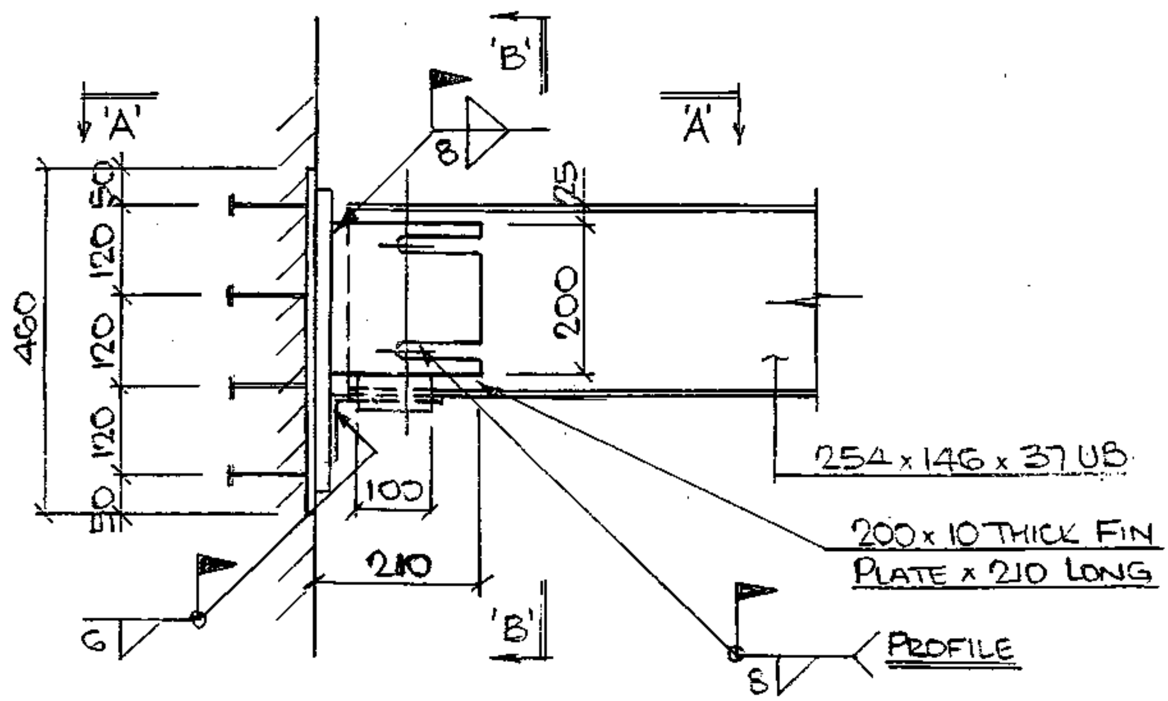
BEAM MARK :— B1-LG

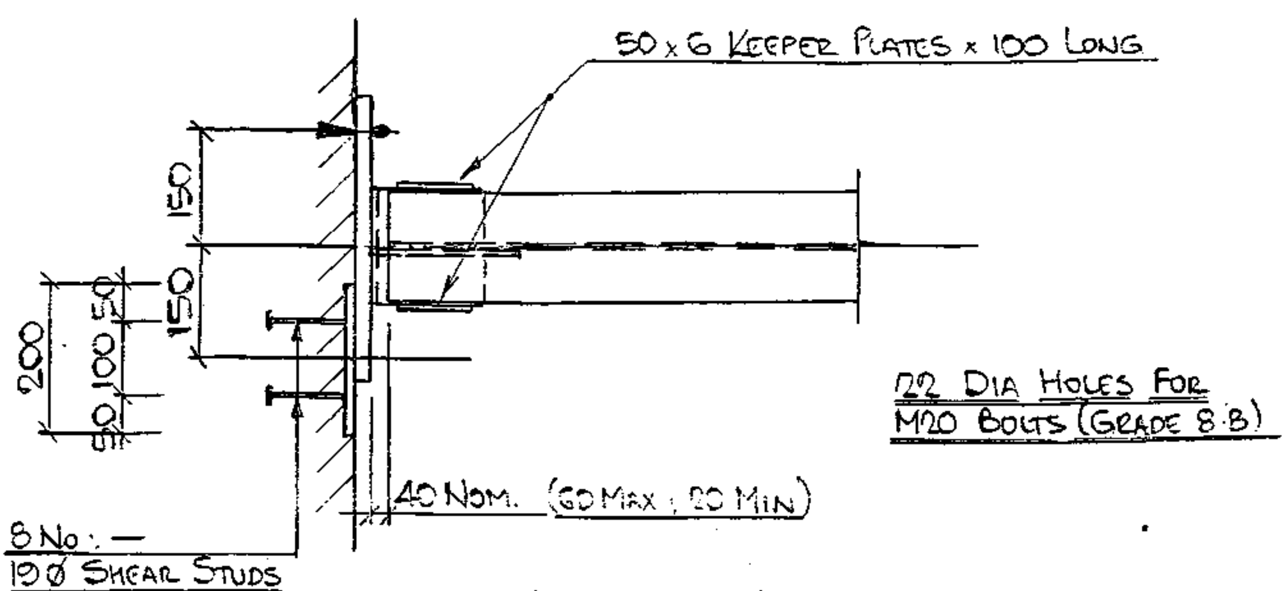
SECTION :— 254 x 146 x 37 UB

LOCATION :— GRID LINES E-F / WEST OF GRID LINE G

MIS-ALIGNMENT 150 MM EAST.

ORIGINAL DESIGN REFER SHEET NOS: — CW/22-24 (INCLUSIVE)





VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SERIES

SHT. No. CW/64 REV.

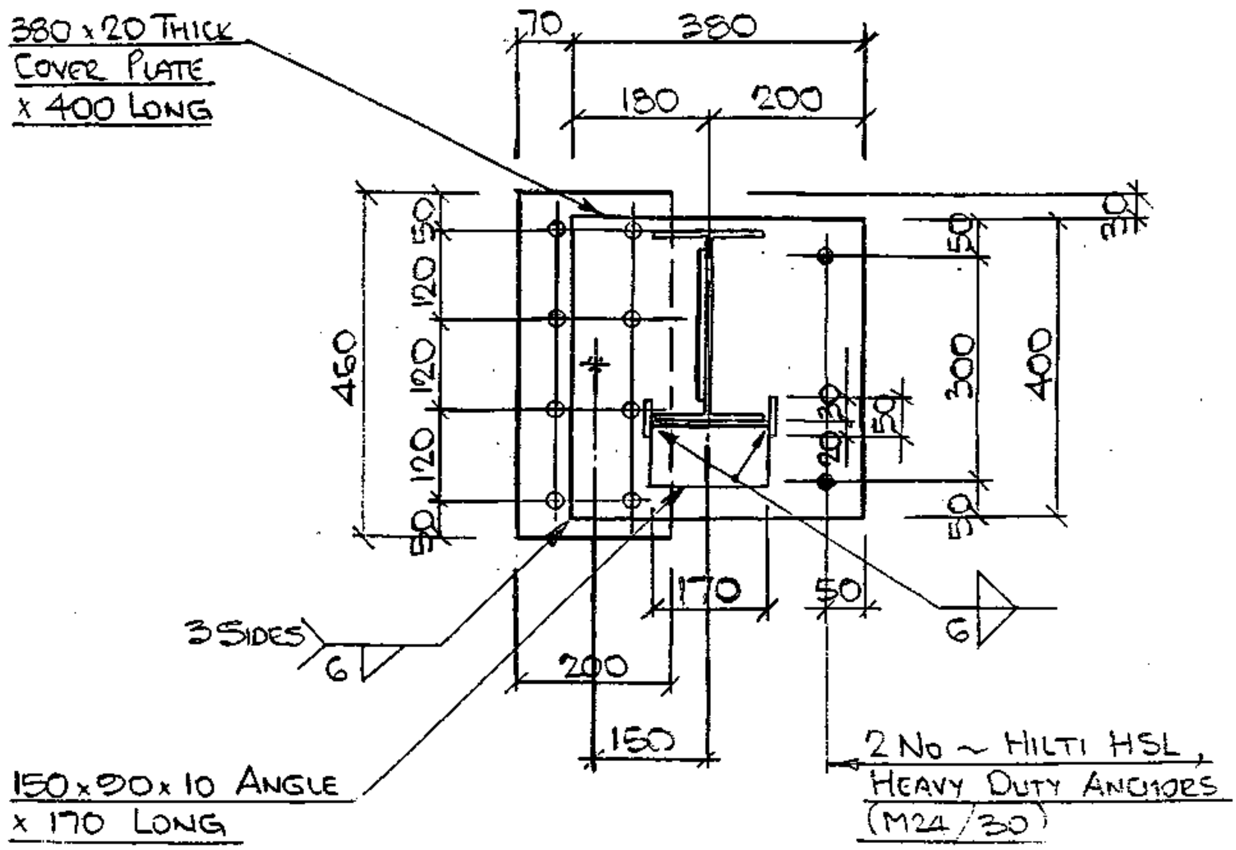
SUBJECT CONNECTIONS TO CONCRETE CORE WALLS

MIS-ALIGNMENT OF EMBEDMENT PLATE

(CN ~ 28)

BY KB DATE 08/98

EX. DATE



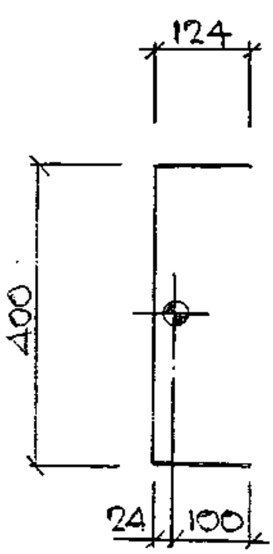
VIEW ON 'B-B'

DESIGN OF WELD FOR COVER PLATE TO EMBEDMENT PLATE

ASSUMING 6 LEG FILLET WELD

ACTUAL HORIZONTAL WELD LENGTH : 130 mm

EFFECTIVE HORIZONTAL WELD LENGTH : 130 - 6 = 124 mm



CONSIDER WELD OF UNIT LEG LENGTH

AREA = $(2 \times 124) + 400 = 648\ \text{mm}^2$

TO FIND NEUTRAL AXIS ~
TAKE MOMENTS ABOUT L.H. SIDE

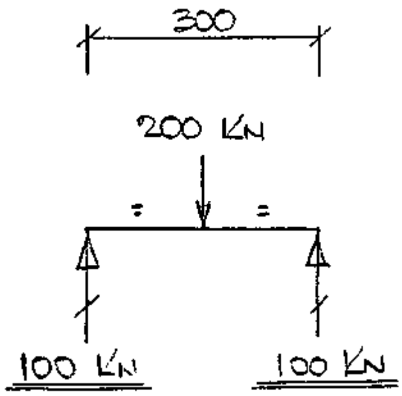
$648\bar{x} = (2 \times 124 \times 62)$

$\bar{x} = 15376 \div 648 = \underline{24\ \text{mm}}$

LOAD ON SHEAR STUDS

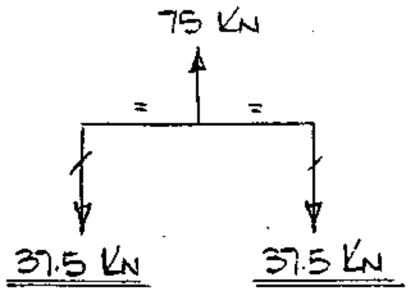
a) SHEAR

INDUCED BENDING MOMENT
DUE TO LOAD ECCENTRICITY
: 1mm IS NEGLIGIBLE



b) TENSION

INDUCED BENDING MOMENT
DUE TO LOAD ECCENTRICITY
: 1mm IS NEGLIGIBLE



BY INSPECTION : —

USE : — 6 LEG FILLET WELD (3 SIDES)

CHECK SHEAR STUDS

8 No ~ 19 Ø x 100 LONG STANDARD STUDS

SHEAR / STUD : 100 ÷ 8 : 12.5 KN

TENSION / STUD : 37.5 ÷ 8 : 4.7 KN.

BY INSPECTION : —

WITH SHEET No CW/5

SHEAR STUDS SATISFACTORY

DESIGN OF COVER PLATE

TRY :— 20 THICK PLATE

TENSION LOAD = 75 KN.

BENDING MOMENT = $75 \times 300 \div 4 = 5625$ KNmm

$Z = 400 \times 20^2 \div 6 = 26666 \text{ mm}^3$

$P_y = 265 \text{ N/mm}^2$ ~ ASSUMING GRADE 43 STEEL

$M_c = 265 \times 26666 \div 10^3$

$= 7066 \text{ KN/mm}^2 > 5625$ KNmm OK

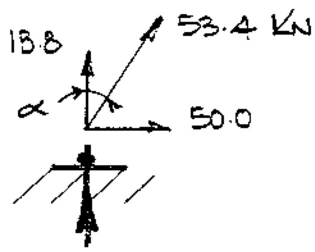
USE :— 20 THICK COVER PLATE (GRADE 43 MINIMUM)

DESIGN OF ANCHOR BOLTS

TRY :— HILTI HSL, HEAVY DUTY ANCHOR

SHEAR LOAD PER ANCHOR = $100 \div 2 = 50$ KN

TENSION LOAD PER ANCHOR = $37.5 \div 2 = 18.8$ KN.



RESULTANT LOAD = $\sqrt{50.0^2 + 18.8^2} = 53.4$ KN

$\alpha = \tan^{-1}\left[\frac{50.0}{18.8}\right] = 69.4°$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES .... SHT. No. CW/67 REV. ....

SUBJECT CONNECTIONS TO CONCRETE CORE WALLS — BY KB DATE 06/98

MIS. ALIGNMENT OF EMBEDMENT PLATE — EX. .... DATE ....

(CN ~ 28)

RECOMMENDED LOAD ; F_{30} = 69.6 KN / BOLT ~ M24 ANCHOR

CONCRETE GRADE : C35

INFLUENCE OF CONCRETE STRENGTH

$$f_B = 1 + 0.02 \times \left(1 - \frac{69.4}{90.0}\right)\left(35 - 30\right) = 1.02$$

INFLUENCE OF EMBEDMENT DEPTH

$$f_T = \frac{166}{155} = 1.07$$

INFLUENCE OF ANCHOR SPACING

$$f_A = \frac{0.15 \times 300}{146} + 0.55 = 0.86$$

$$F_{REC} = 69.6 \times 1.02 \times 1.07 \times 0.86 \times 0.86 \times 1.0$$

$$= \underline{56.2 \text{ KN} > 53.4 \text{ KN}} \text{ OK}$$

USE : — M24/30 ~ HILTI HSL, HEAVY DUTY ANCHORS (2 No)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE PILES

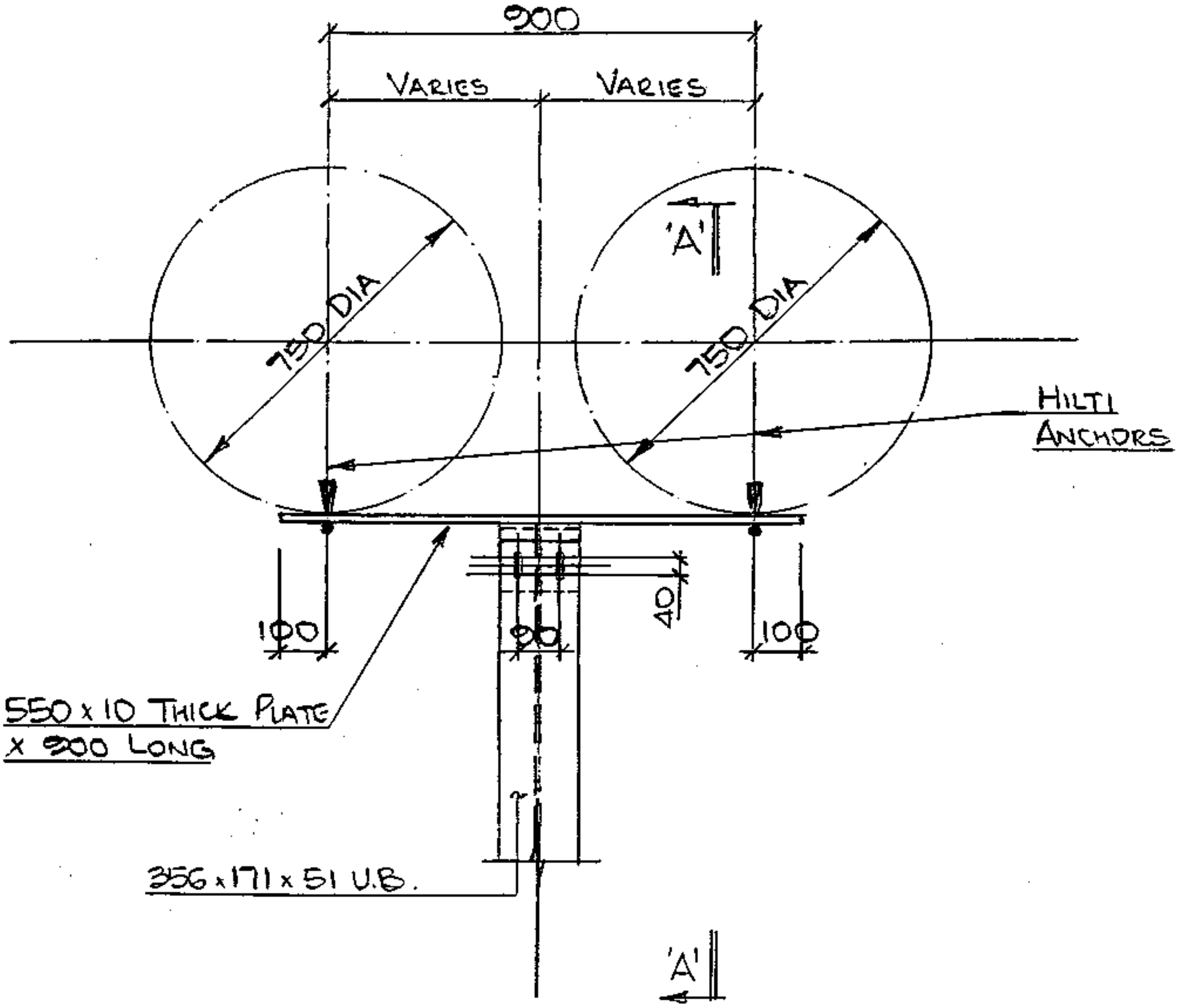
BEAM MARK B5-LG

SERIES ....

SHT. No. CW/68 REV. ....

BY KB DATE 07/98

EX. .... DATE ....



BEAM MARK B5-LG TO CONCRETE PILE

REFERENCE CALCULATIONS : —

SHEET NOS CW/29 - CW/32 (INCLUSIVE)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE PILES

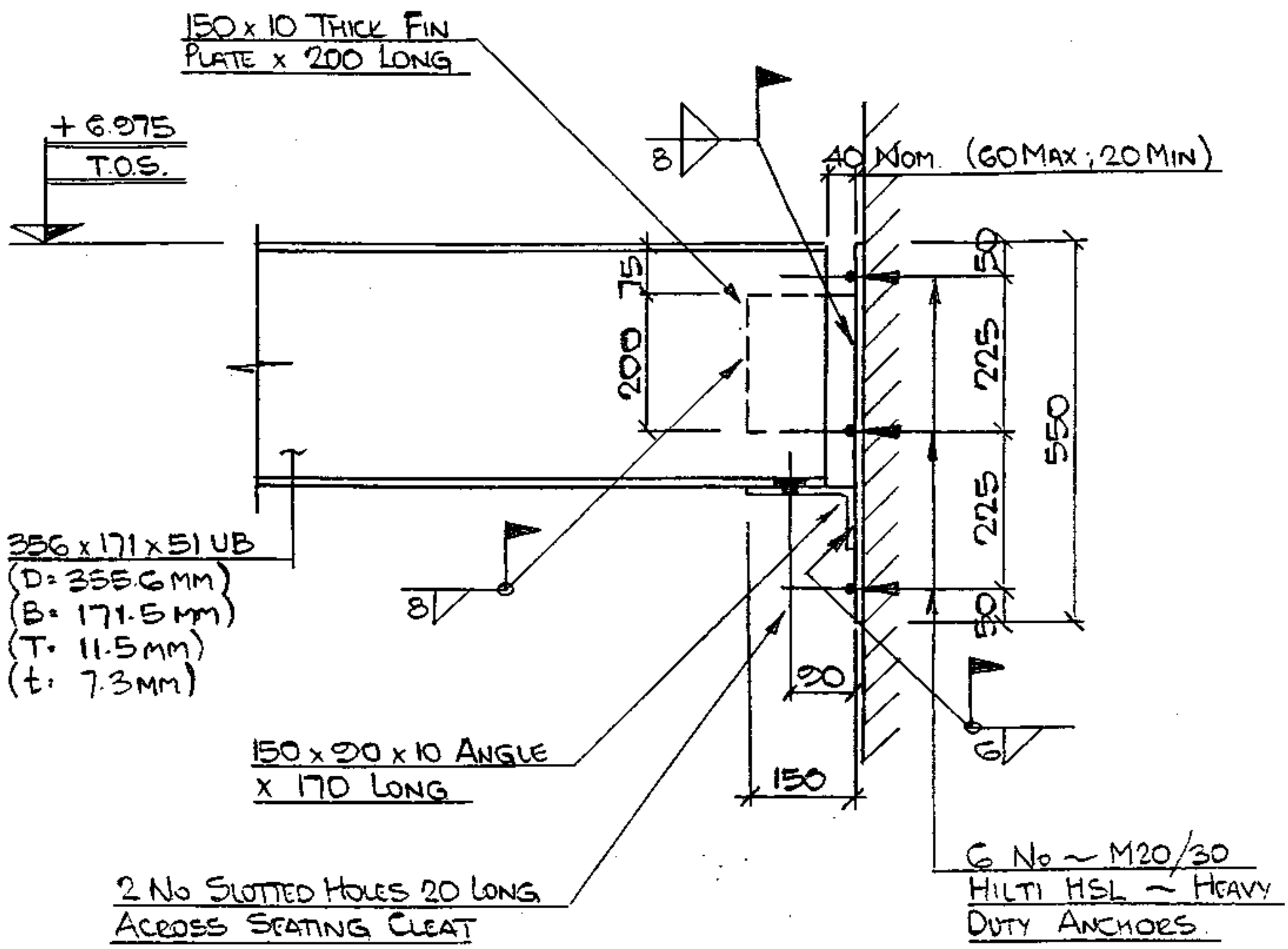
BEAM MARK B5-LG

SERIES

SHT. No. CW/69 REV.

BY KB DATE 07/95

EX. DATE



M20 BOLTS (GRADE 8.8)

VIEW ON 'A-A'

SCALE: — 1:10

BEAM MARK BS-LG

SECTION :— 356 x 171 x 45 KN

END REACTION = 200 KN.

DESIGN OF ANCHORS

TRY :— M20/30 HILTI HSL, HEAVY DUTY ANCHOR

RECOMMENDED SHEAR LOAD = 58.6 KN.

VERTICAL SPACING = 225 mm

REDUCTION FACTOR = 0.80

DESIGN LOAD = 58.6 x 0.80 = 46.8 KN

No ANCHORS REQ^D = 200 ÷ 46.8 = 4.3 ⟶ 6 No.

USE :— 6 No ~ M20/30 HILTI HSL, HEAVY DUTY ANCHORS

END PLATE ~ TRY 10 THICK

A_V = 0.9 x 550 x 10 = 4950 mm^2

P_y = 275 N/mm^2 ~ ASSUME GRADE 43

P_V = 0.6 x 275 x 4950

= 816 KN > 200 KN OK

Z = 10 x 550^2 ÷ 6 = 504166 mm^3

M_C = 275 x 504166 ÷ 10^3 = 138645 KNmm

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE PILES

SERIES

SHT. No. CW/71 REV.

BY KB DATE 07/98

EX. DATE

BM : 200 x 900 ÷ 4

: 45000 KNmm < 138645 KNmm OK

USE :— 10 THICK END PLATE.

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONN TO CONCRETE CORE WALL

SERIES

SHT. No. CW/72 REV.

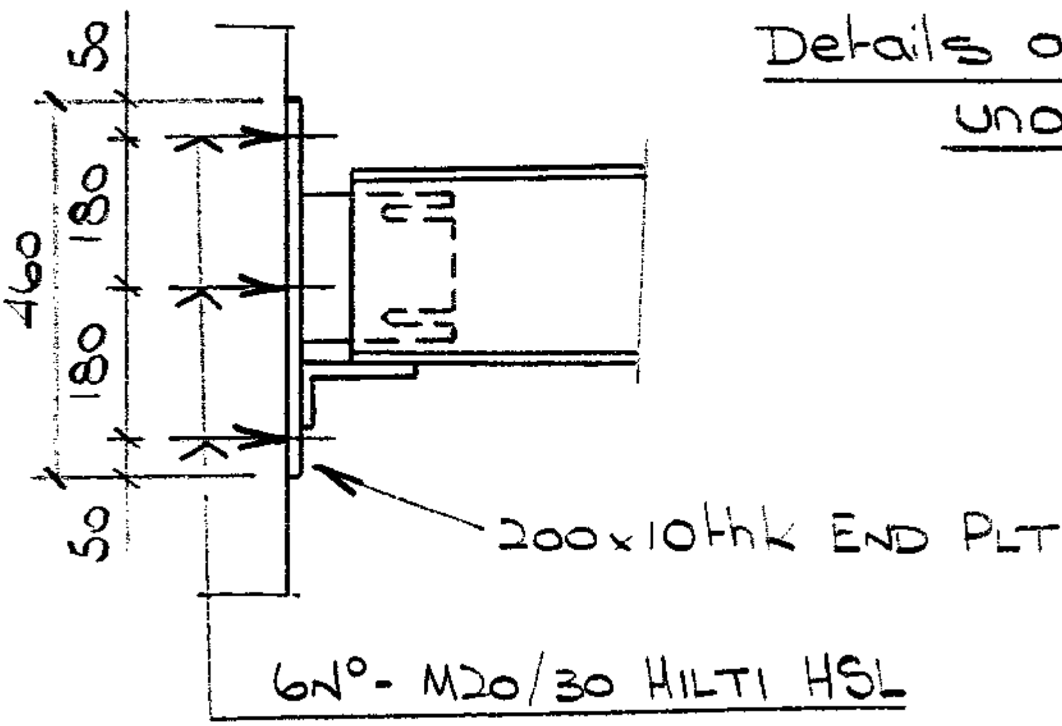
BY KM DATE Sep 98

EX. DATE

BEAM MARK B1-1 (24124), B1-2 (24124), B1-3 (34111) & B1-4 (34111)

SECTION : 254×146×31UB

END SHEAR = 164 kN



Details as CW/22

UNO

DESIGN OF ANCHORS

Try :- M20/30 HILTI HSL, HEAVY DUTY ANCHORS

Recommended Shear Load = 58.6 kN

Vertical Spacing = 180 mm

Reduction Factor = 0.75

Design Load = 58.6 × 0.75 = 44 kN

N° Anchors Req^{d} = 164/44 = 3.7

USE 6N° M20/30 HILTI HSL, HEAVY DUTY ANCHORS

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONN. TO CONCRETE CORE WALL

SERIES

SHT. No. CW/73 REV.

BY KM DATE Sep 98

EX. DATE

Beams B1·1 → 4 cont

Check End Plt.

$A_v = 0.9 \times 460 \times 10 = 4140 \text{ mm}^2$

$p_y = 275 \text{ N/mm}^2$ (Assume Gr 43)

$P_v = 0.6 \times 275 \times 10^{-3} \times 4140 = 683 \text{kN} > 200 \text{kN}$

∴ Plt OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Conn to Concrete Core Wall

SERIES

SHT. No. CW 74 REV.

BY LM DATE Aug 98

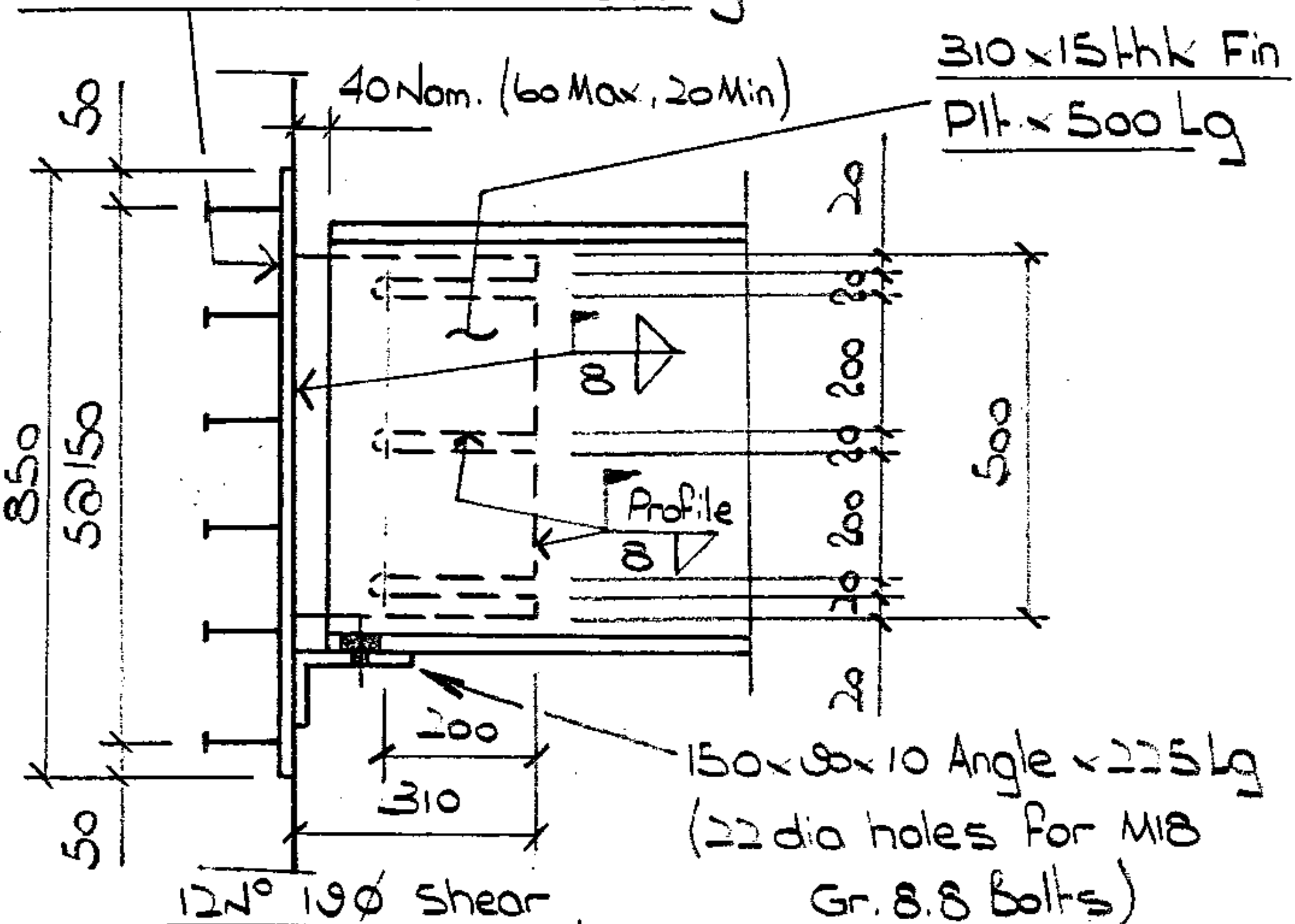
EX. DATE

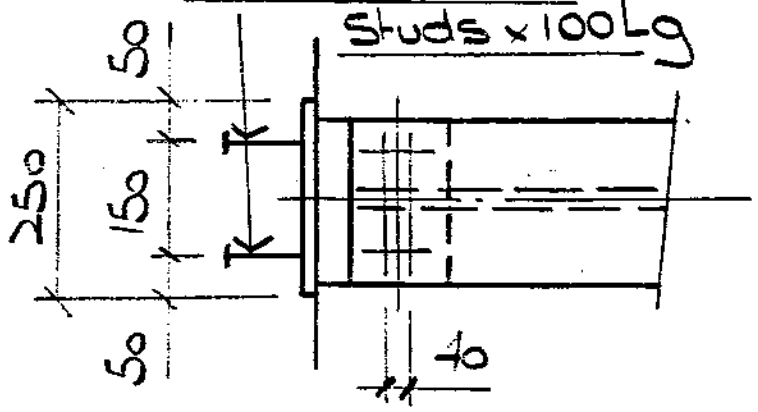
Conn of Beam B8·6 to Concrete Core Wall

Section 610×229×140UB

Shear Load = 1000kN

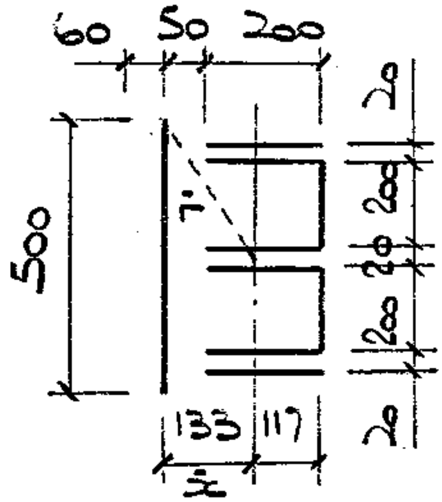
Axial Load = 75kN (Tension)





Beam B8·6 to Core Wall cont.

Design of Fin Plt to Beam Weld



Consider Weld of Unit Leg Length

Area = 500 + 2×200 + 6×200 = 2100 mm²

$$\bar{x} = \frac{2\times200\times250 + 6\times200\times150}{2100}$$

$= \underline{133\,mm}$

$\bar{r} = \sqrt{(133^2 + 250^2)} = \underline{283\,mm}$

$$I_{xx} = \frac{500^3}{12} + \frac{2\times200^3}{12} + \frac{6\times200}{12} + \left[2\times200\times\left(10^2+210^2+230^2\right)\right]$$

$= \underline{50\ 590\ 100\,mm^4}$

$$I_{yy} = \frac{500}{12} + \frac{2\times200}{12} + \frac{6\times200^3}{12} + 500\times133^2 + 2\times200\times117^2 + 6\times200\times17^2$$

$= \underline{18\ 666\ 975\,mm^4}$

$J_p = 50\ 590\ 100 + 18\ 666\ 975 = \underline{69\ 257\ 075\,mm^4}$

$Z_p = 69\ 257\ 075 / 283 = \underline{244\ 725\,mm^3}$

$f_v = 1000/2100 = 0.48\,kN/mm^2$

$f_h = 75/2100 = 0.036\,kN/mm^2$

$f_m = 1000\times(60+133)/244\ 725 = 0.79\,kN/mm^2$

$f_m\,(V_c) = 0.79\times133/283 = 0.37\,kN/mm^2$

$f_m\,(H_c) = 0.79\times250/283 = 0.70\,kN/mm^2$

$f_r = \sqrt{\left[(0.48+0.37)^2 + (0.036+0.70)^2\right]} = \underline{1.124\,kN/mm^2}$

$$\text{Weld size} = \frac{1.124\times10^3}{0.7\times215} = \underline{7.4\,mm}$$

Use 8mm FW

Beam B8·6 to Core Wall cont.

Check Fin Plate

i) Shear

$A_v = 0.9 \times 500 \times 15 = 6750\ mm^2$

$P_v = 0.6 \times 275 \times 10^{-3} \times 6750$

$= \underline{1114\ kN} > 1000\ kN$

ii) Moment

$M = 1000 \times 60 = \underline{60\,000\ kNmm}$

$Z = 15 \times 500^2/6 = 625\,000\ mm^3$

$M_{cx} = 1.2 \times 275 \times 10^{-3} \times 625\,000$

$= \underline{206\,250\ kNmm} > 60\,000\ kNmm$

For a plt. $S_x = S_v$

$S_x = 15 \times 500^2/4 = 937\,500\ mm^3$

$M_{cx} = 275 \times 10^{-3} \times 937\,500 \times \left[1 - \left(\frac{2.5 \times 1000}{1114} - 1.5\right)\right]$

$= \underline{65\,957\ kNmm} > 60\,000\ kNmm$

$\therefore$ Fin Plt OK

Fin Plt Weld

$F_V = 1000/2 \times 500 = 1.0\ kN/mm^2$

$F_H = 75/2 \times 500 = 0.075\ kN/mm^2$

$F_R = \sqrt{(1.0^2 + 0.075^2)} = \underline{1.003\ kN/mm^2}$

Weld size $= \frac{1.003 \times 10^3}{0.7 \times 215} = \underline{6.7\ mm}$

Use 8mm FW

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Conn to Concrete Core Wall

SERIES

SHT. No. CW77 REV.

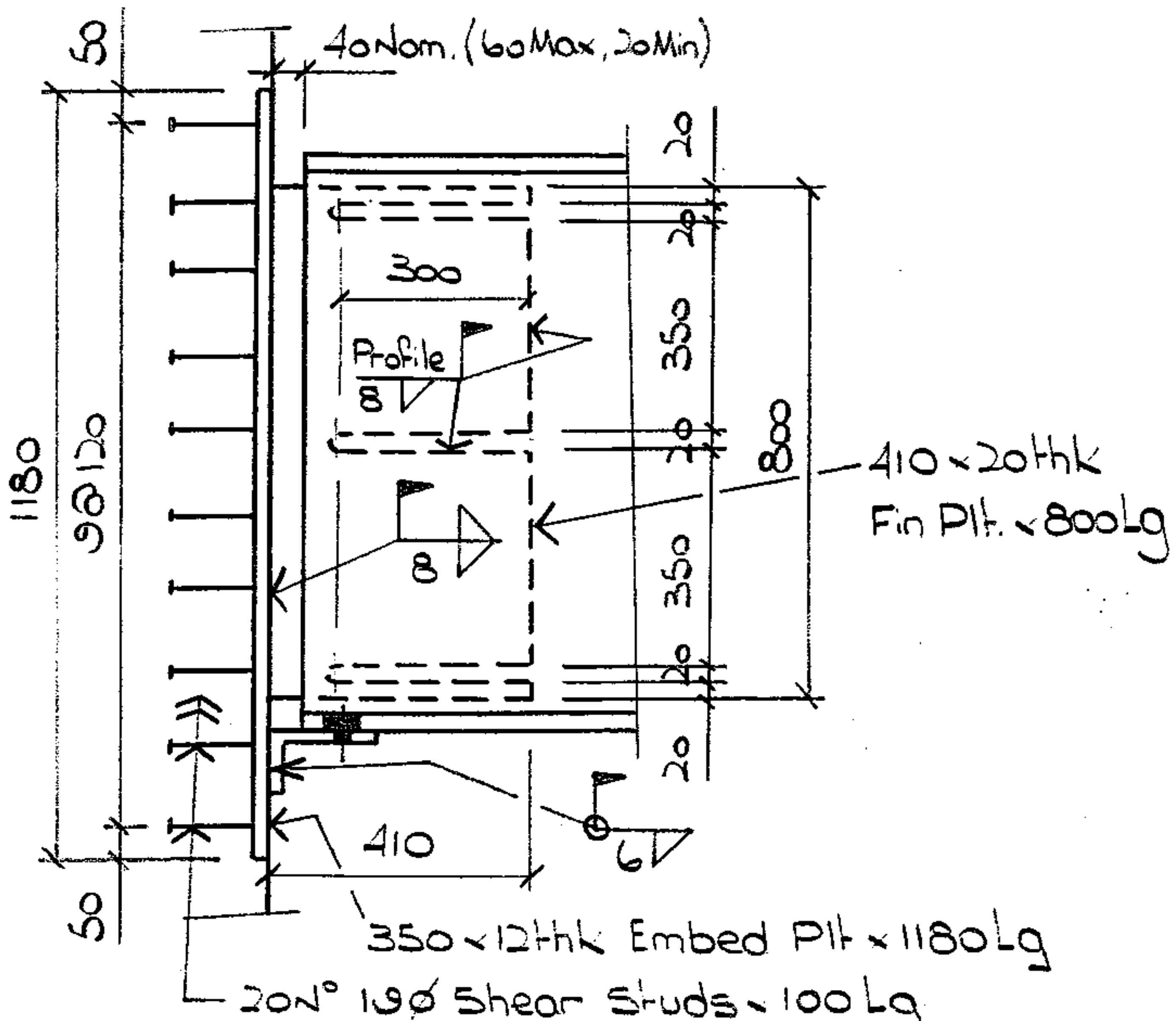
BY KM DATE Aug 98

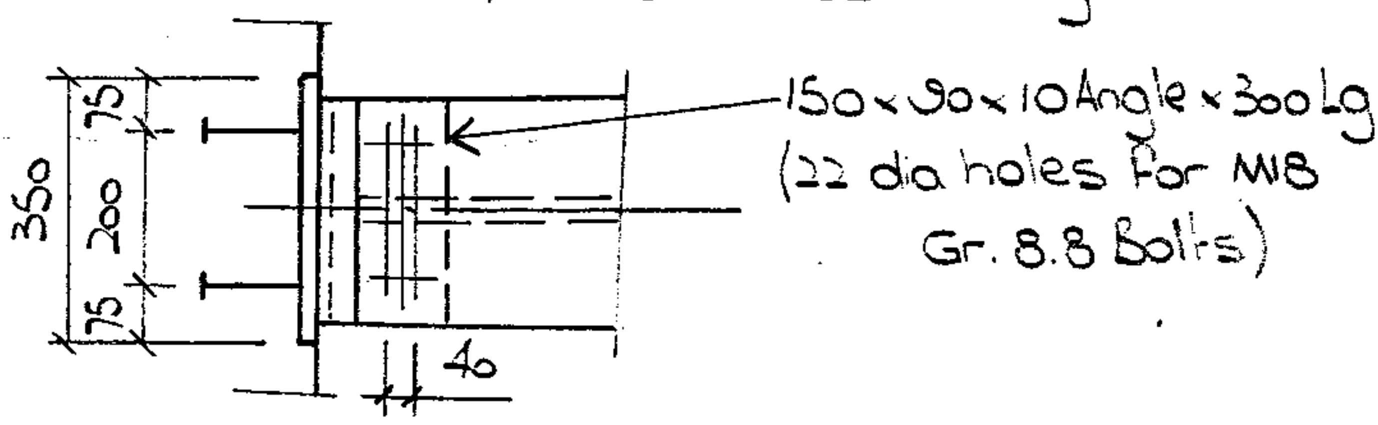
EX. DATE

Conn of Beam B9·6 to Concrete Core Wall

Section : 914 × 305 × 253 UB

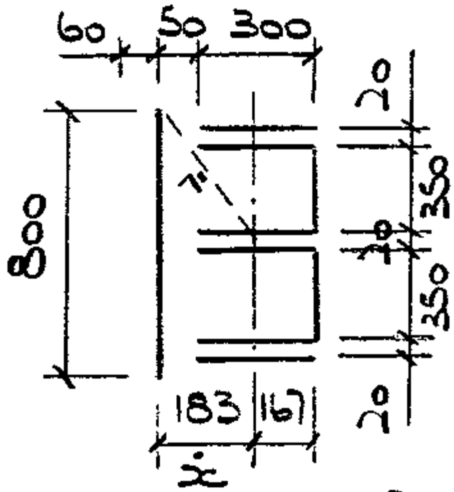
Shear Load = 1800 kN   Axial Load = 75 kN (Tension)





Beam B9·6 to Core Wall cont.

Design of Fin Plt to Beam Weld



Consider Weld of Unit Leg Length

Area $= 800 + 2\times350 + 6\times300 = \underline{3300\text{ mm}^2}$

$\bar{x} = \dfrac{2\times350\times350 + 6\times300\times200}{3300}$

$= \underline{183\text{mm}}$

$\bar{r} = \sqrt{(183^2 + 400^2)} = \underline{440\text{ mm}}$

$I_{xx} = \dfrac{800^3}{12} + \dfrac{2\times350^3}{12} + \dfrac{6\times300}{12} + \left[2\times300\times(10^2+360^2+380^2)\right]$

$= \underline{214\,272\,650\text{ mm}^4}$

$I_{yy} = \dfrac{800}{12} + \dfrac{2\times350}{12} + \dfrac{6\times300^3}{12} + 800\times183^2 + 2\times350\times167^2 + 6\times300\times17^2$

$= \underline{60\,333\,825\text{ mm}^4}$

$I_p = 214\,272\,650 + 60\,333\,825 = \underline{274\,606\,475\text{ mm}^4}$

$Z_p = 274\,606\,475 / 440 = \underline{624\,106\text{ mm}^3}$

$f_v = {}^{1800}/_{3300} = 0.54\text{ kN/mm}^2$

$f_h = {}^{75}/_{3300} = 0.023\text{ kN/mm}^2$

$f_m = {}^{1800\times(60+183)}/_{624\,106} = 0.70\text{ kN/mm}^2$

$f_m\,(V_c) = {}^{0.70\times183}/_{440} = 0.29\text{ kN/mm}^2$

$f_m\,(H_c) = {}^{0.70\times400}/_{440} = 0.64\text{ kN/mm}^2$

$f_r = \sqrt{\left[(0.54+0.29)^2 + (0.023+0.64)^2\right]} = \underline{1.06\text{ kN/mm}^2}$

Weld size $= \dfrac{1.06\times10^3}{0.7\times215} = \underline{7.04\text{mm}}$

<u>Use 8mm FW</u>

Beam B9.6 to Core Wall cont.

Check Fin Plt

i/ Shear $A_v = 0.9 \times 800 \times 20 = 14\,400\ mm^2$

$P_v = 0.6 \times 275 \times 10^{-3} \times 14\,400$

$= \underline{2\,376\ kN} > 1800\ kN$

ii/ Moment $M = 1800 \times 60 = \underline{108\,000\ kNmm}$

$Z = 20 \times 800^2/6 = 2\,133\,333\ mm^3$

$M_{cx} = 1.2 \times 275 \times 10^{-3} \times 2\,133\,333$

$= \underline{704\,000\ kNmm} > 108\,000\ kNmm$

For a plt $S_x = S_v$

$S_x = 20 \times 800^2/4 = 3.2 \times 10^6\ mm^3$

$M_{cx} = 275 \times 10^{-3} \times 3.2 \times 10^6 \left[1 - \left(\frac{2.5 \times 1800}{2376} - 1.5\right)\right]$

$= \underline{533\,333\ kNmm} > 108\,000\ kNmm$

$\therefore$ Fin Plt ok

Fin Plt. Weld

$F_v = 1800/2 \times 800 = 1.125\ kN/mm^2$

$F_H = 75/2 \times 800 = 0.046\ kN/mm^2$

$F_R = \sqrt{(1.125^2 + 0.046^2)} = \underline{1.126\ kN/mm^2}$

Weld size $= \frac{1.126 \times 10^3}{0.7 \times 215} = \underline{7.5\ mm}$

Use 8mm FW

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

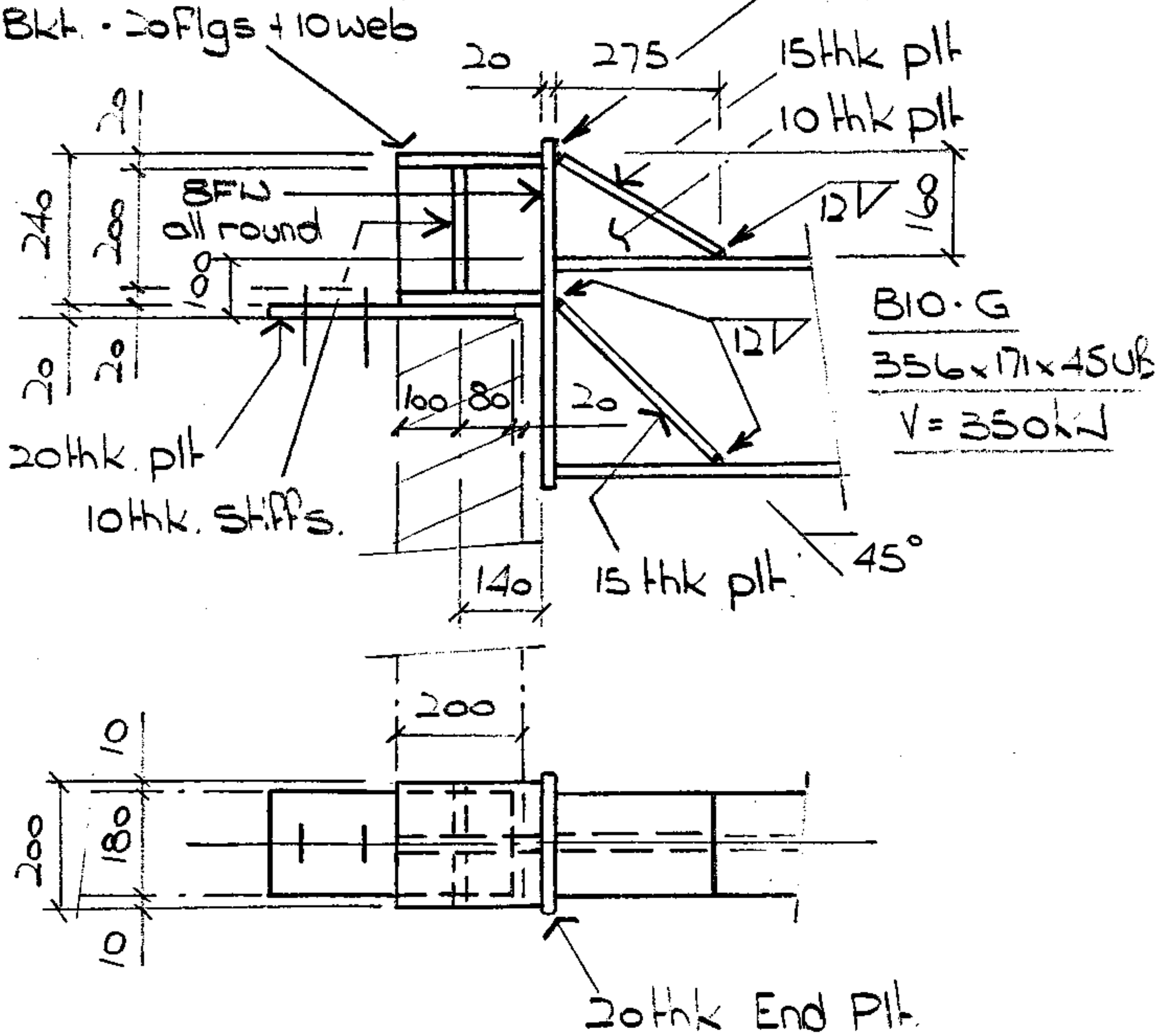
SERIES

SHT. No. CW80 REV.

BY KM DATE Sep 98

EX. DATE

Beam B10·G to Core Wall



All welds 6FW uno

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

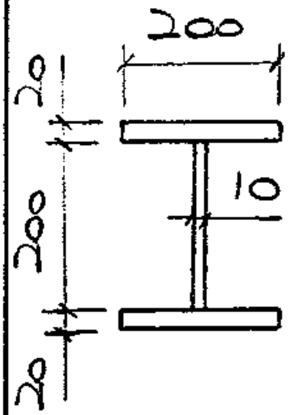
SERIES

SHT. No. CW81 REV.

BY KM DATE Sep 98

EX. DATE

Check Bracket



$$I_{xx} = \frac{10 \times 200^3}{12} + 2 \times 200 \times 20 \times 110^2$$

$$= 103\,466\,667\ mm^4$$

Shear

$$A_v = 0.9 \times 240 \times 10 = 2160\ mm^2$$

$$P_v = 0.6 \times 355 \times 10^{-3} \times 2160 = \underline{460 kN} > 350 kN$$

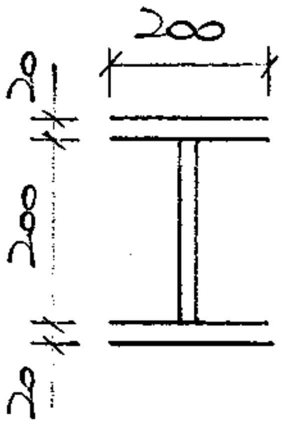
Bending

$$M = 350 \times 140 \times 10^{-3} = \underline{49\ kNm}$$

$$P_b = \frac{103\,466\,667}{120} \times 355 \times 10^{-6} = \underline{306\ kNm} > 49\ kNm$$

$\therefore$ OK

Bracket Weld



$$I_{xx} = \frac{2 \times 200^3}{12} + 2 \times 200 \times 100^2 + 2 \times 200 \times 120^2$$

$$= 11\,093\,333\ mm^4$$

Try 8FW

$$f_v = \frac{350 \times 10^3}{2 \times 200 \times 5.6} = 156\ N/mm^2$$

$$f_b = \frac{140 \times 350 \times 10^3 \times 120}{11\,093\,333 \times 5.6} = 95\ N/mm^2$$

$$f_r = \sqrt{(156^2 + 95^2)} = \underline{183\ N/mm^2} < 255\ N/mm^2$$

$\therefore$ OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDEN

SUBJECT Connection Design

SERIES

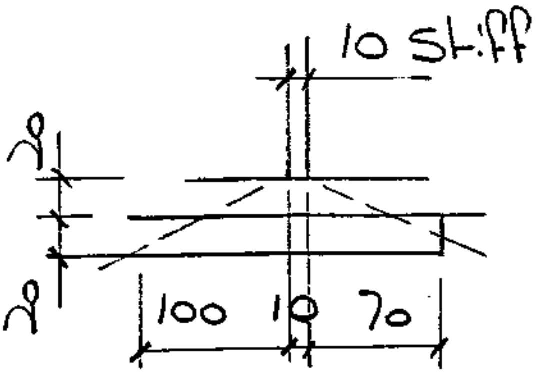
SHT. No. CW82 REV.

BY KM DATE Sep 98

EX. DATE

Beam B10-G to Core Wall cont

Check Bearing on Concrete



Bearing on wall

$= \frac{350 \times 10^3}{180 \times 180}$

$= \underline{10.8\ N/mm^2} < 0.4 \times 35$

$\therefore$ OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALLS

BEAM MARK B13-5

SERIES

SHT. No. CW/83 REV.

BY KB DATE 09/98

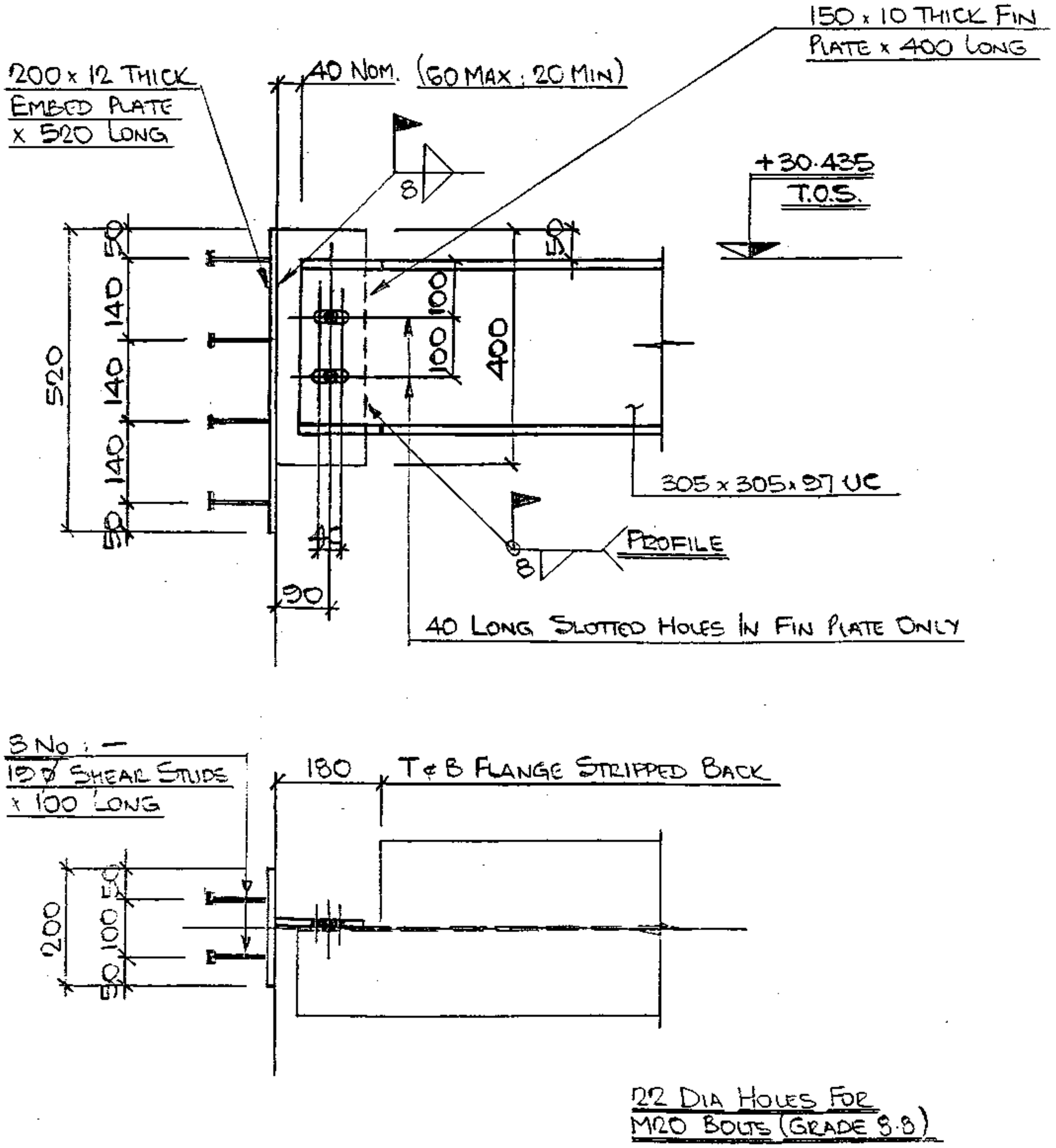
EX. DATE

BEAM MARK :— B13-5

SECTION :— 305 x 305 x 97 UC

SHEAR LOAD : 340 KN

AXIAL LOAD : 75 KN (TENSION)



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALLS

BEAM MARK B13-5

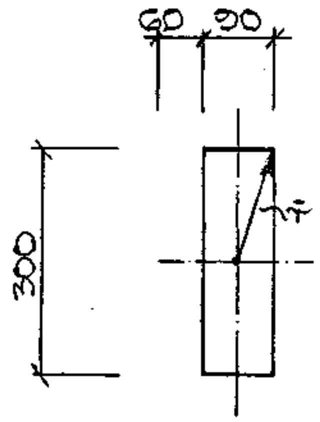
SERIES

SHT. No. CW/84 REV.

BY KB DATE 03/98

EX. DATE

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA: $2 \times (300 + 90) = 780 \text{ mm}^2$

$\bar{r} = \sqrt{45^2 + 150^2} = 156.6 \text{ MM}$

$I_{xx} = \left(\frac{2 \times 300^3}{12}\right) + \left(\frac{2 \times 90}{12}\right) + (2 \times 90 \times 150^2) = 8550015 \text{ mm}^4$

$I_{yy} = \left(\frac{2 \times 300}{12}\right) + \left(\frac{2 \times 90^3}{12}\right) + (2 \times 300 \times 45^2) = 1336550 \text{ mm}^4$

$I_p = 8550015 + 1336550 = 9886565 \text{ mm}^4$

$Z_p = 9886565 \div 156.6 = 63132 \text{ mm}^3$

$f_v = 340 \div 780 = 0.44 \text{ KN/mm}^2$

$f_H = 75 \div 780 = 0.10 \text{ KN/mm}^2$

$f_m = 340 \times 105 \div 63132 = 0.57 \text{ KN/mm}^2$

$f_m(V_c) = 0.57 \times 45 \div 156.6 = 0.17 \text{ KN/mm}^2$

$f_m(H_c) = 0.57 \times 150 \div 156.6 = 0.55 \text{ KN/mm}^2$

$f_{RES} = \sqrt{(0.44 + 0.17)^2 + (0.10 + 0.55)^2} = 0.90 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D $= \frac{0.90 \times 10^3}{0.70 \times 215} = 6.0 \text{ mm} \longrightarrow$ 6 LEG F.W. (MIN^M/)

USE: — 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE : 400 mm

EFFECTIVE WELD LENGTH : 400 − (2×10) : 380 mm

F_V : 340 ÷ (2 × 380) : 0.45 KN/mm²

F_H : 75 ÷ (2 × 380) : 0.10 KN/mm²

F_{RES} : $\sqrt{0.45^2 + 0.10^2}$: 0.46 KN/mm²

WELD STRENGTH (GRADE 43) : 215 N/mm²

WELD SIZE REQ^D = $\frac{0.46 \times 10^3}{0.70 \times 215}$: 3.1 mm ⟶ 6 LEG F.W. (MIN^M)

USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

CHECK FIN PLATE

TRY :— 10 THICK PLATE

a) SHEAR

SHEAR FORCE : 340 KN

P_y : 275 N/mm²

A_V : 0.9 × 10 × 400 : 3600 mm²

P_V : 0.6 × 275 × 3600 ÷ 10³

: 594 KN > 340 KN. OK (FV < 0.6 PV)

b) MOMENT

BENDING MOMENT : 340 × 60 : 20400 KNmm

Z : 10 × 400² ÷ 6 : 266666 mm³

$M_{cx} = 1.2 \times 275 \times 266666 \div 10^3$

$= \underline{88000 \text{ KNmm} > 20400 \text{ KNmm}}$ OK

$S = 10 \times 400^2 \div 4 = 400000 \text{ mm}^3$

$M_{cx} = 275 \times 400000 \div 10^3$

$= \underline{110000 \text{ KNmm} > 20400 \text{ KNmm}}$ OK

USE:— 10 THICK FIN PLATE

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK BS-7 (BG-7 ~ SIMILAR)

(CN ~ 17)

SERIES ...

SHT. No. CW/87 REV. ...

BY KB DATE 09/98

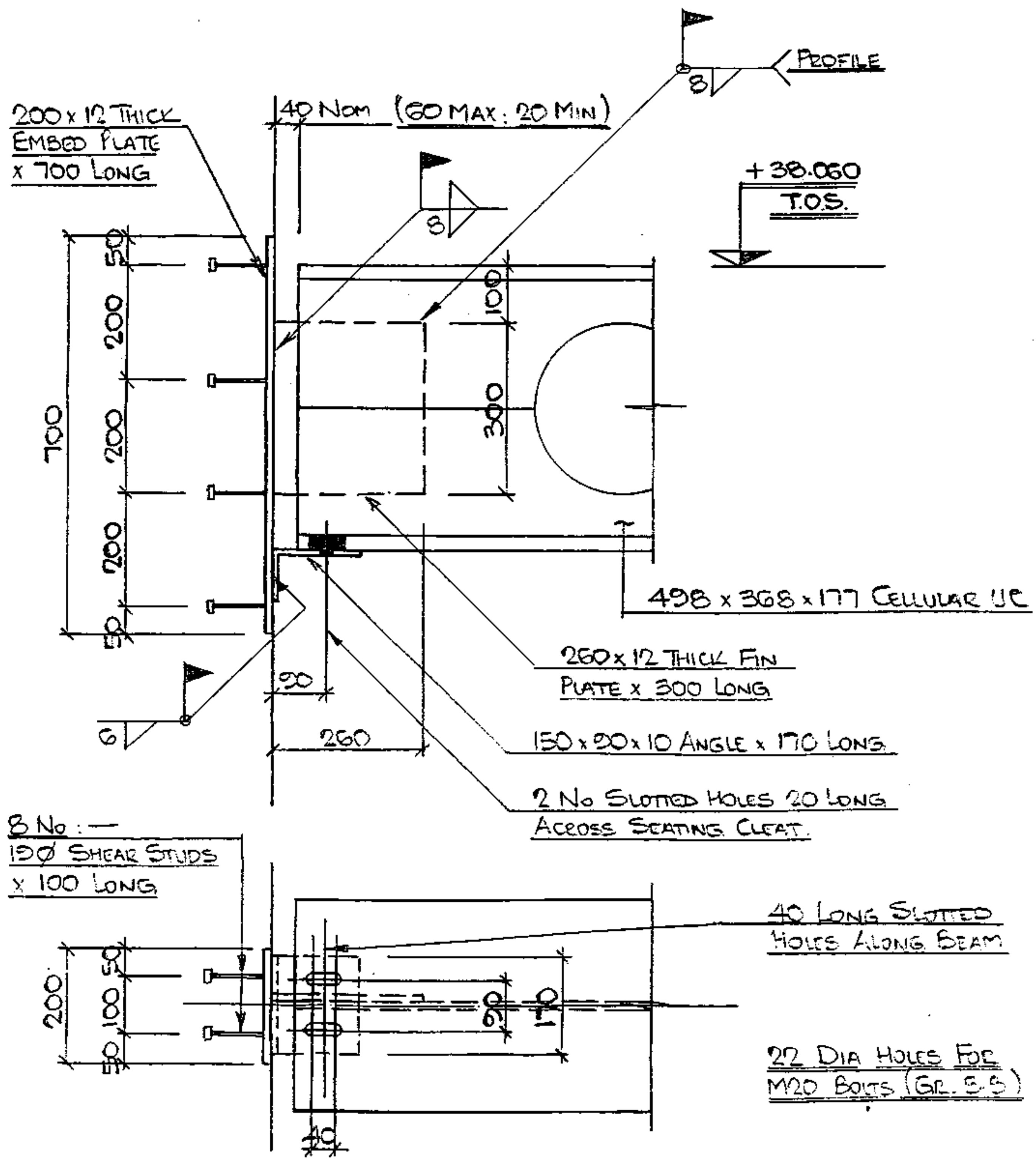
EX. ... DATE ...

BEAM MARK : — BS-7 (BEAM MARK BG-7 ~ SIMILAR)

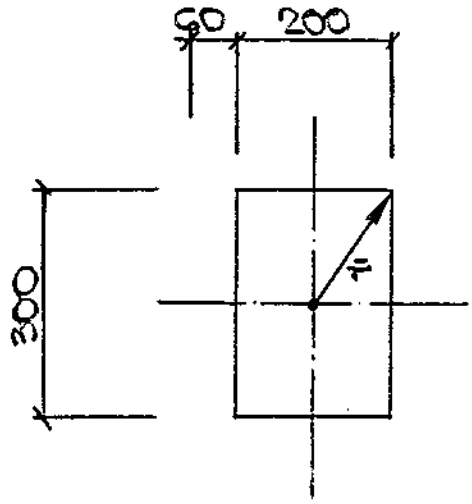
SECTION : — 498 x 368 x 177 CELLULAR UC

SHEAR LOAD : 420 KN ~ BS-7 & BG-7

AXIAL LOAD : 75 KN (TENSION)



DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $2 \times (200 + 300)$

: $1000\ mm^2$

$$I_{xx} : \left(\frac{2 \times 300^3}{12}\right) + \left(\frac{2 \times 200}{12}\right) + (2 \times 200 \times 150^2) = 13500033\ mm^4$$

$$I_{yy} : \left(\frac{2 \times 300}{12}\right) + \left(\frac{2 \times 200^3}{12}\right) + (2 \times 300 \times 100^2) : 7333383\ mm^4$$

$$I_p : 13500033 + 7333383 : 20833416\ mm^4$$

$$r : \sqrt{100^2 + 150^2} : 180.3\ mm$$

$$Z_p : 20833416 \div 180.3 : 115548\ mm^3$$

$$f_V : 420 \div 1000 : 0.42\ KN/mm^2$$

$$f_H : 75 \div 1000 : 0.08\ KN/mm^2$$

$$f_m : 420 \times 160 \div 115548 : 0.59\ KN/mm^2$$

$$f_m\ (V_c) : 0.59 \times 100 \div 180.3 = 0.33\ KN/mm^2$$

$$f_m\ (h_c) : 0.59 \times 150 \div 180.3 = 0.49\ KN/mm^2$$

$$f_{RES} : \sqrt{(0.42 + 0.33)^2 + (0.08 + 0.49)^2} = 0.95\ KN/mm^2$$

WELD STRENGTH (GRADE 43) : 215 N/mm²

WELD SIZE REQ^D $= \dfrac{0.95 \times 10^3}{0.7 \times 215} = 6.4$ mm ⟶ 8 LEG F.W. (MIN^M)

<u>USE :— 8 LEG FILLET WELD</u>

<u>CHECK WELD TO EMBEDMENT PLATE</u>

LENGTH OF FIN PLATE : 300 mm

EFFECTIVE WELD LENGTH : $300 - (2 \times 10) = 280$ mm

$F_V = 420 \div (2 \times 280) = 0.75$ kN/mm^2

$F_H = 75 \div (2 \times 280) = 0.14$ kN/mm^2

$F_{RES} = \sqrt{0.75^2 + 0.14^2} = 0.77$ kN/mm^2

WELD STRENGTH (GRADE 43) : 215 N/mm^2

WELD SIZE REQ^D $= \dfrac{0.77 \times 10^3}{0.7 \times 215} = 5.2$ mm ⟶ 6 LEG F.W. (MIN^M)

<u>USE :— 8 LEG FILLET WELD (DETAIL STANDARDIZATION)</u>

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS | SERIES | SHT. No CW/90 REV.
SUBJECT CONNECTIONS TO CONCRETE CORE WALLS | BY KB DATE 03/98
BEAM MARK BS-7 (BG-7 ~ SIMILAR) | EX. DATE
(CN-17)

CHECK FIN PLATE ~ 12 THICK

a) SHEAR

SHEAR FORCE : 420 KN

P_y : 275 N/mm²

A_v : 0.9 × 12 × 300 : 3240 mm²

P_v : 0.6 × 275 × 3240 ÷ 10³

: 534 KN > 420 KN OK ✓ (FV > 0.6 PV)

b) MOMENT

BENDING MOMENT : 420 × 60 : 25200 KNMM

Z = 12 × 300² ÷ 6 : 180000 mm³

Mcx : 1.2 × P_y × Z

= 1.2 × 275 × 180000 ÷ 10³

: 59400 KNMM > 25200 KNMM OK ✓

FOR A PLATE Sx = Sv

Sx : 12 × 300² ÷ 4 : 270000 mm³

Mcx : 275 × 270000 × (1 - {2.5 × 420 ÷ 534 - 1.5}) ÷ 1000

: 39627 KNMM > 25200 KNMM OK ✓

USE: — 12 THICK FIN PLATE

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B8-7 (B6-7 ~ SIMILAR)

(CN ~ 17)

SERIES ....

SHT. No. CW/51 REV. ....

BY KB DATE 09/98

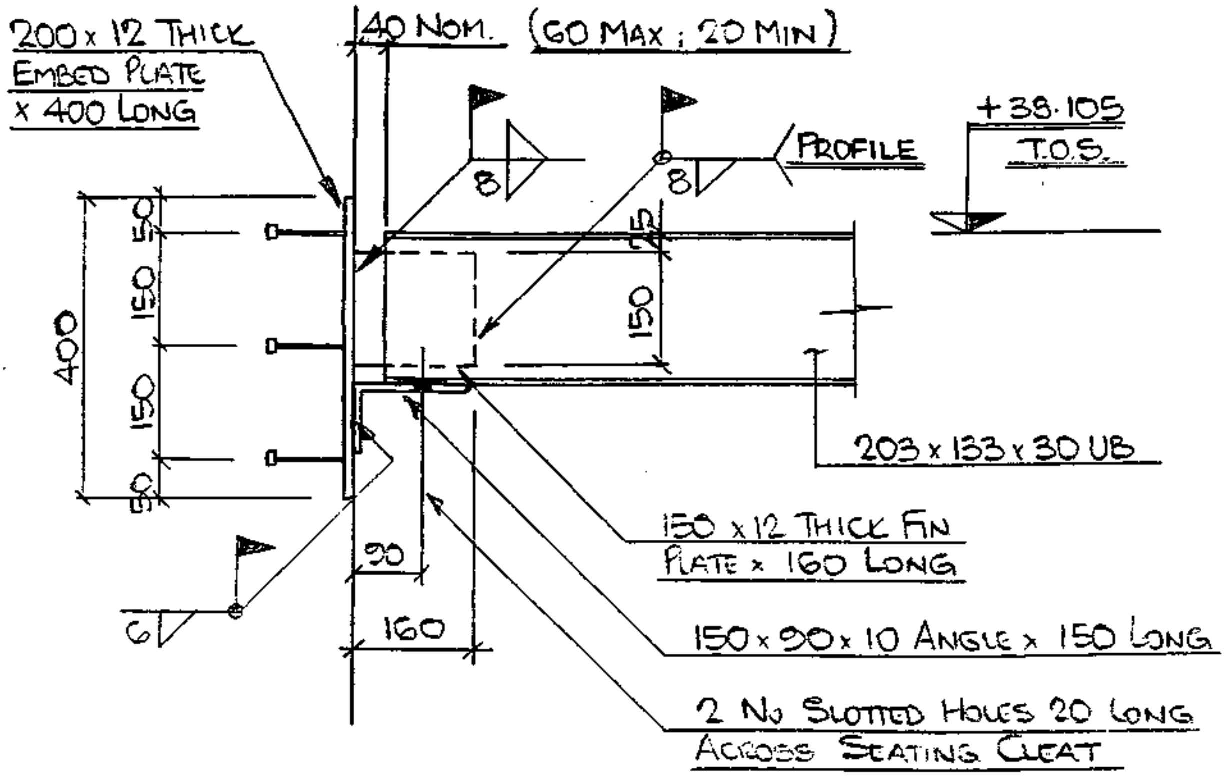
EX. .... DATE ....

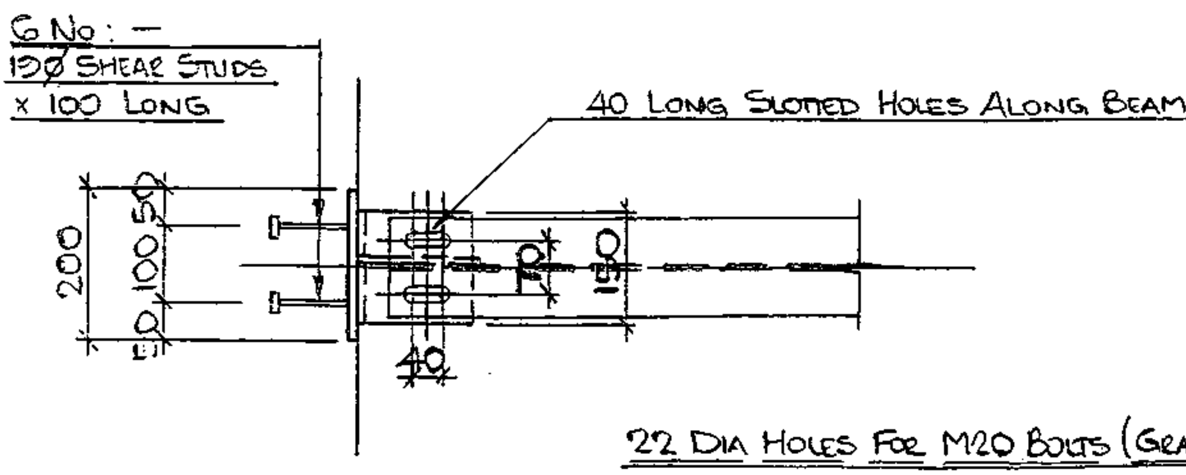
BEAM MARK : — B8-7

SECTION — 203 x 133 x 30 UB

SHEAR LOAD : 170 KN

AXIAL LOAD : 75 KN (TENSION)





22 DIA HOLES FOR M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B8-7

SERIES ......

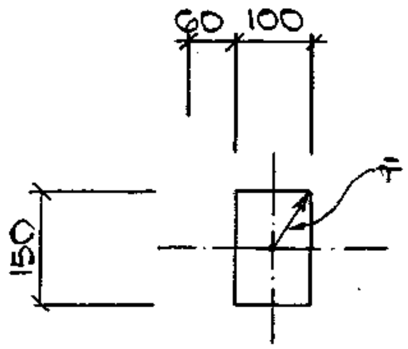
SHT. No. CW/92 REV. ......

BY KB DATE 03/98

EX. ...... DATE ......

(CN ~ 17)

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA = 2 x (150 + 100)

= 500 mm².

$$I_{xx} = \left(\frac{2 \times 150^3}{12}\right) + \left(\frac{2 \times 100}{12}\right) + (2 \times 100 \times 75^2) = 1687516 \text{ mm}^4$$

$$I_{yy} = \left(\frac{2 \times 150}{12}\right) + \left(\frac{2 \times 100^3}{12}\right) + (2 \times 150 \times 50^2) = 916691 \text{ mm}^4$$

$$I_P = 1687516 + 916691 = 2604207 \text{ mm}^4$$

$$\bar{r} = \sqrt{50^2 + 75^2} = 90.2 \text{ mm}$$

$$Z_p = 2604207 \div 90.2 = 28871 \text{ mm}^3.$$

$f_v = 170 \div 500 = 0.34$ KN/mm²

$f_H = 75 \div 500 = 0.15$ KN/mm²

$f_M = 170 \times 110 \div 28871 = 0.65$ KN/mm²

$f_M\,(V_C) = 0.65 \times 50 \div 90.2 = 0.36$ KN/mm²

$f_M\,(H_C) = 0.65 \times 75 \div 90.2 = 0.54$ KN/mm².

$$f_{RES} = \sqrt{(0.34 + 0.36)^2 + (0.15 + 0.54)^2} = 0.98 \text{ KN/mm}^2$$

WELD STRENGTH (GRADE 43) = 215 N/mm².

WELD SIZE REQ^D = $\dfrac{0.98 \times 10^3}{0.7 \times 215}$ = 6.6 mm ⟶ 8 LEG F.W. (MIN^M/)

(CN~17)

USE :— 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE = 150 mm

EFFECTIVE WELD LENGTH = 150 − (2 x 10) = 130 mm

$F_V = 170 \div (2 \times 130) = 0.66 \text{ KN/mm}^2$

$F_H = 75 \div (2 \times 130) = 0.29 \text{ KN/mm}^2$

$F_{RES} = \sqrt{0.66^2 + 0.29^2} = 0.72 \text{ KN/mm}^2$

WELD STRENGTH (GRADE 43) = 215 N/MM²

WELD SIZE REQ^D = $\dfrac{0.72 \times 10^3}{0.7 \times 215}$ = 4.8 mm ⟶ 6 LEG F.W (MIN^{M})

USE :— 8 LEG FILLET WELD (DETAIL STANDARDISATION)

CHECK FIN PLATE ~ 12 THICK

a) SHEAR

SHEAR FORCE = 170 KN

$P_y = 275 \text{ N/mm}^2$

$A_V = 0.9 \times 12 \times 150 = 1620 \text{ mm}^2$

$P_V = 0.6 \times 275 \times 1620 \div 10^3$

= 267 KN > 170 KN OK (FV > 0.6 PV)

b) MOMENT

BENDING MOMENT = 170 x 60 = 10200 KNmm

(CN~17)

$Z = 12 \times 150^2 \div 6 = 45000\ mm^3$

$Mcx = 1.2 \times Py \times Z$

$= 1.2 \times 275 \times 45000 \div 10^3$

$= \underline{14850\ KNmm > 10200\ KNmm}$ OK ✓

FOR A PLATE Sx = SV

$Sx = 12 \times 150^2 \div 4 = 67500\ mm^3$

$Mcx = 275 \times 67500 \times (1 - \{2.5 \times 170 \div 267 - 1.5\}) \div 1000$

$= \underline{18859\ KNmm > 10200\ KNmm}$ OK ✓

<u>USE: — 12 THICK FIN PLATE</u>

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B9-7

SERIES

SHT. No. CW/95 REV.

BY KB DATE 09/98

EX. DATE

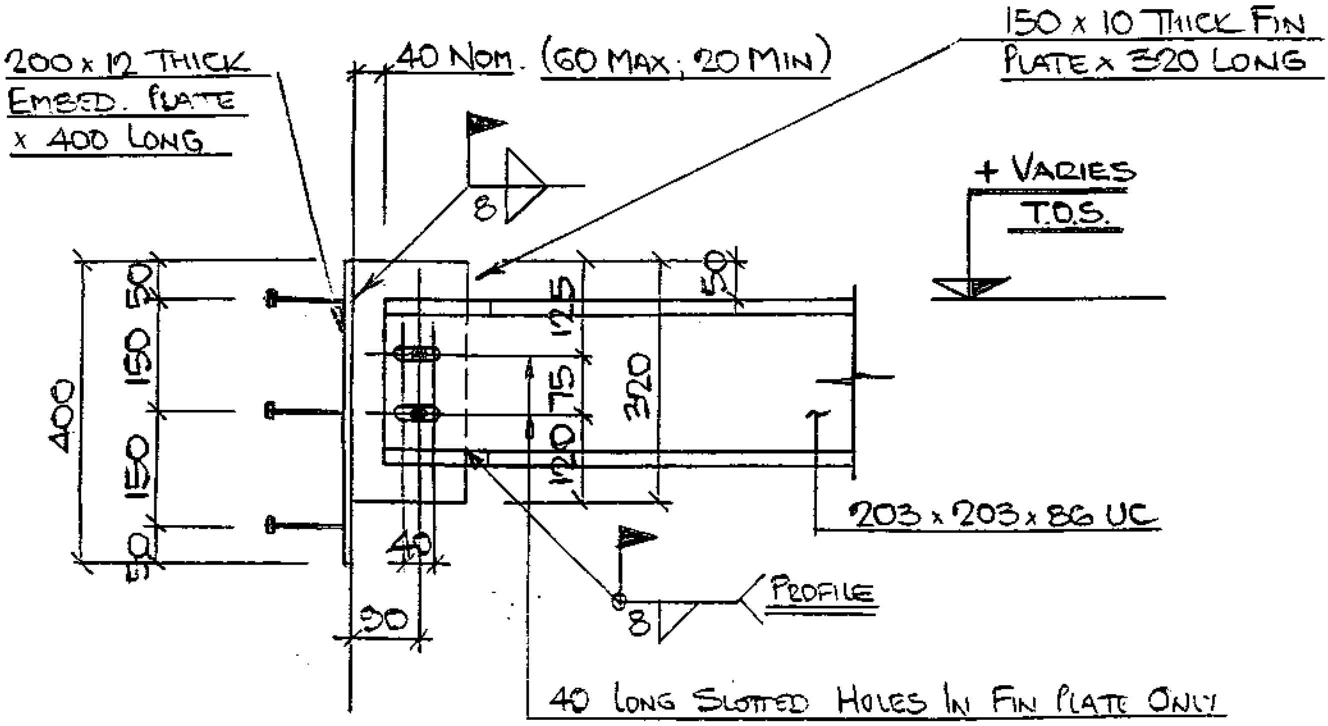
(CN~17)

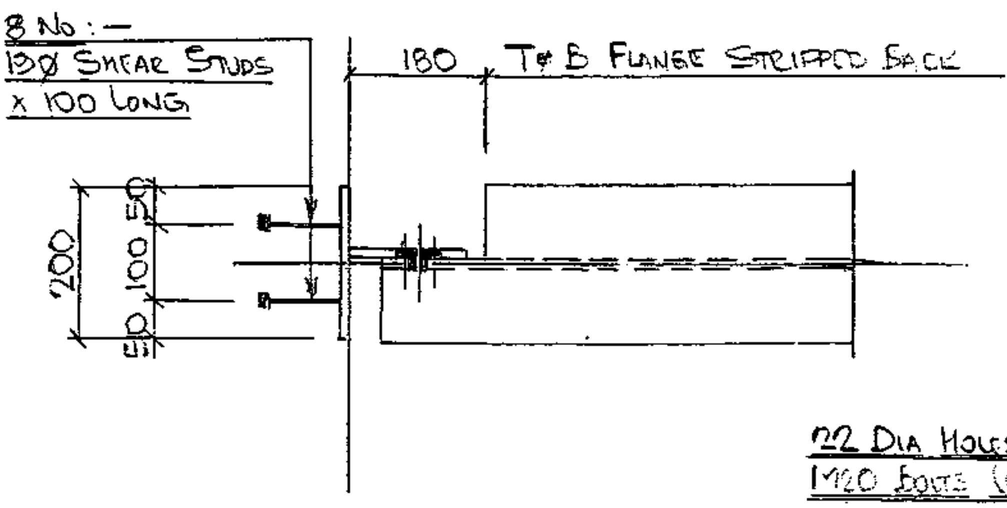
BEAM MARK :— B9-7

SECTION :— 203x203x86 UC

SHEAR LOAD = 200 kN.

AXIAL LOAD : 75 kN (TENSION)





22 DIA HOLES FOR
M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTIONS TO CONCRETE CORE WALL

BEAM MARK B9.7

(CN~17)

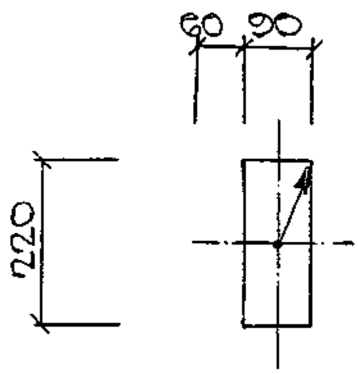
SERIES ....

SHT. No. CW/96 REV. ....

BY KB DATE 09/98

EX. .... DATE ....

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $2 \times (220 + 90) = 620\ mm^2$

$\bar{r} : \sqrt{45^2 + 110^2} = 118.9\ mm$

$I_{xx} : \left(\frac{2 \times 220^3}{12}\right) + \left(\frac{2 \times 90}{12}\right) + (2 \times 90 \times 110^2) = 3952681\ mm^4$

$I_{yy} : \left(\frac{2 \times 220}{12}\right) + \left(\frac{2 \times 90^3}{12}\right) + (2 \times 220 \times 45^2) = 1012536\ mm^4$

$I_p = 3952681 + 1012536 = 4965217\ mm^4$

$Z_p : 4965217 \div 118.9 = 41759\ mm^3$

$f_V : 200 \div 620 = 0.33\ KN/mm^2$

$f_H : 75 \div 620 = 0.12\ KN/mm^2$

$f_M : 200 \times 105 \div 41759 = 0.50\ KN/mm^2$

$f_M(V_c) = 0.50 \times 45 \div 118.9 = 0.19\ KN/mm^2$

$f_M(H_c) : 0.50 \times 110 \div 118.9 = 0.47\ KN/mm^2$

$f_{RES} : \sqrt{(0.33 + 0.19)^2 + (0.12 + 0.47)^2} : 0.79\ KN/mm^2$

WELD STRENGTH (GRADE 43) : $215\ N/mm^2$

WELD SIZE REQ'D : $\frac{0.79 \times 10^3}{0.70 \times 215} : 5.3\ mm$ ⟶ 6 LEG F.W. (MIN^m)

USE : — 8 LEG FILLET WELD

CHECK WELD TO EMBEDMENT PLATE

LENGTH OF FIN PLATE = 320 mm

EFFECTIVE WELD LENGTH = 320 - (2 x 10) = 300 mm

$F_V = 200 \div (2 \times 300) = 0.33\ KN/mm^2$

$F_H = 75 \div (2 \times 300) = 0.13\ KN/mm^2$

$F_{RES} = \sqrt{0.33^2 + 0.13^2} = 0.36\ KN/mm^2$

WELD STRENGTH (GRADE 43) = 215 N/mm^2

WELD SIZE REQ^D = $\dfrac{0.36 \times 10^3}{0.70 \times 215} = 2.4$ mm ⟶ 6 LEG F.W. (MIN^M)

USE: — 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

CHECK FIN PLATE

TRY: — 10 THICK PLATE

a) SHEAR

SHEAR FORCE = 200 KN

$P_y = 275\ N/mm^2$

$A_V = 0.9 \times 10 \times 320 = 2880\ mm^2$

$P_V = 0.6 \times 275 \times 2880 \div 10^3$

$= 475\ KN > 200\ KN$ OK ✓ $(F_V < 0.6 P_V)$

b) MOMENT

BENDING MOMENT = 200 x 60 = 12000 KNmm

$Z = 10 \times 320^2 \div 6 = 170666\ mm^3$

$M_{cx} = 1{\cdot}2 \times 275 \times 170666 \div 10^3$

$= \underline{56320 \text{ KNmm} > 12000 \text{ KNmm}}$ OK✓

$S = 10 \times 320^2 \div 4 = 256000 \text{ mm}^3$

$M_{cx} = 275 \times 256000 \div 10^3$

$= \underline{70400 \text{ KNmm} > 12000 \text{ KNmm}}$ OK✓

USE :— 10 THICK FIN PLATE

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION TO CONCRETE CORE WALL
BEAM MARK B5 - LG (SITE CONDITION)

SERIES ....

SHT. No. CW/99 REV. ....

BY KB DATE 09/98

EX. .... DATE ....

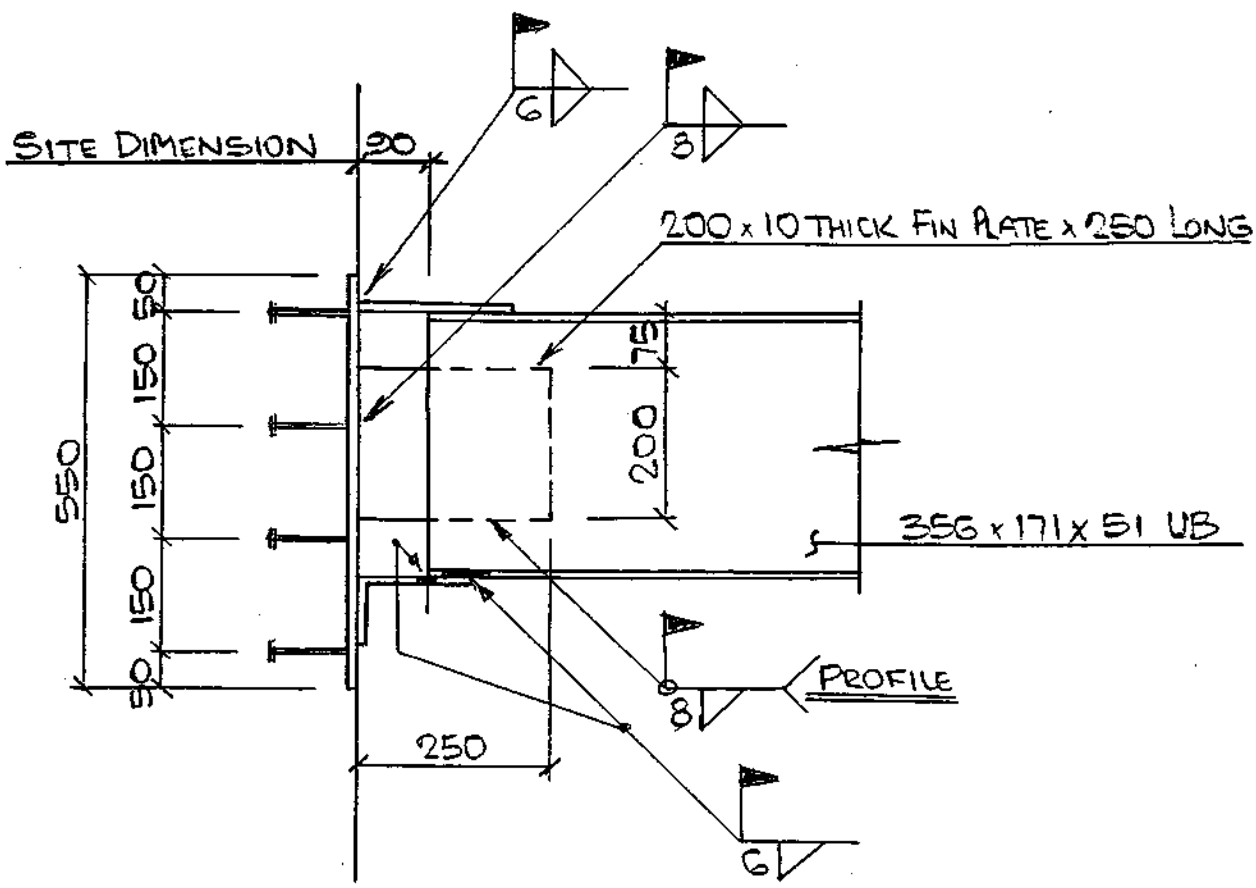
BEAM MARK : — B5-LG

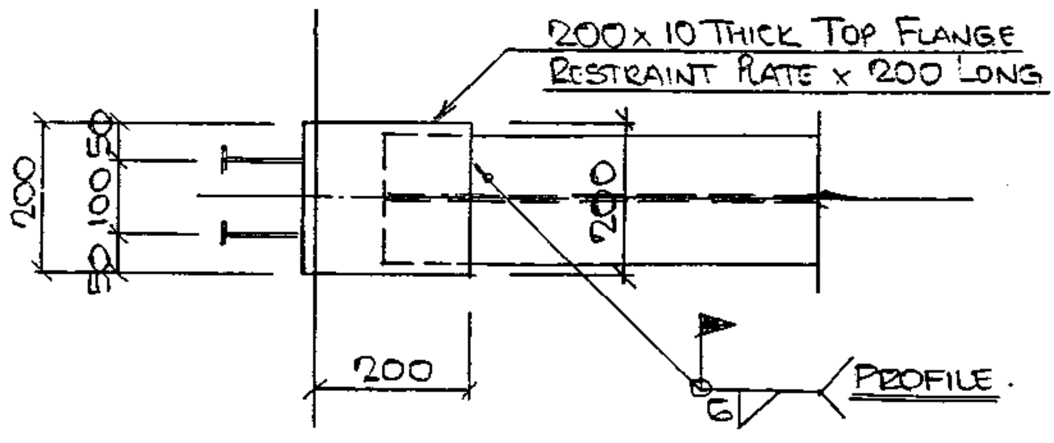
SECTION : — 356 x 171 x 51 UB

SHEAR LOAD = 200 KN

AXIAL LOAD : 75 KN (TENSION)

REFERENCES : — SHEET NOS CW/14 – CW/17 & CW/29 – CW/32





Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONNECTION TO CONCRETE CORE WALL

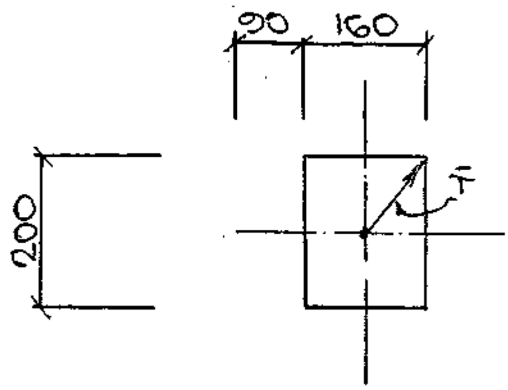
SERIES

SHT. No. CW/100 REV.

BY KB DATE 09/98

EX. DATE

DESIGN OF WELD FOR FIN PLATE TO BEAM



CONSIDER WELD OF UNIT LEG LENGTH

AREA : $2 \times (200 + 160) = 720\ mm^2$

$r = \sqrt{100^2 + 80^2} = 128.0\ mm$

$I_{XX} = \left(\frac{2 \times 200^3}{12}\right) + \left(\frac{2 \times 160}{12}\right) + (2 \times 160 \times 100^2) = 4533360\ mm^4$

$I_{YY} = \left(\frac{2 \times 200}{12}\right) + \left(\frac{2 \times 160^3}{12}\right) + (2 \times 200 \times 80^2) = 3242700\ mm^4$

$I_P = 4533360 + 3242700 = 7776060\ mm^4$

$Z_P = 7776060 \div 128.0 = 60750\ mm^3$

$F_V = 200 \div 720 = 0.28\ kN/mm^2$

$F_H = 75 \div 720 = 0.11\ kN/mm^2$

$F_M = 200 \times 170 \div 60750 = 0.56\ kN/mm^2$

$F_M(V_C) = 0.56 \times 80 \div 128.0 = 0.35\ kN/mm^2$

$F_M(H_C) = 0.56 \times 100 \div 128.0 = 0.44\ kN/mm^2$

$F_{RES} = \sqrt{(0.28 + 0.35)^2 + (0.11 + 0.44)^2} = 0.84\ kN/mm^2$

WELD STRENGTH (GRADE 43) = $215\ N/mm^2$

WELD SIZE REQ^D = $\frac{0.84 \times 10^3}{0.7 \times 215} = 5.6\ mm \longrightarrow$ 6 LEG F.W. (MIN^M)

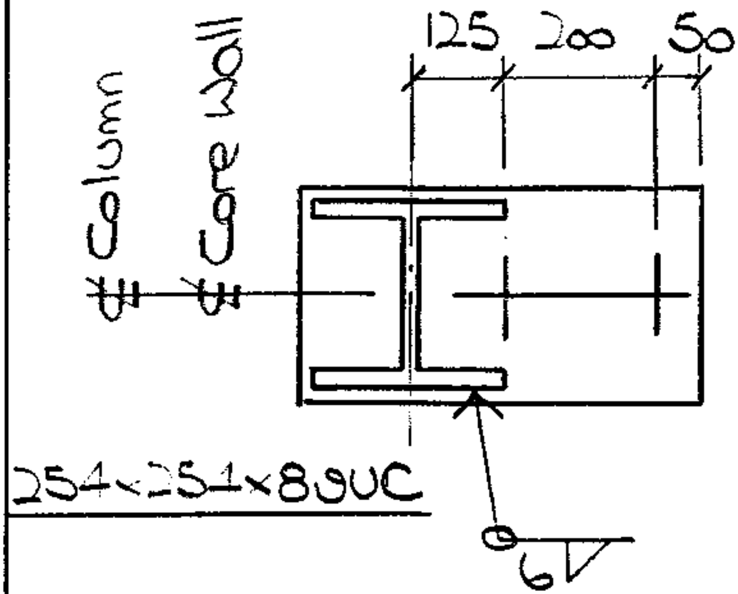
USE : — 8 LEG FILLET WELD (DETAIL STANDARDIZATION)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS
SUBJECT Connection Design

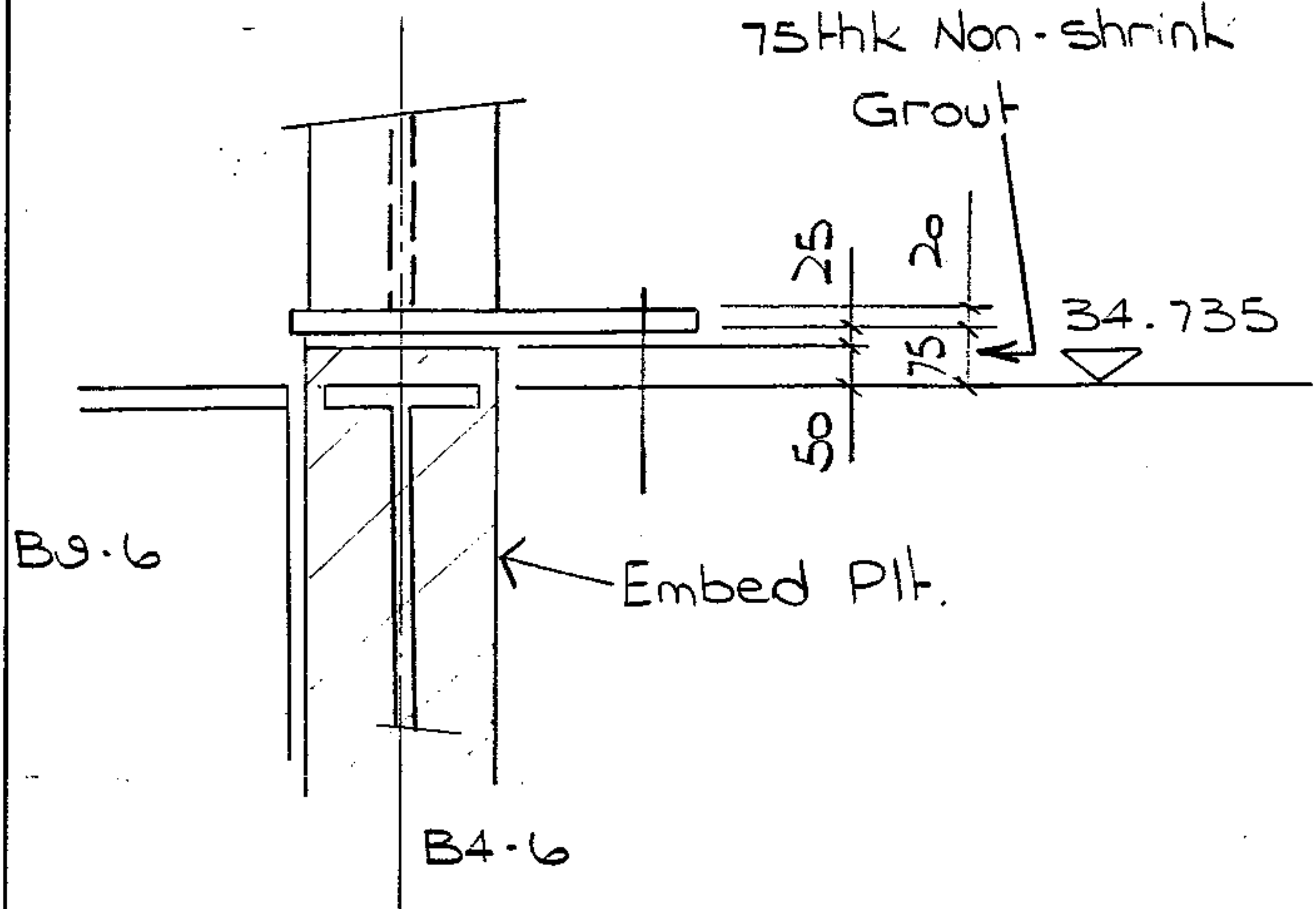
SERIES
SHT. No CW101 REV.
BY KM DATE Oct 98
EX. DATE
CN17

Column C8 to Core Wall - (Grid Line F5)



20thk Base Plt

2N° M20/30 Hilti HSL heavy-duty anchors



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES

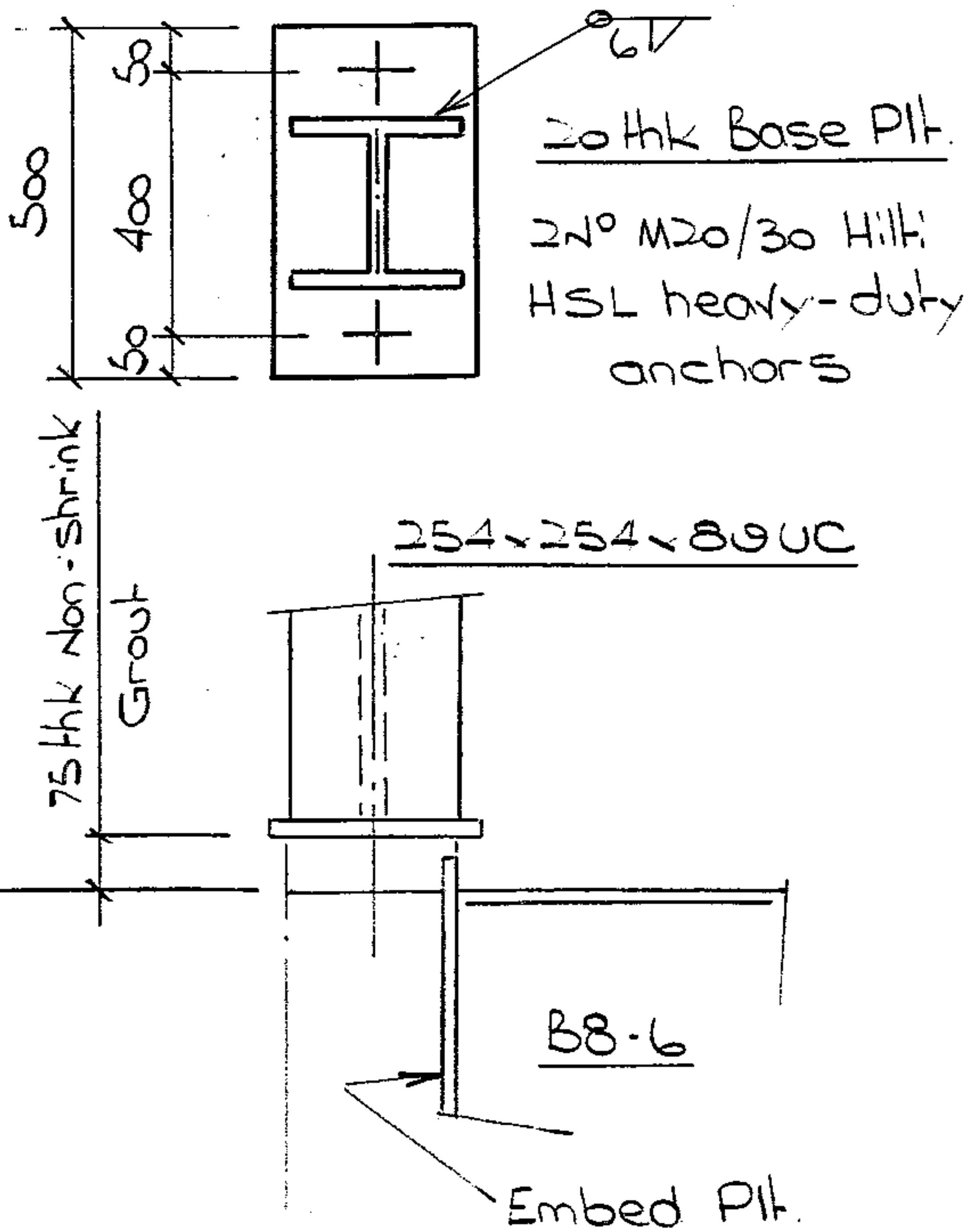
SHT. No. CW102 REV.

BY KM DATE Oct 98

EX. DATE

CN17

Column C8 to Core Wall - (Local to E8)



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection Design

SERIES

SHT. No. CW103 REV. A

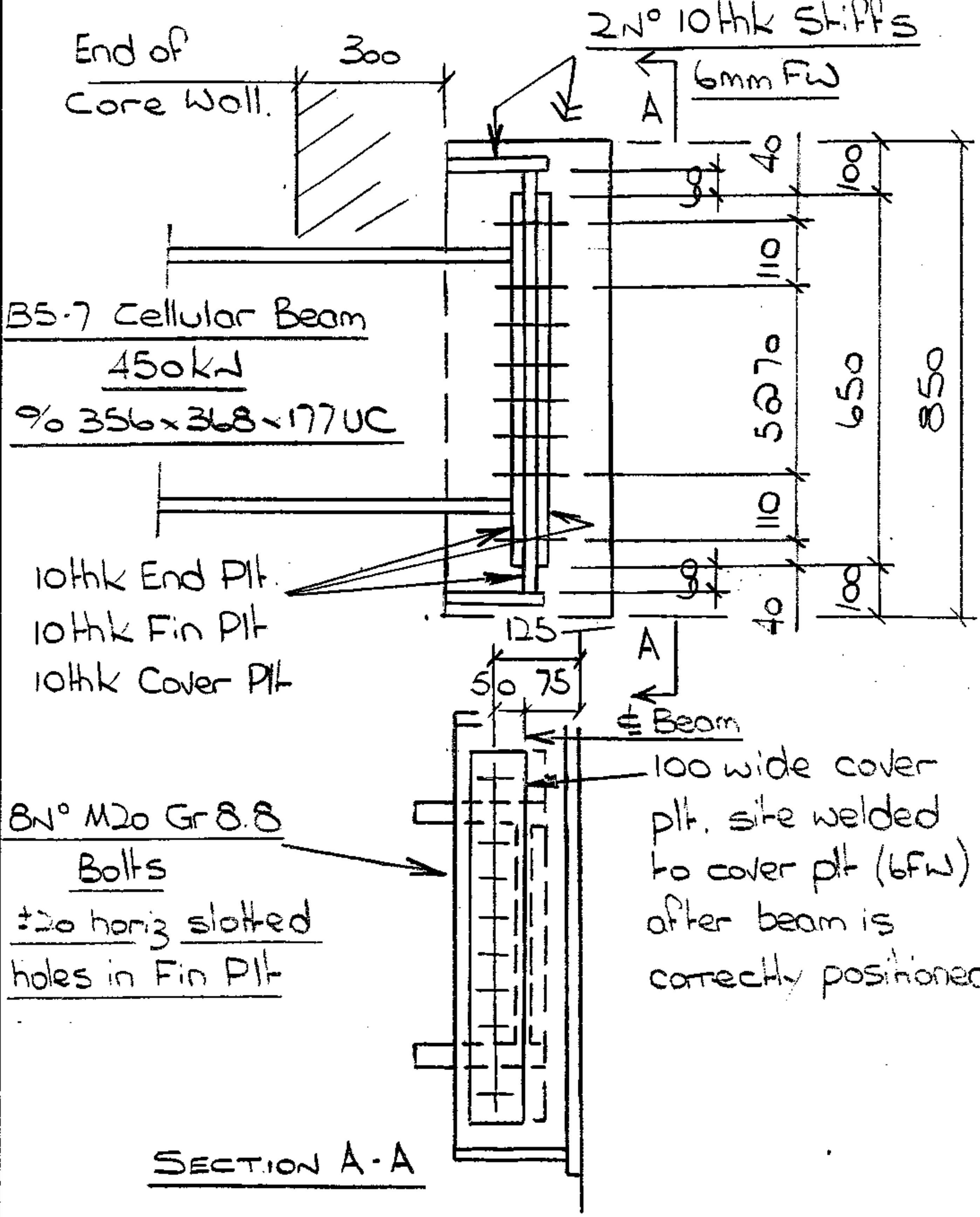
BY KM DATE Oct. 98

EX. DATE

CN17

Rev A 16.10.98 Slotted holes & cover plt added

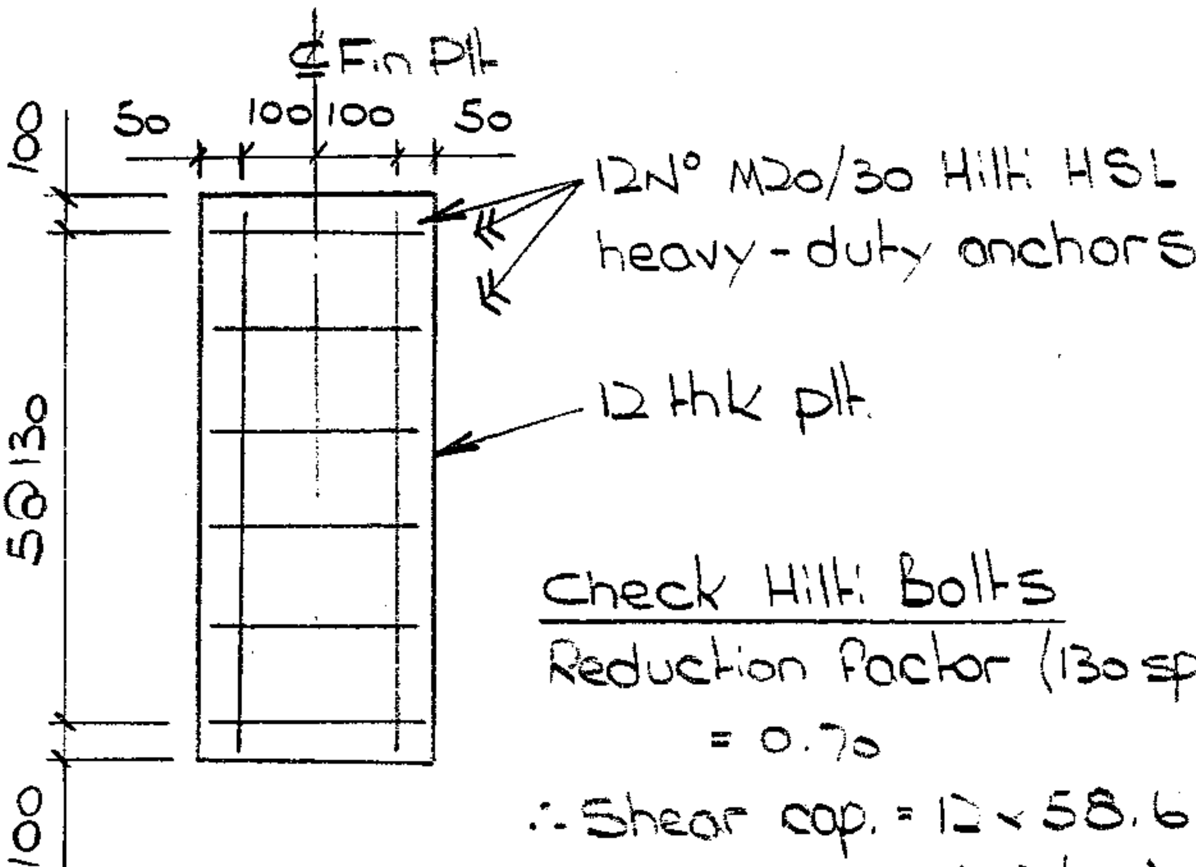
Conn of B5.7 to Core Wall (Local to D8)



SECTION A-A

B5-7 to Core Wall cont.

Conn of Backing Plt to Core Wall



Check Hilti Bolts

Reduction Factor (130 spacing)

= 0.70

∴ Shear cap. = 12 × 58.6 × 0.70

= 492 kN > 450 kN

∴ OK

Check Bolt Bearing

Load per bolt = 450/12 = 37.5 kN

Allowable bearing = $12 \times 20 \times 550 \times 10^{-3}$ = 132 kN > 37.5 kN

∴ OK

B5-7 to Core Wall cont.

Check Fin Conn.

i) Bolts $Z = \frac{2(35^2+105^2+175^2+285^2)}{285} = 871\,mm$

$f_v = 450/8 = 56\,kN$

$f_h = 450 \times 125/871 = 65\,kN$

$f_r = \sqrt{(56^2+65^2)} = \underline{86\,kN} < 92\,kN$

Allowable bearing on end plt / fin plt $= 20 \times 10 \times 550 \times 10^{-3}$

$= \underline{110\,kN} > 86\,kN$

$\therefore$ Bolts OK

ii) Fin Plt

a. Shear

$A_v = 0.9 \times 650 \times 10 = 5850\,mm^2$

$A_{v_{net}} = 5850 - (8 \times 22 \times 10) = 4090\,mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 5850 = \underline{1246\,kN} > 450\,kN$

$P_{v_{net}} = 0.5 \times 490 \times 10^{-3} \times 4090 = \underline{1002\,kN} > 450\,kN$

b. Bending

$Z = 10 \times 650^2/6 = 704\,167\,mm^3$

$M = 450 \times 125 \times 10^{-3} = \underline{56.3\,kNm}$

$P_b = 704\,167 \times 355 \times 10^{-6} = \underline{250\,kNm} > 56.3\,kNm$

$\therefore$ Fin Plt OK

B5·7 to Core Wall cont.

Fin Conn cont.

iii/ Fin Plt. Weld

$P_v = 2 \times 650 \times 4.2 \times 255 \times 10^{-3} = \underline{1392 kN} > 450 kN$

∴ Weld ok

iv/ End Plt to Beam Weld

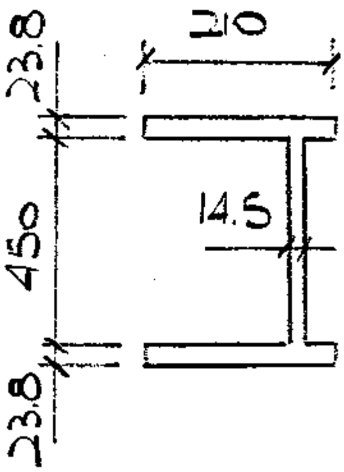
$d = 498 - 2 \times 23.8 = 450 mm$

$P_v = 2 \times 450 \times 4.2 \times 255 \times 10^{-3} = \underline{964 kN} > 450 kN$

∴ Weld ok

v/ Check beam for Notch

Length of Notch = 500 mm Max



$I = 14.5 \times 450^3/12 = 110\,109\,375$

$2 \times 210 \times 23.8^3/12 = 471\,845$

$2 \times 210 \times 23.8 \times 236.9^2 = 560\,991\,614$

$671\,572\,834 mm^4$

a. Shear

$A_v = 0.9 \times 498 \times 14.5 = 6499 mm^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 6499 = \underline{1384 kN} > 450 kN$

b. Bending

$M = 450 \times 500 \times 10^{-3} = \underline{225 kNm}$

$P_b = 671\,572\,834 / 248.8 \times 355 \times 10^{-6}$

$= \underline{958 kNm} > 225 kNm$

∴ Beam OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS SERIES

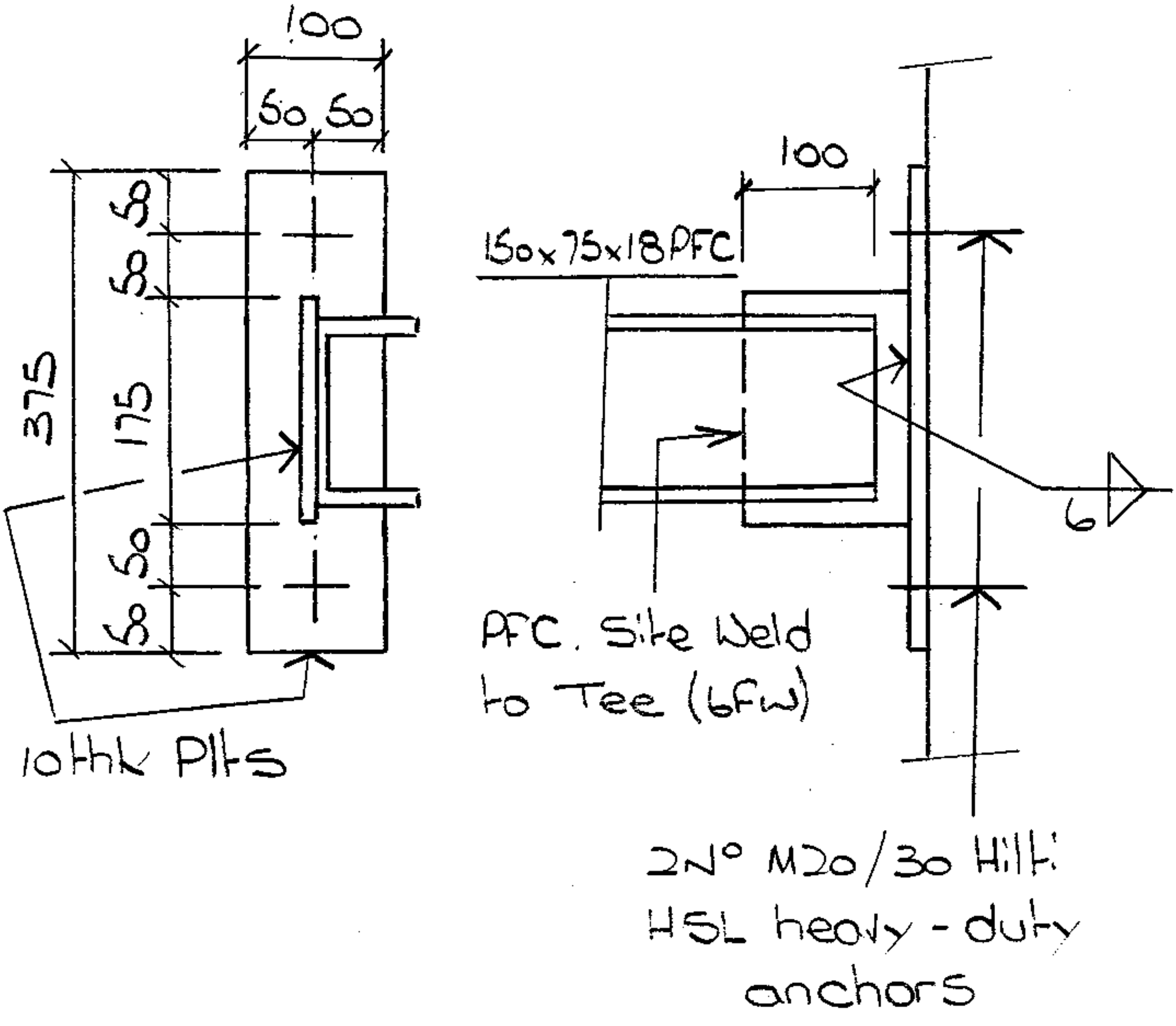
SHT. No CW107 REV.

SUBJECT Connection Design BY KM DATE Oct 98

EX. DATE

CN17

150 x 75 x 18 PFC to Core Wall



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connections to Core Wall

SERIES

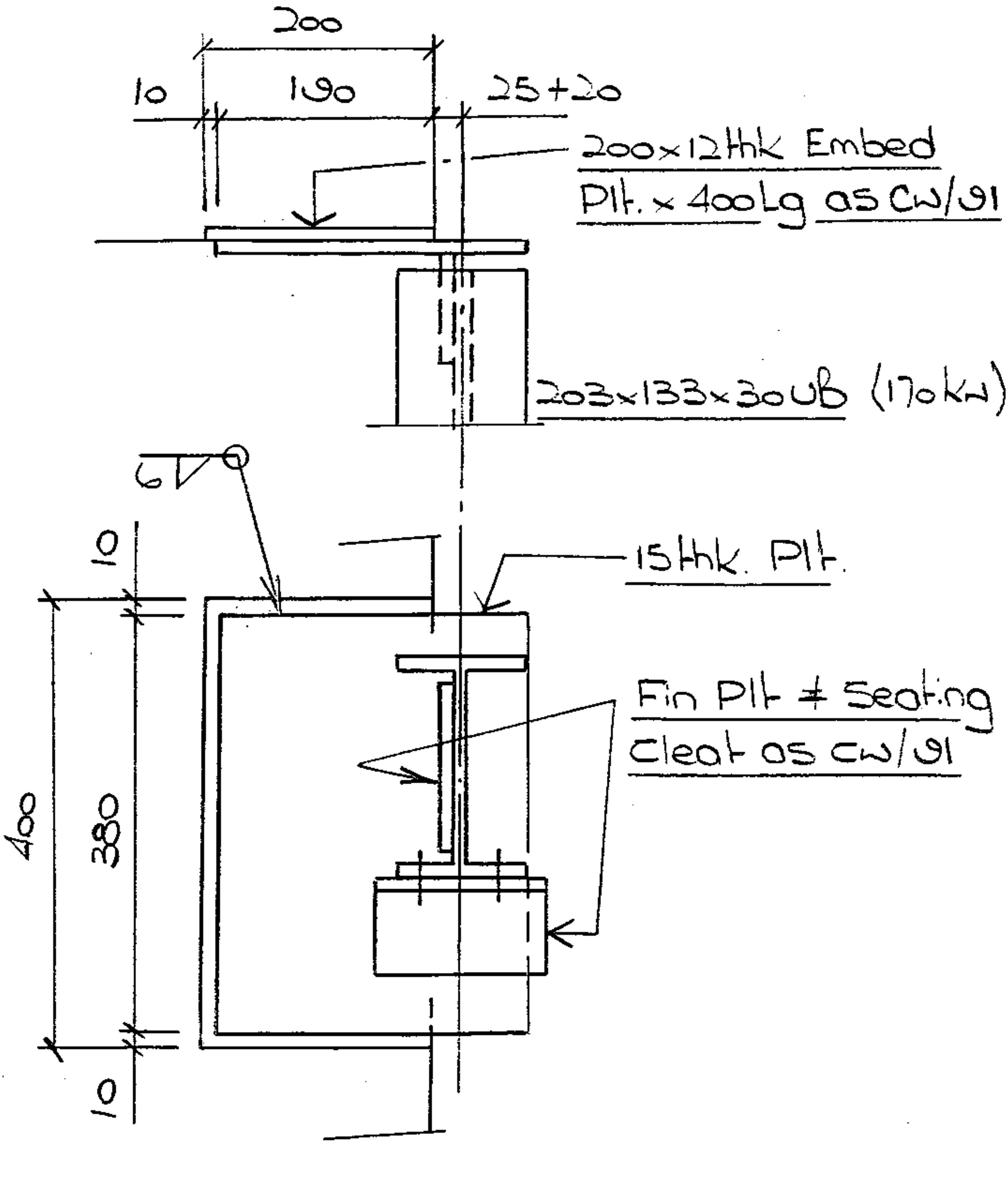
SHT. No. CW108 REV.

BY KM DATE Oct 98

EX. DATE

CN17

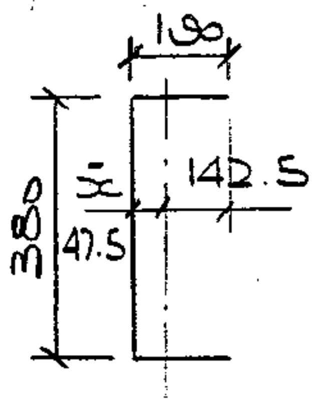
Beam B8·7 to Core Wall (Local to E5)



B8-7 to Core Wall

i) Check 12 thk plt to Embed Plt.

Assume 3 side weld



$\bar{x} = \dfrac{2 \times 190 \times 95}{(2 \times 190 + 380)} = 47.5\,mm$

F_V (vert. weld)

$= 170 \times 10^3 / 380 \times 4.2$

$= \underline{107\ N/mm^2} < 215\ N/mm^2$

F_H (2 Horiz. welds)

$M = (142.5 + 25 + 20) \times 170$

$= 31875\ kNmm$

Load per length of weld $= \dfrac{31875}{380} = \underline{84\ kN}$

$F_H = 84 \times 10^3 / 190 \times 4.2$

$= \underline{105\ N/mm^2} < 215\ N/mm^2$

$\therefore$ Weld ok

Kvaerner Cleveland Bridge Ltd.

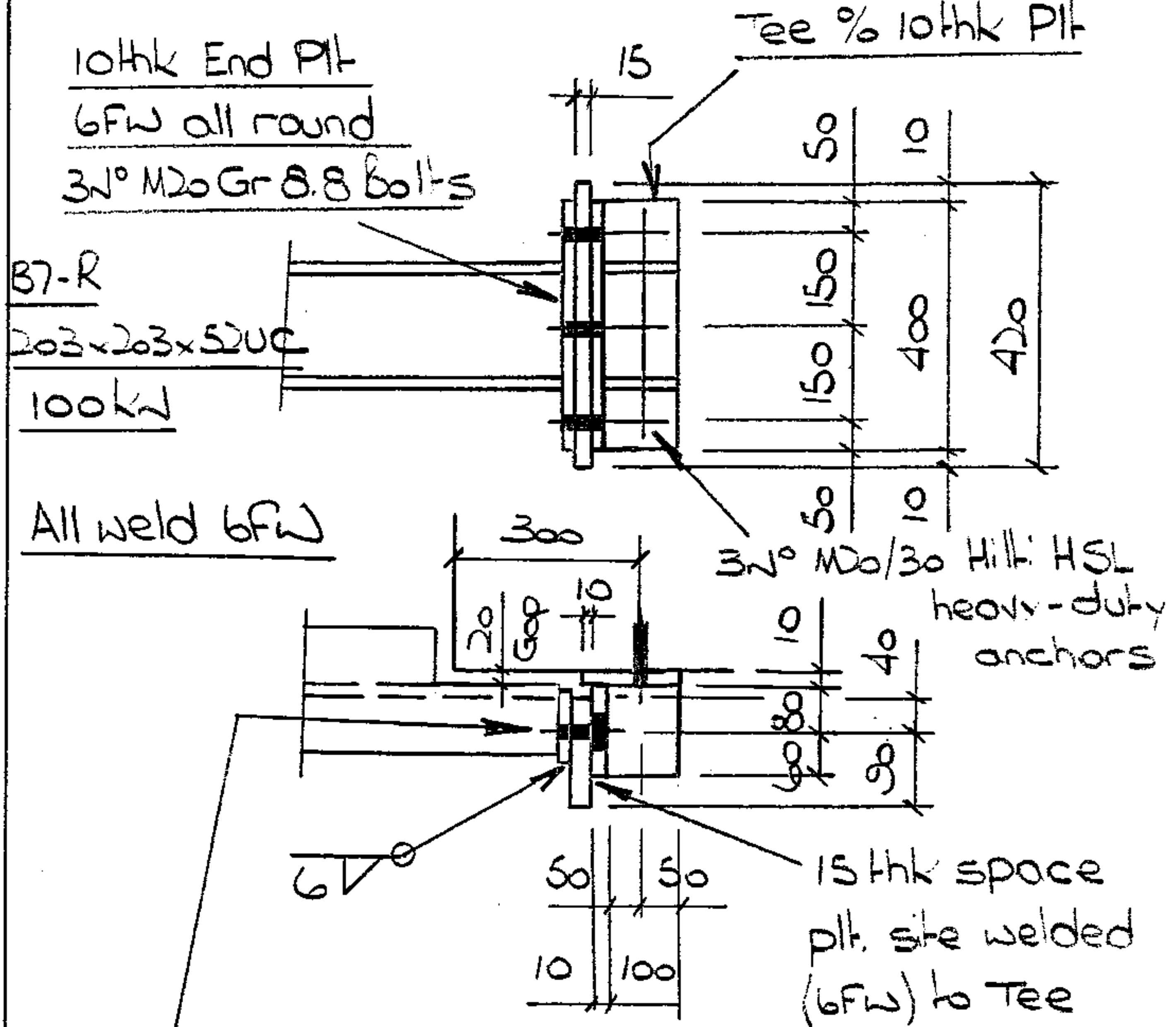
O/No. 325 JOB CARLTON GARDENS — SERIES .... SHT. No. CW110 REV. ....

SUBJECT Connection Design — BY KM DATE Oct 98

EX. .... DATE ....

CW17

Conn. of B7-R to Core Wall (Local to E5)



15 thk space plt. site welded (6FW) to Tee after beam is correctly positioned

22 dia hole in beam end plt & 15 thk space plt.

22 dia × ±20 horiz slotted hole in Tee plt.

B7-R to Core Wall

i) Bolts

$F_v = 100/3 = 33 \text{ kN}$

$F_h = \dfrac{100 \times 90}{300} = 30 \text{ kN}$

$F_r = \sqrt{(33^2 + 30^2)} = \underline{45 \text{ kN}} < 92 \text{ kN}$

Allowable bearing on fin plt. $= 20 \times 10 \times 550 \times 10^{-3}$

$= \underline{110 \text{ kN}} > 45 \text{ kN}$

$\therefore$ Bolts OK

ii) Fin Plt.

a. Shear

$A_v = 0.9 \times 400 \times 10 = 3600 \text{ mm}^2$

$Av_{net} = 3600 - (3 \times 22 \times 10) = 2940 \text{ mm}^2$

$P_v = 0.6 \times 355 \times 10^{-3} \times 3600 = \underline{767 \text{ kN}} > 100 \text{ kN}$

$P_{V_{net}} = 0.5 \times 490 \times 10^{-3} \times 2940 = \underline{720 \text{ kN}} > 100 \text{ kN}$

b. Bending

$Z = 10 \times 400^2/6 = 266\,667 \text{ mm}^3$

$M = 100 \times 90 \times 10^{-3} = \underline{9 \text{ kNm}}$

$P_b = 266\,667 \times 355 \times 10^{-6} = \underline{95 \text{ kNm}} > 9 \text{ kNm}$

$\therefore$ Fin Plt OK

B7-R to Core Wall

iii/ Hilti Bolts

$F_v = 100/3 = 33kN$

$F_h = \frac{100 \times 55}{300} = 18kN$

$F_r = \sqrt{(33^2 + 18^2)} = \underline{38kN}$

Allowable shear $= 0.91 \times 58.6 = \underline{53kN} > 38kN$

$\therefore$ Hilti Bolts OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

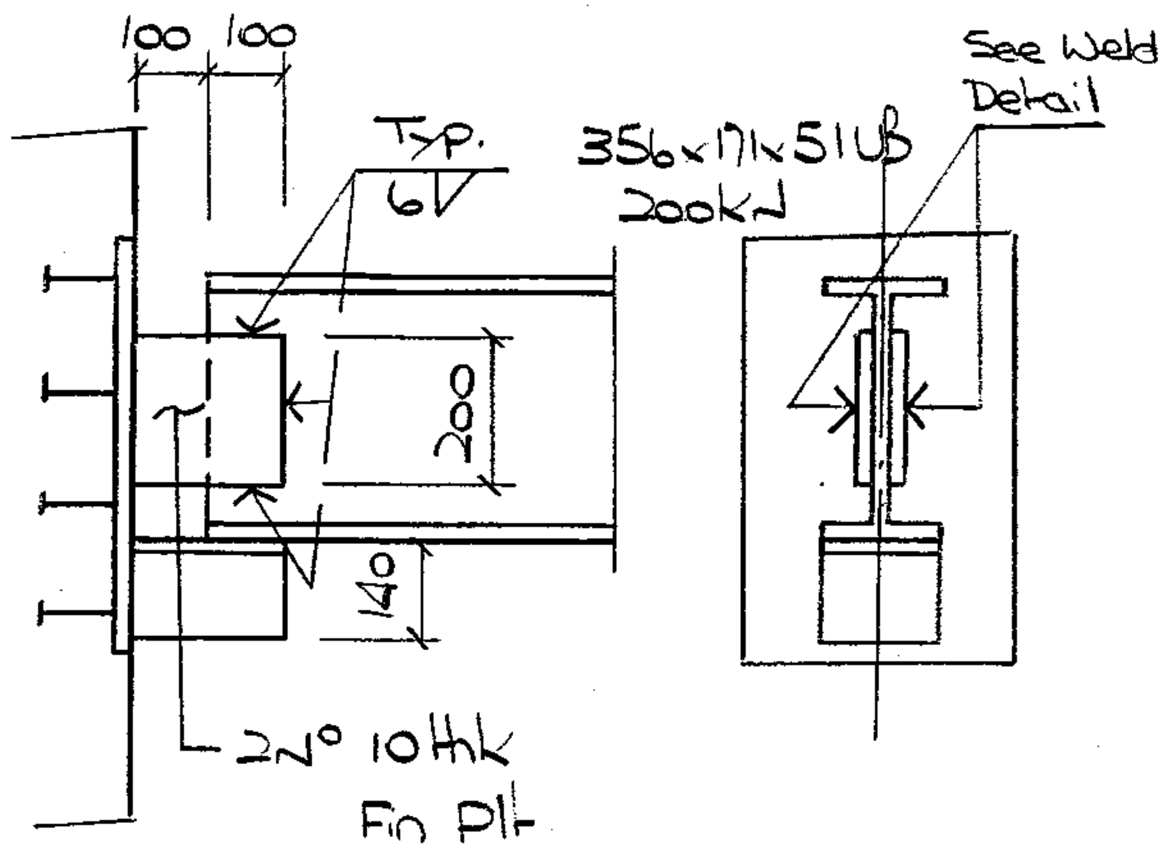
SUBJECT Connection to Core Wall

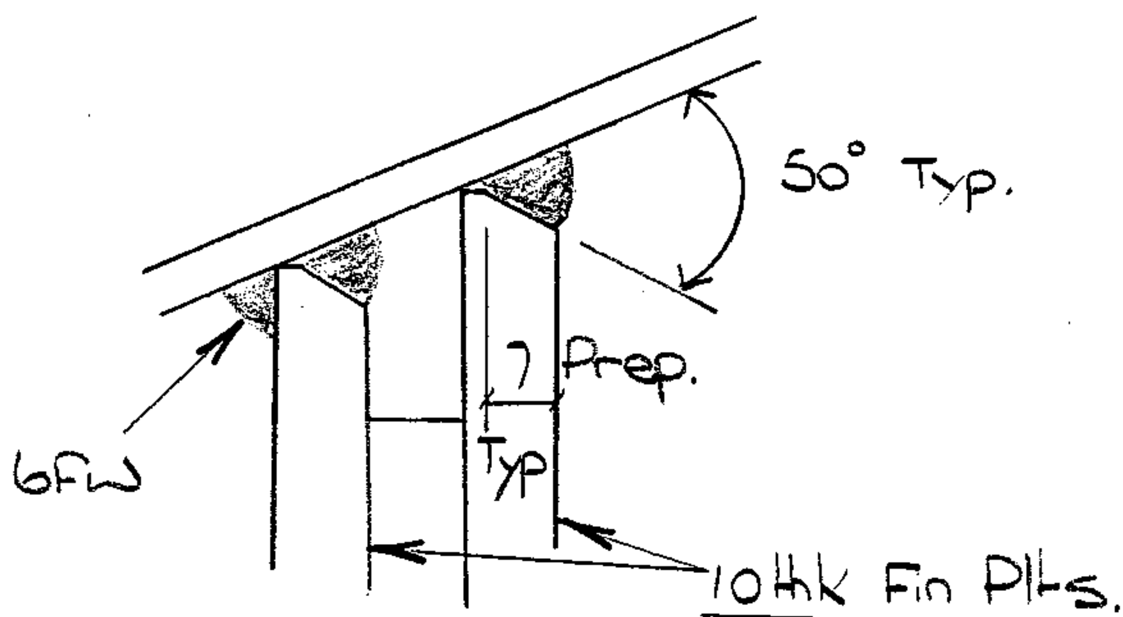
SERIES

SHT. No. CW113 REV.

BY KM DATE Nov 98

EX. DATE





Fin Plt. Weld Detail

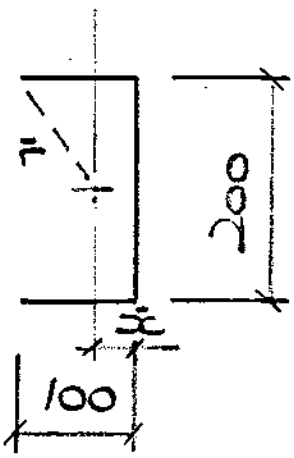
Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS | SERIES ...... | SHT. No. CW114 REV. ......

SUBJECT Connection to Core Wall | BY KM DATE Nov 98

EX. ...... DATE ......

Design of Weld for Fin Plt. to Beam



Consider Weld of Unit Leg Length

$Area = 2(200 + 2\times100) = 800 mm^2$

$\bar{x} = \dfrac{2\times100\times50}{400} = 25 mm$

$r = \sqrt{(75^2 + 100^2)} = 125 mm$

$I_{xx} = \left(\dfrac{2\times200^3}{12}\right) + (4\times100\times100^2) = 5\,333\,333 mm^4$

$I_{yy} = \left(\dfrac{4\times100^3}{12}\right) + (2\times200\times25^2) + (4\times100\times25^2) = 833\,333 mm^4$

$I_p = 5\,333\,333 + 833\,333 = 6\,166\,666 mm^4$

$Z_p = 6\,166\,666/125 = \underline{49\,333 mm^3}$

$f_v = 200/800 = 0.25 kN/mm^2$

$f_h = 75/800 = 0.09 kN/mm^2$

$f_m = 200\times175/49\,333 = 0.71 kN/mm^2$

$f_m(V_c) = 0.71\times25/125 = 0.14 kN/mm^2$

$f_m(H_c) = 0.71\times100/125 = 0.57 kN/mm^2$

$f_{res} = \sqrt{((0.25+0.14)^2 + (0.09+0.57)^2)} = \underline{0.77 kN/mm^2}$

Weld strength $= 215 N/mm^2$

Weld size req^d $= \dfrac{0.77\times10^3}{0.7\times215} = \underline{5.1 mm}$

Use 6mm FW

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES ...... — SHT. No. CW115 REV. ......

SUBJECT Connection to Core Wall — BY KM DATE Nov 98

EX. ...... DATE ......

Check Fin Plts

a) Shear

Shear Force = 200 kN

$P_y = 275\ N/mm^2$

$A_v = 2 \times 0.9 \times 10 \times 200 = 3600\ mm^2$

$P_v = 0.6 \times 3600 \times 275 \times 10^{-3}$

$= \underline{594\ kN} > 200\ kN$

b) Bending

Bending Moment = $200 \times 175 = 35\,000$ kNmm

$Z = 2 \times 10 \times 200^2 / 6 = 133\,333\ mm^3$

$M_{cx} = 1.2 \times P_y \times Z$

$= 1.2 \times 275 \times 133\,333 \times 10^{-3} = \underline{44000\ kNmm}$

> 35000 kNm

For a Plt. $S_x = S_v$

$S_x = 2 \times 10 \times 200^2 / 4 = 200\,000\ mm^3$

$$M_{cx} = 275 \times 200\,000 \times \left[1 - \left(\frac{2.5 \times 200}{594} - 1.5\right)\right] \times 10^{-3}$$

$= \underline{91204\ kNmm} > 35\,000$ kNmm

$\therefore$ Plts OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

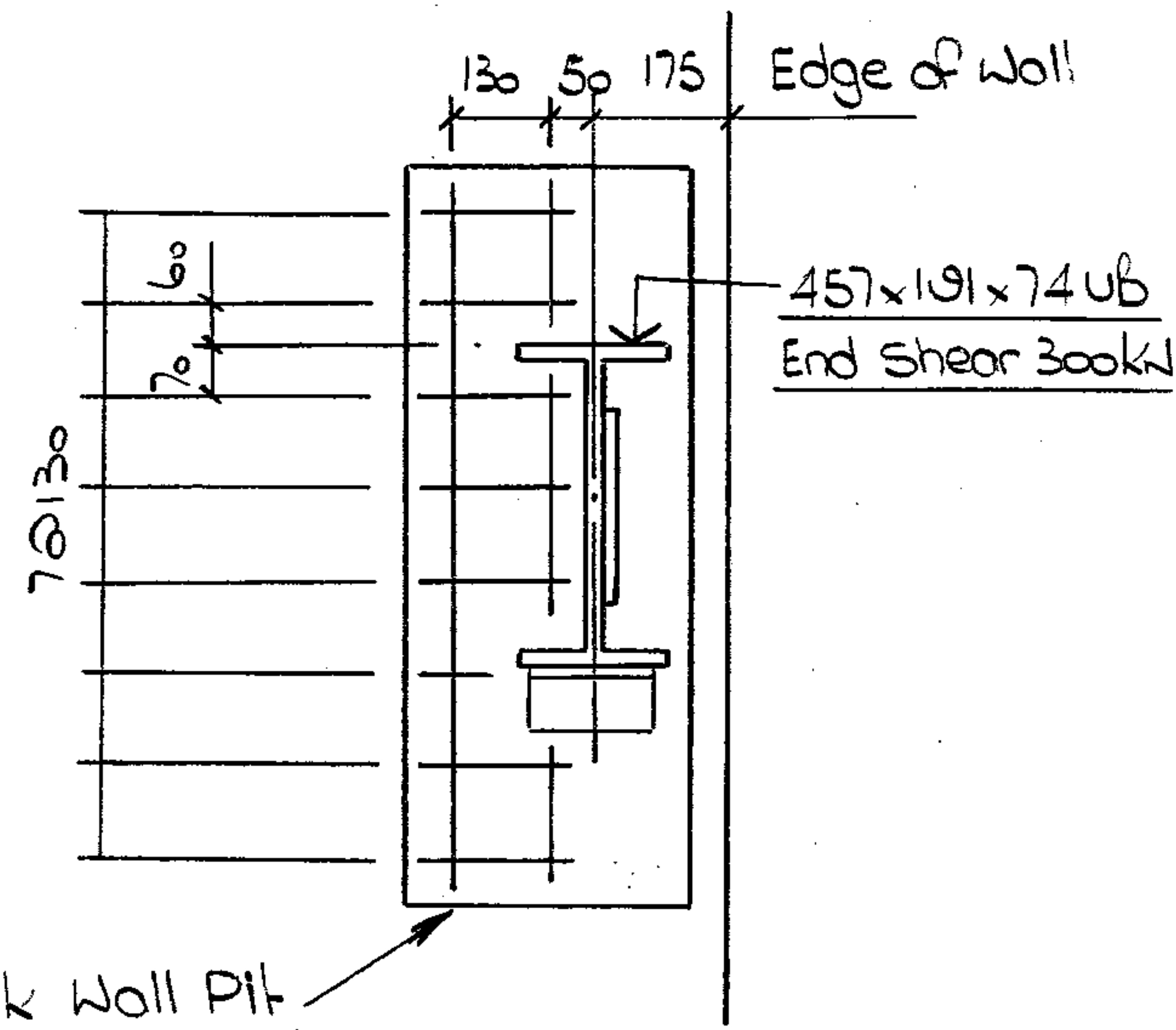
SUBJECT CONN. TO CONCRETE CORE WALL

SERIES

SHT. No. CW116 REV.

BY KM DATE NOV 98

EX. DATE



15thk Wall Plt

15 N° M20/30 Hilti HSL

heavy-duty anchors

Fin Plt & Seating Angle as CW/42

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONN. TO CONCRETE CORE WALL

SERIES

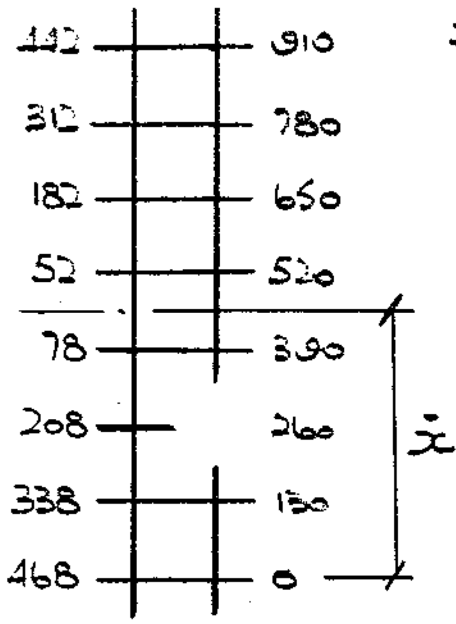
SHT. No. CW117 REV.

BY KM DATE Nov 98

EX. DATE

Check Anchors

$$\text{Capacity} = 58.6 \times 0.64 \times 0.7 = \underline{26.3\,\text{kN}}$$



$$\bar{x} = \frac{2(130+390+520+650+780+910)+260}{15}$$

$$= \underline{468\,\text{mm}}$$

$$Z = \frac{2(468^2+338^2+78^2+52^2+182^2+312^2+442^2)+208^2}{468}$$

$$= 2947\,\text{mm}$$

$$F_v = 300/_{15} = 20\,\text{kN}$$

$$F_h = \frac{300 \times 115}{2947} = 11.7\,\text{kN}$$

$$F_r = \sqrt{(20^2 + 11.7^2)}$$

$$= \underline{23\,\text{kN}} > 26.3\,\text{kN}$$

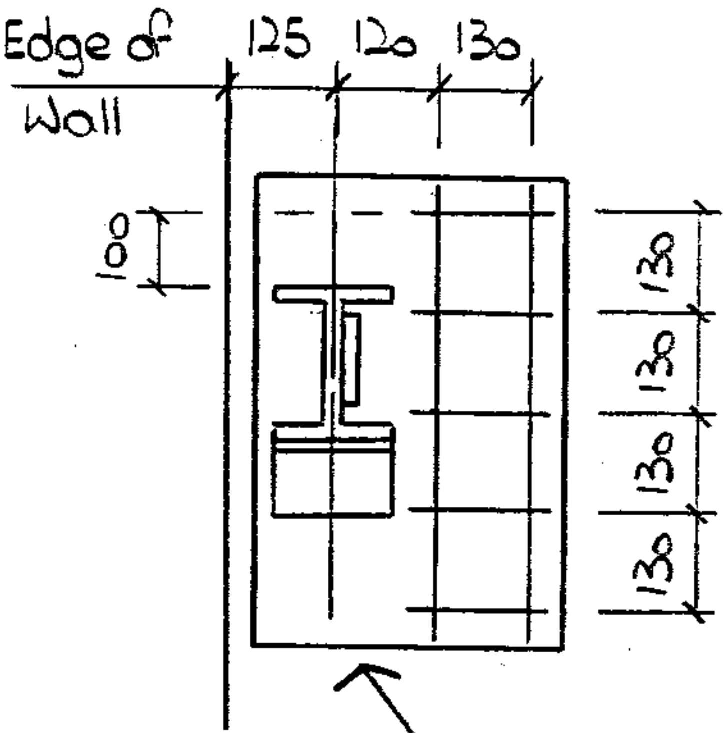
$\therefore$ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES — SHT. No. CW118 REV.

SUBJECT CONN TO CONCRETE CORE WALL — BY KM DATE NOV 98

EX. DATE



203x133x30 UB

End Shear = 150kN

15thk Wall Plt

10N° M20/30 Hilti HSL

heavy-duty anchors

Fin Plt & Seating Angle as CW/91

Check Anchors

$$\text{Capacity} = 58.6 \times 0.875 \times 0.7 = \underline{36kN}$$

$$Z = \frac{4(130^2 + 260^2)}{260} = 1300mm$$

$$F_v = 150/10 = 15kN$$

$$F_h = \frac{150 \times 185}{1300} = 21kN$$

$$F_r = \sqrt{(15^2 + 21^2)} = \underline{26kN} > 36kN$$

$\therefore$ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

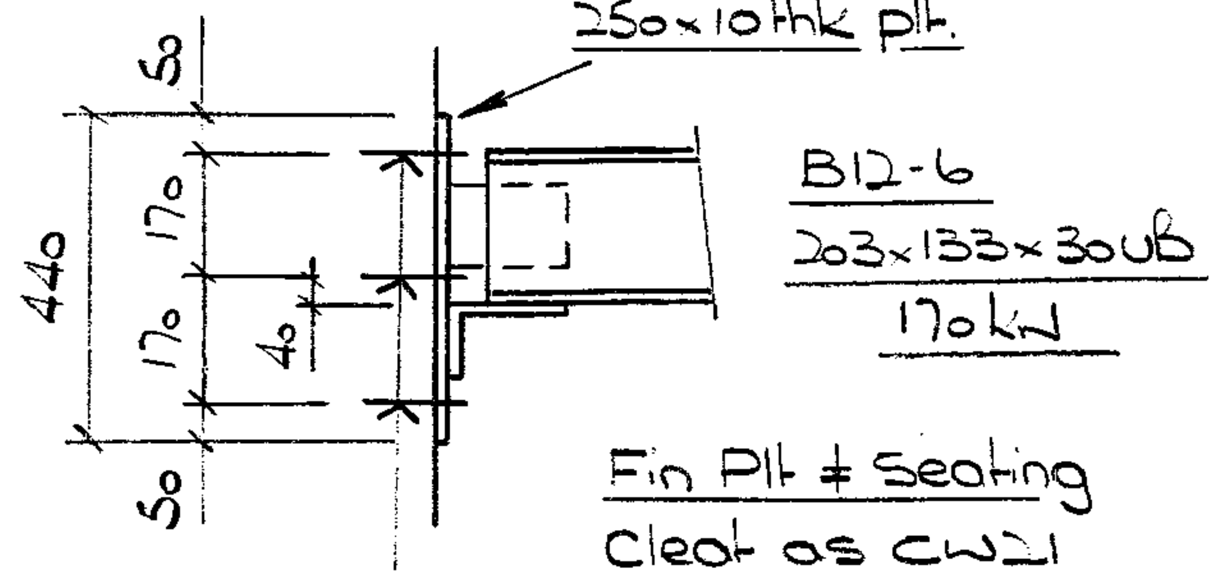
SERIES

SHT. No. CW119 REV.

BY KM DATE Nov 98

EX. DATE

Beam B12-6 to Core Wall



6N° M20/30 Hilti HSL Heavy Duty Anchors @150 c/c

Check Anchors

Vert spacing = 170mm

Reduction Factor = 0.74

∴ Anchor shear capacity = 58.6 × 0.74 = 43kN

N° of anchors req^{d} = 170/43 = 3.95

∴ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

SERIES

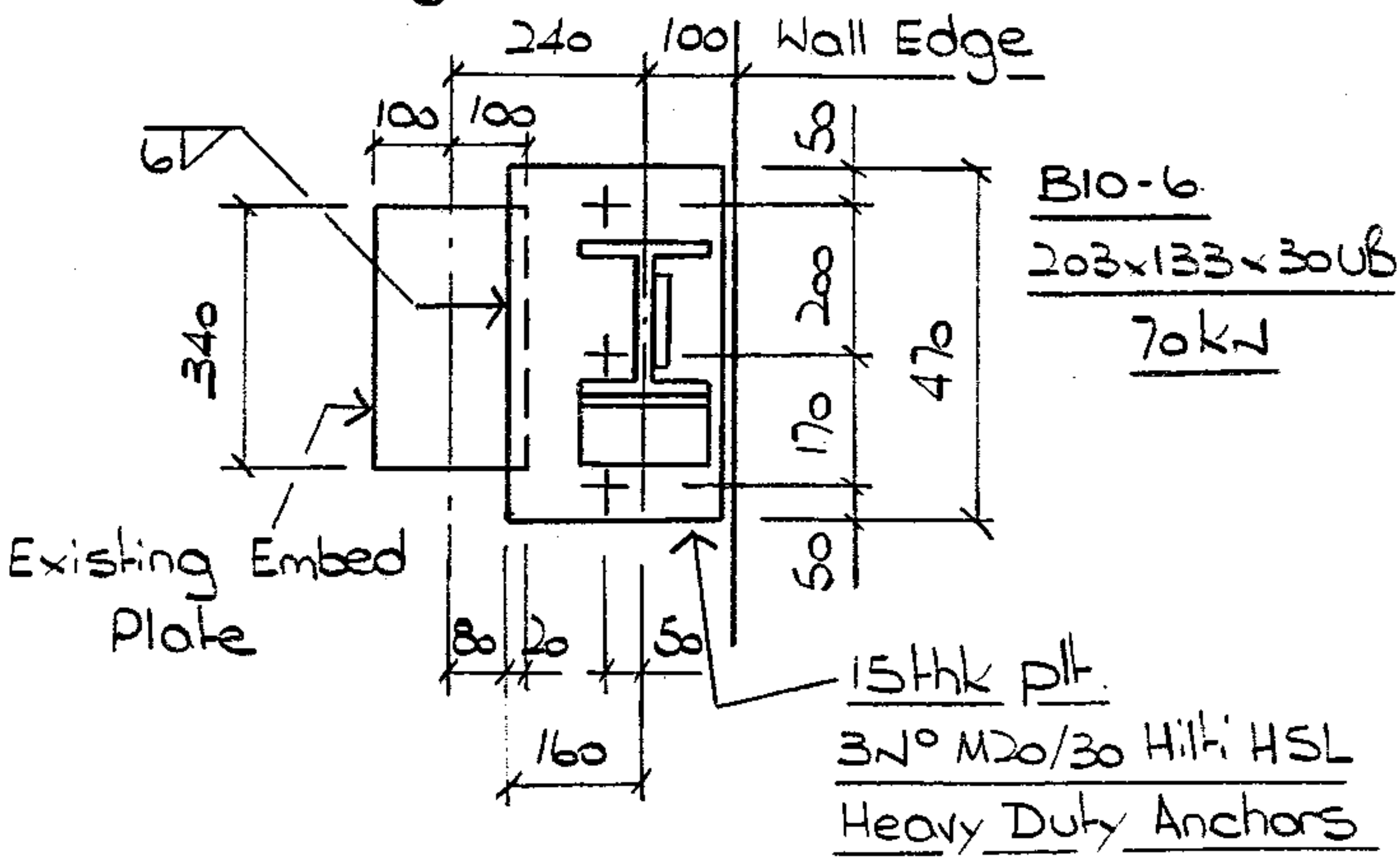
SHT. No. CW120 REV.

BY KM DATE Nov 98

EX. DATE

Beam B10-6 to Core Wall

Wall Plt. Design



Check Weld

$$Z = \frac{4.2 \times 320^2}{6} = 71\,680\ mm^3$$

$$F_v = 70 \times 10^3 / 320 \times 4.2 = 52\ N/mm^2$$

$$F_h = \frac{160 \times 70 \times 10^3}{71\,680} = 156\ N/mm^2$$

$$F_r = \sqrt{(52^2 + 156^2)} = \underline{164\ N/mm^2} < 215\ N/mm^2$$

$\therefore$ Weld OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

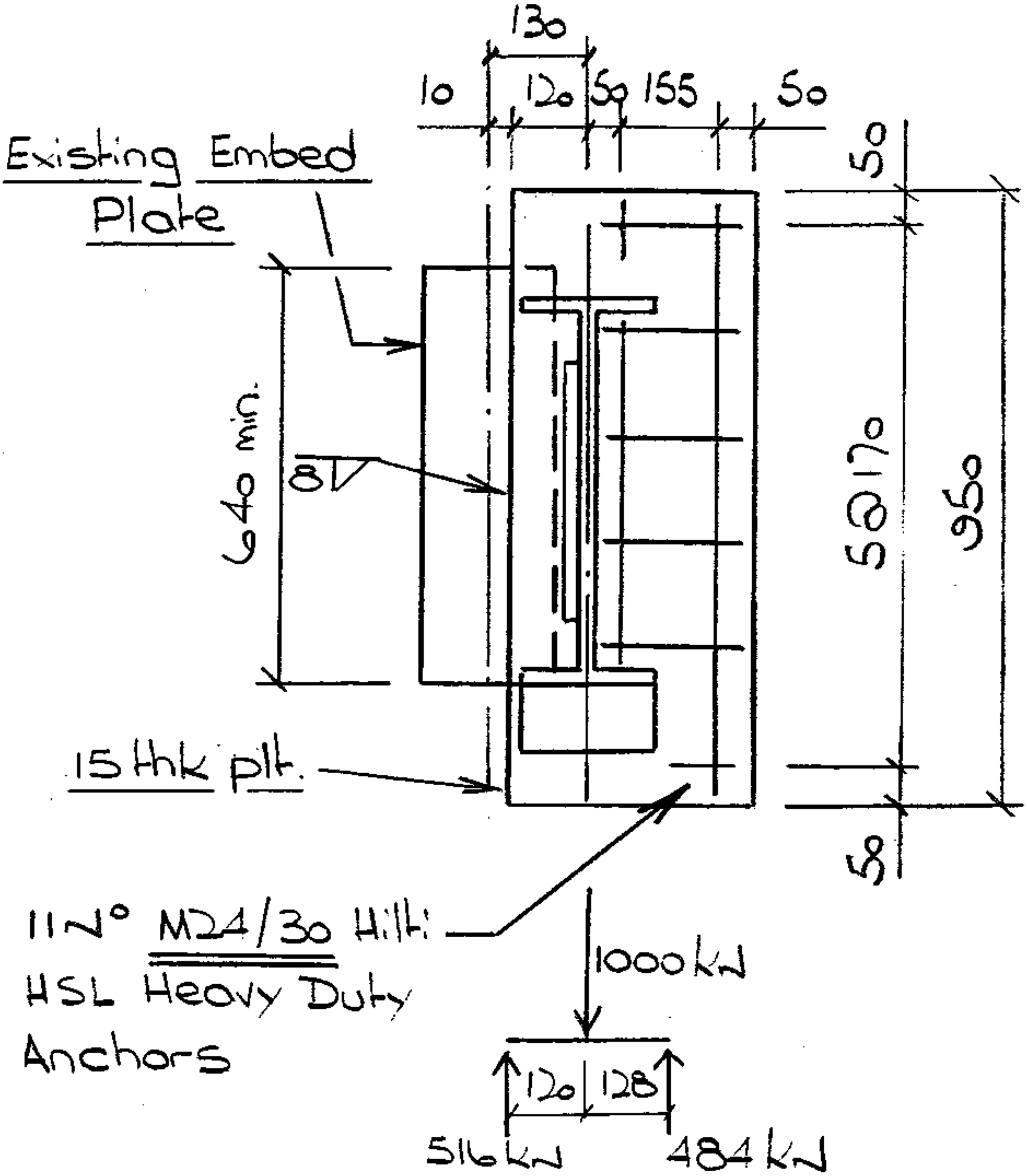
SERIES

SHT. No. CW121 REV.

BY KM DATE Nov 98

EX. DATE

Beam B8-6 to Core Wall



B8-6 - 610 x 229 x 140 UB - 1000 kN

Kvaerner Cleveland Bridge Ltd.

O/No 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

SERIES

SHT. No CW122 REV.

BY KM DATE Nov 98

EX. DATE

B8-6 to Core Wall cont.

Check Weld · 8FW

$$F_v = \frac{516 \times 10^3}{620 \times 5.6} = \underline{149\ N/mm^2} < 215\ N/mm^2$$

$\therefore$ Weld OK

Check Anchor Bolts

Vert spacing = 170 mm

Reduction Factor = 0.71

$\therefore$ Anchor shear capacity = $76.6 \times 0.71 = \underline{54\ kN}$

N° of anchor reqd = $^{484}/_{54} = \underline{9}$

$\therefore$ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

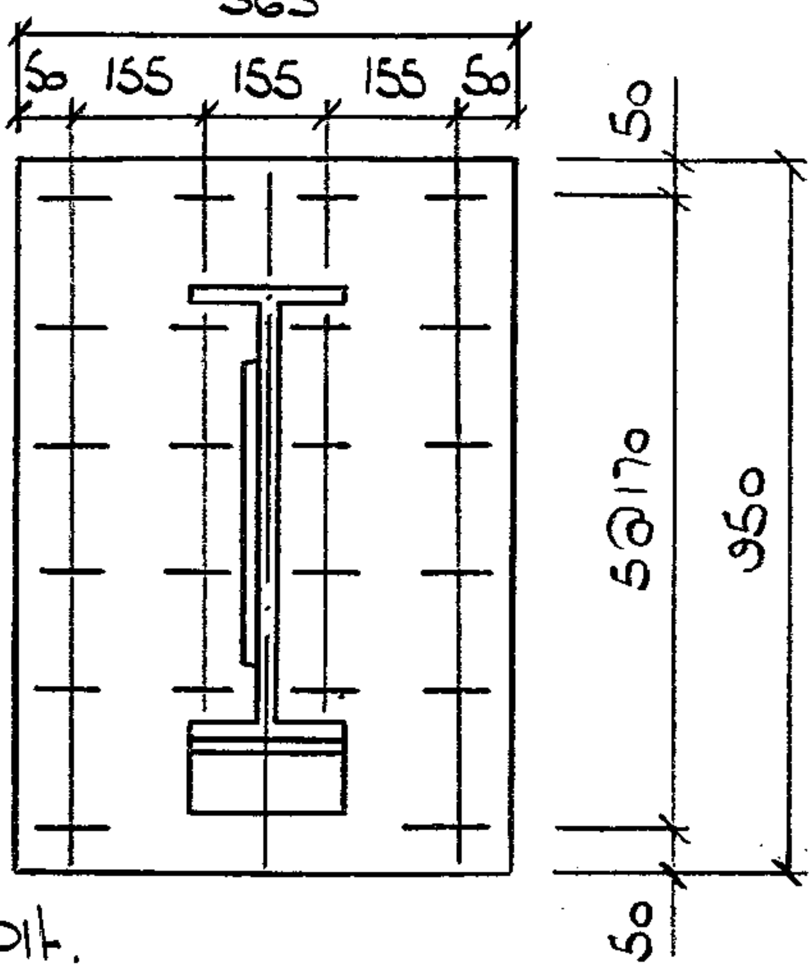
SERIES

SHT. No. CW123 REV.

BY KM DATE Nov 98

EX. DATE

Beam B8·6 to Core Wall



15thk Plt.

22N° M24/30 Hilti HSL
Heavy Duty Anchors

B8·6 - 1000kN
610×229×140UB

Check Anchor Bolts

Anchor Shear capacity = 54kN (As CW122)

No of anchors req^d = $^{1000}/_{54}$ = 18.5

∴ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

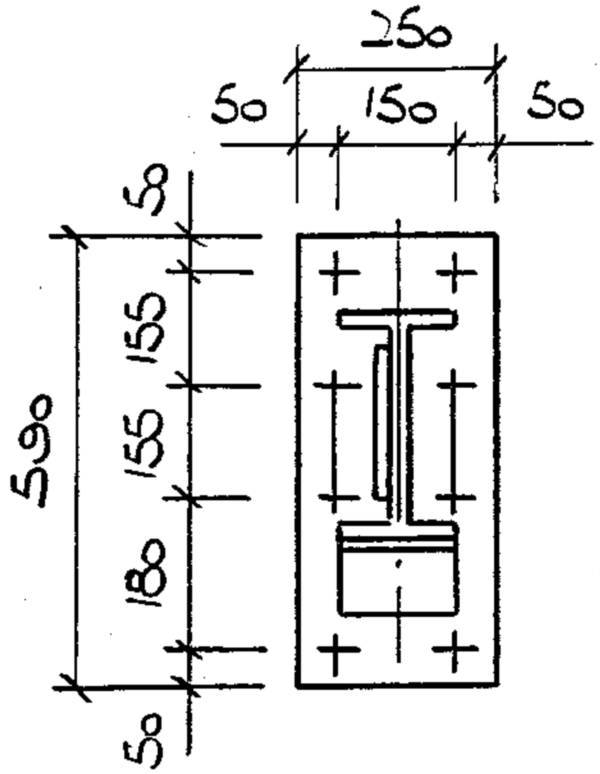
SERIES

SHT. No CW124 REV.

BY KM DATE Nov 98

EX. DATE

Beam B4-6 to Core Wall



B4-6 - 200kN

305×102×33UB

15thk Plt.

8N° M20/30 Hilti HSL Heavy Duty Anchors

Check Anchors

Vert. spacing = 155mm

Reduction factor = 0.72

∴ Anchor shear capacity = 58.6 × 0.72 = 42kN

N° of Anchors reqd = 200/42 = 4.8

∴ Anchors OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Conn to Concrete Core Wall

SERIES

SHT. No. CW125 REV.

BY KM DATE Nov 98

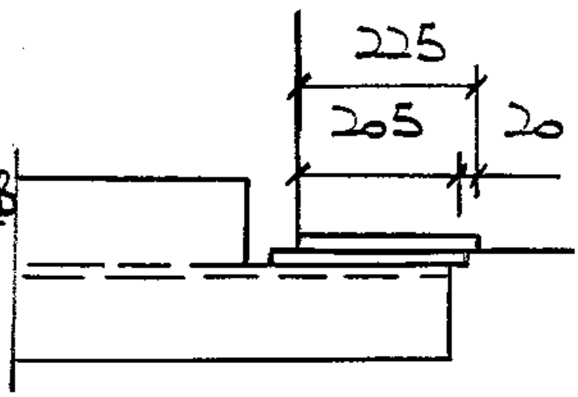
EX. DATE

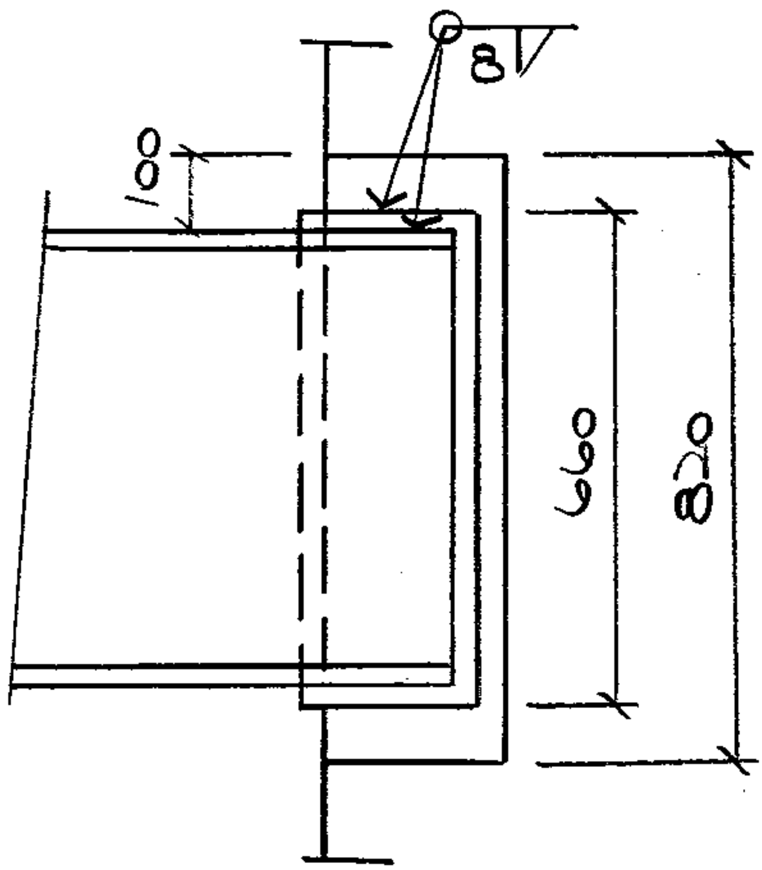
Beam B8·6 to Core Wall Local to 5E

610×229×140UB

550 + (10×0.2)

552kN





Check Weld

$P_v = 640 \times 5.6 \times 215 \times 10^{-3} = \underline{770 kN} > 552 kN$

∴ Weld OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Conn to Concrete Core Wall

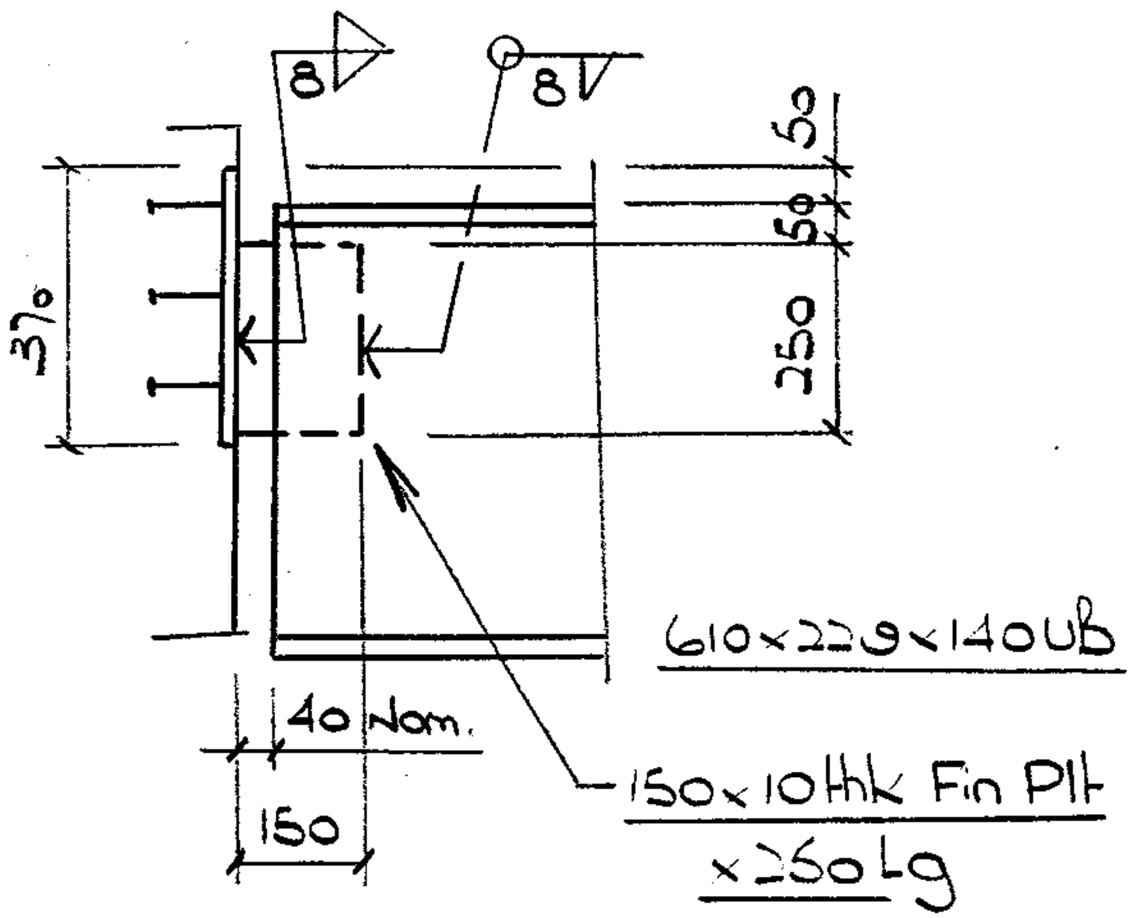
Beam B8-6 local to E5

SERIES

SHT. No. CW126 REV.

BY KM DATE Dec 98

EX. DATE



Fin Plt Similar to CW/29

Shear Load = 200kN
Axial Load = 75kN

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

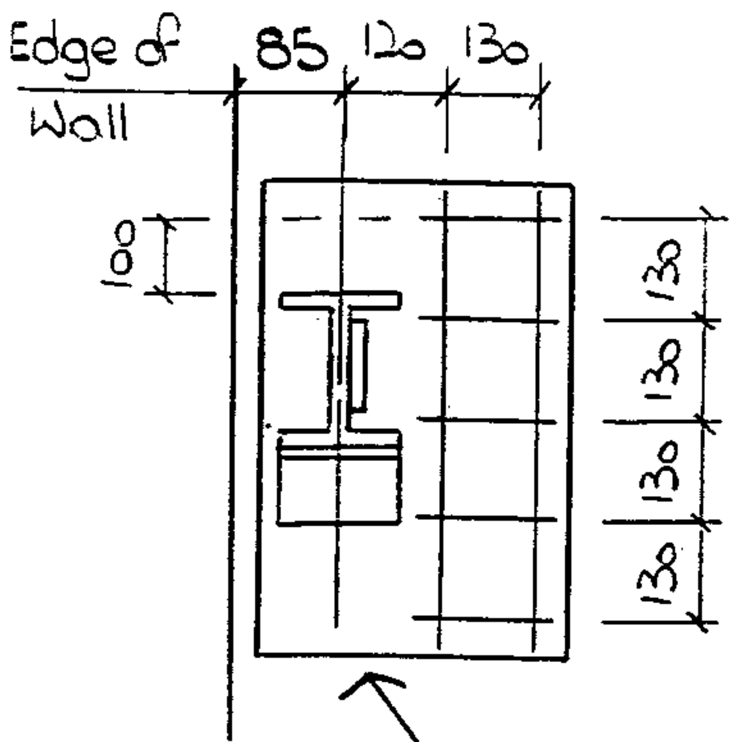
SUBJECT CONN TO CONCRETE CORE WALL

SERIES

SHT. No. CW127 REV.

BY KM DATE Nov 98

EX. DATE



203×133×30 UB

End Shear = 100 kN

15thk Wall Plt.

10N° M20/30 Hilti HSL

heavy-duty anchors

Fin Plt & Seating Angle as CW/91

Check Anchors

Capacity = $58.6 \times 0.55 \times 0.7 = \underline{23\ kN}$

$Z = \dfrac{4(130^2 + 260^2)}{260} = 1300mm$

$F_v = 100/10 = 10\ kN$

$F_h = \dfrac{100 \times 185}{1300} = 14\ kN$

$F_r = \sqrt{(10^2 + 14^2)} = \underline{17\ kN} > 23\ kN$

$\therefore$ Anchors ok

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT CONN TO CONCRETE CORE WALL

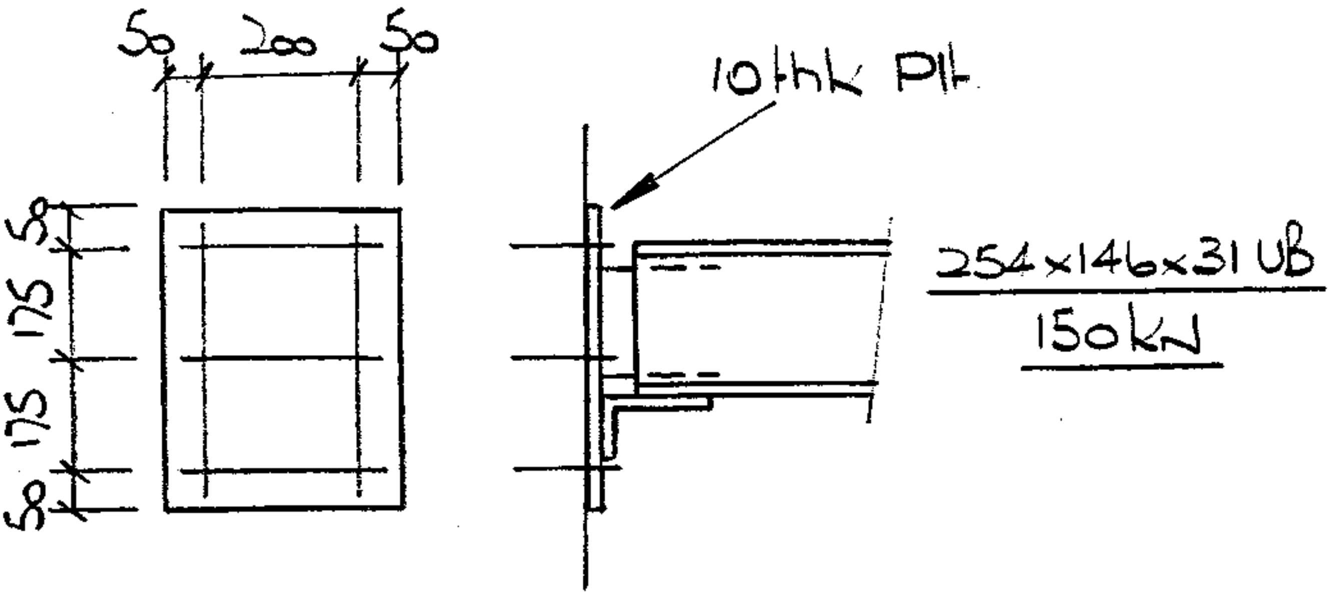
SERIES

SHT. No. CW128 REV.

BY KM DATE Dec 98

EX. DATE

Beam B1-6 Wall Plt



Anchor capacity $58.6 \times 0.75 \times 0.77 = 34\,kN$

N° of Anchor reqd $= 150/34 = \underline{4.4}$

$\therefore$ Anchors OK

Use 6N° M20/30 Hilti HSL Heavy Duty Anchors

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connections to Core Wall

SERIES

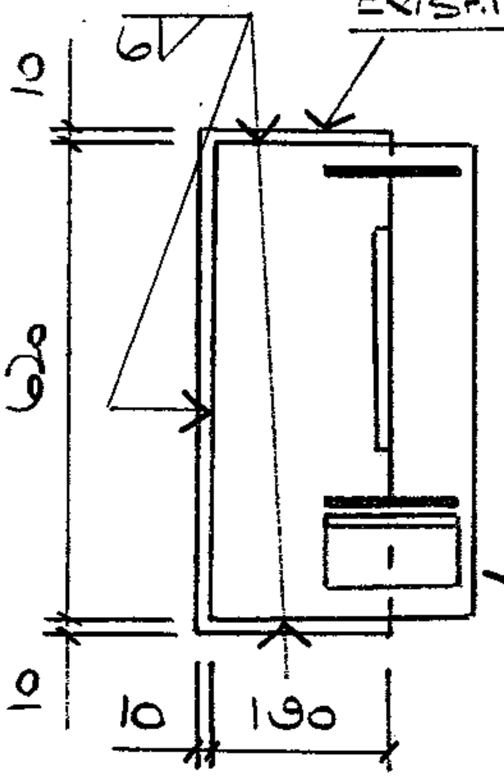
SHT. No. CW29 REV.

BY KM DATE Dec 9

EX. DATE

Beam B3-R to Core Wall

Wall Plt.

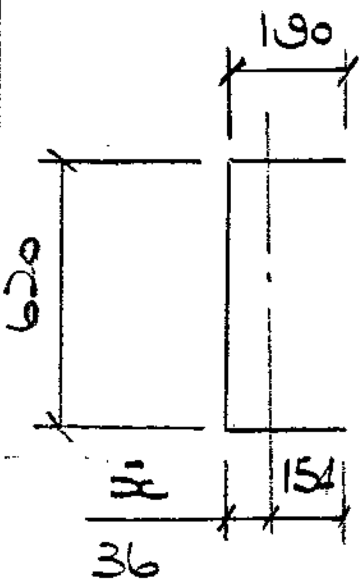


B3-R Cellular Beam

D = 479

End Shear = 200kN

Check Weld



$$\bar{x} = \frac{2 \times 190 \times 95}{(2 \times 190 + 620)} = 36$$

F_v (vert weld)

$$= 200 \times 10^3 / 620 \times 4.2$$

$$= \underline{77 \text{ N/mm}^2} < 215 \text{ N/mm}^2$$

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connections to Core Wall

SERIES

SHT. No. CW130 REV.

BY KM DATE Dec 98

EX. DATE

B3-R to Core Wall cont

Check weld cont.

F_H (2 Horiz welds)

$M = 154 \times 200 = 30\,800$ kNmm

Load per weld $= 30\,800/620 = \underline{50\text{ kN}}$

$F_H = 50 \times 10^3 / 190 \times 4.2 = \underline{63\text{ N/mm}^2} < 215\text{ N/mm}^2$

$\therefore$ Weld OK

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

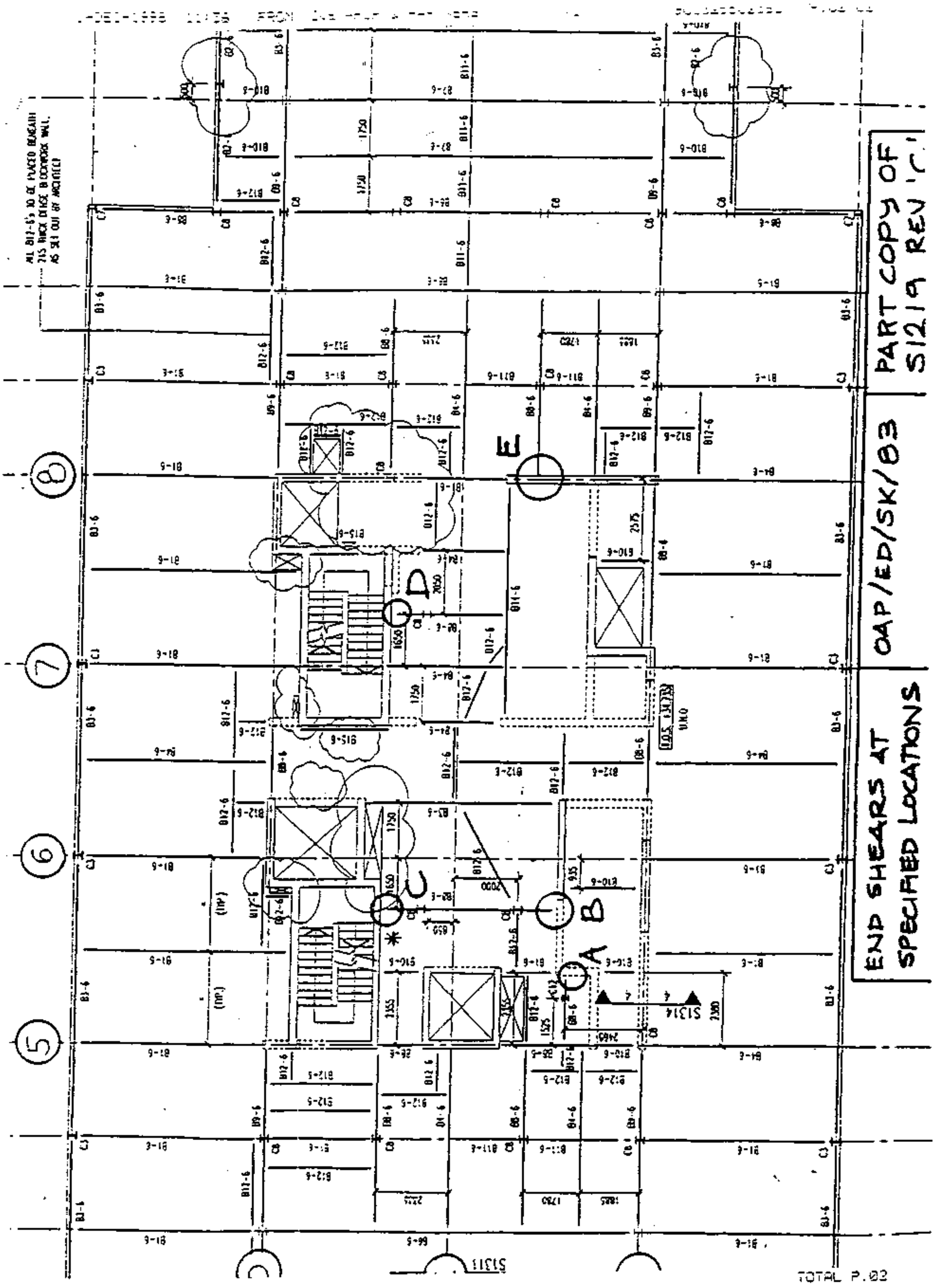
SUBJECT Connection to Core Wall

SERIES ....

SHT. No. CW131 REV. ....

BY KM DATE Dec 9

EX. .... DATE ....



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

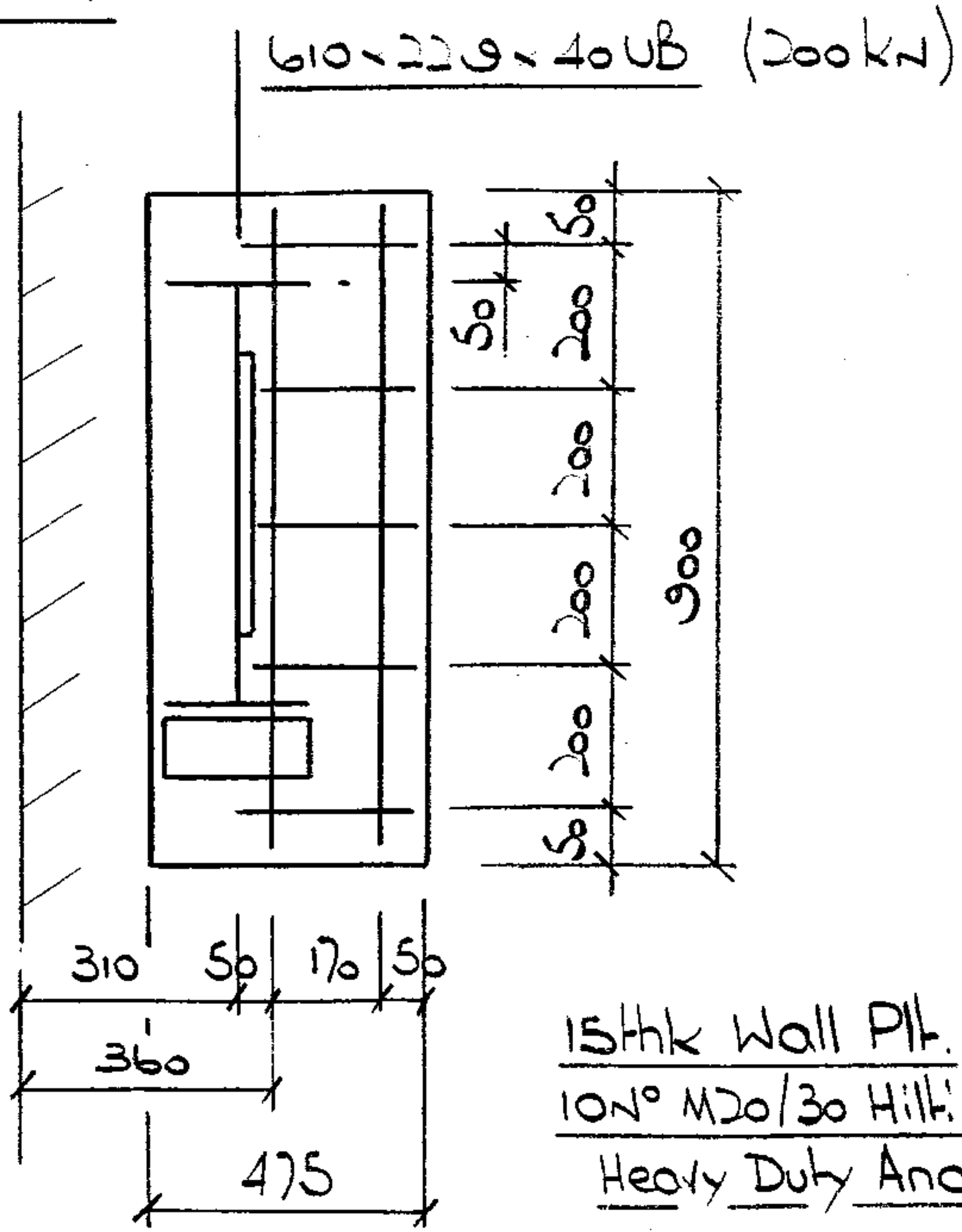
SERIES

SHT. No. CW132 REV.

BY KM DATE Dec 9?

EX. DATE

Conn @ A



15thk Wall Plt.

10N° M20/30 Hilti HSL

Heavy Duty Anchors

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS — SERIES ....

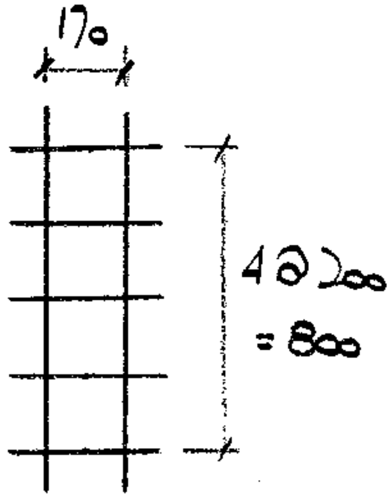
SUBJECT Connection to Core Wall — SHT. No. CW133 REV. ....

BY KM DATE Dec 98

EX. .... DATE ....

Conn @ A cont.

Anchor capacity = $58.6 \times 0.74 \times 0.77$ = <u>33 kN</u>



Load per Anchor

$F_V = 200/10 = 20\,kN$

$F_H = \dfrac{200 \times 135}{800 \times 2} = 17\,kN$

$F_R = \sqrt{(20^2 + 17^2)}$

$= \underline{26\,kN} < 33\,kN$

<u>∴ Anchors ok</u>

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT Connection to Core Wall

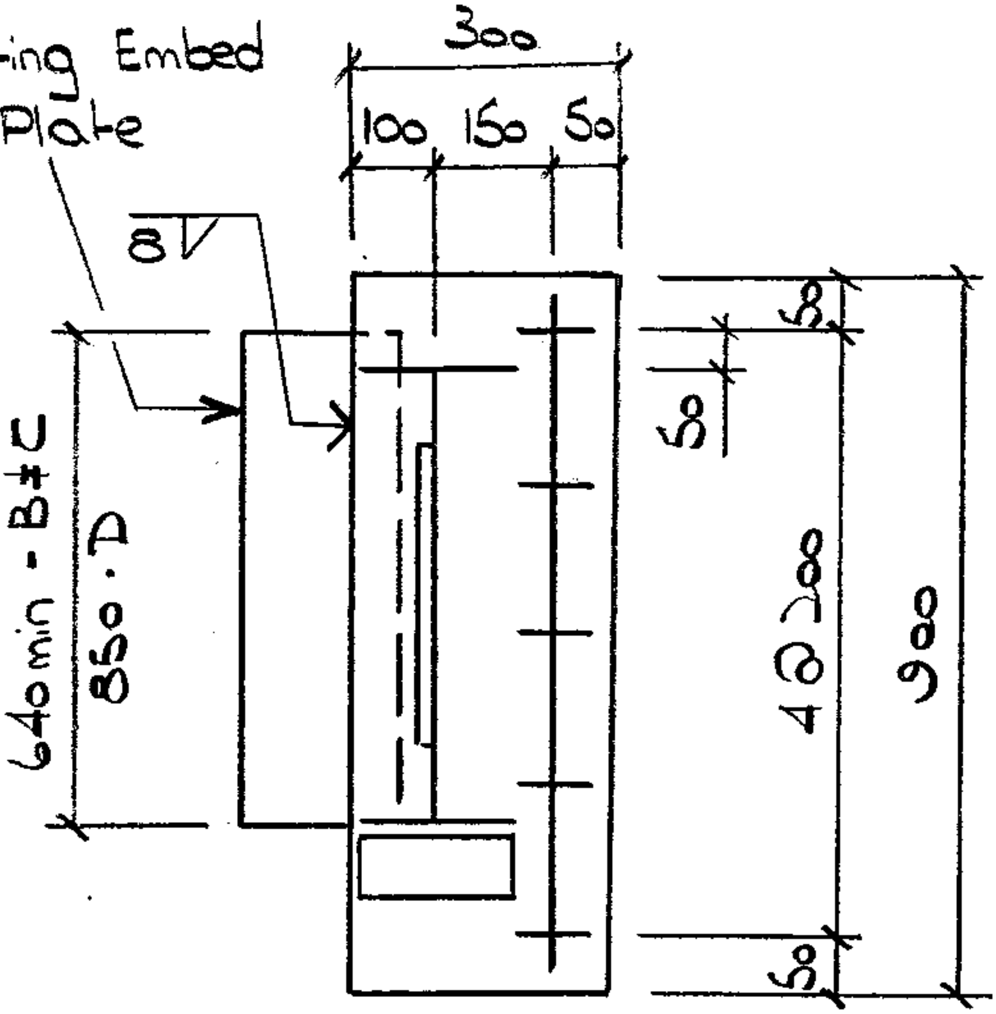
SERIES

SHT. No. CW134 REV.

BY KM DATE Dec 9?

EX. DATE

Conn. @ B, C & D



610 x 229 x 140 UB

End Shear : B & C = 500 kN

D = 600 kN

15 thk Wall Plt.

5 No M20/30 Hilti HSL Heavy Duty Anchors

Conn @ B, C & D cont

Check Wall Plt. to Embed Plt. Weld - 8FW

Conn @ B & C - End Shear = 500kN

Weld $Z = 5.6 \times 620^2/6 = 358\,773 \text{mm}^3$

$F_V = 500 \times 10^3 / 620 \times 5.6 = 144 \text{ N/mm}^2$

$F_H = 500 \times 10^3 \times 100 / 358\,773 = 139 \text{ N/mm}^2$

$F_R = \sqrt{(144^2 + 139^2)} = \underline{200 \text{ N/mm}^2} < 215 \text{ N/mm}^2$

$\therefore$ Weld OK

Conn @ D - End Shear = 600kN

Weld $Z = 5.6 \times 830^2/6 = 642\,973 \text{mm}^3$

$F_V = 600 \times 10^3 / 830 \times 5.6 = 129 \text{ N/mm}^2$

$F_H = 600 \times 10^3 \times 100 / 642\,973 = 93 \text{ N/mm}^2$

$F_R = \sqrt{(129^2 + 93^2)} = \underline{159 \text{ N/mm}^2} < 215 \text{ N/mm}^2$

$\therefore$ Weld OK

Kvaerner Cleveland Bridge Ltd.

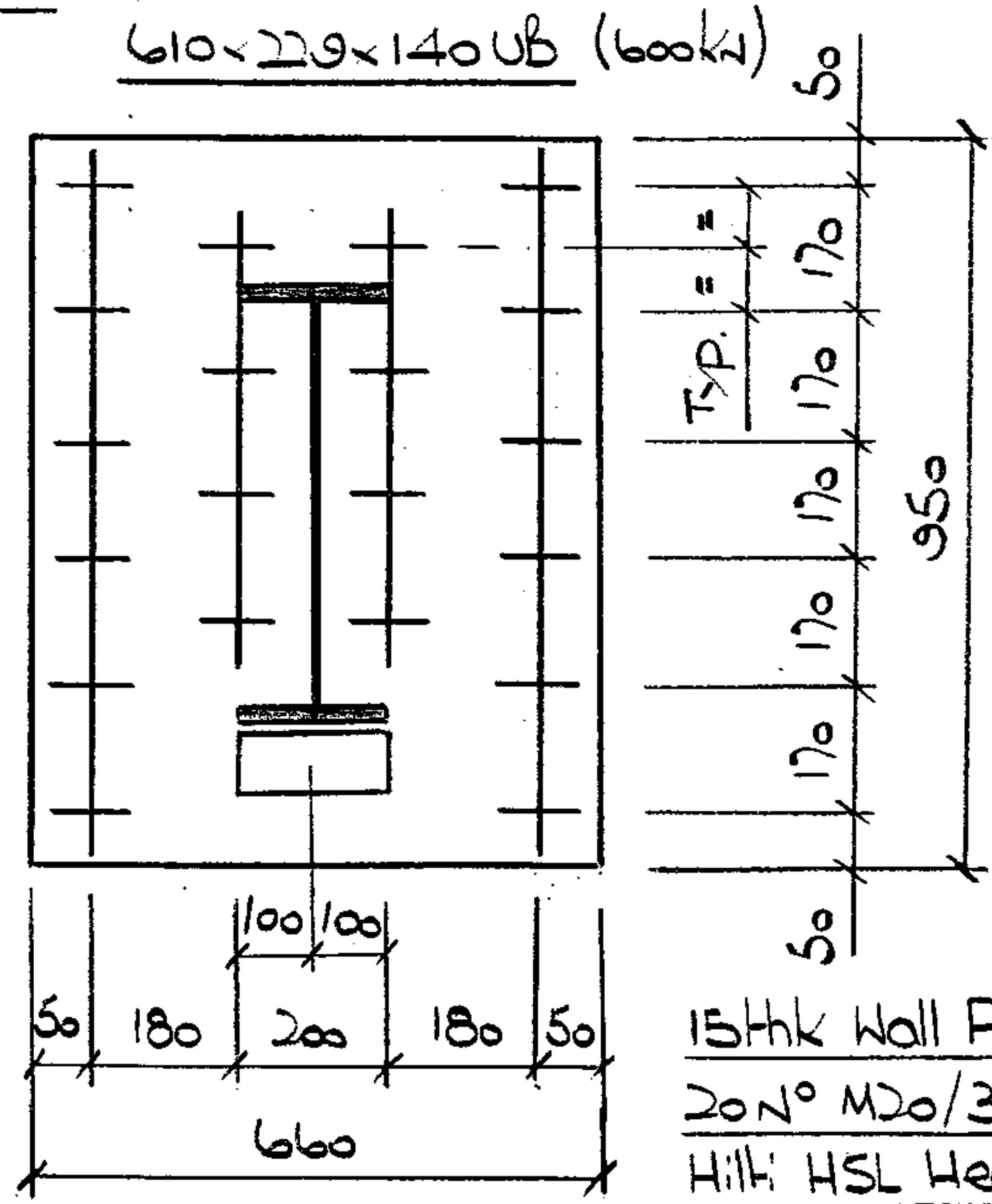
O/No. 325 JOB CARLTON GARDENS — SERIES — SHT. No. CW136 REV.

SUBJECT Connection to Core Wall — BY KM DATE Dec 9?

EX. DATE

Conn @ E

610×229×140 UB (600kN)



15thk Wall Plt.
20 N° M20/30
Hilti HSL Heavy-
Duty Anchors

Check Anchors

Anchor capacity = 58.6 × 0.74 × 0.77 = 33 kN

N° of anchors = 600/33 = 18

∴ 20 N° Anchor OK

Kvaerner Cleveland Bridge Ltd.

O/No: 325 JOB CARLTON GARDENS

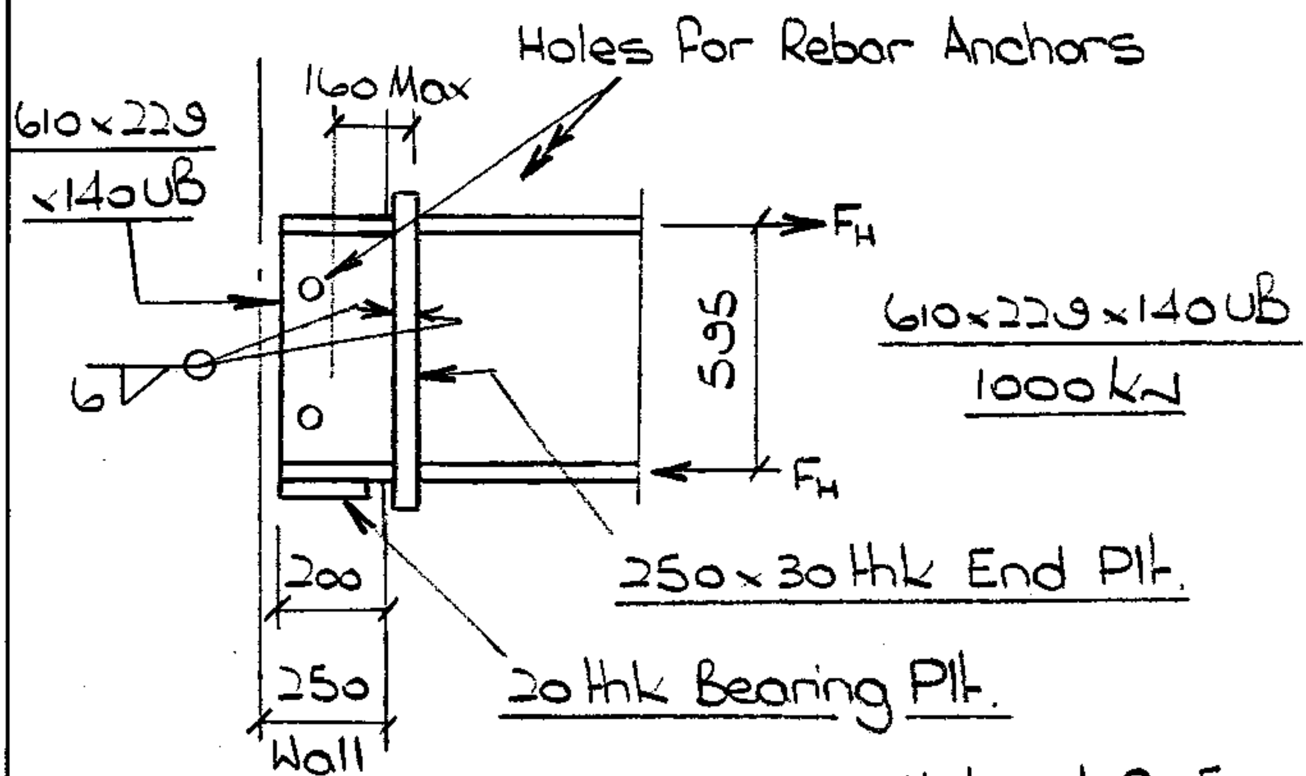
SUBJECT Connection to Core Wall

SERIES .... SHT. No. .... REV. ....

BY .... DATE ....

EX. .... DATE ....

Beam B8-6 to Core Wall



Material Gr 50

Check Weld (6FW)

Flg $F_H = 1000 \times 160 / 595 = \underline{269 kN}$

Flange weld capacity $= 2 \times 200 \times 4.2 \times 255 \times 10^{-3}$

$= \underline{428 kN} > 269 kN$

$\therefore$ OK

Web

Web weld capacity $= 2 \times 550 \times 4.2 \times 255 \times 10^{-3}$

$= \underline{1178 kN} > 1000 kN$

$\therefore$ OK

5

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

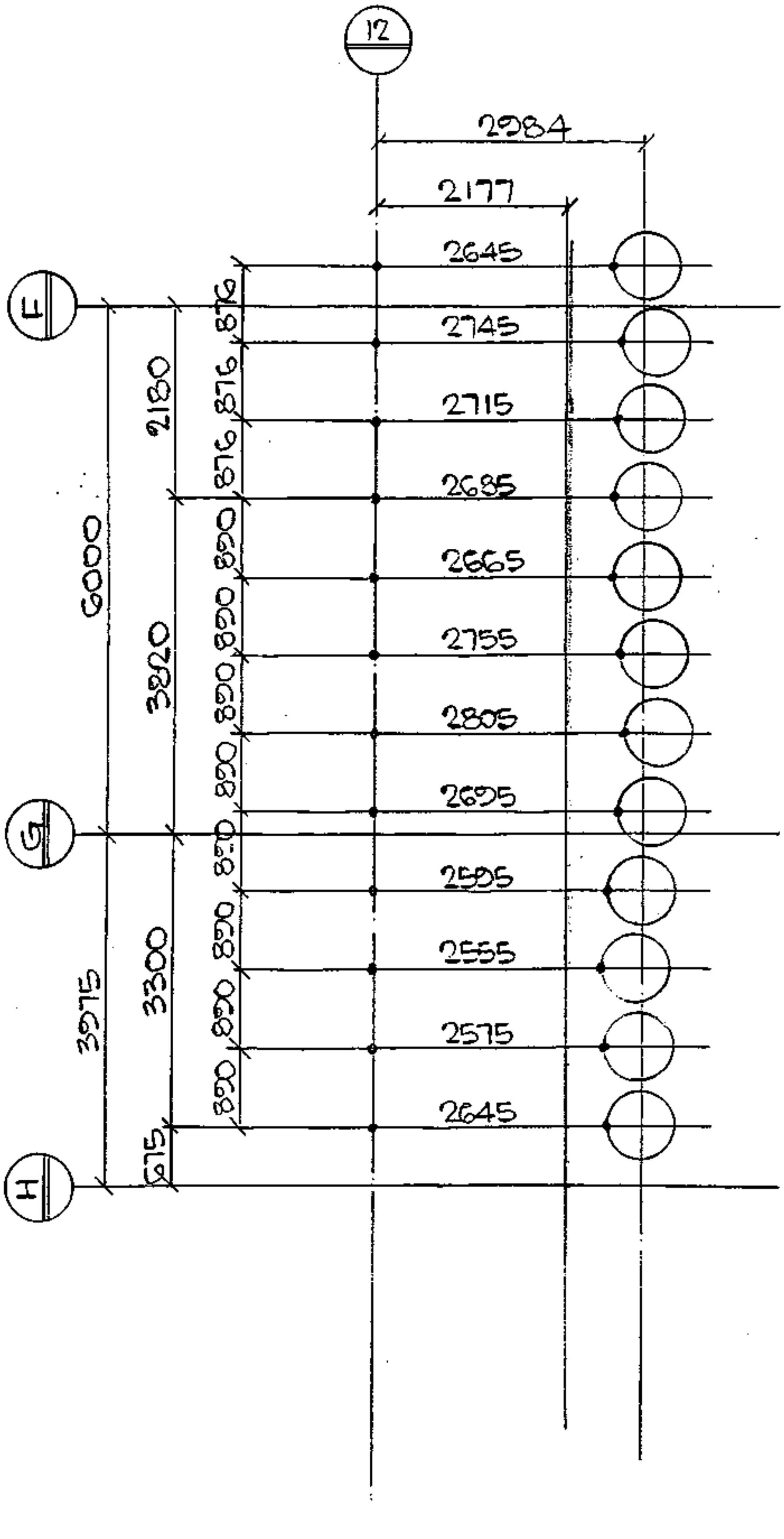
SUBJECT WALING BEAM — SETTING OUT

SERIES

SHT. No. WB REV.

BY KB DATE 08/98

EX. DATE



SETTING - OUT OF WALING BEAM

B-H / EAST OF LINE 12

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

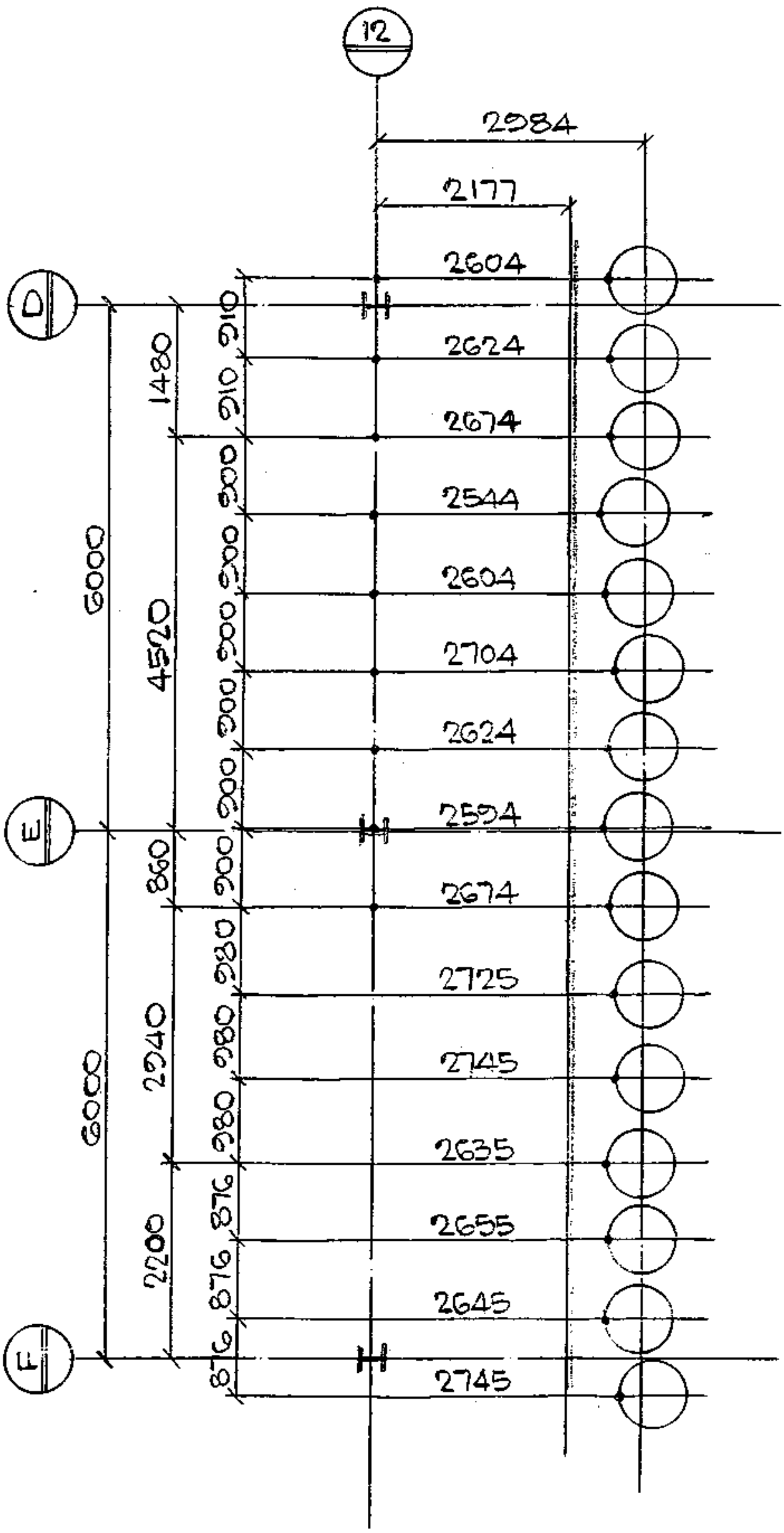
SUBJECT WALING BEAM ~ SETTING OUT

SERIES

SHT. No. WB REV.

BY KB DATE 08/92

EX. DATE



SETTING-OUT OF WALING BEAM

B-H/EAST OF LINE 12

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

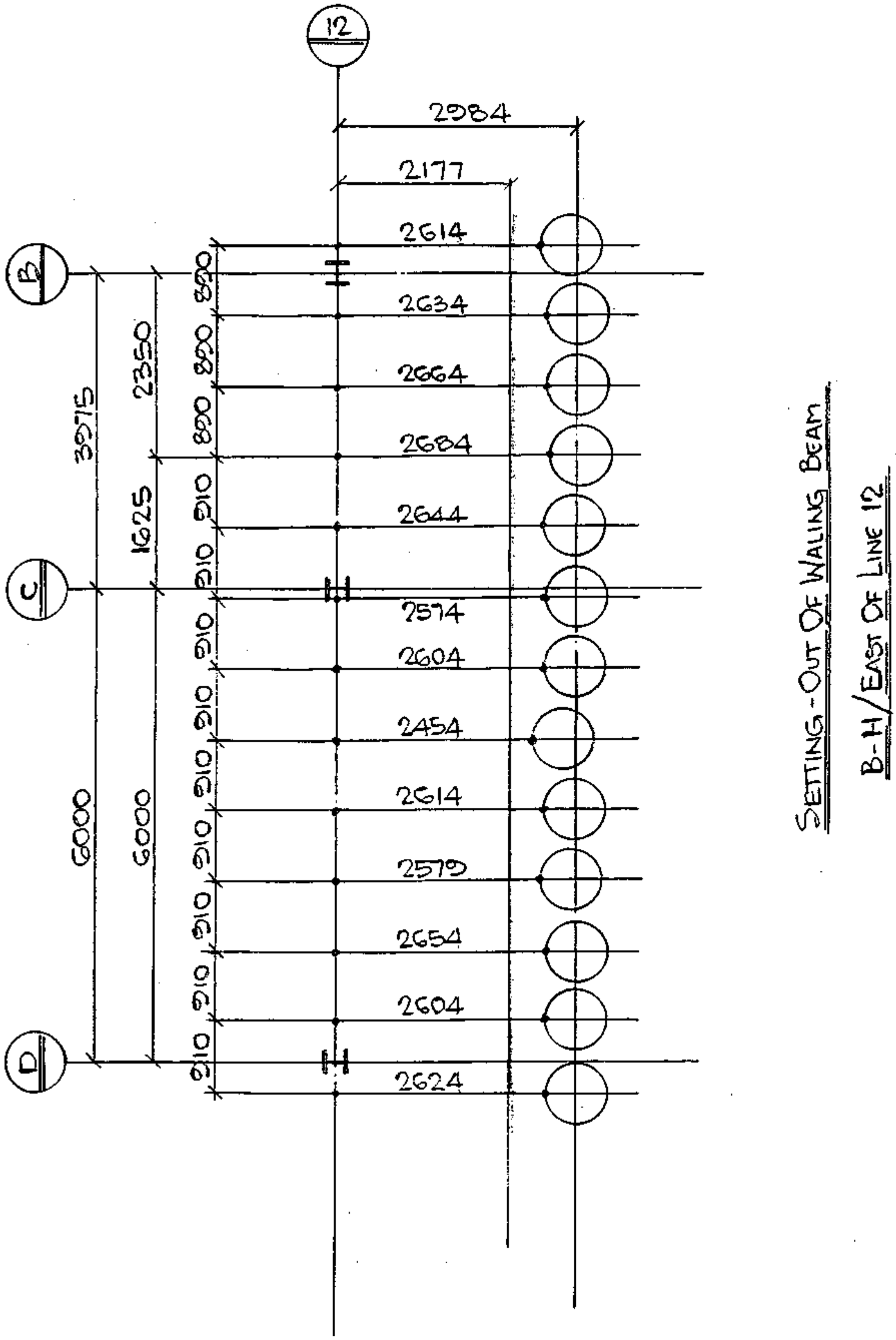
SUBJECT WALING BEAM ~ SETTING OUT

SERIES

SHT. No. WB REV.

BY KB DATE 08/98

EX. DATE



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ SETTING OUT

SERIES

SHT. No. WB1 REV.

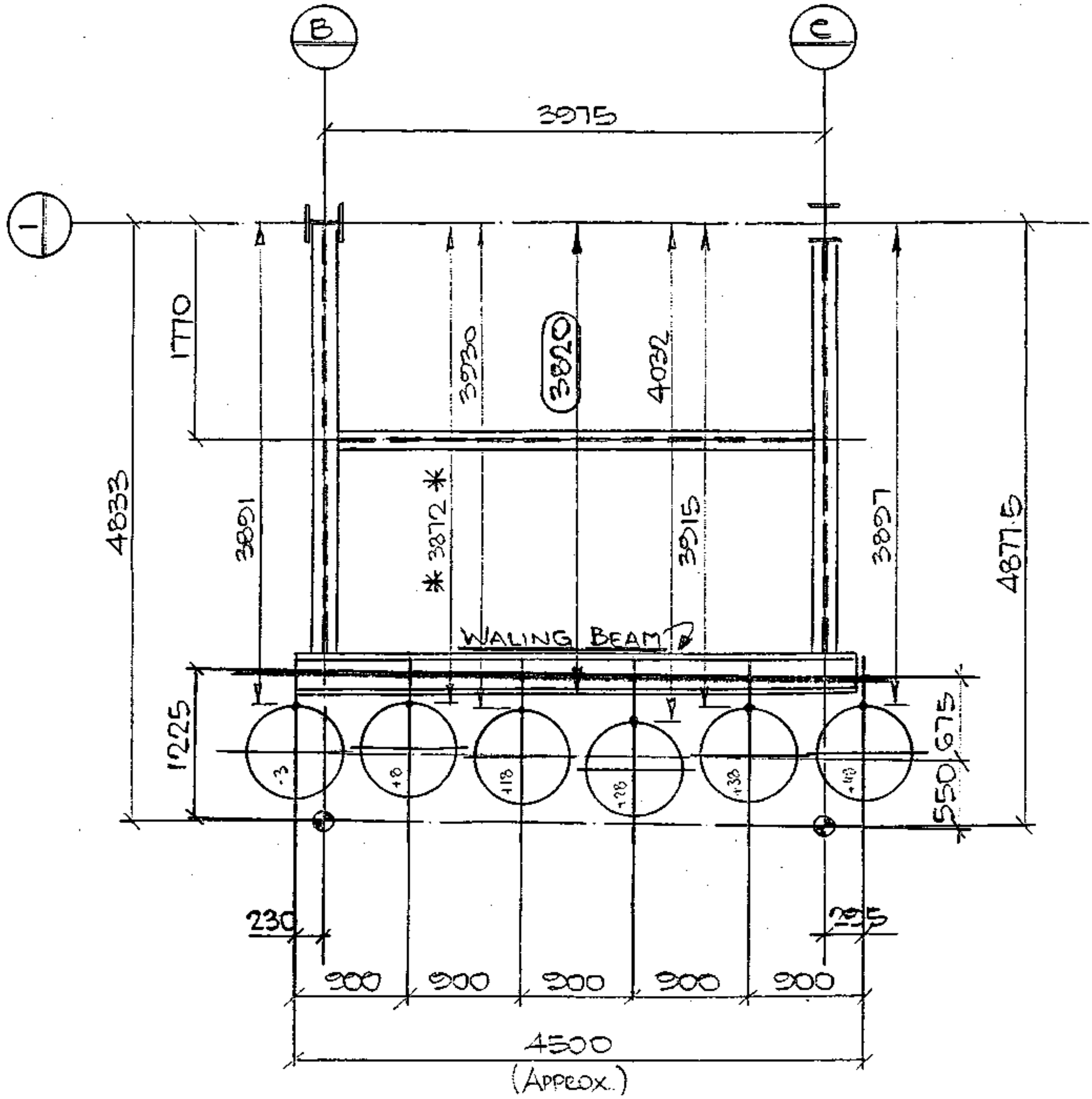
BY KB DATE 07/98

EX. DATE

* SUPERSEDED *

SETTING~OUT WALING BEAM

B-C / WEST OF LINE 1



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM – TEMPORARY SUPPORTS

SERIES

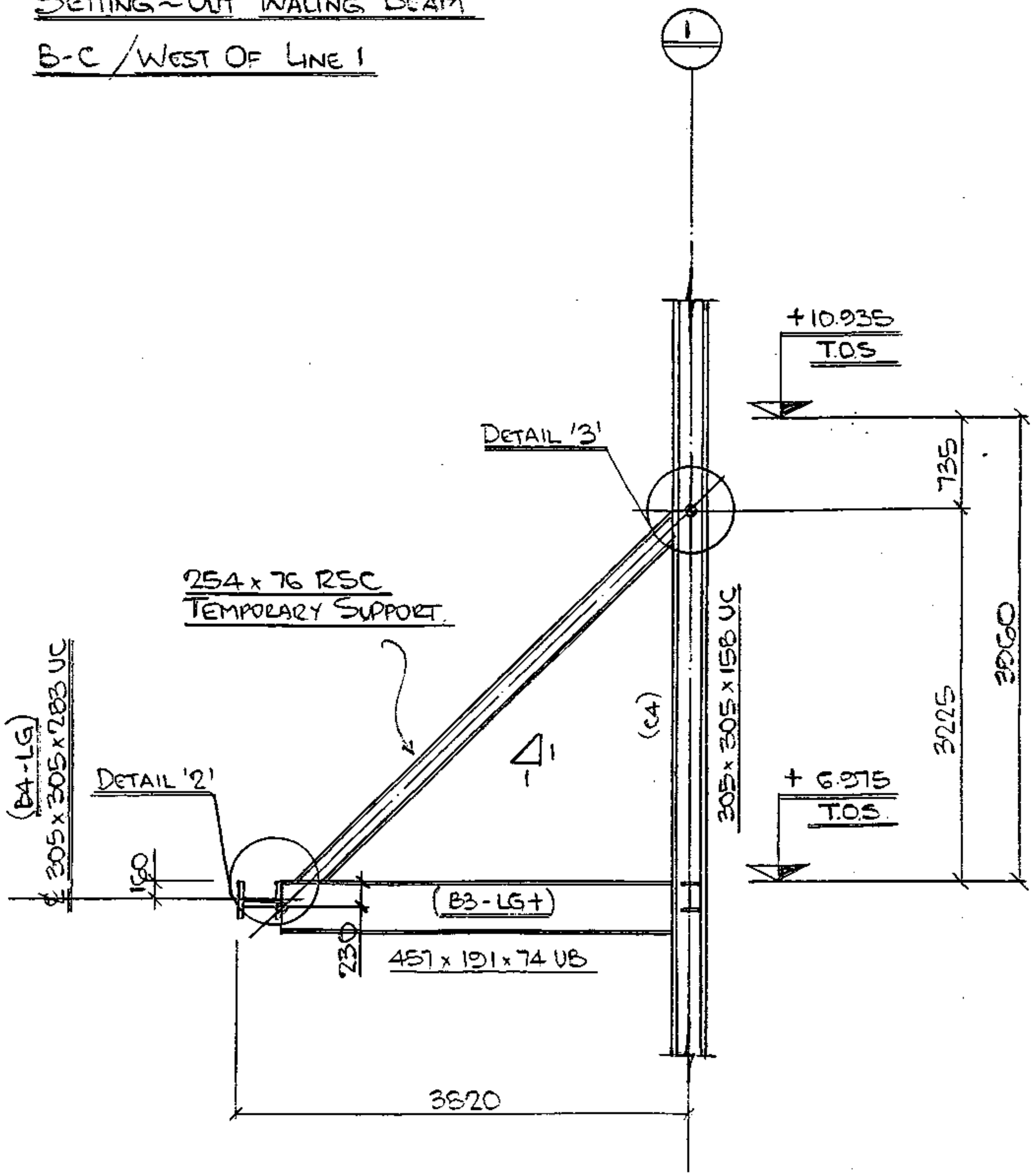
SHT. No. WB3 REV.

BY KB DATE 07/98

EX. DATE

SETTING – OUT WALING BEAM

B-C / WEST OF LINE 1



ELEVATION AT LINE 'C'

(LOOKING NORTH)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ SETTING OUT

SERIES

SHT. No. WB4 REV. A

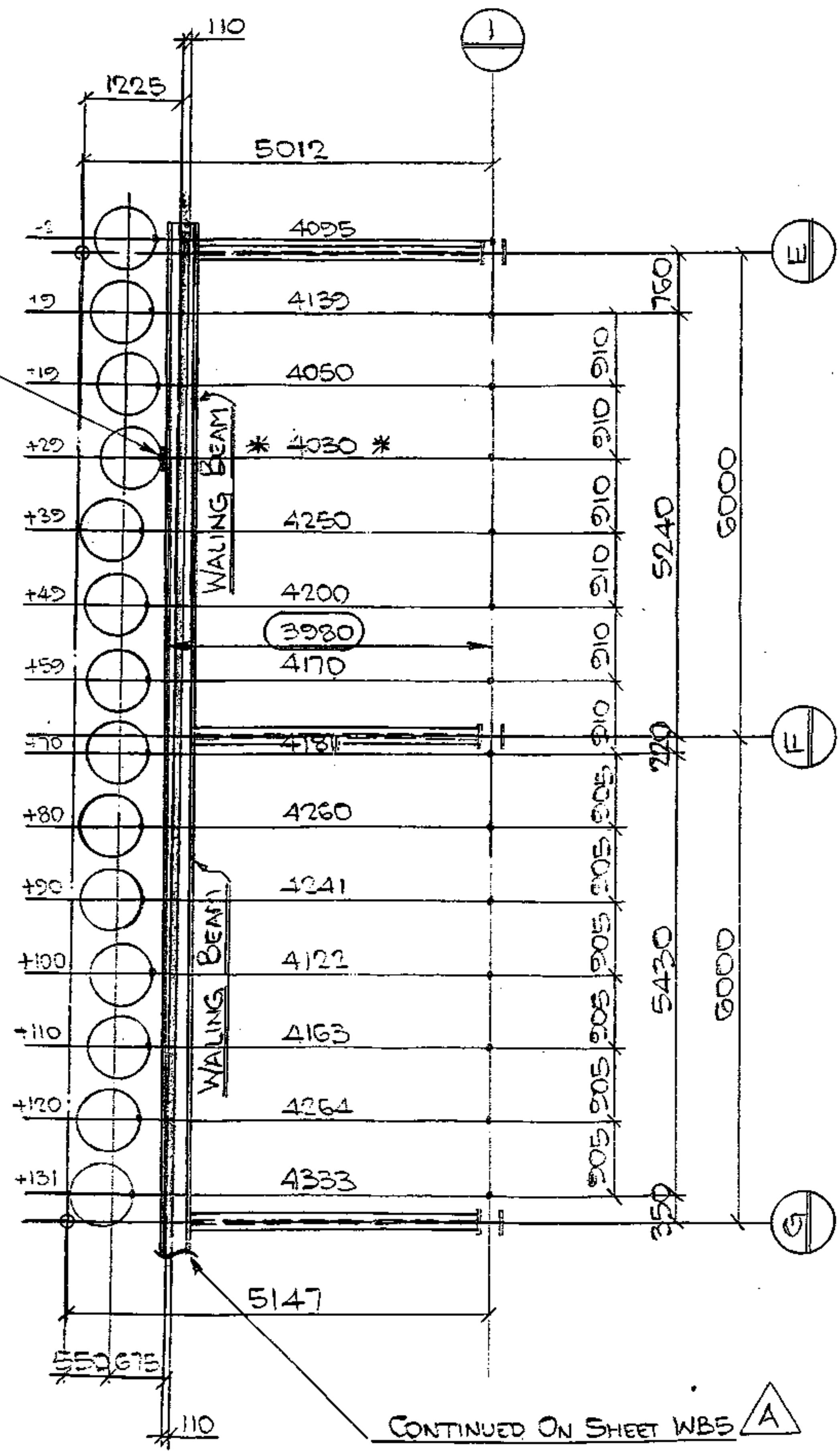
BY KB DATE 05/98

EX. DATE

REV			
A	25/08/98	MIN. PACK THICKNESS ADDED	KB

SETTING-OUT WALING BEAM

E-H / WEST OF LINE I



CONTINUED ON SHEET WB5 A

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

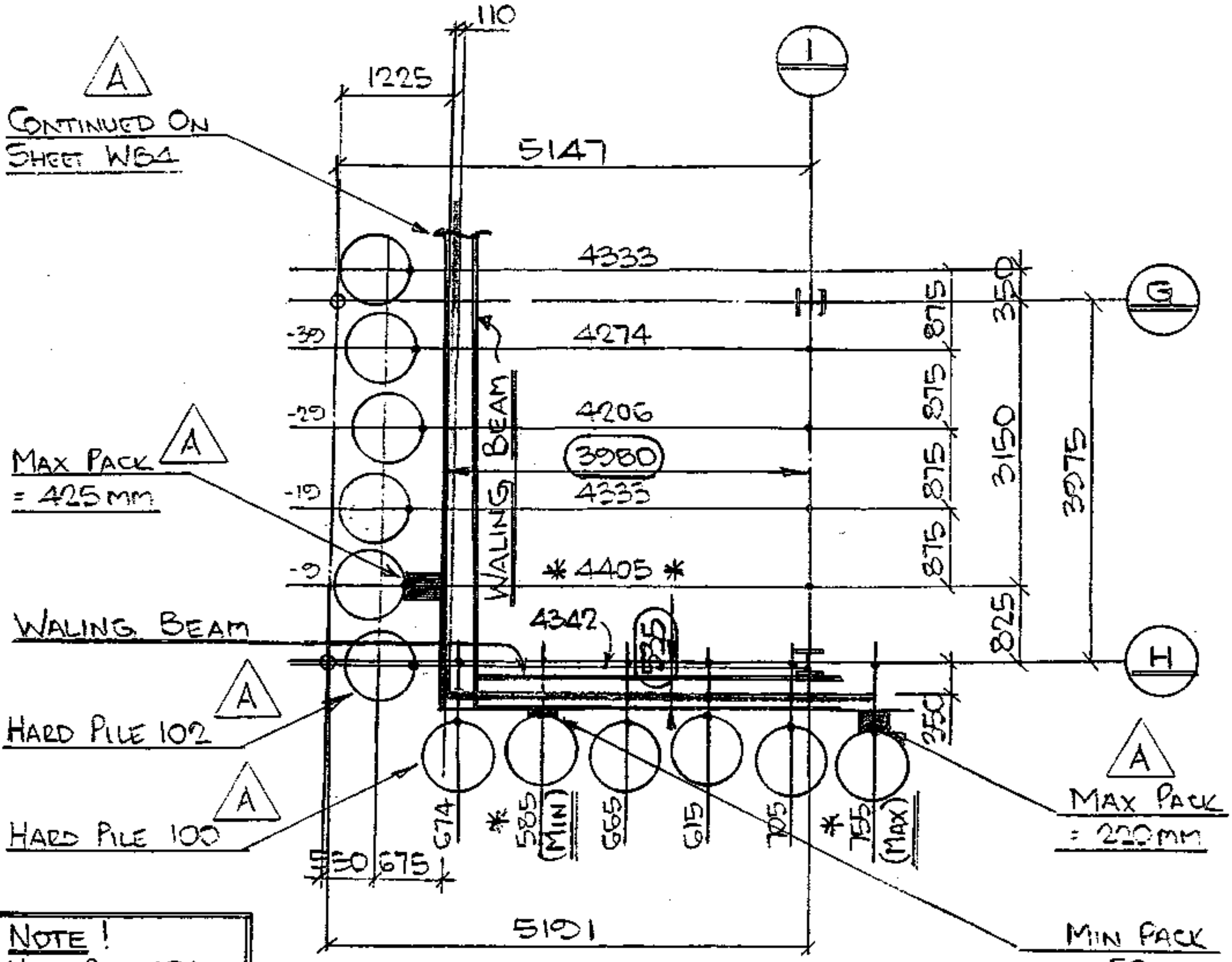
SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

SHT. No. WB5 REV. A

BY KB DATE 05/98

EX. DATE



NOTE !
HARD PILE 101
SITE SURVEY
NOT RECEIVED
BY KCBL

A

SETTING ~ OUT WALING BEAMS

E - H / WEST OF LINE 1

WEST - 1 / SOUTH OF LINE H

REV 'A' | 25/08/98 | NOTE ADDED, HARD PILE 100 ADDED, MAX & MIN PACK THICKNESSES ADDED | KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

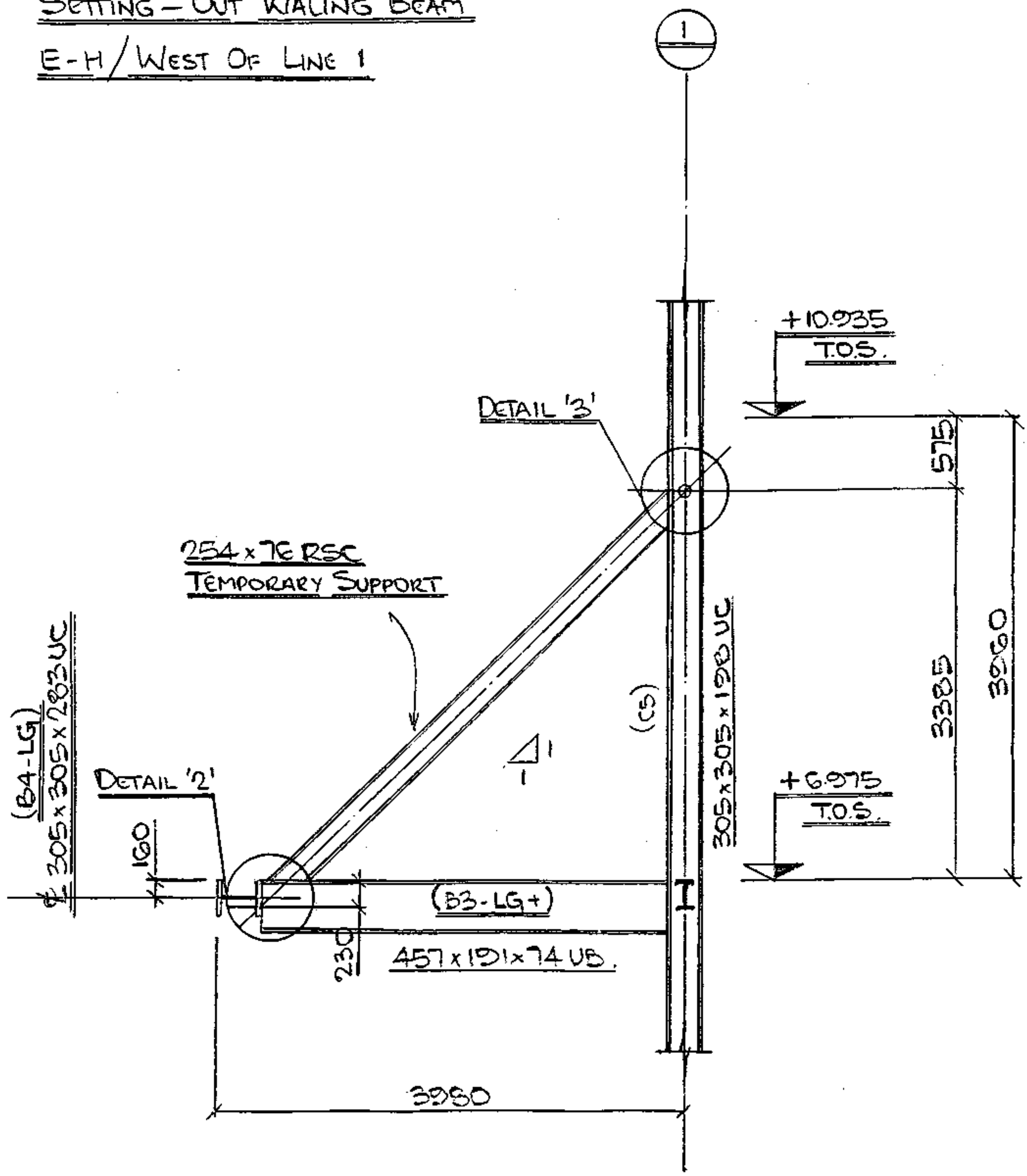
SHT. No. WBG REV.

BY KB DATE 05/98

EX. DATE

SETTING – OUT WALING BEAM

E-H / WEST OF LINE 1



ELEVATION AT LINE 'F'

(LOOKING NORTH)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

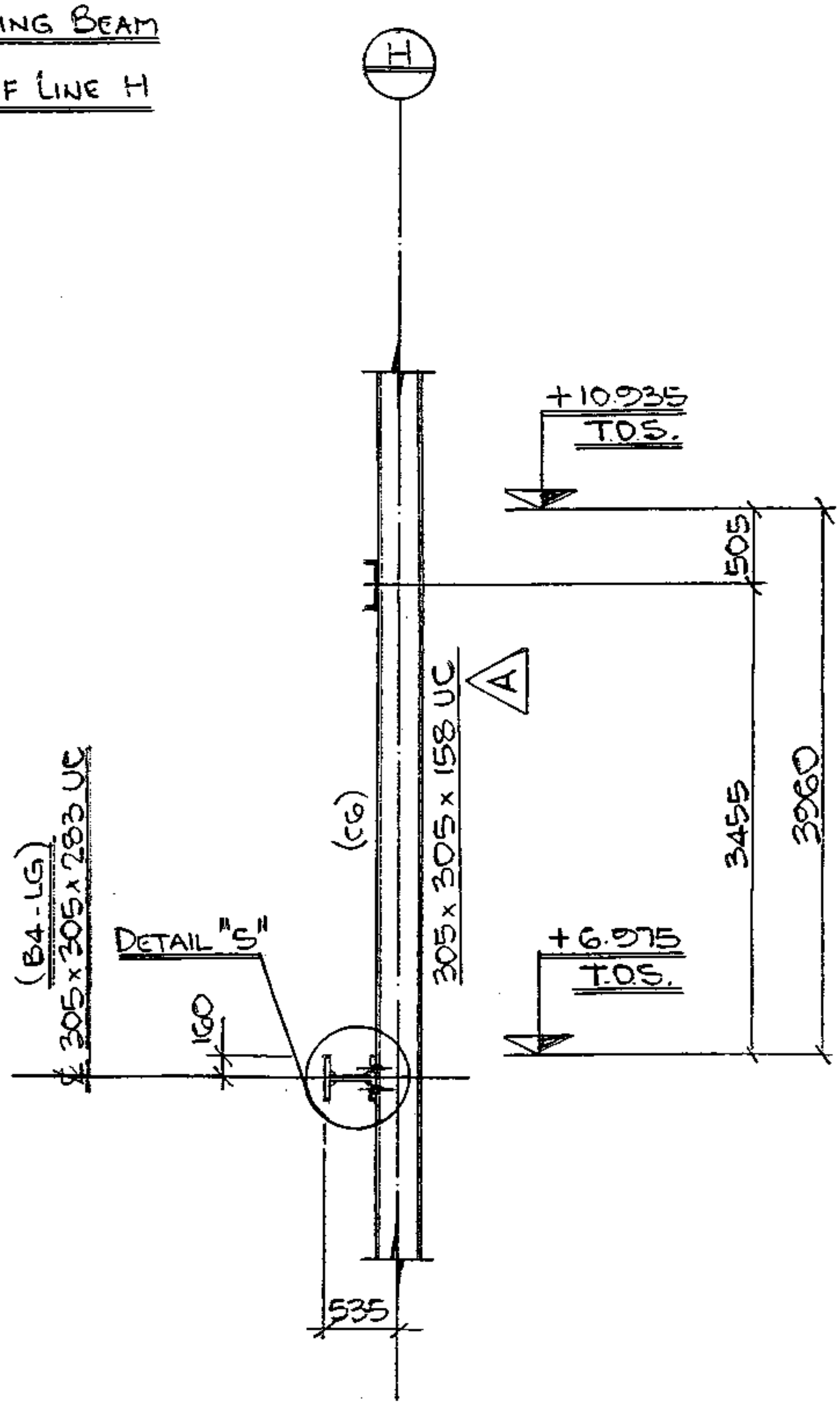
SHT. No. WB7 REV. A

BY KB DATE 08/98

EX. DATE

SETTING ~ OUT WALING BEAM

WEST - 1/ SOUTH OF LINE H



ELEVATION AT LINE '1'

(LOOKING WEST)

REV A | 25/08/98 | COLUMN SECTION AMENDED | KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

SHT. No. WB8 REV. A

BY KB DATE 07/98

EX. DATE

SETTING-OUT WALING BEAM

5-8 / SOUTH OF LINE H.

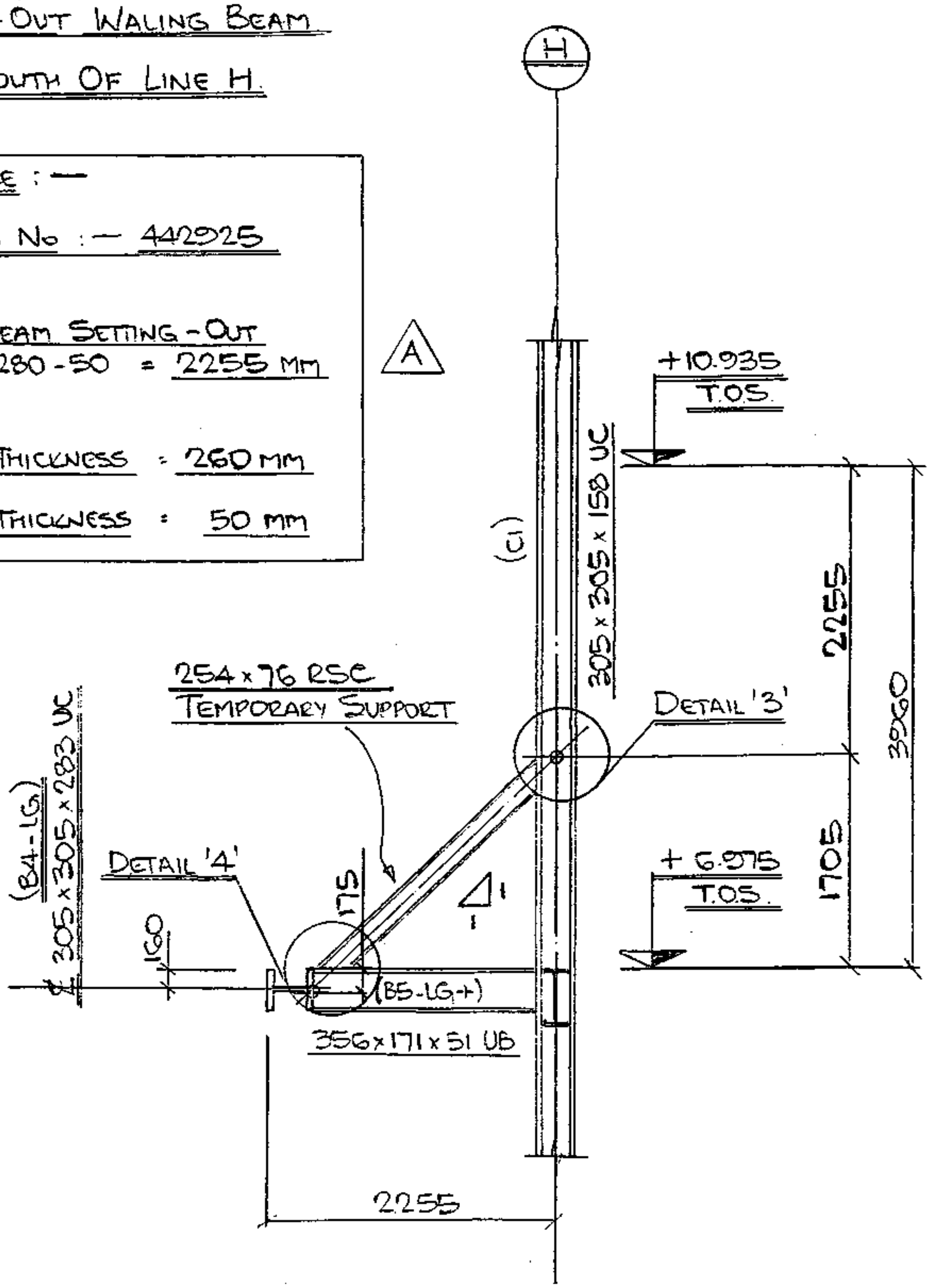
REFERENCE :—

SES DRG No :— 442925

WALING BEAM SETTING-OUT
= 2025 + 280 - 50 = 2255 MM

MAX PACK THICKNESS = 260 MM

MIN PACK THICKNESS : 50 MM



ELEVATION AT LINE '7'

(LOOKING WEST)

REV 'A' 25/08/98 REFERENCES ADDED KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

SHT. No. WB2 REV.

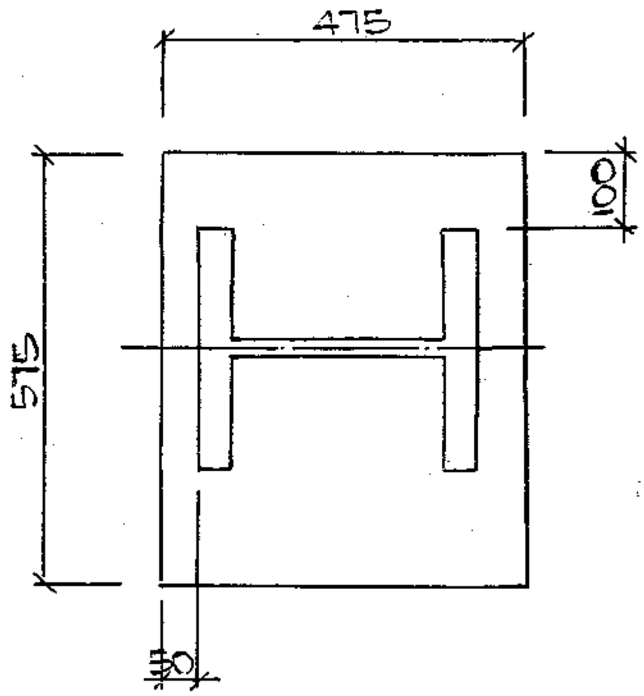
BY KS DATE 07/98

EX. DATE

TEMPORARY SUPPORT OF WALING BEAMS

a) SELF WEIGHT OF ENCASED WALING BEAM

SECTION : — 305 x 305 x 283 UC



SECTION AREA : 57.5 x 47.5 = 2731.25 cm^2

STEEL SECTION AREA = 360.00 cm^2

NET CONCRETE AREA : 2731.25 − 360.00 = 2371.25 cm^2

CONCRETE WEIGHT/METRE = 24.0 x 2371.25 ÷ 10^4 = 5.691 KN/M

STEEL WEIGHT/METRE : 7850 x 9.807 x 360.00 ÷ 10^7 = 2.771 KN/M

SECTION WEIGHT/METRE : 5.69 + 2.77 : 8.46 KN/m

LOAD FACTOR : 1.4

FACTORED LOAD = 8.46 x 1.4 = 11.85 KN/m

MAX SPAN : 9000

SELF WEIGHT : 11.85 x 9.000 = 106.6 KN

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

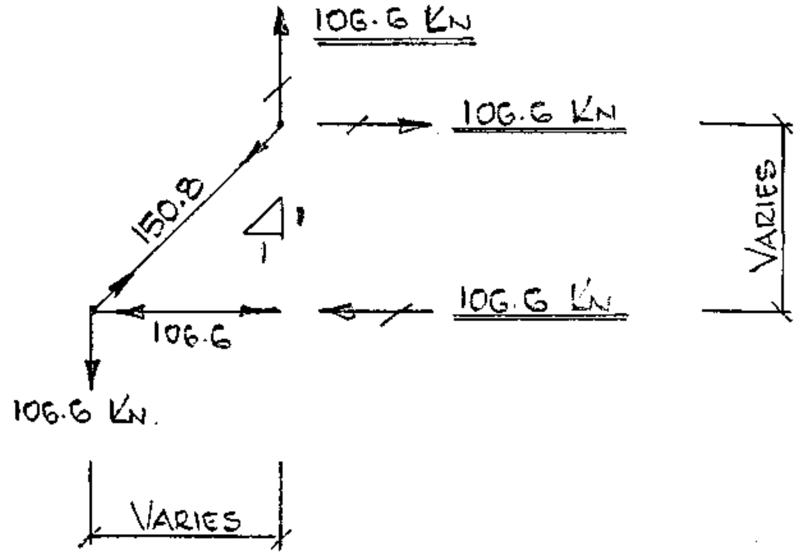
SERIES

SHT. No. WB10 REV.

BY KB DATE 07/98

EX. DATE

b) LOAD IN TEMPORARY SUPPORT.



LOAD IN SUPPORT: $\sqrt{2} \times 106.6 = 150.8$ KN (TENSION)

CONSIDER MATERIAL GRADE 43 FOR TEMPORARY SUPPORTS

$P_y = 275$ N/mm^2

MINIMUM AREA REQ^D = $150.8 \times 10^3 \div 275 = 548$ mm^2 (5.48 cm^2)

TRY ; — 254 x 76 x 28.29 RSC ~ A/C MATERIAL AVAILABILITY

AREA = 36.03 cm^2 > 5.48 cm^2 OK.

USE ; — 254 x 76 x 28.29 RSC (GRADE 43A)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

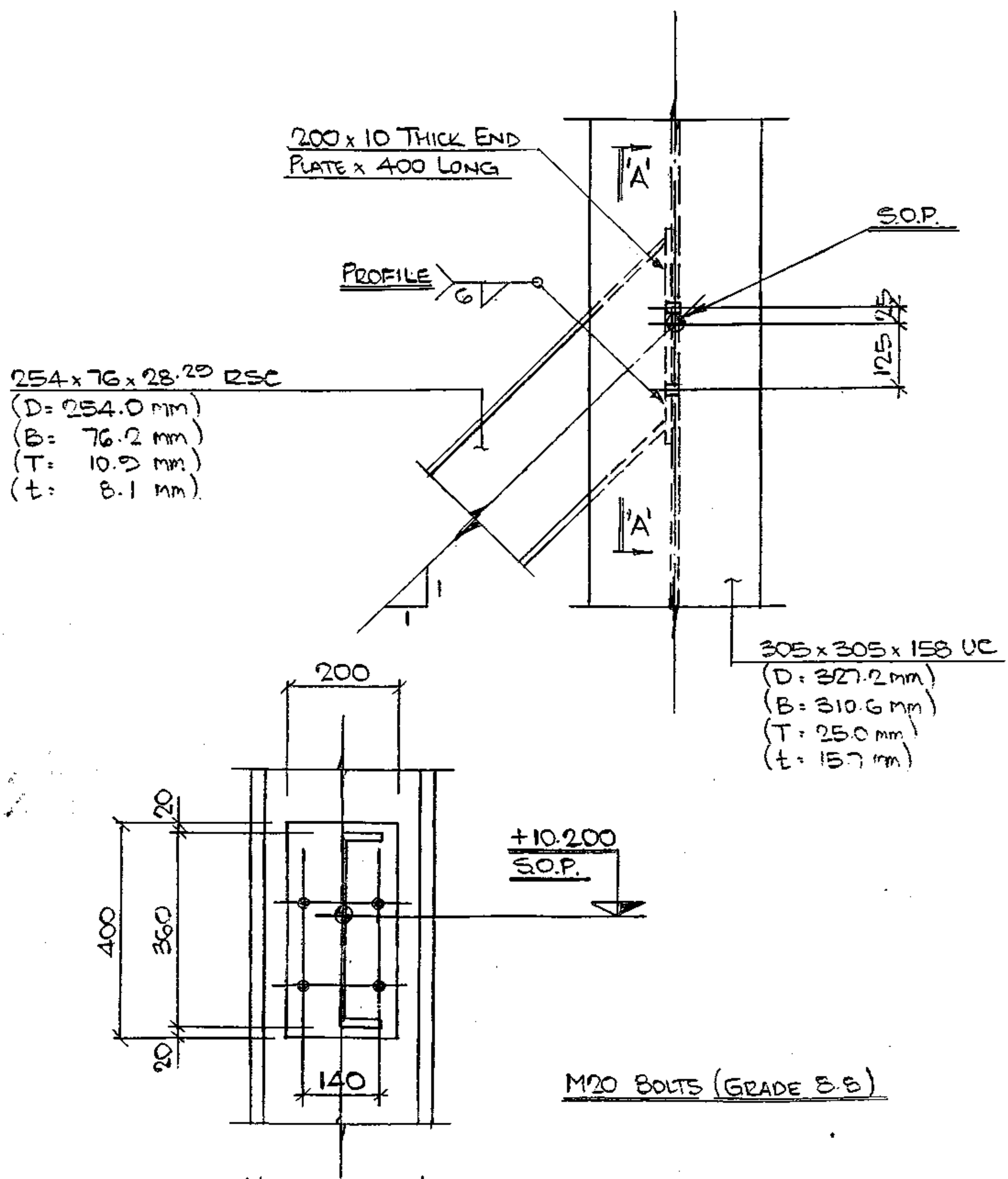
SERIES

SHT. No. WB11 REV.

BY KB DATE 08/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL '1'



VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

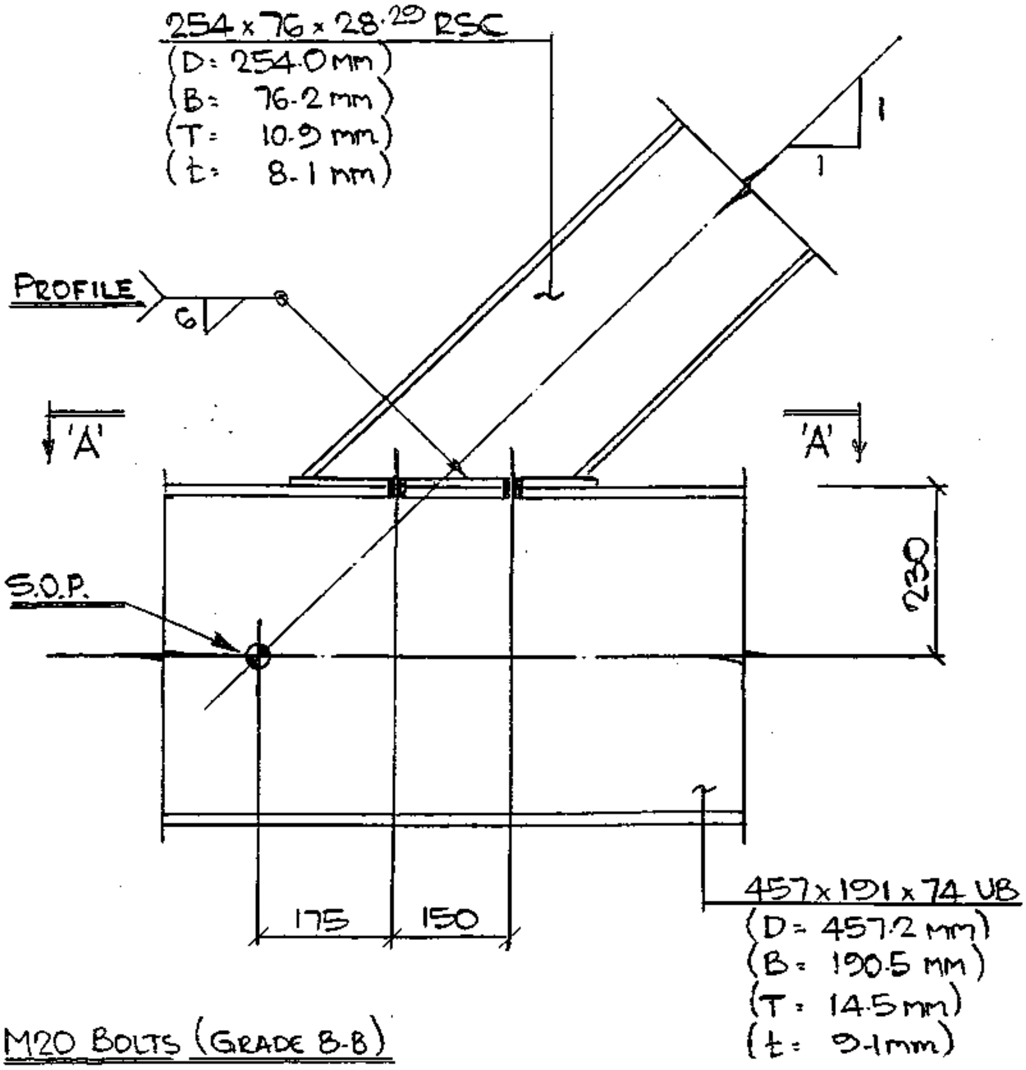
SERIES

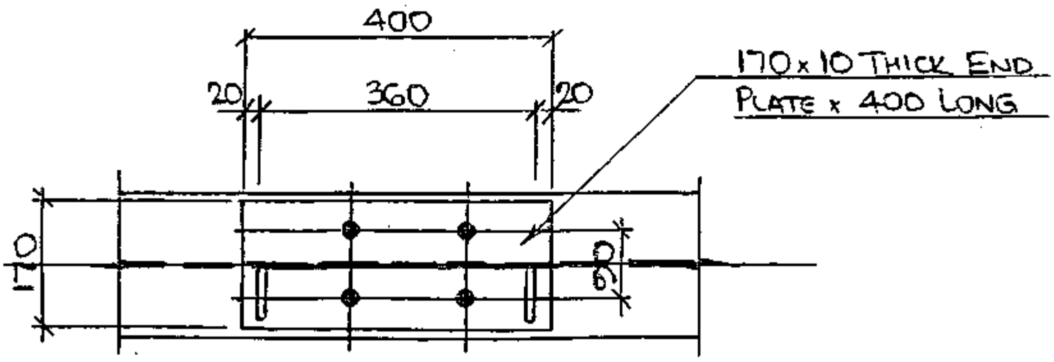
SHT. No. WB12 REV.

BY KB DATE 05/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL '2'





VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

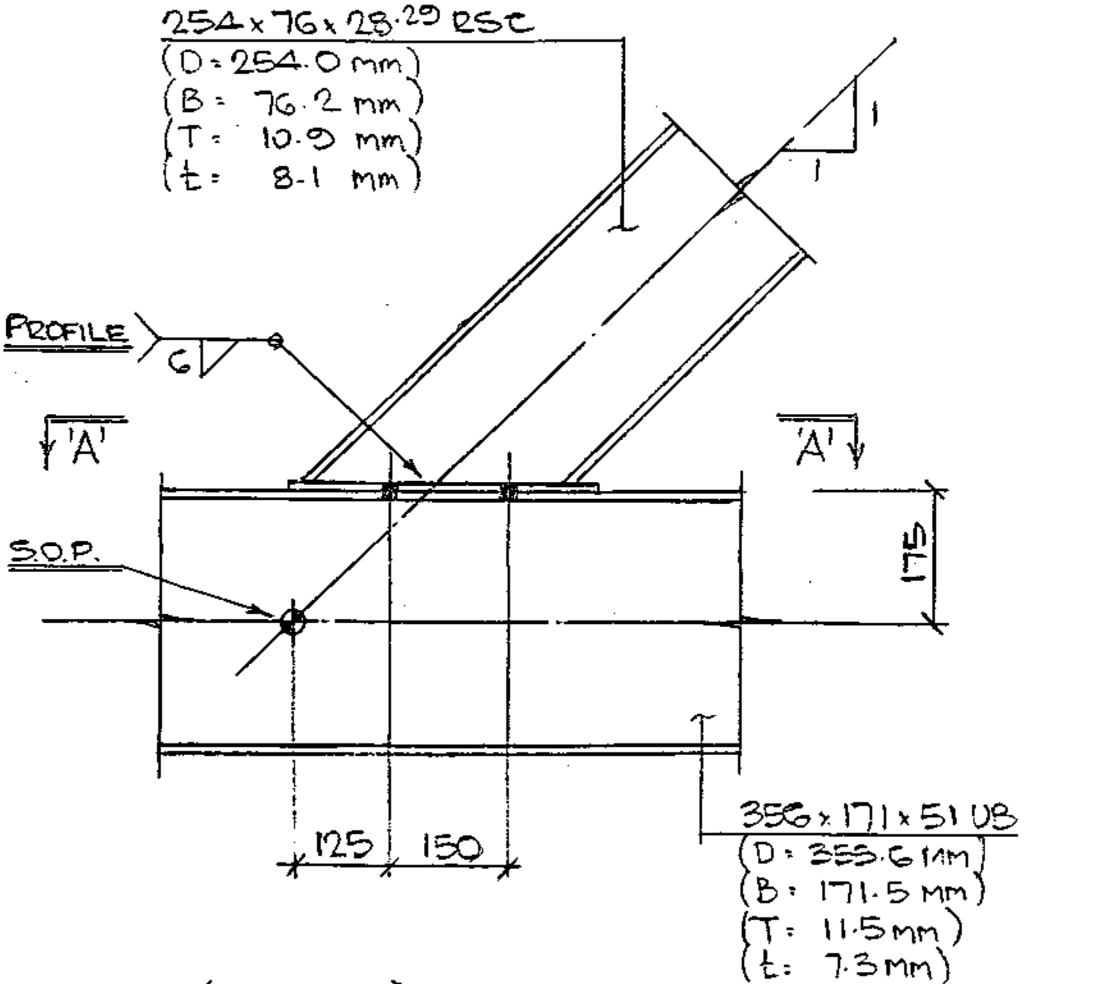
SERIES

SHT. No. WB14 REV.

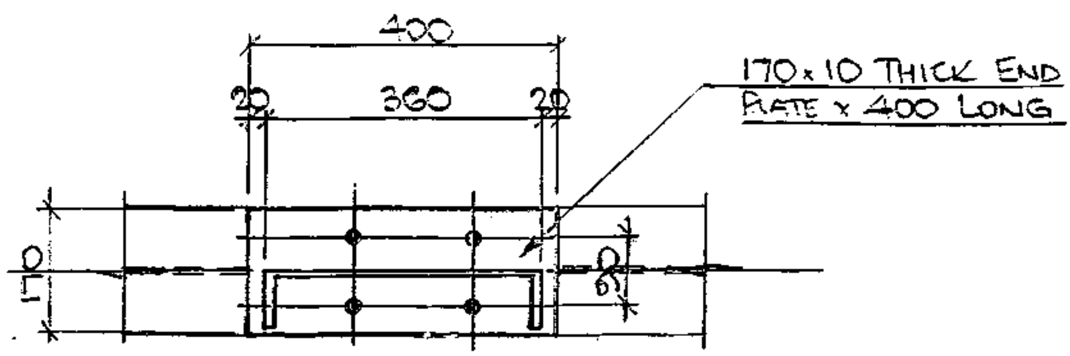
BY KB DATE 07/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL '4'



M20 BOLTS (GRADE 8.8)



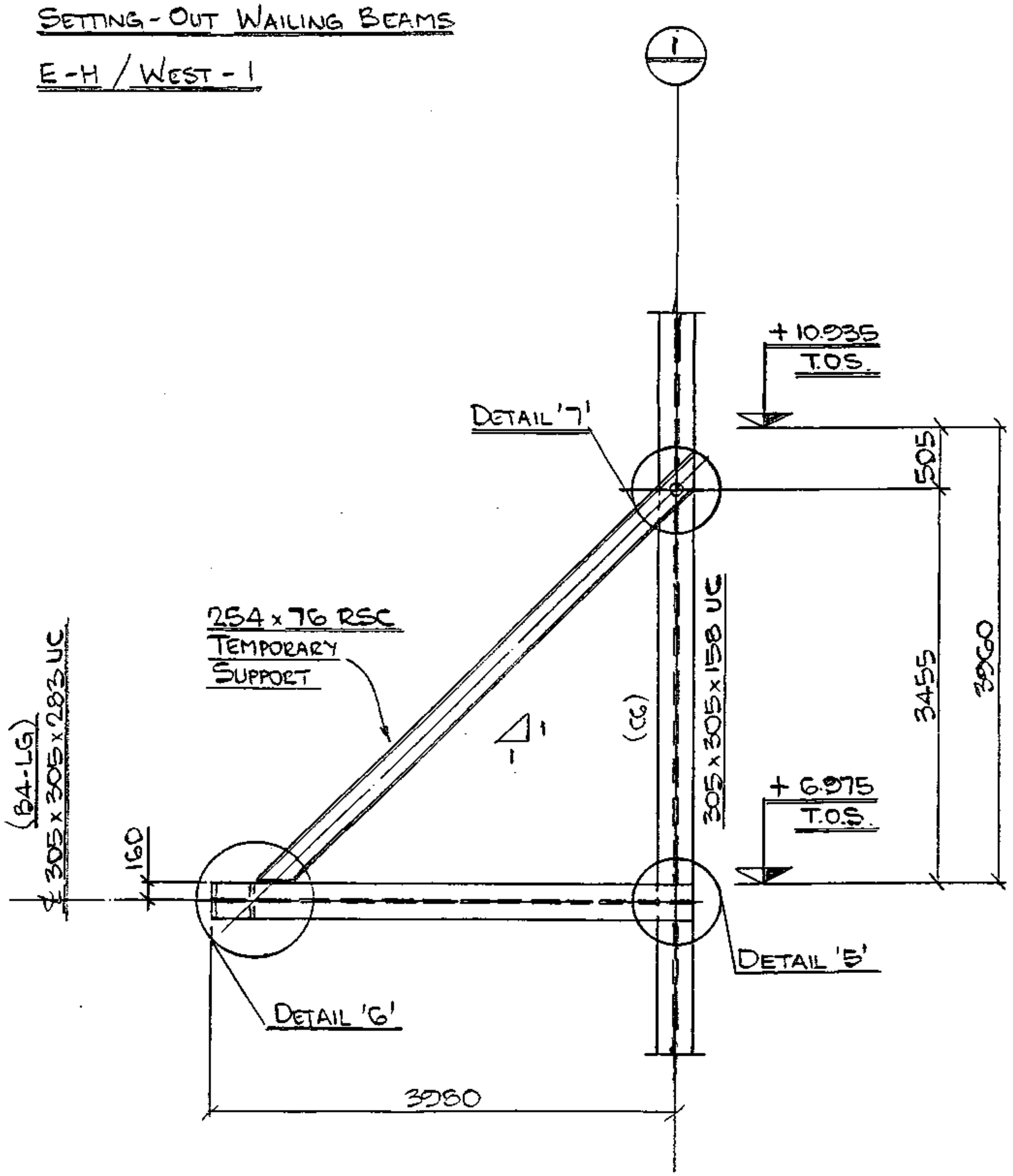
VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS	SERIES
SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS	SHT. No. WB15 REV.
	BY KB DATE 09/98
	EX. DATE

<u>SETTING - OUT WAILING BEAMS</u>

<u>E-H / WEST - 1</u>



Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

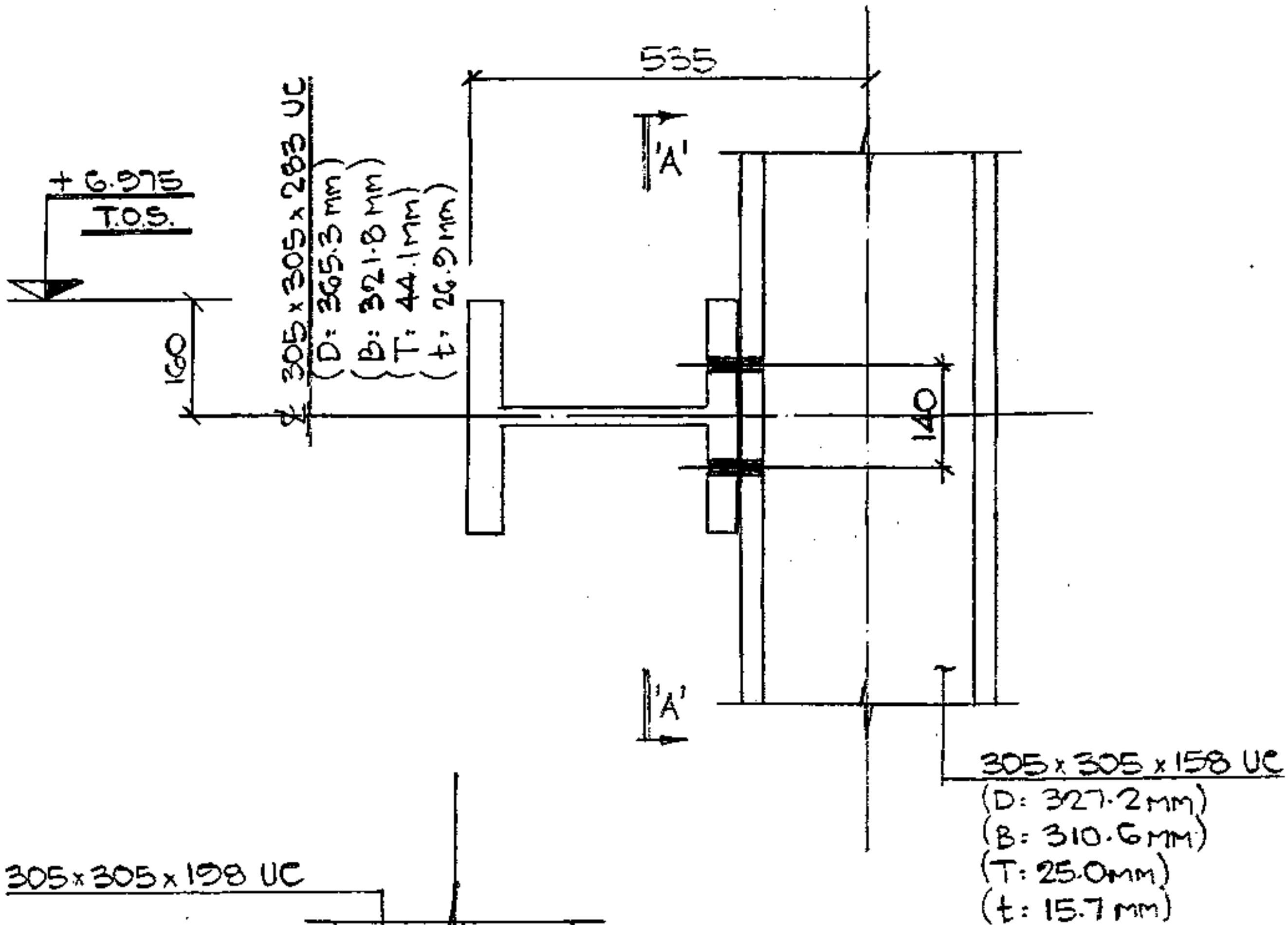
SERIES

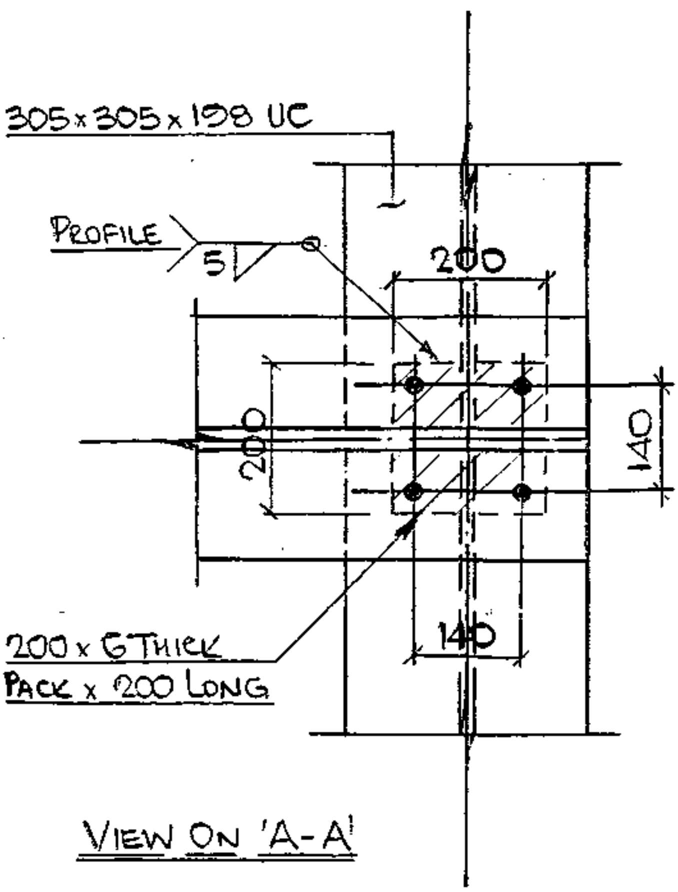
SHT. No. WB16 REV.

BY KB DATE 09/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL '5'





M20 BOLTS (GRADE 8.8)

VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

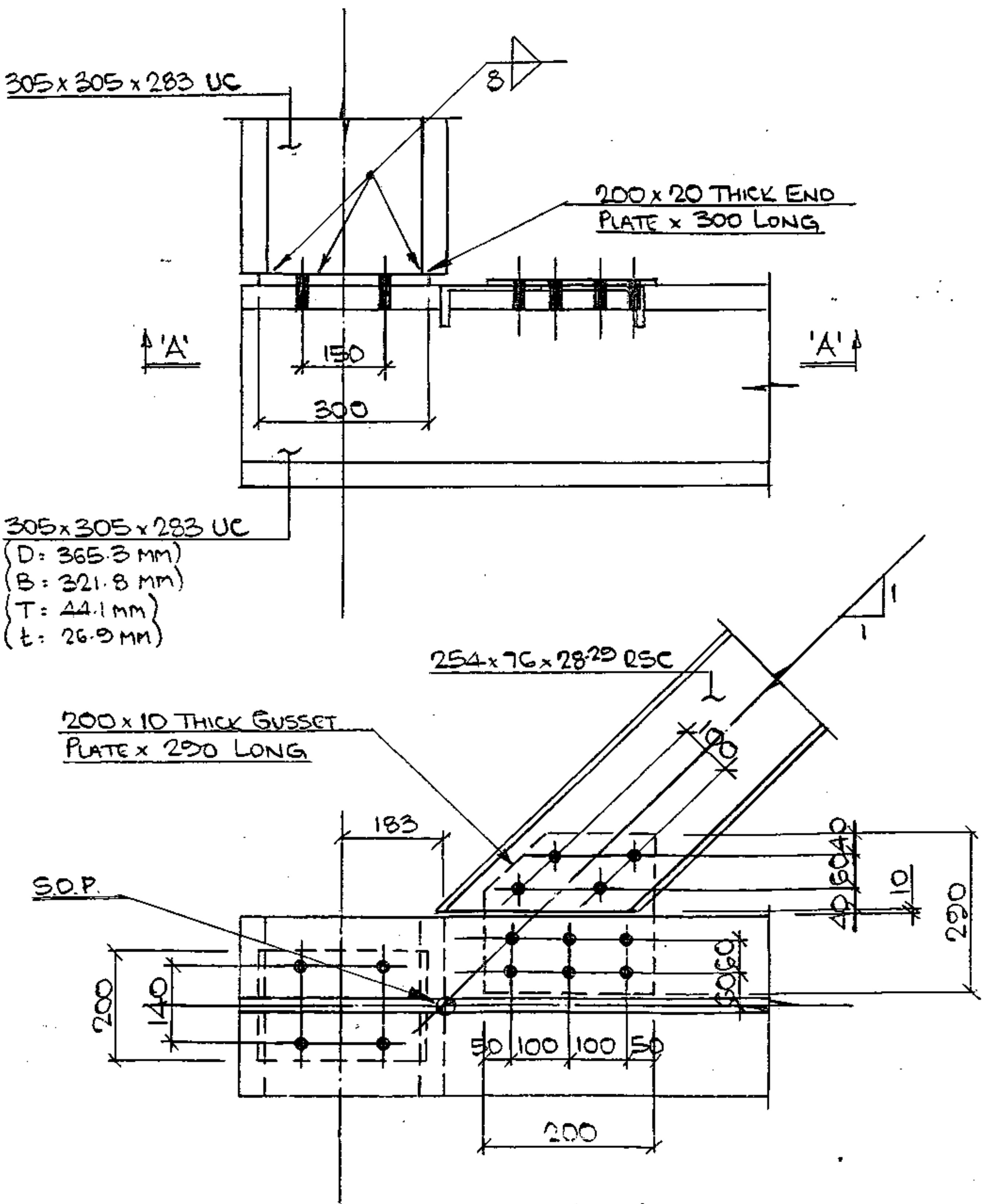
SERIES

SHT. No. WB17 REV.

BY KB DATE 08/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL 'G'



VIEW ON 'A-A'

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

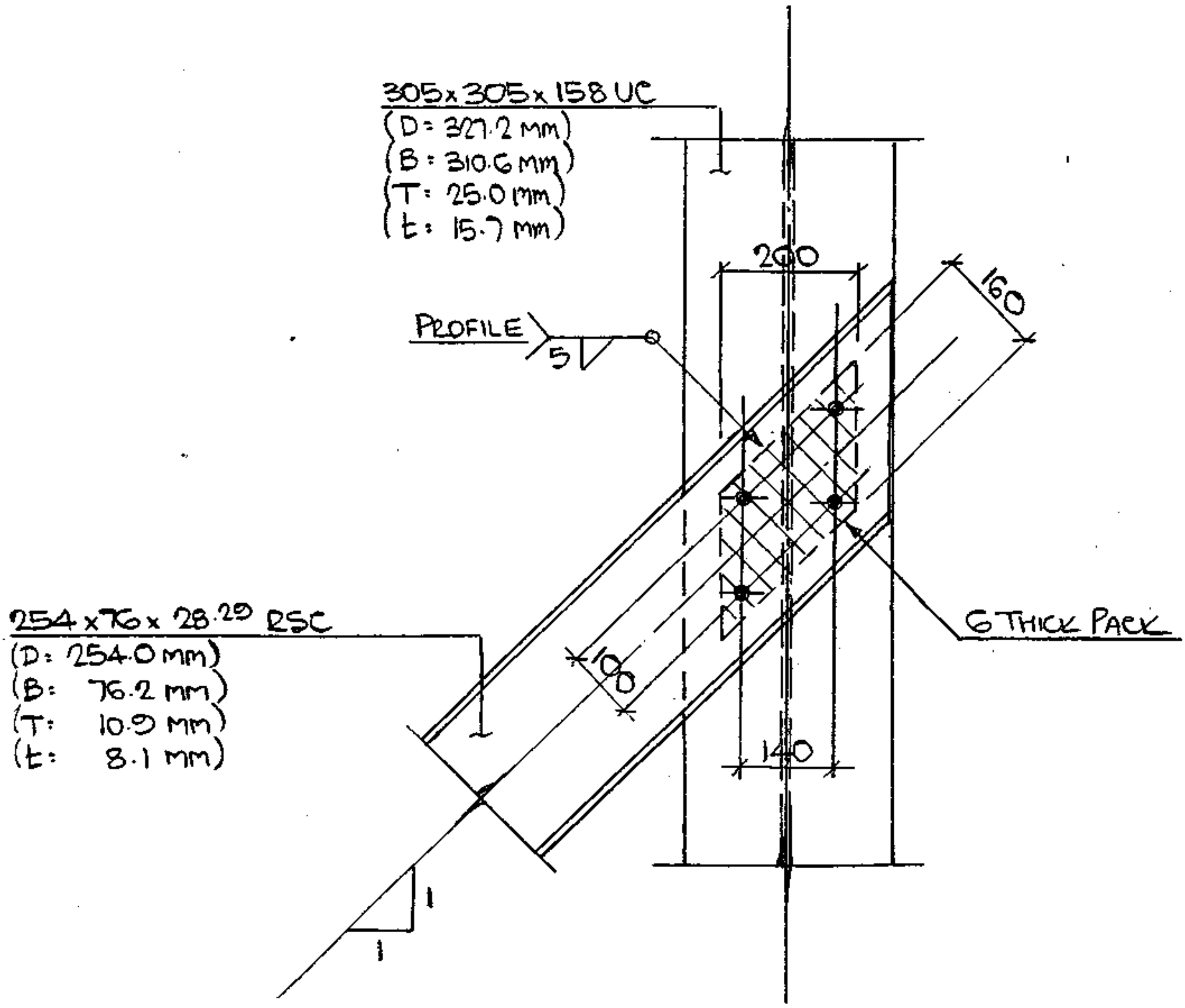
SERIES

SHT. No. WB18 REV.

BY KB DATE 08/98

EX. DATE

TEMPORARY SUPPORT ~ DETAIL '7'



M20 BOLTS (GRADE 8.8)

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ SETTING OUT

SERIES

SHT. No. WB1 REV. B

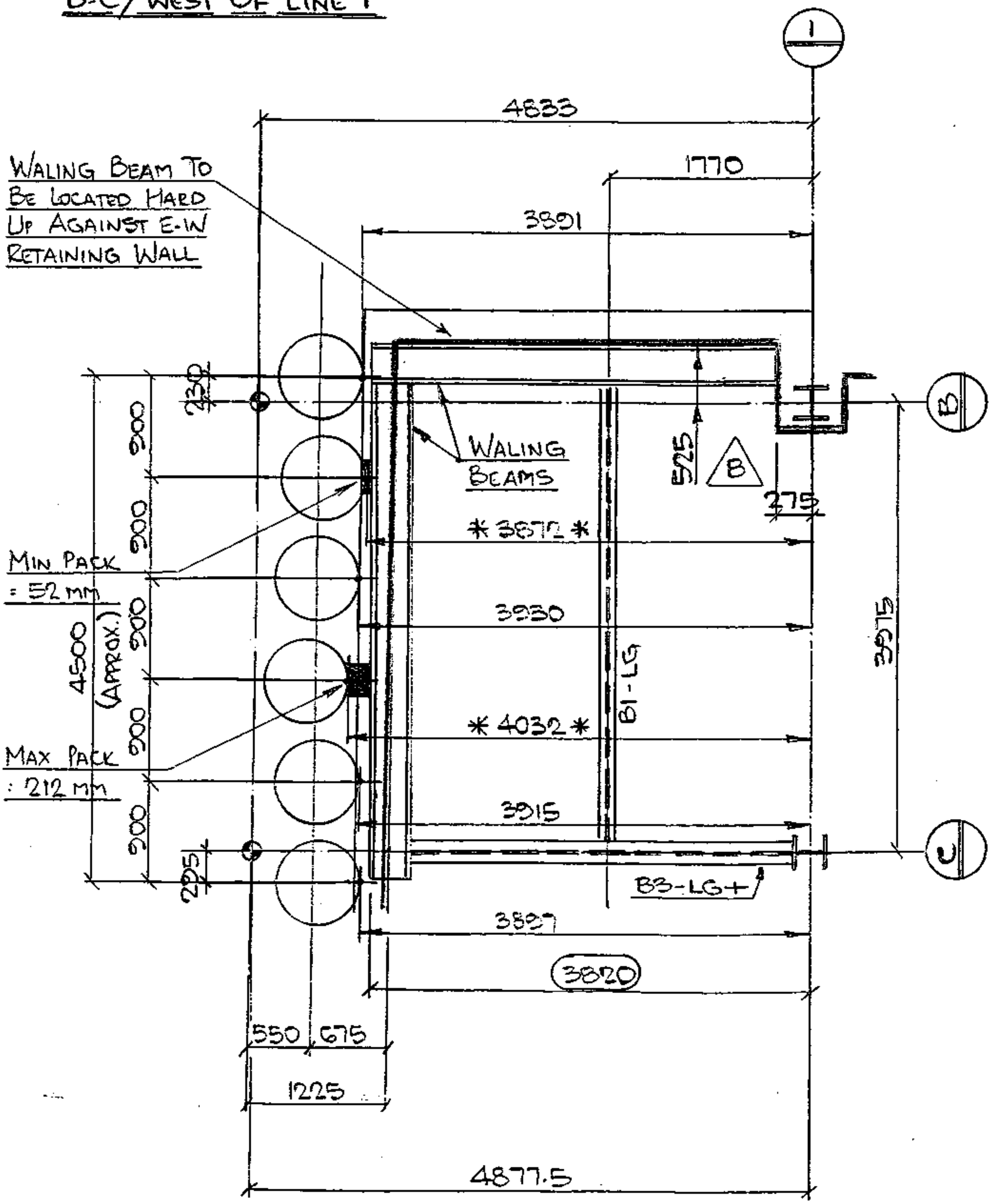
BY KB DATE 08/98

EX. DATE

(CN ~ 32)

SETTING-OUT WALING BEAM

B-C/WEST OF LINE 1



REV	DATE	DESCRIPTION	REF	BY
REV 'B'	14/09/98	REVISED TO SUIT SITE CONDITIONS.	CN~32	KB
REV 'A'	25/08/98	WALING BEAM NORTH OF LINE B/WEST OF LINE 1 ADDED	CN~32	KB

Kvaerner Cleveland Bridge Ltd.

O/No. 325 JOB CARLTON GARDENS

SUBJECT WALING BEAM ~ TEMPORARY SUPPORTS

SERIES

SHT. No. WB2 REV. B

BY KB DATE 08/98

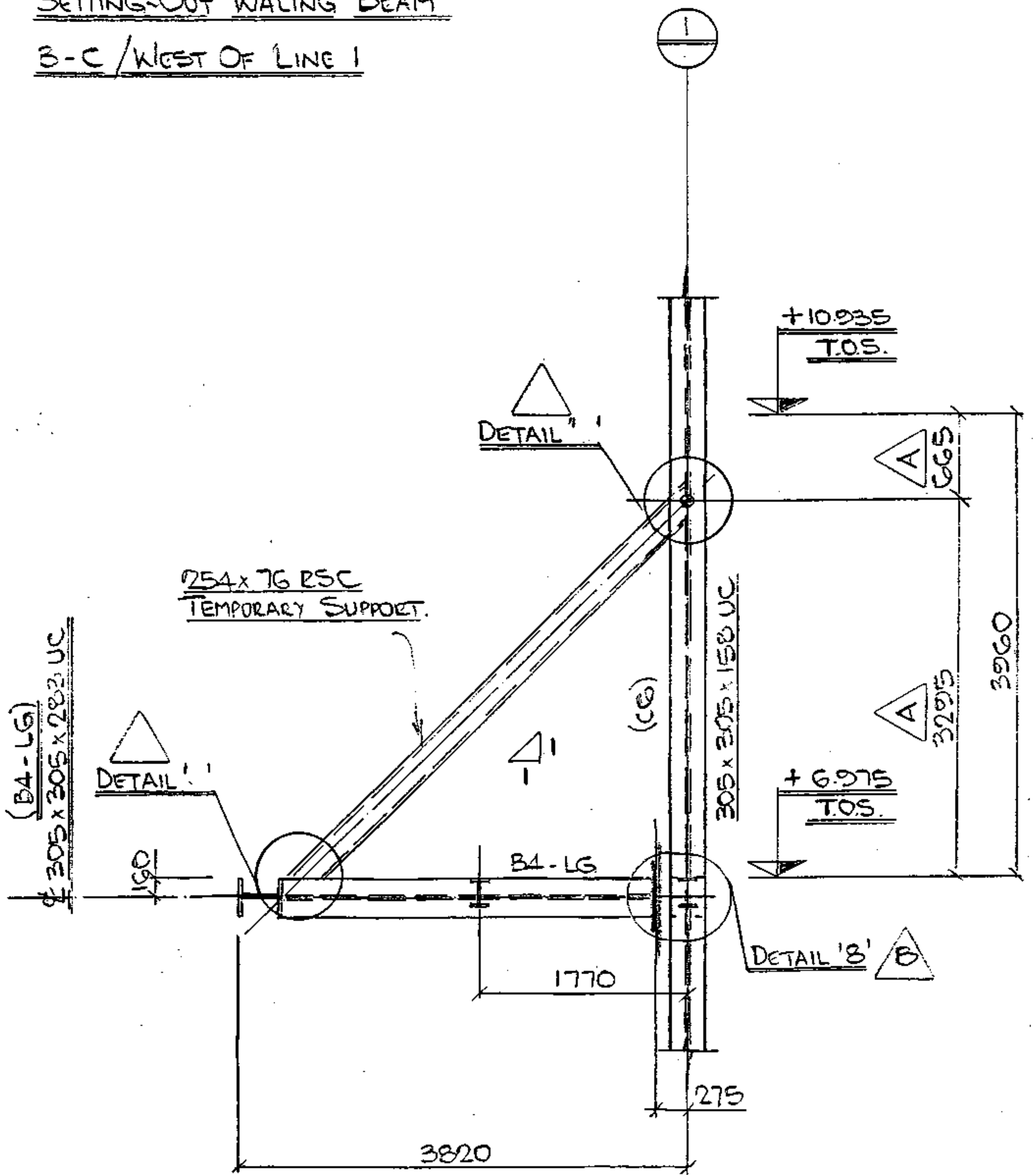
EX. DATE

(CN ~ 32)

REV B	24/09/98	REVISED TO SUIT SITE CONDITIONS	CN~32	KB
REV A	25/08/98	WALING BEAM ADDED		KB

SETTING-OUT WALING BEAM

B-C / WEST OF LINE 1



ELEVATION AT LINE 'B'

(LOOKING NORTH)