C(함) 산운제5

- 2.0 x 2.00 x 1
- 토피 = 7.5m 이하

1. 설계 조건

1) 구조물명 : c (함)

2) 구조형식 : 철근 콘크리트 구조

3) 설계하중 : KRL2012 (복 선)

4) 재료중량

- 철근 콘크리트(γ_c)	=	24.5	kN/m^3
- 무근 콘크리트(γ_{con})	=	23.0	kN/m^3
- 뒷채움 토사(γ_t)	=	20.0	kN/m^3
- 뒷채움 토사(γ_{sub})	=	11.0	kN/m^3
- 지하수(γ_w)	=	10.0	kN/m^3

5) 사용재료의 성질

- 콘크리트 설계기준 강도(fck)	=	30.0	MPa
- 콘크리트 탄성계수(Ec)	=	27537	MPa
- 철근의 항복강도(fy)	=	400.0	MPa
- 철근의 탄성계수(Es)	=	200000	MPa

6) 토 사

- 흙의 내부마찰각 (ϕ)	=	35.0	°
- 정지 토압계수 (Ko)	=	0.426	

7) 기 초 형 식 : 직접기초

8) 사 각(Skew) : 57.00 ° 연 장 (L) : 54.40 m

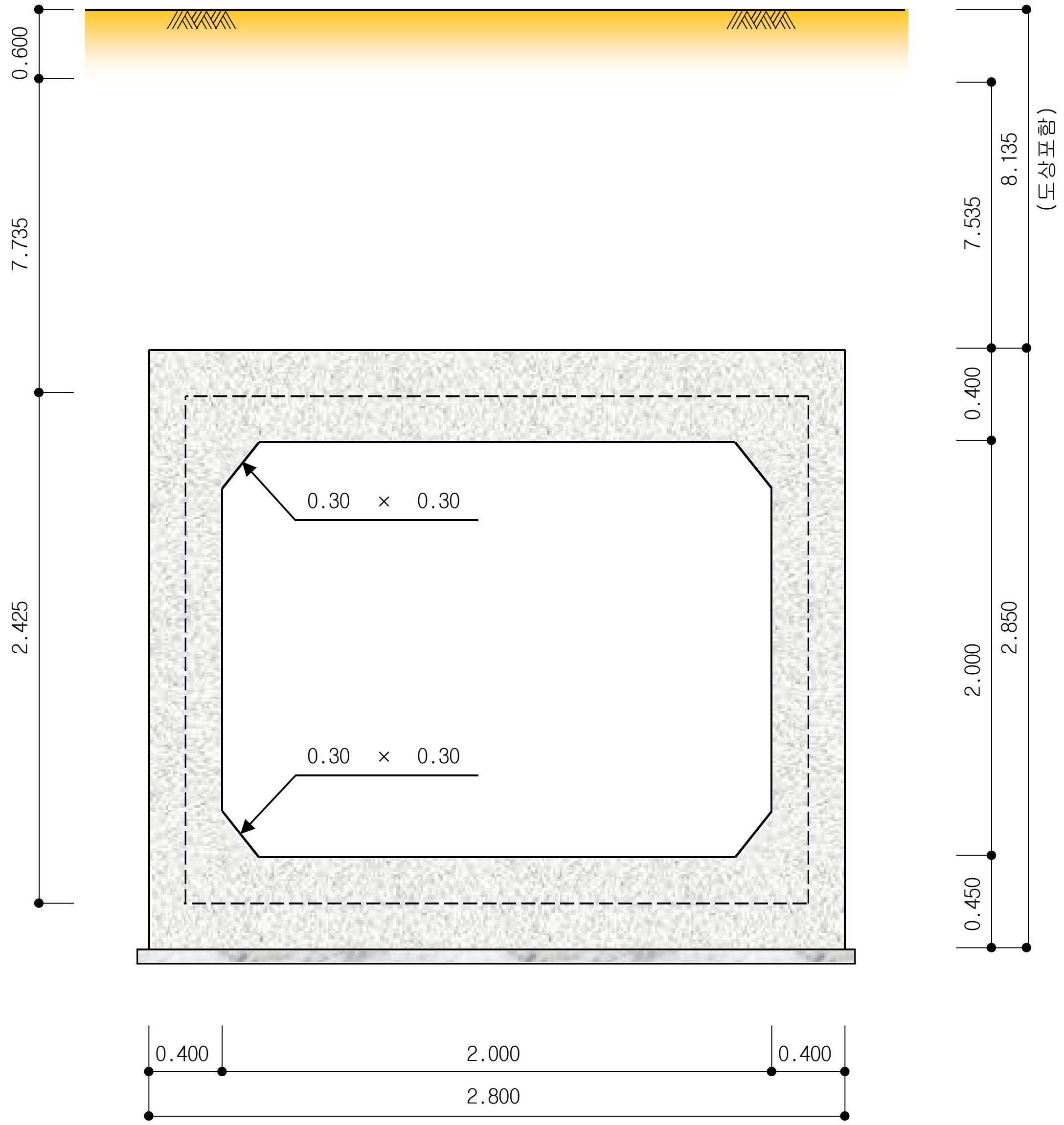
9) 설계방법

- 강도 설계법 : 부재 단면검토(슬래브, 벽체, 보, 기둥 등)
- 허용응력 설계법 : 사용성 검토(처짐, 균열, 우각부 검토)

10) 참고문헌

- 철도설계기준(노반편), 국토해양부, 2013
- 콘크리트 구조설계기준해설, 한국콘크리트 학회, 2012

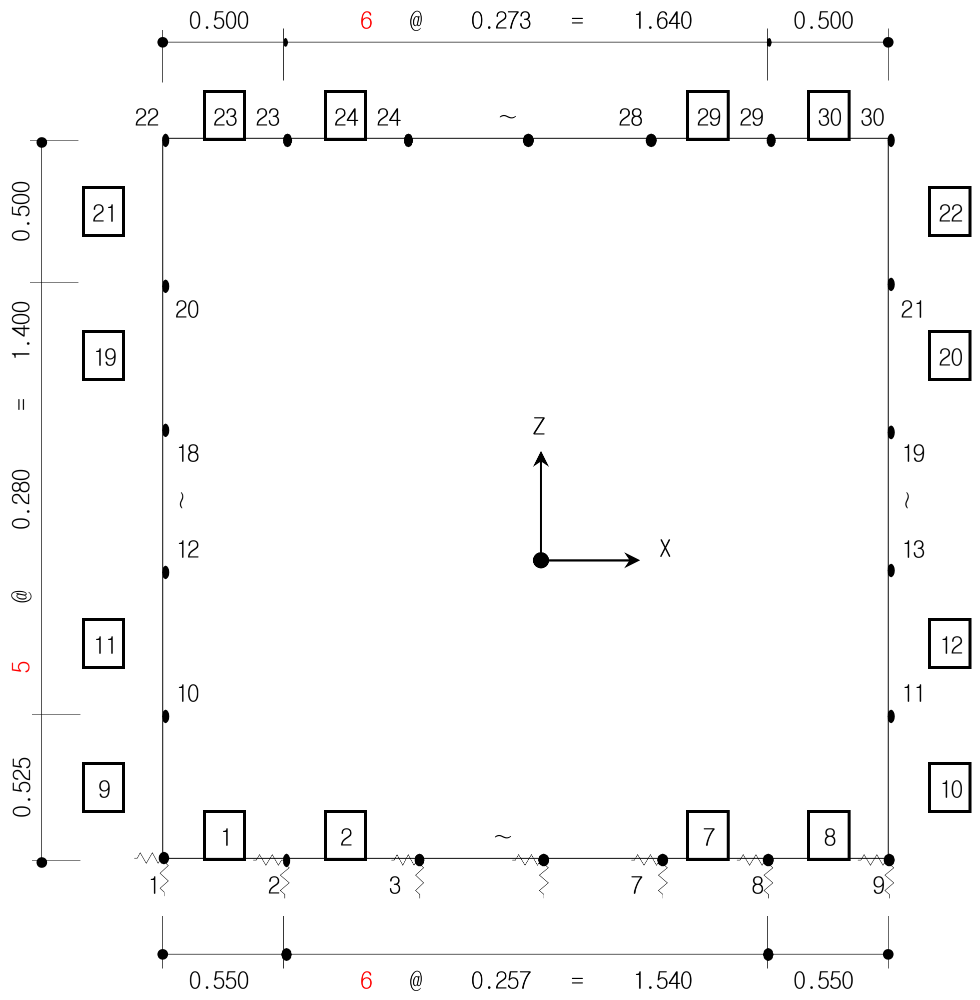
2. 단면 가정



※ 유효경간

Skew =	57.0	°						
Ln =	2.400	m						
Ls =	2.862	m						
Ls/B =	2.862	/	15.000	=	0.191	<	1.500	
L =	2.640	m						

3. 해석모델 및 단면제원



1) 절점 좌표

Joint	좌 표			Joint	좌 표		
	X	Y	Z		X	Y	Z
1	0.000	0.000	0.000	13	2.640	0.000	0.805
2	0.550	0.000	0.000	14	0.000	0.000	1.085
3	0.807	0.000	0.000	15	2.640	0.000	1.085
4	1.063	0.000	0.000	16	0.000	0.000	1.365
5	1.320	0.000	0.000	17	2.640	0.000	1.365
6	1.577	0.000	0.000	18	0.000	0.000	1.645
7	1.833	0.000	0.000	19	2.640	0.000	1.645
8	2.090	0.000	0.000	20	0.000	0.000	1.925
9	2.640	0.000	0.000	21	2.640	0.000	1.925
10	0.000	0.000	0.525	22	0.000	0.000	2.425
11	2.640	0.000	0.525	23	0.500	0.000	2.425
12	0.000	0.000	0.805	24	0.773	0.000	2.425

Joint	좌 표			Joint	좌 표		
	X	Y	Z		X	Y	Z
25	1.047	0.000	2.425	28	1.867	0.000	2.425
26	1.320	0.000	2.425	29	2.140	0.000	2.425
27	1.593	0.000	2.425	30	2.640	0.000	2.425

2) 경계 조건

joint	spring계수 (kN/m²)		joint	spring계수 (kN/m²)	
	z방향(수직)	x방향(수평)		z방향(수직)	x방향(수평)
1 , 9	6074		3 ~ 7	3282	
2 , 8	5157				

3) 단면제원

Element	두께	폭	비 고
23 ~ 30	0.400	1.000	상부슬래브
1 ~ 8	0.450	1.000	하부슬래브
9 ~ 22	0.400	1.000	벽 체

4) 강역설정

(1) 강역설정기준(콘크리트구조설계기준 2012)

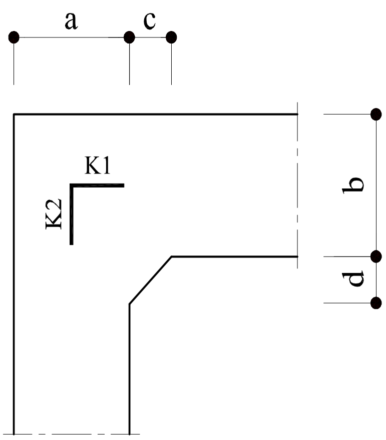
CASE 1 : 부재단부가 다른 부재와 접합할 때는 부재단에서 부재두께의 1/4 안쪽점에서 절점까지

CASE 2 : 부재가 그 축선에 대해 25° 이상 60° 미만인 경사진 헌치를 갖는 경우에는 부재두께가 1.5배가 되는 점에서 부터 절점까지

CASE 3 : 헌치의 경사가 60° 이상의 경우는 헌치의 시점에서 부재두께의 1/4 안쪽 점에서부터 절점까지

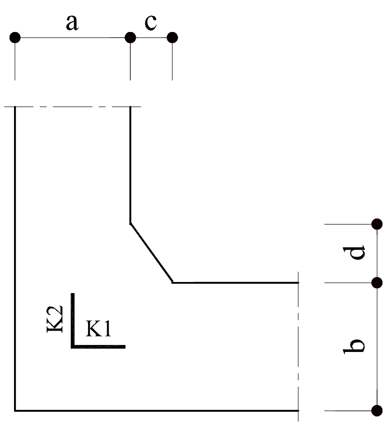
(2) 강역설정

- 절점 22 , 30



a =	400	mm	c =	300	mm
b =	400	mm	d =	300	mm
K1 =	300	mm	K2 =	300	mm

- 절점 1 , 9

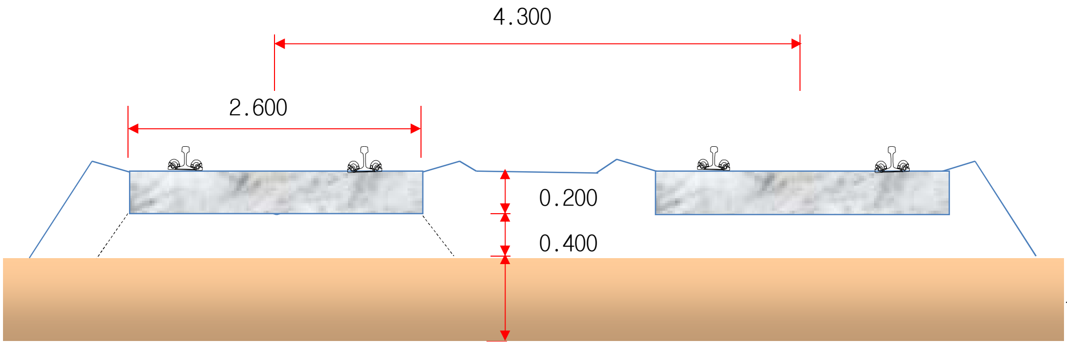


a =	400	mm	c =	300	mm
b =	450	mm	d =	300	mm
K1 =	275	mm	K2 =	325	mm

4. 하중산정

4.1 상시

1) 상부 고정하중(지하수 無)



① 하중 분포폭 산정 (b')

① He < 0.5m 경우 : b + d

b_1 = 2.600 + 0.400 = 3.000 m

② He ≥ 0.5m 경우 : c + b + (d + He)

b_2 = 4.300 + 2.600 + (0.400 + 7.535) = 14.835 m

여기서, He : 시공기면에서의 토피고(m)

a : 동륜간 축간거리(m) c : 복선선로 중심간격(m)

b : 침목길이(m) d : 침목하면의 도상두께(m)

H : 상부슬래브 두께

He ≥ 0.5 m

∴ b' = 14.835 m

② 구조물 자중 : 프로그램 상에서 Self Weight 기능을 이용하여 자동 고려

③ 궤도 중량

- 레일 : 레일당 1.5kN/m를 적용

q_1 = 4 × 1.500 / 14.835 = 0.404 kN/㎡

- 침목 : 고속철도용 5.0kN/m

q_2 = 5.000 × 2 / 14.835 = 0.674 kN/㎡

= 1.078 kN/㎡

④ 도상 (q_2)

TCL = 0.200 × 19.000 × 1.3 = 4.940 kN/㎡

HSB = 0.400 × 19.000 × 1.3 = 9.880 kN/㎡

= 14.820 kN/㎡

⑤ 복토하중 (q_3) = 20.000 × 7.535 = 150.700 kN/㎡

∴ $q_1 + q_2 + q_3$ = 1.078 + 14.820 + 150.700 = 166.598 kN/㎡

2) 활하중

① 활하중 산정 : KRL 2012

$$W_{HL} = \frac{2P}{a \times b'} = \frac{2\times(220+80x3)}{3.000 \times 14.835} = 20.672 \text{ kN/㎡}$$

∴ W = 20.672 kN/㎡

② 충격계수

1) 단선 (i_0) (철도설계기준 노반편(2013) p.5-7)

지 간 L (m)	0	5	10	20	30	40	50	70	100
충격계수 (i)	0.60	0.48	0.43	0.37	0.34	0.32	0.30	0.27	0.24

* 상기표에 없는 지간의 충격 계수는 보간법으로 구한다.

지 간 L (m) = 2.640 m

따라서 i_0 = 0.537 (보간법에 의하여 산정된 충격계수)

2) 구조물 상면의 복토높이에 따른 충격 계수

H ≥ 1.0일 경우 i = io·{(2.5 - H)/1.5}

여기서, H ≤ 1.0 m 에서는 i = i_o

H ≥ 2.5 m 에서는 i = 0

따라서, 본 구조물의 H = 8.135 m i = 0.000

W_L = 20.672 × (1 + 0.000) = 20.672 kN/㎡

③ 활하중에 의한 측압 (충격비고려) (철도설계기준 노반편(2013) p.5-6)

W_{LH} = 20.672 × 0.4 = 8.269 kN/㎡

3) 토 압

- 지하수가 없는 경우 작용 토압

q_1 = 0.426 × (15.898 + 20.000 × 7.735) = 72.675 kN/㎡

q_2 = 72.675 + 0.426 × (20.000 × 0.500) = 76.935 kN/㎡

q_3 = 76.935 + 0.426 × (20.000 × 0.280) = 79.321 kN/㎡

q_4 = 79.321 + 0.426 × (20.000 × 0.280) = 81.707 kN/㎡

q_5 = 81.707 + 0.426 × (20.000 × 0.280) = 84.093 kN/㎡

q_6 = 84.093 + 0.426 × (20.000 × 0.280) = 86.479 kN/㎡

q_7 = 86.479 + 0.426 × (20.000 × 0.280) = 88.865 kN/㎡

q_8 = 88.865 + 0.426 × (20.000 × 0.525) = 93.338 kN/㎡

- 시공중 편토압 산정 (지하수가 없는 경우 작용 토압)

q_1 =	0.426	×(	0.000	+	20.000	×	0.200	)	=	1.704 kN/㎡
q_2 =	1.704	+	0.426	×(	20.000	×	0.500	)	=	5.964 kN/㎡
q_3 =	5.964	+	0.426	×(	20.000	×	0.280	)	=	8.350 kN/㎡
q_4 =	8.350	+	0.426	×(	20.000	×	0.280	)	=	10.736 kN/㎡
q_5 =	10.736	+	0.426	×(	20.000	×	0.280	)	=	13.122 kN/㎡
q_6 =	13.122	+	0.426	×(	20.000	×	0.280	)	=	15.508 kN/㎡
q_7 =	15.508	+	0.426	×(	20.000	×	0.280	)	=	17.894 kN/㎡
q_8 =	17.894	+	0.426	×(	20.000	×	0.525	)	=	22.367 kN/㎡

4) 온도하중

토피가 1.0m 이상일 경우에는 온도변화를 고려치 않으며, 토피가 1.0m 이하일 경우에는 다음식을 따른다.

$t = a \times t_0$ = 0.000 × 15.0 = 0.000 ℃

t : 박스 구조물의 상부 슬래브에 작용하는 온도변화

a : 토피심도(m)에 따른 저감률 $a=1-h_0$ (h_0:토피심도)

t_0 : 지표면에서의 기본온도 변화

상부슬래브의 두께가 700mm 이상인 경우 t_0 = ±10℃

상부슬래브의 두께가 700mm 미만인 경우 t_0 = ±15℃

토피고 H > 1.0m 이므로 온도변화는 고려하지 않는다.

5) 건조수축

$$\Delta Tx = \frac{\varepsilon}{\alpha} = \frac{-0.00015}{0.00001} = -15\,^{\circ}C$$

6) 장대레일 하중

장대레일 종하중은 필요시에 고려하여야 하며, 1궤도당 10 kN/m로 하고 작용위치는 상부슬래브 축선으로 한다. (단, 토피가 1.0m 이상인 경우는 이를 무시한다.)

토피가 8.135 m 이므로 장대레일 하중을 무시한다.

H_r = 0.000 × 4 / 14.835 = 0.000 kN/㎡

7) 상시 지반반력 계수

지지 지반 N치 = 7

Eo = 2800 × N = 2800 × 7 = 19600

Kvo = 1 / 0.3 x α x Eo = 1 / 0.3 × 1 x 19600 = 65333

Bv = √ Av = √ (2.640 × 2.640) = 2.640

Kv = Kvo (Bv/0.3)^-3/4 = 12787 kN/㎡

12787 × (0.400 / 2 + 0.550 / 2) = 6074 kN/m² - 저판 단부 스프링

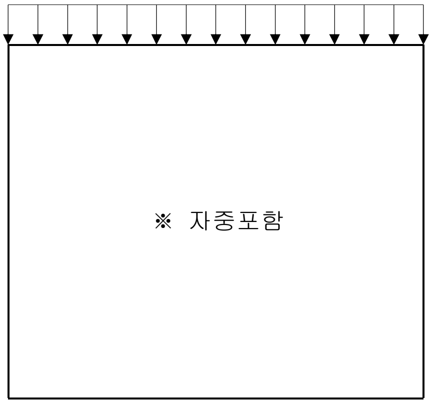
12787 × (0.550 / 2 + 0.257 / 2) = 5157 kN/m² - 저판 전단검토위치

12787 × (0.257 / 2 + 0.257 / 2) = 3282 kN/m² - 저판 중앙부 스프링

5. 하중재하도

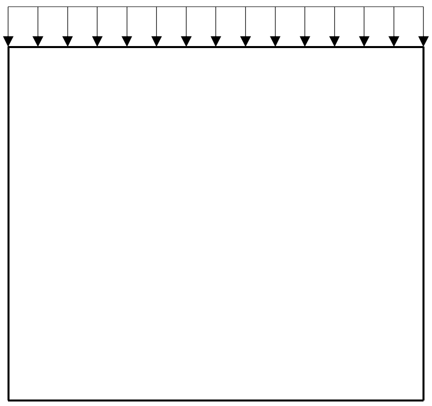
① 고정하중

W_D = 166.598 kN/㎡



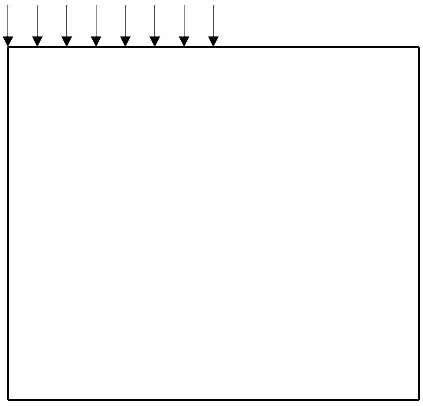
② 활하중(만재하)

W_L = 20.672 kN/㎡

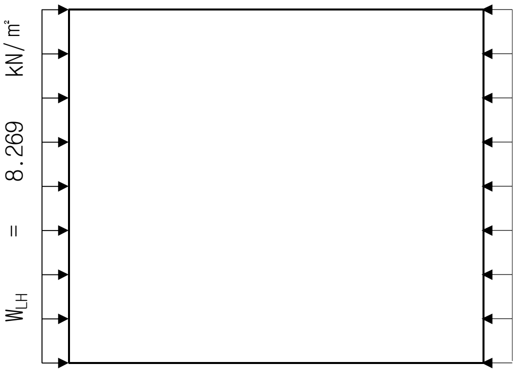


③ 활하중(편재하)

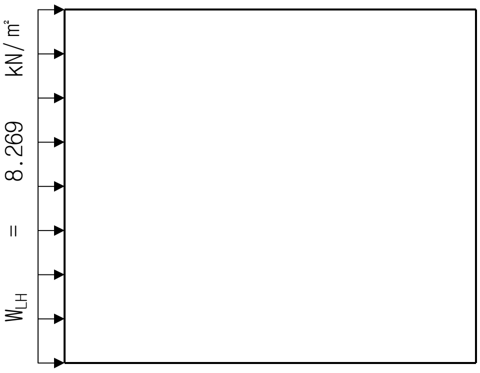
W_L = 20.672 kN/㎡



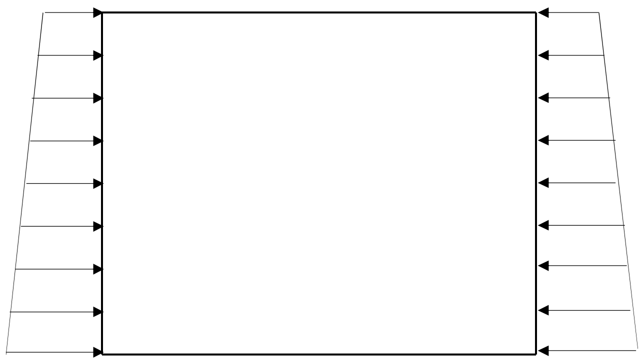
④ 활하중에 의한 측압(만재하)



⑤ 활하중에 의한 측압(편재하)



⑥ 수평토압(지하수 없는 경우)

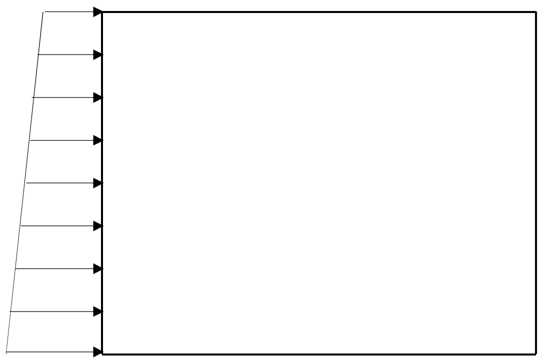


q_1 = 72.675 kN/㎡

q_7 = 93.338 kN/㎡

⑦ 편측토압(지하수 없는 경우)

q_1 = 1.704 kN/㎡



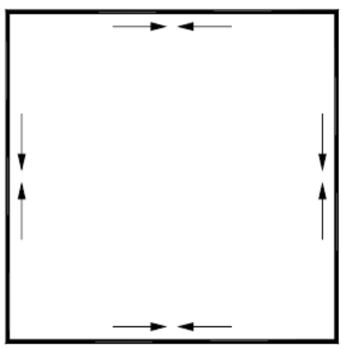
q_7 = 22.367 kN/㎡

⑧ 온도하중

△T = 0 °

⑨ 건조수축

△T = -15 °



⑩ 장대레일 종하중

Hr = 0.000 kN/m^2

6. 하중계수 및 하중조합

1) 극한하중 검토시

구분	고정 하중	활하중		활하중 측압		수평토압 (지하수無)		온도 하중		건조 수축	장대 레일	비고
				양측재하	편측재하	양측재하	편측재하					
		전경간	1/2	수평	수평	수평	수평	온도상승	온도하강			
	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	LC9	LC10	LC11	
1	1.35	1.85		1.85		1.6						
2	1.35	1.85		1.85		0.6						
3	1.35		1.85		1.85	1.6						
4	1.35		1.85		1.85	0.6						
5	1.6	1.60		1.60		1.6						
6	1.6	1.60		1.60		0.6						
7	1.6		1.6		1.6	1.6						
8	1.6		1.6		1.6	0.6						
9	1.35	1.40		1.40		1.6		1.35		1.35		
10	1.35	1.40		1.40		0.6		1.35		1.35		
11	1.35	1.40		1.40		1.6			1.35	1.35	1.40	
12	1.35	1.40		1.40		0.6			1.35	1.35	1.40	
13	1.35		1.40		1.40	1.6		1.35		1.35		
14	1.35		1.40		1.40	0.6		1.35		1.35		
15	1.35		1.40		1.40	1.6			1.35	1.35	1.4	
16	1.35		1.40		1.40	0.6			1.35	1.35	1.4	
17	1.35		1.85		1.85		1.60					
18	1.35		1.85		1.85		0.60					
19	1.60		1.60		1.60		1.60					
20	1.60		1.60		1.60		0.60					

2) 사용하중 검토시

구분	고정 하중	활하중		활하중 측압		수평토압 (지하수無)		온도 하중		건조 수축	장대 레일	비고
				양측재하	편측재하	양측재하	편측재하					
		전경간	1/2	수평	수평	수평	수평	온도상승	온도하강			
	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	LC9	LC10	LC11	
21	1.0	1.0		1.0		1.0						
22	1.0	1.0		1.0		0.5						
23	1.0		1.0		1.0	1.0						
24	1.0		1.0		1.0	0.5						
25	1.0	1.0		1.0		1.0		1.0			1.0	
26	1.0	1.0		1.0		0.5		1.0			1.0	
27	1.0	1.0		1.0		1.0			1.0	1.0	1.0	
28	1.0	1.0		1.0		0.5			1.0	1.0	1.0	
29	1.0		1.0		1.0	1.0		1.0			1.0	
30	1.0		1.0		1.0	0.5		1.0			1.0	
31	1.0		1.0		1.0	1.0			1.0	1.0	1.0	
32	1.0		1.0		1.0	0.5			1.0	1.0	1.0	
33	1.0					1.0						
34	1.0		1.0		1.0		1.0					
35	1.0		1.0		1.0		0.5					

7. INPUT DATA

```
*VERSION
   8.0.0

*UNIT    ; Unit System
; FORCE, LENGTH, HEAT, TEMPER
   KN   , M, J, C

*STRUCTYPE    ; Structure Type
; iSTYP, iMASS, iSMAS, bMASSOFFSET, bSELFWEIGHT, GRAV, TEMPER, bALIGNBEAM, bALIGNSLAB
     1, 1, 1, NO, YES, 9.806, 0, NO, NO

*REBAR-MATL-CODE    ; Rebar Material Code
; CONC_CODE, CONC_MDB, SRC_CODE, SRC_MDB
 KS01-Civil(RC), SD300, KS01-Civil(RC), SD300

*NODE    ; Nodes
; iNO, X, Y, Z
     1, 0, 0, 0
     2, 0.55, 0, 0
     3, 0.807, 0, 0
     4, 1.063, 0, 0
     5, 1.32, 0, 0
     6, 1.577, 0, 0
     7, 1.833, 0, 0
     8, 2.09, 0, 0
     9, 2.64, 0, 0
    10, 0, 0, 0.525
    11, 2.64, 0, 0.525
    12, 0, 0, 0.805
    13, 2.64, 0, 0.805
    14, 0, 0, 1.085
    15, 2.64, 0, 1.085
    16, 0, 0, 1.365
    17, 2.64, 0, 1.365
    18, 0, 0, 1.645
    19, 2.64, 0, 1.645
    20, 0, 0, 1.925
    21, 2.64, 0, 1.925
    22, 0, 0, 2.425
    23, 0.5, 0, 2.425
    24, 0.773, 0, 2.425
    25, 1.047, 0, 2.425
    26, 1.32, 0, 2.425
    27, 1.593, 0, 2.425
    28, 1.867, 0, 2.425
    29, 2.14, 0, 2.425
    30, 2.64, 0, 2.425

*ELEMENT    ; Elements
; iEL, TYPE, iMAT, iPRO, iN1, iN2, ANGLE, iSUB, EXVAL, iOPT(EXVAL2) ; Frame  Element
; iEL, TYPE, iMAT, iPRO, iN1, iN2, ANGLE, iSUB, EXVAL, EXVAL2, bLMT ; Comp/Tens Truss
; iEL, TYPE, iMAT, iPRO, iN1, iN2, iN3, iN4, iSUB, iWID             ; Planar Element
; iEL, TYPE, iMAT, iPRO, iN1, iN2, iN3, iN4, iN5, iN6, iN7, iN8     ; Solid  Element
; iEL, TYPE, iMAT, iPRO, iN1, iN2, REF, RPX, RPY, RPZ, iSUB, EXVAL  ; Frame(Ref. Point)
     1, BEAM  ,    2,     2,     1,     2,   180
     2, BEAM  ,    2,     2,     2,     3,   180
     3, BEAM  ,    2,     2,     3,     4,   180
     4, BEAM  ,    2,     2,     4,     5,   180
     5, BEAM  ,    2,     2,     5,     6,   180
     6, BEAM  ,    2,     2,     6,     7,   180
     7, BEAM  ,    2,     2,     7,     8,   180
     8, BEAM  ,    2,     2,     8,     9,   180
     9, BEAM  ,    2,     3,     1,    10,   180
    10, BEAM  ,    2,     3,     9,    11,     0
    11, BEAM  ,    2,     3,    10,    12,   180
    12, BEAM  ,    2,     3,    11,    13,     0
    13, BEAM  ,    2,     3,    12,    14,   180
    14, BEAM  ,    2,     3,    13,    15,     0
    15, BEAM  ,    2,     3,    14,    16,   180
    16, BEAM  ,    2,     3,    15,    17,     0
    17, BEAM  ,    2,     3,    16,    18,   180
    18, BEAM  ,    2,     3,    17,    19,     0
    19, BEAM  ,    2,     3,    18,    20,   180
    20, BEAM  ,    2,     3,    19,    21,     0
    21, BEAM  ,    2,     3,    20,    22,   180
    22, BEAM  ,    2,     3,    21,    30,     0
    23, BEAM  ,    2,     1,    22,    23,     0
    24, BEAM  ,    2,     1,    23,    24,     0
    25, BEAM  ,    2,     1,    24,    25,     0
    26, BEAM  ,    2,     1,    25,    26,     0
    27, BEAM  ,    2,     1,    26,    27,     0
    28, BEAM  ,    2,     1,    27,    28,     0
    29, BEAM  ,    2,     1,    28,    29,     0
    30, BEAM  ,    2,     1,    29,    30,     0
```

```
*MATERIAL    ; Material
; iMAT, TYPE, MNAME, SPHEAT, HEATCO, PLAST, TUNIT, bMASS, DAMPRATIO, [DATA1]           ; STEEL, CONC, USER
; iMAT, TYPE, MNAME, SPHEAT, HEATCO, PLAST, TUNIT, bMASS, DAMPRATIO, [DATA2], [DATA2] ; SRC
; [DATA1] : 1, DB, NAME, CODE
; [DATA1] : 2, ELAST, POISN, THERMAL, DEN, MASS
; [DATA1] : 3, Ex, Ey, Ez, Tx, Ty, Tz, Sxy, Sxz, Syz, Pxy, Pxz, Pyz, DEN, MASS   ; Orthotropic
; [DATA2] : 1, DB, NAME, CODE or 2, ELAST, POISN, THERMAL, DEN, MASS
    1, USER , STEEL            , 0, 0, , C, NO, 0, 2, 2.0000e+008,   0.3, 1.1700e-005, 76.8196,     0
    2, USER , CONC             , 0, 0, , C, NO, 0, 2, 2.7537e+007,  0.18, 1.0000e-005,  24.5,     0

*MATL-COLOR
; iMAT, W_R, W_G, W_B, HF_R, HF_G, HF_B, HE_R, HE_G, HE_B, bBLEND, FACT
    1, 255,   0,   0,    0, 255,   0,    0,   0, 255,  NO, 0.5
    2, 255,   0,   0,    0, 255,   0,    0,   0, 255,  NO, 0.5


*SECTION    ; Section
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, [DATA1], [DATA2]                    ; 1st line - DB/USER
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, BLT, D1, ..., D8, iCEL              ; 1st line - VALUE
;       AREA, ASy, ASz, Ixx, Iyy, Izz                                          ; 2nd line
;       CyP, CyM, CzP, CzM, QyB, QzB, PERI_OUT, PERI_IN, Cy, Cz                ; 3rd line
;       Y1, Y2, Y3, Y4, Z1, Z2, Z3, Z4, Zyy, Zzz                               ; 4th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, ELAST, DEN, POIS, POIC, SF          ; 1st line - SRC
;       D1, D2, [SRC]                                                          ; 2nd line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, 1, DB, NAME1, NAME2, D1, D2         ; 1st line - COMBINED
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, 2, D11, D12, D13, D14, D15, D21, D22, D23, D24
; iSEC, TYPE, SNAME, [OFFSET2], bSD, SHAPE, iyVAR, izVAR, STYPE                ; 1st line - TAPERED
;       DB, NAME1, NAME2                                                       ; 2nd line(STYPE=DB)
;       [DIM1], [DIM2]                                                         ; 2nd line(STYPE=USER)
;       D11, D12, D13, D14, D15, D16, D17, D18                                 ; 2nd line(STYPE=VALUE)
;       AREA1, ASy1, ASz1, Ixx1, Iyy1, Izz1                                    ; 3rd line(STYPE=VALUE)
;       CyP1, CyM1, CzP1, CzM1, QyB1, QzB1, PERI_OUT1, PERI_IN1, Cy1, Cz1      ; 4th line(STYPE=VALUE)
;       Y11, Y12, Y13, Y14, Z11, Z12, Z13, Z14, Zyy1, Zyy2                     ; 5th line(STYPE=VALUE)
;       D21, D22, D23, D24, D25, D26, D27, D28                                 ; 6th line(STYPE=VALUE)
;       AREA2, ASy2, ASz2, Ixx2, Iyy2, Izz2                                    ; 7th line(STYPE=VALUE)
;       CyP2, CyM2, CzP2, CzM2, QyB2, QzB2, PERI_OUT2, PERI_IN2, Cy2, Cz2      ; 8th line(STYPE=VALUE)
;       Y21, Y22, Y23, Y24, Z21, Z22, Z23, Z24, Zyy2, Zzz2                     ; 9th line(STYPE=VALUE)
;       OPT1, OPT2, [JOINT]                                                    ; 2nd line(STYPE=PSC)
;       ELAST, DEN, POIS, POIC                                                 ; 2nd line(STYPE=PSC-CMPW)
;       bSHEARCHK, [SCHK-I], [SCHK-J], [WT-I], [WT-J], WI, WJ, bSYM, bSIDEHOLE ; 3rd line(STYPE=PSC)
;       bSHEARCHK, bSYM, bHUNCH, [CMPWEB-I], [CMPWEB-J]                        ; 3rd line(STYPE=PSC-CMPW)
;       bUSERDEFMESHSIZE, MESHSIZE, bUSERINPSTIFF, [STIFF-I], [STIFF-J]        ; 4th line(STYPE=PSC)
;       [SIZE-A]-i                                                             ; 5th line(STYPE=PSC)
;       [SIZE-B]-i                                                             ; 6th line(STYPE=PSC)
;       [SIZE-C]-i                                                             ; 7th line(STYPE=PSC)
;       [SIZE-D]-i                                                             ; 8th line(STYPE=PSC)
;       [SIZE-A]-j                                                             ; 9th line(STYPE=PSC)
;       [SIZE-B]-j                                                             ; 10th line(STYPE=PSC)
;       [SIZE-C]-j                                                             ; 11th line(STYPE=PSC)
;       [SIZE-D]-j                                                             ; 12th line(STYPE=PSC)
;       GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMULTI, EsEc-L, EsEc-S        ; 2nd line(STYPE=CMP-B/I)
;       SW_i, Hw_i, tw_i, B_i, Bf1_i, tf1_i, B2_i, Bf2_i, tf2_i                ; 3rd line(STYPE=CMP-B/I)
;       SW_j, Hw_j, tw_j, B_j, Bf1_j, tf1_j, B2_j, Bf2_j, tf2_j                ; 4th line(STYPE=CMP-B/I)
;       N1, N2, Hr, Hr2, tr1, tr2                                              ; 5th line(STYPE=CMP-B)
;       GN, CTC, Bc, Tc, Hh, EgdEsb, DgdDsb, Pgd, Psb, bSYM, SW_i, SW_j        ; 2nd line(STYPE=CMP-CI/CT)
;       OPT1, OPT2, [JOINT]                                                    ; 3rd line(STYPE=CMP-CI/CT)
;       [SIZE-A]-i                                                             ; 4th line(STYPE=CMP-CI/CT)
;       [SIZE-B]-i                                                             ; 5th line(STYPE=CMP-CI/CT)
;       [SIZE-C]-i                                                             ; 6th line(STYPE=CMP-CI/CT)
;       [SIZE-D]-i                                                             ; 7th line(STYPE=CMP-CI/CT)
;       [SIZE-A]-j                                                             ; 8th line(STYPE=CMP-CI/CT)
;       [SIZE-B]-j                                                             ; 9th line(STYPE=CMP-CI/CT)
;       [SIZE-C]-j                                                             ; 10th line(STYPE=CMP-CI/CT)
;       [SIZE-D]-j                                                             ; 11th line(STYPE=CMP-CI/CT)
; iSEC, TYPE, SNAME, [OFFSET], bSD, STYPE1, STYPE2                             ; 1st line - CONSTRUCT
;       SHAPE, ...(same with other type data from shape)                       ; Before (STYPE1)
;       SHAPE, ...(same with other type data from shape)                       ; After  (STYPE2)
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-SB
;       Hw, tw, B, Bf1, tf1, B2, Bf2, tf2                                      ; 2nd line
;       N1, N2, Hr, Hr2, tr1, tr2                                              ; 3rd line
;       SW, GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMulti, Elong, Esh        ; 4th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-SI
;       Hw, tw, B, tf1, B2, tf2                                                ; 2nd line
;       SW, GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMulti, Elong, Esh        ; 3rd line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-CI/CT
;       OPT1, OPT2, [JOINT]                                                    ; 2nd line
;       [SIZE-A]                                                               ; 3rd line
;       [SIZE-B]                                                               ; 4th line
;       [SIZE-C]                                                               ; 5th line
;       [SIZE-D]                                                               ; 6th line
;       SW, GN, CTC, Bc, Tc, Hh, EgdEsb, DgdDsb, Pgd, Psb                      ; 7th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - PSC
;       OPT1, OPT2, [JOINT]                                                    ; 2nd line
;       bSHEARCHK, [SCHK], [WT], WIDTH, bSYM, bSIDEHOLE                        ; 3rd line
;       bUSERDEFMESHSIZE, MESHSIZE, bUSERINPSTIFF, [STIFF]                     ; 4th line
;       [SIZE-A]                                                               ; 5th line
;       [SIZE-B]                                                               ; 6th line
;       [SIZE-C]                                                               ; 7th line
;       [SIZE-D]                                                               ; 8th line
```

```
; [DATA1] : 1, DB, NAME or 2, D1, D2, D3, D4, D5, D6, D7, D8, D9, D10
; [DATA2] : CCSHAPE or iCEL or iN1, iN2
; [SRC]  : 1, DB, NAME1, NAME2 or 2, D1, D2, D3, D4, D5, D6, D7, D8, D9, D10, iN1, iN2
; [DIM1], [DIM2] : D1, D2, D3, D4, D5, D6, D7, D8
; [OFFSET] : OFFSET, iCENT, iREF, iHORZ, HUSER, iVERT, VUSER
; [OFFSET2]: OFFSET, iCENT, iREF, iHORZ, HUSERI, HUSERJ, iVERT, VUSERI, VUSERJ
; [JOINT]  :  8(1CELL, 2CELL), 13(3CELL),  9(PSCM),  8(PSCH), 9(PSCT),  2(PSCB), 0(nCELL),  2(nCEL2)
; [SIZE-A] :  6(1CELL, 2CELL), 10(3CELL), 10(PSCM),  6(PSCH), 8(PSCT), 10(PSCB), 5(nCELL), 11(nCEL2)
; [SIZE-B] :  6(1CELL, 2CELL), 12(3CELL),  6(PSCM),  6(PSCH), 8(PSCT),  6(PSCB), 8(nCELL), 18(nCEL2)
; [SIZE-C] : 10(1CELL, 2CELL), 13(3CELL),  9(PSCM), 10(PSCH), 7(PSCT),  8(PSCB), 0(nCELL), 11(nCEL2)
; [SIZE-D] :  8(1CELL, 2CELL), 13(3CELL),  6(PSCM),  7(PSCH), 8(PSCT),  5(PSCB), 0(nCELL), 18(nCEL2)
; [STIFF]  : AREA, ASy, ASz, Ixx, Iyy, Izz
; [SCHK]   : bAUTO_Z1, Z1, bAUTO_Z3, Z3
; [WT]     : bAUTO_TOR, TOR, bAUTO_SHR1, SHR1, bAUTO_SHR2, SHR2, bAUTO_SHR3, SHR3
; [CMPWEB] : EFD, LRF, A, B, H, T


    1, DBUSER    , TOPSLAB           , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.4, 1, 0, 0, 0, 0, 0, 0, 0, 0
    2, DBUSER    , BOTSLAB           , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.45, 1, 0, 0, 0, 0, 0, 0, 0, 0
    3, DBUSER    , WALL              , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.4, 1, 0, 0, 0, 0, 0, 0, 0, 0

*SECT-COLOR
; iSEC, W_R, W_G, W_B, HF_R, HF_G, HF_B, HE_R, HE_G, HE_B, bBLEND, FACT
    1, 255,   0,   0,    0, 255,   0,    0,   0, 255,  NO, 0.5
    2, 255,   0,   0,    0, 255,   0,    0,   0, 255,  NO, 0.5
    3, 255,   0,   0,    0, 255,   0,    0,   0, 255,  NO, 0.5

*DGN-SECT
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, [DATA1], [DATA2]                    ; 1st line - DB/USER
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, BLT, D1, ..., D8, iCEL              ; 1st line - VALUE
;       AREA, ASy, ASz, Ixx, Iyy, Izz                                          ; 2nd line
;       CyP, CyM, CzP, CzM, QyB, QzB, PERI_OUT, PERI_IN, Cy, Cz                ; 3rd line
;       Y1, Y2, Y3, Y4, Z1, Z2, Z3, Z4, Zyy, Zzz                               ; 4th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, ELAST, DEN, POIS, POIC, SF          ; 1st line - SRC
;       D1, D2, [SRC]                                                          ; 2nd line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, 1, DB, NAME1, NAME2, D1, D2         ; 1st line - COMBINED
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE, 2, D11, D12, D13, D14, D15, D21, D22, D23, D24
; iSEC, TYPE, SNAME, [OFFSET2], bSD, SHAPE, iyVAR, izVAR, STYPE                ; 1st line - TAPERED
;       DB, NAME1, NAME2                                                       ; 2nd line(STYPE=DB)
;       [DIM1], [DIM2]                                                         ; 2nd line(STYPE=USER)
;       D11, D12, D13, D14, D15, D16, D17, D18                                 ; 2nd line(STYPE=VALUE)
;       AREA1, ASy1, ASz1, Ixx1, Iyy1, Izz1                                    ; 3rd line(STYPE=VALUE)
;       CyP1, CyM1, CzP1, CzM1, QyB1, QzB1, PERI_OUT1, PERI_IN1, Cy1, Cz1      ; 4th line(STYPE=VALUE)
;       Y11, Y12, Y13, Y14, Z11, Z12, Z13, Z14, Zyy1, Zyy2                     ; 5th line(STYPE=VALUE)
;       D21, D22, D23, D24, D25, D26, D27, D28                                 ; 6th line(STYPE=VALUE)
;       AREA2, ASy2, ASz2, Ixx2, Iyy2, Izz2                                    ; 7th line(STYPE=VALUE)
;       CyP2, CyM2, CzP2, CzM2, QyB2, QzB2, PERI_OUT2, PERI_IN2, Cy2, Cz2      ; 8th line(STYPE=VALUE)
;       Y21, Y22, Y23, Y24, Z21, Z22, Z23, Z24, Zyy2, Zzz2                     ; 9th line(STYPE=VALUE)
;       OPT1, OPT2, [JOINT]                                                    ; 2nd line(STYPE=PSC)
;       ELAST, DEN, POIS, POIC                                                 ; 2nd line(STYPE=PSC-CMPW)
;       bSHEARCHK, [SCHK-I], [SCHK-J], [WT-I], [WT-J], WI, WJ, bSYM, bSIDEHOLE ; 3rd line(STYPE=PSC)
;       bSHEARCHK, bSYM, bHUNCH, [CMPWEB-I], [CMPWEB-J]                        ; 3rd line(STYPE=PSC-CMPW)
;       bUSERDEFMESHSIZE, MESHSIZE, bUSERINPSTIFF, [STIFF-I], [STIFF-J]        ; 4th line(STYPE=PSC)
;       [SIZE-A]-i                                                             ; 5th line(STYPE=PSC)
;       [SIZE-B]-i                                                             ; 6th line(STYPE=PSC)
;       [SIZE-C]-i                                                             ; 7th line(STYPE=PSC)
;       [SIZE-D]-i                                                             ; 8th line(STYPE=PSC)
;       [SIZE-A]-j                                                             ; 9th line(STYPE=PSC)
;       [SIZE-B]-j                                                             ; 10th line(STYPE=PSC)
;       [SIZE-C]-j                                                             ; 11th line(STYPE=PSC)
;       [SIZE-D]-j                                                             ; 12th line(STYPE=PSC)
;       GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMULTI, EsEc-L, EsEc-S        ; 2nd line(STYPE=CMP-B/I)
;       SW_i, Hw_i, tw_i, B_i, Bf1_i, tf1_i, B2_i, Bf2_i, tf2_i                ; 3rd line(STYPE=CMP-B/I)
;       SW_j, Hw_j, tw_j, B_j, Bf1_j, tf1_j, B2_j, Bf2_j, tf2_j                ; 4th line(STYPE=CMP-B/I)
;       N1, N2, Hr, Hr2, tr1, tr2                                              ; 5th line(STYPE=CMP-B)
;       GN, CTC, Bc, Tc, Hh, EgdEsb, DgdDsb, Pgd, Psb, bSYM, SW_i, SW_j        ; 2nd line(STYPE=CMP-CI/CT)
;       OPT1, OPT2, [JOINT]                                                    ; 3rd line(STYPE=CMP-CI/CT)
;       [SIZE-A]-i                                                             ; 4th line(STYPE=CMP-CI/CT)
;       [SIZE-B]-i                                                             ; 5th line(STYPE=CMP-CI/CT)
;       [SIZE-C]-i                                                             ; 6th line(STYPE=CMP-CI/CT)
;       [SIZE-D]-i                                                             ; 7th line(STYPE=CMP-CI/CT)
;       [SIZE-A]-j                                                             ; 8th line(STYPE=CMP-CI/CT)
;       [SIZE-B]-j                                                             ; 9th line(STYPE=CMP-CI/CT)
;       [SIZE-C]-j                                                             ; 10th line(STYPE=CMP-CI/CT)
;       [SIZE-D]-j                                                             ; 11th line(STYPE=CMP-CI/CT)
; iSEC, TYPE, SNAME, [OFFSET], bSD, STYPE1, STYPE2                             ; 1st line - CONSTRUCT
;       SHAPE, ...(same with other type data from shape)                       ; Before (STYPE1)
;       SHAPE, ...(same with other type data from shape)                       ; After  (STYPE2)
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-SB
;       Hw, tw, B, Bf1, tf1, B2, Bf2, tf2                                      ; 2nd line
;       N1, N2, Hr, Hr2, tr1, tr2                                              ; 3rd line
;       SW, GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMulti, Elong, Esh        ; 4th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-SI
;       Hw, tw, B, tf1, B2, tf2                                                ; 2nd line
;       SW, GN, CTC, Bc, Tc, Hh, EsEc, DsDc, Ps, Pc, bMulti, Elong, Esh        ; 3rd line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                      ; 1st line - COMPOSITE-CI/CT
;       OPT1, OPT2, [JOINT]                                                    ; 2nd line
;       [SIZE-A]                                                               ; 3rd line
;       [SIZE-B]                                                               ; 4th line
```

```
;       [SIZE-C]                                                                    ; 5th line
;       [SIZE-D]                                                                    ; 6th line
;       SW, GN, CTC, Bc, Tc, Hh, EgdEsb, DgdDsb, Pgd, Psb                           ; 7th line
; iSEC, TYPE, SNAME, [OFFSET], bSD, SHAPE                                           ; 1st line - PSC
;       OPT1, OPT2, [JOINT]                                                         ; 2nd line
;       bSHEARCHK, [SCHK], [WT], WIDTH, bSYM, bSIDEHOLE                             ; 3rd line
;       bUSERDEFMESHSIZE, MESHSIZE, bUSERINPSTIFF, [STIFF]                          ; 4th line
;       [SIZE-A]                                                                    ; 5th line
;       [SIZE-B]                                                                    ; 6th line
;       [SIZE-C]                                                                    ; 7th line
;       [SIZE-D]                                                                    ; 8th line
; [DATA1] : 1, DB, NAME or 2, D1, D2, D3, D4, D5, D6, D7, D8, D9, D10
; [DATA2] : CCSHAPE or iCEL or iN1, iN2
; [SRC]  : 1, DB, NAME1, NAME2 or 2, D1, D2, D3, D4, D5, D6, D7, D8, D9, D10, iN1, iN2
; [DIM1], [DIM2] : D1, D2, D3, D4, D5, D6, D7, D8
; [OFFSET] : OFFSET, iCENT, iREF, iHORZ, HUSER, iVERT, VUSER
; [OFFSET2]: OFFSET, iCENT, iREF, iHORZ, HUSERI, HUSERJ, iVERT, VUSERI, VUSERJ
; [JOINT]  :  8(1CELL, 2CELL), 13(3CELL),  9(PSCM),  8(PSCH), 9(PSCT),  2(PSCB), 0(nCELL),  2(nCEL2)
; [SIZE-A] :  6(1CELL, 2CELL), 10(3CELL), 10(PSCM),  6(PSCH), 8(PSCT), 10(PSCB), 5(nCELL), 11(nCEL2)
; [SIZE-B] :  6(1CELL, 2CELL), 12(3CELL),  6(PSCM),  6(PSCH), 8(PSCT),  6(PSCB), 8(nCELL), 18(nCEL2)
; [SIZE-C] : 10(1CELL, 2CELL), 13(3CELL),  9(PSCM), 10(PSCH), 7(PSCT),  8(PSCB), 0(nCELL), 11(nCEL2)
; [SIZE-D] :  8(1CELL, 2CELL), 13(3CELL),  6(PSCM),  7(PSCH), 8(PSCT),  5(PSCB), 0(nCELL), 18(nCEL2)
; [STIFF]  : AREA, ASy, ASz, Ixx, Iyy, Izz
; [SCHK]   : bAUTO_Z1, Z1, bAUTO_Z3, Z3
; [WT]     : bAUTO_TOR, TOR, bAUTO_SHR1, SHR1, bAUTO_SHR2, SHR2, bAUTO_SHR3, SHR3
; [CMPWEB] : EFD, LRF, A, B, H, T


    1, DBUSER    , TOPSLAB           , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.4, 1, 0, 0, 0, 0, 0, 0, 0, 0
    2, DBUSER    , BOTSLAB           , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.45, 1, 0, 0, 0, 0, 0, 0, 0, 0
    3, DBUSER    , WALL              , CC, 0, 0, 0, 0, 0, 0, YES, SB , 2, 0.4, 1, 0, 0, 0, 0, 0, 0, 0, 0


*STLDCASE    ; Static Load Cases
; LCNAME, LCTYPE, DESC
   LC1  , USER,
   LC2  , USER,
   LC3  , USER,
   LC4  , USER,
   LC5  , USER,
   LC6  , USER,
   LC7  , USER,
   LC8  , USER,
   LC9  , USER,
   LC10 , USER,
   LC11 , USER,


*SPRING    ; Point Spring Supports
; NODE_LIST, Type, SDx, SDy, SDz, SRx, SRy, SRz, GROUP                                                        ; LINEAR
; NODE_LIST, Type, Direction, Vx, Vy, Vz, Stiffness, GROUP                                                    ; COMP, TENS
; NODE_LIST, Type, Multi-Linear Type, Direction, Vx, Vy, Vz, ax, ay, bx, by, cx, cy, dx, dy, ex, ey, fx, fy, GROUP  ; MULTI
   9, LINEAR, 0, 0, 6074, 0, 0, 0,
   1, LINEAR, 1e+010, 0, 6074, 0, 0, 0,
   2 8, LINEAR, 0, 0, 5157, 0, 0, 0,
   3to7, LINEAR, 0, 0, 3282, 0, 0, 0,

*OFFSET    ; Beam End Offsets
; ELEM_LIST, TYPE, RGDXi, RGDYi, RGDZi, RGDXj, RGDYj, RGDZj, GROUP  ; TYPE=GLOBAL
; ELEM_LIST, TYPE, RGDi, RGDj, GROUP                                ; TYPE=ELEMENT
     1, ELEMENT, 0.275, 0,
     8, ELEMENT, 0, 0.275,
     9, ELEMENT, 0.325, 0,
    10, ELEMENT, 0.325, 0,
    21, ELEMENT, 0, 0.3,
    22, ELEMENT, 0, 0.3,
    23, ELEMENT, 0.3, 0,
    30, ELEMENT, 0, 0.3,

*USE-STLD, LC1

; *SELFWEIGHT, X, Y, Z, GROUP
*SELFWEIGHT, 0, 0, -1,

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
    23, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    24, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    25, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    26, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    27, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    28, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    29, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
    30, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -166.74, 1, -166.74, 0, 0, 0, 0, , NO, 0, 0, NO
```

```
; End of data for load case [LC1] -------------------------

*USE-STLD, LC2

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
    23, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    24, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    25, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    26, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    27, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    28, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    29, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    30, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO

; End of data for load case [LC2] -------------------------

*USE-STLD, LC3

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
    23, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    24, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    25, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO
    26, BEAM   , UNILOAD, GZ, NO , NO, aDir[1], , , , 0, -21.019, 1, -21.019, 0, 0, 0, 0, , NO, 0, 0, NO

; End of data for load case [LC3] -------------------------

*USE-STLD, LC4

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
     9, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    10, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    11, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    12, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    13, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    14, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    15, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    16, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    17, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    18, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    19, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    20, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    21, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    22, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -8.408, 1, -8.408, 0, 0, 0, 0, , NO, 0, 0, NO

; End of data for load case [LC4] -------------------------

*USE-STLD, LC5

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
     9, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    11, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    13, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    15, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    17, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    19, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO
    21, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 8.408, 1, 8.408, 0, 0, 0, 0, , NO, 0, 0, NO

; End of data for load case [LC5] -------------------------

*USE-STLD, LC6

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST, CMD, TYPE, DIR, bPROJ, [ECCEN], [VALUE], GROUP
; ELEM_LIST, CMD, TYPE, TYPE, DIR, VX, VY, VZ, bPROJ, [ECCEN], [VALUE], GROUP
; [VALUE]       : D1, P1, D2, P2, D3, P3, D4, P4
; [ECCEN]       : bECCEN, ECCDIR, I-END, J-END, bJ-END
; [ADDITIONAL]  : bADDITIONAL, ADDITIONAL_I-END, ADDITIONAL_J-END, bADDITIONAL_J-END
     9, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 93.398, 1, 88.925, 0, 0, 0, 0, , NO, 0, 0, NO
    10, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -93.398, 1, -88.925, 0, 0, 0, 0, , NO, 0, 0, NO
```

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    11, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 88.925, 1, 86.539, 0, 0, 0, 0, , NO, 0, 0, NO
    12, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -88.925, 1, -86.539, 0, 0, 0, 0, , NO, 0, 0, NO
    13, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 86.539, 1, 84.153, 0, 0, 0, 0, , NO, 0, 0, NO
    14, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -86.539, 1, -84.153, 0, 0, 0, 0, , NO, 0, 0, NO
    15, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 84.153, 1, 81.767, 0, 0, 0, 0, , NO, 0, 0, NO
    16, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -84.153, 1, -81.767, 0, 0, 0, 0, , NO, 0, 0, NO
    17, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 81.767, 1, 79.381, 0, 0, 0, 0, , NO, 0, 0, NO
    18, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -81.767, 1, -79.381, 0, 0, 0, 0, , NO, 0, 0, NO
    19, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 79.381, 1, 76.995, 0, 0, 0, 0, , NO, 0, 0, NO
    20, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, -79.381, 1, -76.995, 0, 0, 0, 0, , NO, 0, 0, NO
    21, BEAM   , UNILOAD, GX, NO , NO, aDir[1], , , , 0, 76.995, 1, 72.735, 0, 0, 0, 0, , NO, 0, 0, NO
    22. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. -76.995. 1. -72.735. 0. 0. 0. 0. . NO. 0. 0. N

; End of data for load case [LC6] ------------------------

*USE-STLD. LC7

*BEAMLOAD    ; Element Beam Loads
; ELEM_LIST. CMD. TYPE. DIR. bPROJ. [ECCEN]. [VALUE]. GROUP
; ELEM_LIST. CMD. TYPE. TYPE. DIR. VX. VY. VZ. bPROJ. [ECCEN]. [VALUE]. GROUP
; [VALUE]       : D1. P1. D2. P2. D3. P3. D4. P4
; [ECCEN]       : bECCEN. ECCDIR. I-END. J-END. bJ-END
; [ADDITIONAL]  : bADDITIONAL. ADDITIONAL_I-END. ADDITIONAL_J-END. bADDITIONAL_J-END
     9. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 22.367. 1. 17.894. 0. 0. 0. 0. . NO. 0. 0. NO
    11. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 17.894. 1. 15.508. 0. 0. 0. 0. . NO. 0. 0. NO
    13. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 15.508. 1. 13.122. 0. 0. 0. 0. . NO. 0. 0. NO
    15. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 13.122. 1. 10.736. 0. 0. 0. 0. . NO. 0. 0. NO
    17. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 10.736. 1. 8.35. 0. 0. 0. 0. . NO. 0. 0. NO
    19. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 8.35. 1. 5.964. 0. 0. 0. 0. . NO. 0. 0. NO
    21. BEAM   . UNILOAD. GX. NO . NO. aDir[1]. . . . 0. 5.964. 1. 1.704. 0. 0. 0. 0. . NO. 0. 0. NO

; End of data for load case [LC7] ------------------------

*USE-STLD. LC10

*ELTEMPER    ; Element Temperatures
; ELEM_LIST. TEMPER. GROUP
     1. -15.
     2. -15.
     3. -15.
     4. -15.
     5. -15.
     6. -15.
     7. -15.
     8. -15.
     9. -15.
    10. -15.
    11. -15.
    12. -15.
    13. -15.
    14. -15.
    15. -15.
    16. -15.
    17. -15.
    18. -15.
    19. -15.
    20. -15.
    21. -15.
    22. -15.
    23. -15.
    24. -15.
    25. -15.
    26. -15.
    27. -15.
    28. -15.
    29. -15.
    30. -15.

; End of data for load case [LC10] ------------------------


*LOADCOMB    ; Combinations
; NAME=NAME. KIND. ACTIVE. bES. iTYPE. DESC. iSERV-TYPE. nLCOMTYPE   ; line 1
;      ANAL1. LCNAME1. FACT1. ...                                    ; from line 2
   NAME=COMB1. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.35. ST. LC2. 1.85. ST. LC4. 1.85. ST. LC6. 1.6
   NAME=COMB2. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.35. ST. LC2. 1.85. ST. LC4. 1.85. ST. LC6. 0.6
   NAME=COMB3. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.35. ST. LC3. 1.85. ST. LC5. 1.85. ST. LC6. 1.6
   NAME=COMB4. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.35. ST. LC3. 1.85. ST. LC5. 1.85. ST. LC6. 0.6
   NAME=COMB5. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.6. ST. LC2. 1.6. ST. LC4. 1.6. ST. LC6. 1.6
   NAME=COMB6. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.6. ST. LC2. 1.6. ST. LC4. 1.6. ST. LC6. 0.6
   NAME=COMB7. GEN. ACTIVE. 0. 0. . 0. 0
        ST. LC1. 1.6. ST. LC3. 1.6. ST. LC5. 1.6. ST. LC6. 1.6
   NAME=COMB8. GEN. ACTIVE. 0. 0. . 0. 0
```

```
        ST, LC1, 1.6, ST, LC3, 1.6, ST, LC5, 1.6, ST, LC6, 0.6
   NAME=COMB9, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC2, 1.4, ST, LC4, 1.4, ST, LC6, 1.6, ST, LC8, 1.35
        ST, LC11, 1.35
   NAME=COMB10, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC2, 1.4, ST, LC4, 1.4, ST, LC6, 0.6, ST, LC8, 1.35
        ST, LC11, 1.35
   NAME=COMB11, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC2, 1.4, ST, LC4, 1.4, ST, LC6, 1.6, ST, LC9, 1.35
        ST, LC10, 1.35, ST, LC11, 1.4
   NAME=COMB12, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC2, 1.4, ST, LC4, 1.4, ST, LC6, 0.6, ST, LC9, 1.35
        ST, LC10, 1.35, ST, LC11, 1.4
   NAME=COMB13, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.4, ST, LC5, 1.4, ST, LC6, 1.6, ST, LC8, 1.35
        ST, LC11, 1.35
   NAME=COMB14, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.4, ST, LC5, 1.4, ST, LC6, 0.6, ST, LC8, 1.35
        ST, LC11, 1.35
   NAME=COMB15, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.4, ST, LC5, 1.4, ST, LC6, 1.6, ST, LC9, 1.35
        ST, LC10, 1.35, ST, LC11, 1.4
   NAME=COMB16, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.4, ST, LC5, 1.4, ST, LC6, 0.6, ST, LC9, 1.35
        ST, LC10, 1.35, ST, LC11, 1.4
   NAME=COMB17, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.85, ST, LC5, 1.85, ST, LC7, 1.6
   NAME=COMB18, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.35, ST, LC3, 1.85, ST, LC5, 1.85, ST, LC7, 0.6
   NAME=COMB19, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.6, ST, LC3, 1.6, ST, LC5, 1.6, ST, LC7, 1.6
   NAME=COMB20, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1.6, ST, LC3, 1.6, ST, LC5, 1.6, ST, LC7, 0.6
   NAME=COMB21, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 1
   NAME=COMB22, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 0.5
   NAME=COMB23, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 1
   NAME=COMB24, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 0.5
   NAME=COMB25, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 1, ST, LC8, 1, ST, LC11, 1
   NAME=COMB26, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 0.5, ST, LC8, 1
        ST, LC11, 1
   NAME=COMB27, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 1, ST, LC9, 1, ST, LC10, 1
        ST, LC11, 1
   NAME=COMB28, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC2, 1, ST, LC4, 1, ST, LC6, 0.5, ST, LC9, 1
        ST, LC10, 1, ST, LC11, 1
   NAME=COMB29, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 1, ST, LC8, 1, ST, LC11, 1
   NAME=COMB30, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 0.5, ST, LC8, 1
        ST, LC11, 1
   NAME=COMB31, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 1, ST, LC9, 1, ST, LC10, 1
        ST, LC11, 1
   NAME=COMB32, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC6, 0.5, ST, LC9, 1
        ST, LC10, 1, ST, LC11, 1
   NAME=COMB33, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC6, 1
   NAME=COMB34, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC7, 1
   NAME=COMB35, GEN, ACTIVE, 0, 0, , 0, 0
        ST, LC1, 1, ST, LC3, 1, ST, LC5, 1, ST, LC7, 0.5
   NAME=USD, GEN, ACTIVE, 0, 1, , 0, 0
        CB, COMB1, 1, CB, COMB2, 1, CB, COMB3, 1, CB, COMB4, 1, CB, COMB5, 1
        CB, COMB6, 1, CB, COMB7, 1, CB, COMB8, 1, CB, COMB9, 1, CB, COMB10, 1
        CB, COMB11, 1, CB, COMB12, 1, CB, COMB13, 1, CB, COMB14, 1
        CB, COMB15, 1, CB, COMB16, 1, CB, COMB17, 1, CB, COMB18, 1
        CB, COMB19, 1, CB, COMB20, 1
   NAME=WSD, GEN, ACTIVE, 0, 1, , 0, 0
        CB, COMB21, 1, CB, COMB22, 1, CB, COMB23, 1, CB, COMB24, 1
        CB, COMB25, 1, CB, COMB26, 1, CB, COMB27, 1, CB, COMB28, 1
        CB, COMB29, 1, CB, COMB30, 1, CB, COMB31, 1, CB, COMB32, 1
        CB, COMB33, 1, CB, COMB34, 1, CB, COMB35, 1

*LC-COLOR    ; Diagram Color for Load Case
; ANAL, LCNAME, iR1(ALL), iG1(ALL), iB1(ALL), iR2(MIN), iG2(MIN), iB2(MIN), iR3(MAX), iG2(MAX), iB2(MAX
 CB, COMB1, 255, 0, 192, 192, 0, 128, 163, 255, 160
 CB, COMB2, 0, 128, 57, 160, 192, 255, 255, 0, 192
 CB, COMB3, 210, 210, 210, 255, 0, 192, 0, 128, 128
 CB, COMB4, 255, 255, 255, 93, 255, 87, 160, 255, 255
 CB, COMB5, 0, 157, 192, 85, 0, 192, 0, 128, 192
 CB, COMB6, 148, 87, 255, 255, 255, 255, 255, 87, 128
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 CB. COMB7. 255. 192. 87. 192. 0. 128. 210. 210. 210
 CB. COMB8. 146. 0. 255. 212. 160. 255. 0. 128. 128
 CB. COMB9. 192. 192. 0. 0. 157. 192. 0. 192. 192
 CB. COMB10. 85. 192. 0. 163. 160. 255. 192. 128. 0
 CB. COMB11. 0. 192. 192. 255. 192. 87. 78. 0. 255
 CB. COMB12. 160. 192. 255. 255. 255. 255. 255. 255. 87
 CB. COMB13. 0. 157. 192. 93. 255. 87. 85. 192. 0
 CB. COMB14. 148. 87. 255. 192. 128. 0. 255. 255. 87
 CB. COMB15. 85. 192. 0. 93. 255. 87. 148. 87. 255
 CB. COMB16. 78. 0. 255. 0. 192. 128. 192. 0. 192
 CB. COMB17. 192. 128. 0. 146. 0. 255. 255. 128. 0
 CB. COMB18. 163. 160. 255. 192. 128. 0. 0. 128. 255
 CB. COMB19. 78. 0. 255. 212. 160. 255. 255. 128. 0
 CB. COMB20. 255. 87. 128. 85. 0. 192. 212. 160. 255
 CB. COMB21. 255. 255. 87. 78. 0. 255. 85. 0. 192
 CB. COMB22. 192. 192. 0. 255. 87. 87. 0. 128. 128
 CB. COMB23. 163. 255. 160. 255. 87. 87. 255. 192. 87
 CB. COMB24. 255. 255. 255. 210. 210. 210. 93. 255. 87
 CB. COMB25. 128. 192. 0. 128. 192. 0. 255. 0. 128
 CB. COMB26. 0. 192. 128. 0. 128. 255. 255. 255. 255
 CB. COMB27. 146. 0. 255. 78. 0. 255. 255. 192. 160
 CB. COMB28. 212. 160. 255. 255. 128. 0. 192. 0. 128
 CB. COMB29. 0. 157. 192. 255. 87. 87. 163. 255. 160
 CB. COMB30. 78. 0. 255. 212. 160. 255. 192. 128. 0
 CB. COMB31. 255. 255. 87. 146. 0. 255. 163. 160. 255
 CB. COMB32. 255. 128. 0. 148. 87. 255. 255. 0. 128
 CB. COMB33. 255. 192. 160. 255. 255. 87. 255. 160. 255
 CB. COMB34. 255. 87. 87. 163. 160. 255. 210. 210. 210
 CB. COMB35. 255. 192. 160. 255. 128. 0. 255. 0. 192
 CB. USD. 192. 128. 0. 192. 192. 0. 0. 128. 255
 CB. WSD. 255. 255. 87. 160. 192. 255. 78. 0. 255
 ST. LC1. 255. 128. 0. 163. 255. 160. 192. 128. 0
 ST. LC2. 163. 160. 255. 192. 0. 128. 192. 192. 0
 ST. LC3. 255. 87. 128. 255. 0. 192. 163. 255. 160
 ST. LC4. 255. 160. 255. 78. 0. 255. 212. 160. 255
 ST. LC5. 192. 192. 0. 160. 255. 255. 148. 87. 255
 ST. LC6. 255. 192. 160. 0. 128. 255. 0. 157. 192
 ST. LC7. 192. 128. 0. 146. 0. 255. 192. 0. 128
 ST. LC8. 93. 255. 87. 0. 192. 192. 163. 160. 255
 ST. LC9. 0. 192. 192. 192. 0. 192. 255. 128. 0
 ST. LC10. 192. 0. 128. 192. 0. 192. 192. 72. 0
 ST. LC11. 0. 192. 192. 255. 0. 192. 192. 128. 0

*ANAL-CTRL
; bARDC. bANRC. iITER. TOL. bCSECF. bTRS. bCRBAR. bTPF. bCSO. bSCS. bCLATS
   YES. YES. 20. 0.001. YES. YES. YES. NO. NO. NO. NO


*ENDDATA
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8.OUPUT

FRAME ELEMENT FORCES

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
1	USD(max)	I[1]	32.120	0.000	-256.170	0.00E+00	-17.030	0.000
1	USD(max)	1/4	32.120	0.000	-257.190	0.00E+00	3.990	0.000
1	USD(max)	2/4	32.120	0.000	-258.220	0.00E+00	25.090	0.000
1	USD(max)	3/4	32.120	0.000	-259.240	0.00E+00	46.270	0.000
1	USD(max)	J[2]	32.120	0.000	-260.260	0.00E+00	67.540	0.000
1	USD(min)	I[1]	-176.020	0.000	-309.530	0.00E+00	-60.110	0.000
1	USD(min)	1/4	-176.020	0.000	-310.740	0.00E+00	-41.960	0.000
1	USD(min)	2/4	-176.020	0.000	-311.950	0.00E+00	-23.750	0.000
1	USD(min)	3/4	-176.020	0.000	-313.160	0.00E+00	-5.470	0.000
1	USD(min)	J[2]	-176.020	0.000	-314.380	0.00E+00	12.890	0.000
1	WSD(max)	I[1]	20.080	0.000	-174.220	0.00E+00	-11.950	0.000
1	WSD(max)	1/4	20.080	0.000	-174.980	0.00E+00	1.230	0.000
1	WSD(max)	2/4	20.080	0.000	-175.740	0.00E+00	14.450	0.000
1	WSD(max)	3/4	20.080	0.000	-176.500	0.00E+00	27.730	0.000
1	WSD(max)	J[2]	20.080	0.000	-177.250	0.00E+00	41.060	0.000
1	WSD(min)	I[1]	-107.110	0.000	-193.450	0.00E+00	-36.030	0.000
1	WSD(min)	1/4	-107.110	0.000	-194.210	0.00E+00	-22.980	0.000
1	WSD(min)	2/4	-107.110	0.000	-194.970	0.00E+00	-9.880	0.000
1	WSD(min)	3/4	-107.110	0.000	-195.730	0.00E+00	3.270	0.000
1	WSD(min)	J[2]	-107.110	0.000	-196.490	0.00E+00	16.470	0.000
2	USD(max)	I[2]	32.120	0.000	-157.960	0.00E+00	67.540	0.000
2	USD(max)	1/4	32.120	0.000	-158.920	0.00E+00	79.950	0.000
2	USD(max)	2/4	32.120	0.000	-159.880	0.00E+00	92.430	0.000
2	USD(max)	3/4	32.120	0.000	-160.830	0.00E+00	104.980	0.000
2	USD(max)	J[3]	32.120	0.000	-161.790	0.00E+00	117.600	0.000
2	USD(min)	I[2]	-176.020	0.000	-200.340	0.00E+00	12.890	0.000
2	USD(min)	1/4	-176.020	0.000	-201.480	0.00E+00	23.540	0.000
2	USD(min)	2/4	-176.020	0.000	-202.610	0.00E+00	34.250	0.000
2	USD(min)	3/4	-176.020	0.000	-203.740	0.00E+00	45.020	0.000
2	USD(min)	J[3]	-176.020	0.000	-204.880	0.00E+00	55.850	0.000
2	WSD(max)	I[2]	20.080	0.000	-105.330	0.00E+00	41.060	0.000
2	WSD(max)	1/4	20.080	0.000	-106.040	0.00E+00	48.880	0.000
2	WSD(max)	2/4	20.080	0.000	-106.750	0.00E+00	56.740	0.000
2	WSD(max)	3/4	20.080	0.000	-107.460	0.00E+00	64.640	0.000
2	WSD(max)	J[3]	20.080	0.000	-108.170	0.00E+00	72.600	0.000
2	WSD(min)	I[2]	-107.110	0.000	-125.210	0.00E+00	16.470	0.000
2	WSD(min)	1/4	-107.110	0.000	-125.920	0.00E+00	24.060	0.000
2	WSD(min)	2/4	-107.110	0.000	-126.630	0.00E+00	31.690	0.000
2	WSD(min)	3/4	-107.110	0.000	-127.340	0.00E+00	39.370	0.000
2	WSD(min)	J[3]	-107.110	0.000	-128.050	0.00E+00	47.100	0.000
3	USD(max)	I[3]	32.120	0.000	-93.880	0.00E+00	117.600	0.000
3	USD(max)	1/4	32.120	0.000	-94.840	0.00E+00	125.450	0.000
3	USD(max)	2/4	32.120	0.000	-95.790	0.00E+00	133.360	0.000
3	USD(max)	3/4	32.120	0.000	-96.740	0.00E+00	141.350	0.000
3	USD(max)	J[4]	32.120	0.000	-97.690	0.00E+00	149.410	0.000
3	USD(min)	I[3]	-176.020	0.000	-131.250	0.00E+00	55.850	0.000
3	USD(min)	1/4	-176.020	0.000	-132.380	0.00E+00	62.530	0.000
3	USD(min)	2/4	-176.020	0.000	-133.510	0.00E+00	69.070	0.000
3	USD(min)	3/4	-176.020	0.000	-134.640	0.00E+00	75.570	0.000
3	USD(min)	J[4]	-176.020	0.000	-135.770	0.00E+00	82.130	0.000
3	WSD(max)	I[3]	20.080	0.000	-62.560	0.00E+00	72.600	0.000
3	WSD(max)	1/4	20.080	0.000	-63.270	0.00E+00	77.570	0.000
3	WSD(max)	2/4	20.080	0.000	-63.970	0.00E+00	82.600	0.000
3	WSD(max)	3/4	20.080	0.000	-64.680	0.00E+00	87.660	0.000
3	WSD(max)	J[4]	20.080	0.000	-65.380	0.00E+00	92.780	0.000
3	WSD(min)	I[3]	-107.110	0.000	-82.030	0.00E+00	47.100	0.000
3	WSD(min)	1/4	-107.110	0.000	-82.740	0.00E+00	51.790	0.000
3	WSD(min)	2/4	-107.110	0.000	-83.440	0.00E+00	56.110	0.000
3	WSD(min)	3/4	-107.110	0.000	-84.150	0.00E+00	60.220	0.000
3	WSD(min)	J[4]	-107.110	0.000	-84.850	0.00E+00	64.380	0.000
4	USD(max)	I[4]	32.120	0.000	-29.990	0.00E+00	149.410	0.000
4	USD(max)	1/4	32.120	0.000	-30.950	0.00E+00	152.730	0.000
4	USD(max)	2/4	32.120	0.000	-31.900	0.00E+00	156.120	0.000
4	USD(max)	3/4	32.120	0.000	-32.860	0.00E+00	159.580	0.000
4	USD(max)	J[5]	32.120	0.000	-33.820	0.00E+00	163.110	0.000
4	USD(min)	I[4]	-176.020	0.000	-60.970	0.00E+00	82.130	0.000
4	USD(min)	1/4	-176.020	0.000	-62.100	0.00E+00	84.640	0.000
4	USD(min)	2/4	-176.020	0.000	-63.240	0.00E+00	87.210	0.000
4	USD(min)	3/4	-176.020	0.000	-64.370	0.00E+00	89.840	0.000
4	USD(min)	J[5]	-176.020	0.000	-65.500	0.00E+00	92.540	0.000
4	WSD(max)	I[4]	20.080	0.000	-19.890	0.00E+00	92.780	0.000
4	WSD(max)	1/4	20.080	0.000	-20.600	0.00E+00	94.930	0.000
4	WSD(max)	2/4	20.080	0.000	-21.310	0.00E+00	97.130	0.000
4	WSD(max)	3/4	20.080	0.000	-22.020	0.00E+00	99.370	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
4	WSD(max)	J[5]	20.080	0.000	-22.730	0.00E+00	101.660	0.000
4	WSD(min)	I[4]	-107.110	0.000	-38.110	0.00E+00	64.380	0.000
4	WSD(min)	1/4	-107.110	0.000	-38.810	0.00E+00	65.690	0.000
4	WSD(min)	2/4	-107.110	0.000	-39.520	0.00E+00	67.030	0.000
4	WSD(min)	3/4	-107.110	0.000	-40.230	0.00E+00	68.420	0.000
4	WSD(min)	J[5]	-107.110	0.000	-40.940	0.00E+00	69.860	0.000
5	USD(max)	I[5]	32.120	0.000	40.020	0.00E+00	163.110	0.000
5	USD(max)	1/4	32.120	0.000	38.880	0.00E+00	161.830	0.000
5	USD(max)	2/4	32.120	0.000	37.750	0.00E+00	160.610	0.000
5	USD(max)	3/4	32.120	0.000	36.620	0.00E+00	159.470	0.000
5	USD(max)	J[6]	32.120	0.000	35.480	0.00E+00	158.660	0.000
5	USD(min)	I[5]	-176.020	0.000	3.450	0.00E+00	92.540	0.000
5	USD(min)	1/4	-176.020	0.000	2.500	0.00E+00	91.140	0.000
5	USD(min)	2/4	-176.020	0.000	1.540	0.00E+00	89.800	0.000
5	USD(min)	3/4	-176.020	0.000	0.590	0.00E+00	88.530	0.000
5	USD(min)	J[6]	-176.020	0.000	-0.370	0.00E+00	87.320	0.000
5	WSD(max)	I[5]	20.080	0.000	25.010	0.00E+00	101.660	0.000
5	WSD(max)	1/4	20.080	0.000	24.300	0.00E+00	100.930	0.000
5	WSD(max)	2/4	20.080	0.000	23.590	0.00E+00	100.250	0.000
5	WSD(max)	3/4	20.080	0.000	22.890	0.00E+00	99.620	0.000
5	WSD(max)	J[6]	20.080	0.000	22.180	0.00E+00	99.160	0.000
5	WSD(min)	I[5]	-107.110	0.000	6.640	0.00E+00	69.860	0.000
5	WSD(min)	1/4	-107.110	0.000	5.940	0.00E+00	68.420	0.000
5	WSD(min)	2/4	-107.110	0.000	5.230	0.00E+00	67.030	0.000
5	WSD(min)	3/4	-107.110	0.000	4.520	0.00E+00	65.690	0.000
5	WSD(min)	J[6]	-107.110	0.000	3.810	0.00E+00	64.380	0.000
6	USD(max)	I[6]	32.120	0.000	115.590	0.00E+00	158.660	0.000
6	USD(max)	1/4	32.120	0.000	114.460	0.00E+00	153.330	0.000
6	USD(max)	2/4	32.120	0.000	113.330	0.00E+00	148.080	0.000
6	USD(max)	3/4	32.120	0.000	112.200	0.00E+00	142.900	0.000
6	USD(max)	J[7]	32.120	0.000	111.070	0.00E+00	137.790	0.000
6	USD(min)	I[6]	-176.020	0.000	66.560	0.00E+00	87.320	0.000
6	USD(min)	1/4	-176.020	0.000	65.600	0.00E+00	82.020	0.000
6	USD(min)	2/4	-176.020	0.000	64.650	0.00E+00	76.780	0.000
6	USD(min)	3/4	-176.020	0.000	63.700	0.00E+00	71.450	0.000
6	USD(min)	J[7]	-176.020	0.000	62.750	0.00E+00	65.400	0.000
6	WSD(max)	I[6]	20.080	0.000	72.240	0.00E+00	99.160	0.000
6	WSD(max)	1/4	20.080	0.000	71.540	0.00E+00	95.830	0.000
6	WSD(max)	2/4	20.080	0.000	70.830	0.00E+00	92.550	0.000
6	WSD(max)	3/4	20.080	0.000	70.130	0.00E+00	89.310	0.000
6	WSD(max)	J[7]	20.080	0.000	69.420	0.00E+00	86.120	0.000
6	WSD(min)	I[6]	-107.110	0.000	52.340	0.00E+00	64.380	0.000
6	WSD(min)	1/4	-107.110	0.000	51.640	0.00E+00	60.220	0.000
6	WSD(min)	2/4	-107.110	0.000	50.930	0.00E+00	56.110	0.000
6	WSD(min)	3/4	-107.110	0.000	50.220	0.00E+00	52.030	0.000
6	WSD(min)	J[7]	-107.110	0.000	49.520	0.00E+00	48.010	0.000
7	USD(max)	I[7]	32.120	0.000	191.380	0.00E+00	137.790	0.000
7	USD(max)	1/4	32.120	0.000	190.240	0.00E+00	127.640	0.000
7	USD(max)	2/4	32.120	0.000	189.110	0.00E+00	117.570	0.000
7	USD(max)	3/4	32.120	0.000	187.980	0.00E+00	107.560	0.000
7	USD(max)	J[8]	32.120	0.000	186.840	0.00E+00	97.630	0.000
7	USD(min)	I[7]	-176.020	0.000	131.360	0.00E+00	65.400	0.000
7	USD(min)	1/4	-176.020	0.000	130.410	0.00E+00	55.010	0.000
7	USD(min)	2/4	-176.020	0.000	129.450	0.00E+00	44.690	0.000
7	USD(min)	3/4	-176.020	0.000	128.490	0.00E+00	34.420	0.000
7	USD(min)	J[8]	-176.020	0.000	127.540	0.00E+00	24.220	0.000
7	WSD(max)	I[7]	20.080	0.000	119.610	0.00E+00	86.120	0.000
7	WSD(max)	1/4	20.080	0.000	118.900	0.00E+00	79.780	0.000
7	WSD(max)	2/4	20.080	0.000	118.190	0.00E+00	73.480	0.000
7	WSD(max)	3/4	20.080	0.000	117.490	0.00E+00	67.230	0.000
7	WSD(max)	J[8]	20.080	0.000	116.780	0.00E+00	61.020	0.000
7	WSD(min)	I[7]	-107.110	0.000	99.100	0.00E+00	48.010	0.000
7	WSD(min)	1/4	-107.110	0.000	98.390	0.00E+00	41.080	0.000
7	WSD(min)	2/4	-107.110	0.000	97.680	0.00E+00	34.200	0.000
7	WSD(min)	3/4	-107.110	0.000	96.970	0.00E+00	27.360	0.000
7	WSD(min)	J[8]	-107.110	0.000	96.270	0.00E+00	20.570	0.000
8	USD(max)	I[8]	32.120	0.000	313.500	0.00E+00	97.630	0.000
8	USD(max)	1/4	32.120	0.000	312.280	0.00E+00	78.310	0.000
8	USD(max)	2/4	32.120	0.000	311.070	0.00E+00	59.080	0.000
8	USD(max)	3/4	32.120	0.000	309.860	0.00E+00	39.930	0.000
8	USD(max)	J[9]	32.120	0.000	308.640	0.00E+00	21.900	0.000
8	USD(min)	I[8]	-176.020	0.000	238.230	0.00E+00	24.220	0.000
8	USD(min)	1/4	-176.020	0.000	237.200	0.00E+00	6.000	0.000
8	USD(min)	2/4	-176.020	0.000	236.180	0.00E+00	-12.150	0.000
8	USD(min)	3/4	-176.020	0.000	235.160	0.00E+00	-30.760	0.000
8	USD(min)	J[9]	-176.020	0.000	234.130	0.00E+00	-49.350	0.000
8	WSD(max)	I[8]	20.080	0.000	195.930	0.00E+00	61.020	0.000
8	WSD(max)	1/4	20.080	0.000	195.180	0.00E+00	48.940	0.000
8	WSD(max)	2/4	20.080	0.000	194.420	0.00E+00	36.920	0.000
8	WSD(max)	3/4	20.080	0.000	193.660	0.00E+00	24.960	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
8	WSD(max)	J[9]	20.080	0.000	192.900	0.00E+00	13.040	0.000
8	WSD(min)	I[8]	-107.110	0.000	175.980	0.00E+00	20.570	0.000
8	WSD(min)	1/4	-107.110	0.000	175.220	0.00E+00	8.410	0.000
8	WSD(min)	2/4	-107.110	0.000	174.460	0.00E+00	-3.700	0.000
8	WSD(min)	3/4	-107.110	0.000	173.710	0.00E+00	-16.930	0.000
8	WSD(min)	J[9]	-107.110	0.000	172.950	0.00E+00	-30.220	0.000
9	USD(max)	I[1]	-371.290	0.000	-13.910	0.00E+00	-72.090	0.000
9	USD(max)	1/4	-369.550	0.000	-12.660	0.00E+00	-66.260	0.000
9	USD(max)	2/4	-367.820	0.000	-11.430	0.00E+00	-60.820	0.000
9	USD(max)	3/4	-366.080	0.000	-10.220	0.00E+00	-55.770	0.000
9	USD(max)	J[10]	-364.350	0.000	-9.010	0.00E+00	-51.110	0.000
9	USD(min)	I[1]	-454.240	0.000	-133.690	0.00E+00	-99.280	0.000
9	USD(min)	1/4	-452.180	0.000	-125.700	0.00E+00	-97.760	0.000
9	USD(min)	2/4	-450.120	0.000	-117.740	0.00E+00	-96.360	0.000
9	USD(min)	3/4	-448.060	0.000	-109.810	0.00E+00	-95.060	0.000
9	USD(min)	J[10]	-446.000	0.000	-101.920	0.00E+00	-93.860	0.000
9	WSD(max)	I[1]	-256.610	0.000	-10.880	0.00E+00	-48.190	0.000
9	WSD(max)	1/4	-255.320	0.000	-9.990	0.00E+00	-44.890	0.000
9	WSD(max)	2/4	-254.040	0.000	-9.100	0.00E+00	-41.800	0.000
9	WSD(max)	3/4	-252.750	0.000	-8.220	0.00E+00	-38.950	0.000
9	WSD(max)	J[10]	-251.470	0.000	-7.360	0.00E+00	-36.310	0.000
9	WSD(min)	I[1]	-283.900	0.000	-80.200	0.00E+00	-62.050	0.000
9	WSD(min)	1/4	-282.610	0.000	-75.270	0.00E+00	-61.100	0.000
9	WSD(min)	2/4	-281.320	0.000	-70.360	0.00E+00	-60.220	0.000
9	WSD(min)	3/4	-280.040	0.000	-65.470	0.00E+00	-59.410	0.000
9	WSD(min)	J[10]	-278.750	0.000	-60.600	0.00E+00	-58.660	0.000
10	USD(max)	I[9]	-359.760	0.000	32.120	0.00E+00	-52.030	0.000
10	USD(max)	1/4	-358.020	0.000	32.120	0.00E+00	-53.580	0.000
10	USD(max)	2/4	-356.290	0.000	32.120	0.00E+00	-50.030	0.000
10	USD(max)	3/4	-354.550	0.000	32.120	0.00E+00	-45.850	0.000
10	USD(max)	J[11]	-352.810	0.000	32.120	0.00E+00	-42.030	0.000
10	USD(min)	I[9]	-454.240	0.000	-123.230	0.00E+00	-86.150	0.000
10	USD(min)	1/4	-452.180	0.000	-115.230	0.00E+00	-84.180	0.000
10	USD(min)	2/4	-450.120	0.000	-107.280	0.00E+00	-82.370	0.000
10	USD(min)	3/4	-448.060	0.000	-99.350	0.00E+00	-80.730	0.000
10	USD(min)	J[11]	-446.000	0.000	-91.460	0.00E+00	-79.260	0.000
10	WSD(max)	I[9]	-256.610	0.000	20.080	0.00E+00	-40.630	0.000
10	WSD(max)	1/4	-255.320	0.000	20.080	0.00E+00	-41.630	0.000
10	WSD(max)	2/4	-254.040	0.000	20.080	0.00E+00	-39.440	0.000
10	WSD(max)	3/4	-252.750	0.000	20.080	0.00E+00	-36.860	0.000
10	WSD(max)	J[11]	-251.470	0.000	20.080	0.00E+00	-34.510	0.000
10	WSD(min)	I[9]	-283.900	0.000	-74.540	0.00E+00	-53.750	0.000
10	WSD(min)	1/4	-282.610	0.000	-69.610	0.00E+00	-52.040	0.000
10	WSD(min)	2/4	-281.320	0.000	-64.700	0.00E+00	-50.460	0.000
10	WSD(min)	3/4	-280.040	0.000	-59.810	0.00E+00	-49.020	0.000
10	WSD(min)	J[11]	-278.750	0.000	-54.940	0.00E+00	-47.710	0.000
11	USD(max)	I[10]	-364.350	0.000	-9.010	0.00E+00	-51.110	0.000
11	USD(max)	1/4	-363.420	0.000	-7.350	0.00E+00	-45.220	0.000
11	USD(max)	2/4	-362.490	0.000	-5.710	0.00E+00	-40.090	0.000
11	USD(max)	3/4	-361.570	0.000	-4.090	0.00E+00	-35.690	0.000
11	USD(max)	J[12]	-360.640	0.000	-2.500	0.00E+00	-32.040	0.000
11	USD(min)	I[10]	-446.000	0.000	-101.920	0.00E+00	-93.860	0.000
11	USD(min)	1/4	-444.910	0.000	-90.930	0.00E+00	-92.370	0.000
11	USD(min)	2/4	-443.810	0.000	-80.010	0.00E+00	-91.080	0.000
11	USD(min)	3/4	-442.710	0.000	-69.150	0.00E+00	-90.730	0.000
11	USD(min)	J[12]	-441.610	0.000	-58.360	0.00E+00	-90.500	0.000
11	WSD(max)	I[10]	-251.470	0.000	-7.360	0.00E+00	-36.310	0.000
11	WSD(max)	1/4	-250.780	0.000	-6.160	0.00E+00	-33.000	0.000
11	WSD(max)	2/4	-250.090	0.000	-4.990	0.00E+00	-30.120	0.000
11	WSD(max)	3/4	-249.410	0.000	-3.830	0.00E+00	-27.670	0.000
11	WSD(max)	J[12]	-248.720	0.000	-2.700	0.00E+00	-25.640	0.000
11	WSD(min)	I[10]	-278.750	0.000	-60.600	0.00E+00	-58.660	0.000
11	WSD(min)	1/4	-278.070	0.000	-53.820	0.00E+00	-57.730	0.000
11	WSD(min)	2/4	-277.380	0.000	-47.080	0.00E+00	-56.920	0.000
11	WSD(min)	3/4	-276.690	0.000	-40.390	0.00E+00	-56.610	0.000
11	WSD(min)	J[12]	-276.010	0.000	-33.740	0.00E+00	-56.380	0.000
12	USD(max)	I[11]	-352.810	0.000	32.120	0.00E+00	-42.030	0.000
12	USD(max)	1/4	-351.890	0.000	32.120	0.00E+00	-37.280	0.000
12	USD(max)	2/4	-350.960	0.000	32.120	0.00E+00	-33.220	0.000
12	USD(max)	3/4	-350.030	0.000	32.120	0.00E+00	-29.840	0.000
12	USD(max)	J[13]	-349.110	0.000	32.120	0.00E+00	-27.150	0.000
12	USD(min)	I[11]	-446.000	0.000	-91.460	0.00E+00	-79.260	0.000
12	USD(min)	1/4	-444.910	0.000	-80.470	0.00E+00	-77.810	0.000
12	USD(min)	2/4	-443.810	0.000	-69.550	0.00E+00	-79.720	0.000
12	USD(min)	3/4	-442.710	0.000	-58.690	0.00E+00	-81.630	0.000
12	USD(min)	J[13]	-441.610	0.000	-47.900	0.00E+00	-83.540	0.000
12	WSD(max)	I[11]	-251.470	0.000	20.080	0.00E+00	-34.510	0.000
12	WSD(max)	1/4	-250.780	0.000	20.080	0.00E+00	-31.580	0.000
12	WSD(max)	2/4	-250.090	0.000	20.080	0.00E+00	-29.090	0.000
12	WSD(max)	3/4	-249.410	0.000	20.080	0.00E+00	-27.030	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
12	WSD(max)	J[13]	-248.720	0.000	20.080	0.00E+00	-25.390	0.000
12	WSD(min)	I[11]	-278.750	0.000	-54.940	0.00E+00	-47.710	0.000
12	WSD(min)	1/4	-278.070	0.000	-48.170	0.00E+00	-48.120	0.000
12	WSD(min)	2/4	-277.380	0.000	-41.430	0.00E+00	-49.350	0.000
12	WSD(min)	3/4	-276.690	0.000	-34.730	0.00E+00	-50.590	0.000
12	WSD(min)	J[13]	-276.010	0.000	-28.080	0.00E+00	-51.820	0.000
13	USD(max)	I[12]	-360.640	0.000	-2.500	0.00E+00	-32.040	0.000
13	USD(max)	1/4	-359.720	0.000	-0.940	0.00E+00	-29.000	0.000
13	USD(max)	2/4	-358.790	0.000	0.600	0.00E+00	-26.250	0.000
13	USD(max)	3/4	-357.860	0.000	4.230	0.00E+00	-24.240	0.000
13	USD(max)	J[14]	-356.940	0.000	8.700	0.00E+00	-22.940	0.000
13	USD(min)	I[12]	-441.610	0.000	-58.360	0.00E+00	-90.500	0.000
13	USD(min)	1/4	-440.520	0.000	-47.640	0.00E+00	-90.380	0.000
13	USD(min)	2/4	-439.420	0.000	-36.980	0.00E+00	-90.370	0.000
13	USD(min)	3/4	-438.320	0.000	-26.390	0.00E+00	-90.460	0.000
13	USD(min)	J[14]	-437.220	0.000	-15.870	0.00E+00	-90.660	0.000
13	WSD(max)	I[12]	-248.720	0.000	-2.700	0.00E+00	-25.640	0.000
13	WSD(max)	1/4	-248.040	0.000	-1.590	0.00E+00	-24.040	0.000
13	WSD(max)	2/4	-247.350	0.000	-0.500	0.00E+00	-22.860	0.000
13	WSD(max)	3/4	-246.660	0.000	0.570	0.00E+00	-22.100	0.000
13	WSD(max)	J[14]	-245.980	0.000	3.970	0.00E+00	-21.750	0.000
13	WSD(min)	I[12]	-276.010	0.000	-33.740	0.00E+00	-56.380	0.000
13	WSD(min)	1/4	-275.320	0.000	-27.120	0.00E+00	-56.230	0.000
13	WSD(min)	2/4	-274.640	0.000	-20.550	0.00E+00	-56.160	0.000
13	WSD(min)	3/4	-273.950	0.000	-14.030	0.00E+00	-56.160	0.000
13	WSD(min)	J[14]	-273.260	0.000	-7.540	0.00E+00	-56.240	0.000
14	USD(max)	I[13]	-349.110	0.000	32.120	0.00E+00	-27.150	0.000
14	USD(max)	1/4	-348.180	0.000	32.120	0.00E+00	-25.080	0.000
14	USD(max)	2/4	-347.260	0.000	32.120	0.00E+00	-23.570	0.000
14	USD(max)	3/4	-346.330	0.000	32.120	0.00E+00	-22.720	0.000
14	USD(max)	J[15]	-345.400	0.000	32.120	0.00E+00	-22.540	0.000
14	USD(min)	I[13]	-441.610	0.000	-47.900	0.00E+00	-83.540	0.000
14	USD(min)	1/4	-440.520	0.000	-37.180	0.00E+00	-85.450	0.000
14	USD(min)	2/4	-439.420	0.000	-26.520	0.00E+00	-87.360	0.000
14	USD(min)	3/4	-438.320	0.000	-15.930	0.00E+00	-89.280	0.000
14	USD(min)	J[15]	-437.220	0.000	-5.470	0.00E+00	-91.190	0.000
14	WSD(max)	I[13]	-248.720	0.000	20.080	0.00E+00	-25.390	0.000
14	WSD(max)	1/4	-248.040	0.000	20.080	0.00E+00	-24.040	0.000
14	WSD(max)	2/4	-247.350	0.000	20.080	0.00E+00	-22.860	0.000
14	WSD(max)	3/4	-246.660	0.000	20.080	0.00E+00	-22.100	0.000
14	WSD(max)	J[15]	-245.980	0.000	20.080	0.00E+00	-21.750	0.000
14	WSD(min)	I[13]	-276.010	0.000	-28.080	0.00E+00	-51.820	0.000
14	WSD(min)	1/4	-275.320	0.000	-21.470	0.00E+00	-53.060	0.000
14	WSD(min)	2/4	-274.640	0.000	-14.900	0.00E+00	-54.300	0.000
14	WSD(min)	3/4	-273.950	0.000	-8.370	0.00E+00	-55.530	0.000
14	WSD(min)	J[15]	-273.260	0.000	-2.010	0.00E+00	-56.770	0.000
15	USD(max)	I[14]	-356.940	0.000	8.700	0.00E+00	-22.940	0.000
15	USD(max)	1/4	-356.010	0.000	13.140	0.00E+00	-22.360	0.000
15	USD(max)	2/4	-355.080	0.000	17.560	0.00E+00	-22.490	0.000
15	USD(max)	3/4	-354.160	0.000	27.720	0.00E+00	-23.330	0.000
15	USD(max)	J[16]	-353.230	0.000	37.830	0.00E+00	-24.870	0.000
15	USD(min)	I[14]	-437.220	0.000	-15.870	0.00E+00	-90.660	0.000
15	USD(min)	1/4	-436.130	0.000	-5.410	0.00E+00	-90.970	0.000
15	USD(min)	2/4	-435.030	0.000	-0.360	0.00E+00	-91.370	0.000
15	USD(min)	3/4	-433.930	0.000	2.020	0.00E+00	-91.880	0.000
15	USD(min)	J[16]	-432.830	0.000	4.320	0.00E+00	-92.480	0.000
15	WSD(max)	I[14]	-245.980	0.000	3.970	0.00E+00	-21.750	0.000
15	WSD(max)	1/4	-245.290	0.000	7.480	0.00E+00	-21.820	0.000
15	WSD(max)	2/4	-244.610	0.000	10.970	0.00E+00	-22.290	0.000
15	WSD(max)	3/4	-243.920	0.000	17.320	0.00E+00	-23.170	0.000
15	WSD(max)	J[16]	-243.230	0.000	23.640	0.00E+00	-24.450	0.000
15	WSD(min)	I[14]	-273.260	0.000	-7.540	0.00E+00	-56.240	0.000
15	WSD(min)	1/4	-272.580	0.000	-1.100	0.00E+00	-56.390	0.000
15	WSD(min)	2/4	-271.890	0.000	1.970	0.00E+00	-56.610	0.000
15	WSD(min)	3/4	-271.210	0.000	3.370	0.00E+00	-56.900	0.000
15	WSD(min)	J[16]	-270.520	0.000	4.720	0.00E+00	-57.260	0.000
16	USD(max)	I[15]	-345.400	0.000	32.120	0.00E+00	-22.540	0.000
16	USD(max)	1/4	-344.480	0.000	32.120	0.00E+00	-23.020	0.000
16	USD(max)	2/4	-343.550	0.000	32.120	0.00E+00	-24.150	0.000
16	USD(max)	3/4	-342.630	0.000	33.620	0.00E+00	-25.930	0.000
16	USD(max)	J[17]	-341.700	0.000	42.800	0.00E+00	-28.360	0.000
16	USD(min)	I[15]	-437.220	0.000	-5.470	0.00E+00	-91.190	0.000
16	USD(min)	1/4	-436.130	0.000	4.730	0.00E+00	-93.100	0.000
16	USD(min)	2/4	-435.030	0.000	14.860	0.00E+00	-95.010	0.000
16	USD(min)	3/4	-433.930	0.000	19.160	0.00E+00	-96.920	0.000
16	USD(min)	J[17]	-432.830	0.000	23.410	0.00E+00	-98.830	0.000
16	WSD(max)	I[15]	-245.980	0.000	20.080	0.00E+00	-21.750	0.000
16	WSD(max)	1/4	-245.290	0.000	20.080	0.00E+00	-21.820	0.000
16	WSD(max)	2/4	-244.610	0.000	20.080	0.00E+00	-22.290	0.000
16	WSD(max)	3/4	-243.920	0.000	21.010	0.00E+00	-23.170	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
16	WSD(max)	J[17]	-243.230	0.000	26.750	0.00E+00	-24.450	0.000
16	WSD(min)	I[15]	-273.260	0.000	-2.010	0.00E+00	-56.770	0.000
16	WSD(min)	1/4	-272.580	0.000	3.850	0.00E+00	-58.010	0.000
16	WSD(min)	2/4	-271.890	0.000	9.670	0.00E+00	-59.240	0.000
16	WSD(min)	3/4	-271.210	0.000	14.440	0.00E+00	-60.480	0.000
16	WSD(min)	J[17]	-270.520	0.000	17.660	0.00E+00	-61.720	0.000
17	USD(max)	I[16]	-353.230	0.000	37.830	0.00E+00	-24.870	0.000
17	USD(max)	1/4	-352.310	0.000	47.870	0.00E+00	-27.110	0.000
17	USD(max)	2/4	-351.380	0.000	57.850	0.00E+00	-30.040	0.000
17	USD(max)	3/4	-350.450	0.000	67.760	0.00E+00	-33.600	0.000
17	USD(max)	J[18]	-349.530	0.000	77.600	0.00E+00	-37.870	0.000
17	USD(min)	I[16]	-432.830	0.000	4.320	0.00E+00	-92.480	0.000
17	USD(min)	1/4	-431.740	0.000	6.560	0.00E+00	-93.180	0.000
17	USD(min)	2/4	-430.640	0.000	8.740	0.00E+00	-93.980	0.000
17	USD(min)	3/4	-429.540	0.000	10.540	0.00E+00	-94.870	0.000
17	USD(min)	J[18]	-428.440	0.000	11.980	0.00E+00	-95.840	0.000
17	WSD(max)	I[16]	-243.230	0.000	23.640	0.00E+00	-24.450	0.000
17	WSD(max)	1/4	-242.550	0.000	29.920	0.00E+00	-26.140	0.000
17	WSD(max)	2/4	-241.860	0.000	36.160	0.00E+00	-28.220	0.000
17	WSD(max)	3/4	-241.180	0.000	42.350	0.00E+00	-30.570	0.000
17	WSD(max)	J[18]	-240.490	0.000	48.500	0.00E+00	-33.350	0.000
17	WSD(min)	I[16]	-270.520	0.000	4.720	0.00E+00	-57.260	0.000
17	WSD(min)	1/4	-269.830	0.000	6.030	0.00E+00	-57.680	0.000
17	WSD(min)	2/4	-269.150	0.000	7.300	0.00E+00	-58.170	0.000
17	WSD(min)	3/4	-268.460	0.000	8.370	0.00E+00	-58.730	0.000
17	WSD(min)	J[18]	-267.780	0.000	9.250	0.00E+00	-59.350	0.000
18	USD(max)	I[17]	-341.700	0.000	42.800	0.00E+00	-28.360	0.000
18	USD(max)	1/4	-340.770	0.000	51.920	0.00E+00	-31.430	0.000
18	USD(max)	2/4	-339.850	0.000	60.970	0.00E+00	-35.130	0.000
18	USD(max)	3/4	-338.920	0.000	69.960	0.00E+00	-39.330	0.000
18	USD(max)	J[19]	-338.000	0.000	78.870	0.00E+00	-44.190	0.000
18	USD(min)	I[17]	-432.830	0.000	23.410	0.00E+00	-98.830	0.000
18	USD(min)	1/4	-431.740	0.000	26.260	0.00E+00	-100.740	0.000
18	USD(min)	2/4	-430.640	0.000	26.260	0.00E+00	-102.910	0.000
18	USD(min)	3/4	-429.540	0.000	26.260	0.00E+00	-105.160	0.000
18	USD(min)	J[19]	-428.440	0.000	26.260	0.00E+00	-107.410	0.000
18	WSD(max)	I[17]	-243.230	0.000	26.750	0.00E+00	-24.450	0.000
18	WSD(max)	1/4	-242.550	0.000	32.450	0.00E+00	-26.140	0.000
18	WSD(max)	2/4	-241.860	0.000	38.110	0.00E+00	-28.220	0.000
18	WSD(max)	3/4	-241.180	0.000	43.720	0.00E+00	-30.690	0.000
18	WSD(max)	J[19]	-240.490	0.000	49.300	0.00E+00	-33.560	0.000
18	WSD(min)	I[17]	-270.520	0.000	17.660	0.00E+00	-61.720	0.000
18	WSD(min)	1/4	-269.830	0.000	17.660	0.00E+00	-62.950	0.000
18	WSD(min)	2/4	-269.150	0.000	17.660	0.00E+00	-64.320	0.000
18	WSD(min)	3/4	-268.460	0.000	17.660	0.00E+00	-65.720	0.000
18	WSD(min)	J[19]	-267.780	0.000	17.660	0.00E+00	-67.130	0.000
19	USD(max)	I[18]	-349.530	0.000	77.600	0.00E+00	-37.870	0.000
19	USD(max)	1/4	-348.600	0.000	87.380	0.00E+00	-42.830	0.000
19	USD(max)	2/4	-347.680	0.000	97.090	0.00E+00	-48.480	0.000
19	USD(max)	3/4	-346.750	0.000	106.730	0.00E+00	-54.820	0.000
19	USD(max)	J[20]	-345.820	0.000	116.310	0.00E+00	-61.850	0.000
19	USD(min)	I[18]	-428.440	0.000	11.980	0.00E+00	-95.840	0.000
19	USD(min)	1/4	-427.340	0.000	13.390	0.00E+00	-96.910	0.000
19	USD(min)	2/4	-426.250	0.000	14.770	0.00E+00	-98.070	0.000
19	USD(min)	3/4	-425.150	0.000	16.130	0.00E+00	-99.310	0.000
19	USD(min)	J[20]	-424.050	0.000	17.460	0.00E+00	-103.260	0.000
19	WSD(max)	I[18]	-240.490	0.000	48.500	0.00E+00	-33.350	0.000
19	WSD(max)	1/4	-239.800	0.000	54.610	0.00E+00	-36.570	0.000
19	WSD(max)	2/4	-239.120	0.000	60.680	0.00E+00	-40.210	0.000
19	WSD(max)	3/4	-238.430	0.000	66.710	0.00E+00	-44.270	0.000
19	WSD(max)	J[20]	-237.750	0.000	72.690	0.00E+00	-48.750	0.000
19	WSD(min)	I[18]	-267.780	0.000	9.250	0.00E+00	-59.350	0.000
19	WSD(min)	1/4	-267.090	0.000	10.110	0.00E+00	-60.020	0.000
19	WSD(min)	2/4	-266.400	0.000	10.950	0.00E+00	-60.760	0.000
19	WSD(min)	3/4	-265.720	0.000	11.770	0.00E+00	-61.560	0.000
19	WSD(min)	J[20]	-265.030	0.000	12.570	0.00E+00	-62.590	0.000
20	USD(max)	I[19]	-338.000	0.000	78.870	0.00E+00	-44.190	0.000
20	USD(max)	1/4	-337.070	0.000	87.720	0.00E+00	-49.720	0.000
20	USD(max)	2/4	-336.140	0.000	97.090	0.00E+00	-55.930	0.000
20	USD(max)	3/4	-335.220	0.000	106.730	0.00E+00	-62.810	0.000
20	USD(max)	J[21]	-334.290	0.000	116.310	0.00E+00	-70.350	0.000
20	USD(min)	I[19]	-428.440	0.000	26.260	0.00E+00	-107.410	0.000
20	USD(min)	1/4	-427.340	0.000	26.260	0.00E+00	-109.660	0.000
20	USD(min)	2/4	-426.250	0.000	26.260	0.00E+00	-111.900	0.000
20	USD(min)	3/4	-425.150	0.000	26.260	0.00E+00	-114.150	0.000
20	USD(min)	J[21]	-424.050	0.000	26.260	0.00E+00	-116.400	0.000
20	WSD(max)	I[19]	-240.490	0.000	49.300	0.00E+00	-33.560	0.000
20	WSD(max)	1/4	-239.800	0.000	54.830	0.00E+00	-36.820	0.000
20	WSD(max)	2/4	-239.120	0.000	60.680	0.00E+00	-40.460	0.000
20	WSD(max)	3/4	-238.430	0.000	66.710	0.00E+00	-44.480	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
20	WSD(max)	J[21]	-237.750	0.000	72.690	0.00E+00	-48.890	0.000
20	WSD(min)	I[19]	-267.780	0.000	17.660	0.00E+00	-67.130	0.000
20	WSD(min)	1/4	-267.090	0.000	17.660	0.00E+00	-68.530	0.000
20	WSD(min)	2/4	-266.400	0.000	17.660	0.00E+00	-69.940	0.000
20	WSD(min)	3/4	-265.720	0.000	17.660	0.00E+00	-71.350	0.000
20	WSD(min)	J[21]	-265.030	0.000	17.660	0.00E+00	-72.750	0.000
21	USD(max)	I[20]	-345.820	0.000	116.310	0.00E+00	-61.850	0.000
21	USD(max)	1/4	-344.170	0.000	123.110	0.00E+00	-67.280	0.000
21	USD(max)	2/4	-342.520	0.000	129.870	0.00E+00	-73.030	0.000
21	USD(max)	3/4	-340.860	0.000	136.600	0.00E+00	-79.100	0.000
21	USD(max)	J[22]	-339.210	0.000	143.300	0.00E+00	-85.120	0.000
21	USD(min)	I[20]	-424.050	0.000	17.460	0.00E+00	-103.260	0.000
21	USD(min)	1/4	-422.090	0.000	18.400	0.00E+00	-106.360	0.000
21	USD(min)	2/4	-420.130	0.000	19.320	0.00E+00	-109.600	0.000
21	USD(min)	3/4	-418.170	0.000	20.240	0.00E+00	-112.990	0.000
21	USD(min)	J[22]	-416.210	0.000	21.140	0.00E+00	-116.520	0.000
21	WSD(max)	I[20]	-237.750	0.000	72.690	0.00E+00	-48.750	0.000
21	WSD(max)	1/4	-236.520	0.000	76.940	0.00E+00	-52.210	0.000
21	WSD(max)	2/4	-235.300	0.000	81.170	0.00E+00	-55.830	0.000
21	WSD(max)	3/4	-234.070	0.000	85.380	0.00E+00	-59.590	0.000
21	WSD(max)	J[22]	-232.850	0.000	89.560	0.00E+00	-63.530	0.000
21	WSD(min)	I[20]	-265.030	0.000	12.570	0.00E+00	-62.590	0.000
21	WSD(min)	1/4	-263.810	0.000	13.130	0.00E+00	-64.880	0.000
21	WSD(min)	2/4	-262.580	0.000	13.670	0.00E+00	-67.290	0.000
21	WSD(min)	3/4	-261.360	0.000	14.210	0.00E+00	-69.820	0.000
21	WSD(min)	J[22]	-260.130	0.000	14.740	0.00E+00	-72.460	0.000
22	USD(max)	I[21]	-334.290	0.000	116.310	0.00E+00	-70.350	0.000
22	USD(max)	1/4	-332.640	0.000	123.110	0.00E+00	-76.120	0.000
22	USD(max)	2/4	-330.980	0.000	129.870	0.00E+00	-82.100	0.000
22	USD(max)	3/4	-329.330	0.000	136.600	0.00E+00	-88.380	0.000
22	USD(max)	J[30]	-327.680	0.000	143.300	0.00E+00	-94.960	0.000
22	USD(min)	I[21]	-424.050	0.000	26.260	0.00E+00	-116.400	0.000
22	USD(min)	1/4	-422.090	0.000	26.260	0.00E+00	-118.010	0.000
22	USD(min)	2/4	-420.130	0.000	26.260	0.00E+00	-119.610	0.000
22	USD(min)	3/4	-418.170	0.000	26.260	0.00E+00	-121.220	0.000
22	USD(min)	J[30]	-416.210	0.000	26.260	0.00E+00	-122.820	0.000
22	WSD(max)	I[21]	-237.750	0.000	72.690	0.00E+00	-48.890	0.000
22	WSD(max)	1/4	-236.520	0.000	76.940	0.00E+00	-52.260	0.000
22	WSD(max)	2/4	-235.300	0.000	81.170	0.00E+00	-55.830	0.000
22	WSD(max)	3/4	-234.070	0.000	85.380	0.00E+00	-59.590	0.000
22	WSD(max)	J[30]	-232.850	0.000	89.560	0.00E+00	-63.540	0.000
22	WSD(min)	I[21]	-265.030	0.000	17.660	0.00E+00	-72.750	0.000
22	WSD(min)	1/4	-263.810	0.000	17.660	0.00E+00	-73.750	0.000
22	WSD(min)	2/4	-262.580	0.000	17.660	0.00E+00	-74.760	0.000
22	WSD(min)	3/4	-261.360	0.000	17.660	0.00E+00	-75.760	0.000
22	WSD(min)	J[30]	-260.130	0.000	17.660	0.00E+00	-76.770	0.000
23	USD(max)	I[22]	-26.260	0.000	-259.090	0.00E+00	-3.050	0.000
23	USD(max)	1/4	-26.260	0.000	-245.730	0.00E+00	9.620	0.000
23	USD(max)	2/4	-26.260	0.000	-232.380	0.00E+00	23.060	0.000
23	USD(max)	3/4	-26.260	0.000	-218.840	0.00E+00	36.210	0.000
23	USD(max)	J[23]	-26.260	0.000	-205.020	0.00E+00	48.570	0.000
23	USD(min)	I[22]	-183.150	0.000	-321.620	0.00E+00	-51.890	0.000
23	USD(min)	1/4	-183.150	0.000	-305.850	0.00E+00	-37.090	0.000
23	USD(min)	2/4	-183.150	0.000	-290.090	0.00E+00	-24.030	0.000
23	USD(min)	3/4	-183.150	0.000	-274.320	0.00E+00	-11.660	0.000
23	USD(min)	J[23]	-183.150	0.000	-258.560	0.00E+00	-0.250	0.000
23	WSD(max)	I[22]	-17.660	0.000	-179.930	0.00E+00	-3.500	0.000
23	WSD(max)	1/4	-17.660	0.000	-171.110	0.00E+00	5.700	0.000
23	WSD(max)	2/4	-17.660	0.000	-162.290	0.00E+00	14.410	0.000
23	WSD(max)	3/4	-17.660	0.000	-153.470	0.00E+00	22.630	0.000
23	WSD(max)	J[23]	-17.660	0.000	-144.650	0.00E+00	30.360	0.000
23	WSD(min)	I[22]	-114.230	0.000	-201.010	0.00E+00	-32.430	0.000
23	WSD(min)	1/4	-114.230	0.000	-191.160	0.00E+00	-22.630	0.000
23	WSD(min)	2/4	-114.230	0.000	-181.300	0.00E+00	-13.320	0.000
23	WSD(min)	3/4	-114.230	0.000	-171.450	0.00E+00	-4.500	0.000
23	WSD(min)	J[23]	-114.230	0.000	-161.600	0.00E+00	3.240	0.000
24	USD(max)	I[23]	-26.260	0.000	-205.020	0.00E+00	48.570	0.000
24	USD(max)	1/4	-26.260	0.000	-186.160	0.00E+00	64.170	0.000
24	USD(max)	2/4	-26.260	0.000	-167.290	0.00E+00	78.300	0.000
24	USD(max)	3/4	-26.260	0.000	-148.430	0.00E+00	90.960	0.000
24	USD(max)	J[24]	-26.260	0.000	-129.570	0.00E+00	102.190	0.000
24	USD(min)	I[23]	-183.150	0.000	-258.560	0.00E+00	-0.250	0.000
24	USD(min)	1/4	-183.150	0.000	-237.040	0.00E+00	14.080	0.000
24	USD(min)	2/4	-183.150	0.000	-215.520	0.00E+00	27.160	0.000
24	USD(min)	3/4	-183.150	0.000	-194.000	0.00E+00	39.000	0.000
24	USD(min)	J[24]	-183.150	0.000	-172.480	0.00E+00	49.590	0.000
24	WSD(max)	I[23]	-17.660	0.000	-144.650	0.00E+00	30.360	0.000
24	WSD(max)	1/4	-17.660	0.000	-132.610	0.00E+00	40.100	0.000
24	WSD(max)	2/4	-17.660	0.000	-120.570	0.00E+00	48.940	0.000
24	WSD(max)	3/4	-17.660	0.000	-108.530	0.00E+00	56.850	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
24	WSD(max)	J[24]	-17.660	0.000	-95.780	0.00E+00	63.870	0.000
24	WSD(min)	I[23]	-114.230	0.000	-161.600	0.00E+00	3.240	0.000
24	WSD(min)	1/4	-114.230	0.000	-148.150	0.00E+00	12.700	0.000
24	WSD(min)	2/4	-114.230	0.000	-134.700	0.00E+00	21.340	0.000
24	WSD(min)	3/4	-114.230	0.000	-121.250	0.00E+00	29.160	0.000
24	WSD(min)	J[24]	-114.230	0.000	-107.800	0.00E+00	36.160	0.000
25	USD(max)	I[24]	-26.260	0.000	-129.570	0.00E+00	102.190	0.000
25	USD(max)	1/4	-26.260	0.000	-110.640	0.00E+00	112.120	0.000
25	USD(max)	2/4	-26.260	0.000	-91.700	0.00E+00	120.570	0.000
25	USD(max)	3/4	-26.260	0.000	-72.770	0.00E+00	127.540	0.000
25	USD(max)	J[25]	-26.260	0.000	-53.840	0.00E+00	133.030	0.000
25	USD(min)	I[24]	-183.150	0.000	-172.480	0.00E+00	49.590	0.000
25	USD(min)	1/4	-183.150	0.000	-150.880	0.00E+00	58.970	0.000
25	USD(min)	2/4	-183.150	0.000	-129.280	0.00E+00	67.100	0.000
25	USD(min)	3/4	-183.150	0.000	-107.680	0.00E+00	73.320	0.000
25	USD(min)	J[25]	-183.150	0.000	-86.080	0.00E+00	78.030	0.000
25	WSD(max)	I[24]	-17.660	0.000	-95.780	0.00E+00	63.870	0.000
25	WSD(max)	1/4	-17.660	0.000	-82.280	0.00E+00	70.050	0.000
25	WSD(max)	2/4	-17.660	0.000	-68.780	0.00E+00	75.310	0.000
25	WSD(max)	3/4	-17.660	0.000	-55.280	0.00E+00	79.640	0.000
25	WSD(max)	J[25]	-17.660	0.000	-41.780	0.00E+00	83.050	0.000
25	WSD(min)	I[24]	-114.230	0.000	-107.800	0.00E+00	36.160	0.000
25	WSD(min)	1/4	-114.230	0.000	-94.300	0.00E+00	42.350	0.000
25	WSD(min)	2/4	-114.230	0.000	-80.800	0.00E+00	47.720	0.000
25	WSD(min)	3/4	-114.230	0.000	-67.300	0.00E+00	52.260	0.000
25	WSD(min)	J[25]	-114.230	0.000	-53.800	0.00E+00	55.970	0.000
26	USD(max)	I[25]	-26.260	0.000	-53.840	0.00E+00	133.030	0.000
26	USD(max)	1/4	-26.260	0.000	-34.980	0.00E+00	137.030	0.000
26	USD(max)	2/4	-26.260	0.000	-16.110	0.00E+00	139.560	0.000
26	USD(max)	3/4	-26.260	0.000	2.750	0.00E+00	140.620	0.000
26	USD(max)	J[26]	-26.260	0.000	21.610	0.00E+00	140.210	0.000
26	USD(min)	I[25]	-183.150	0.000	-86.080	0.00E+00	78.030	0.000
26	USD(min)	1/4	-183.150	0.000	-64.560	0.00E+00	81.480	0.000
26	USD(min)	2/4	-183.150	0.000	-43.040	0.00E+00	83.680	0.000
26	USD(min)	3/4	-183.150	0.000	-21.520	0.00E+00	84.630	0.000
26	USD(min)	J[26]	-183.150	0.000	0.000	0.00E+00	84.340	0.000
26	WSD(max)	I[25]	-17.660	0.000	-41.780	0.00E+00	83.050	0.000
26	WSD(max)	1/4	-17.660	0.000	-28.330	0.00E+00	85.530	0.000
26	WSD(max)	2/4	-17.660	0.000	-14.880	0.00E+00	87.090	0.000
26	WSD(max)	3/4	-17.660	0.000	-1.430	0.00E+00	87.730	0.000
26	WSD(max)	J[26]	-17.660	0.000	12.020	0.00E+00	87.460	0.000
26	WSD(min)	I[25]	-114.230	0.000	-53.800	0.00E+00	55.970	0.000
26	WSD(min)	1/4	-114.230	0.000	-40.350	0.00E+00	58.850	0.000
26	WSD(min)	2/4	-114.230	0.000	-26.900	0.00E+00	60.900	0.000
26	WSD(min)	3/4	-114.230	0.000	-13.450	0.00E+00	62.130	0.000
26	WSD(min)	J[26]	-114.230	0.000	0.000	0.00E+00	62.550	0.000
27	USD(max)	I[26]	-26.260	0.000	21.610	0.00E+00	140.210	0.000
27	USD(max)	1/4	-26.260	0.000	38.490	0.00E+00	138.410	0.000
27	USD(max)	2/4	-26.260	0.000	57.760	0.00E+00	135.300	0.000
27	USD(max)	3/4	-26.260	0.000	77.020	0.00E+00	130.870	0.000
27	USD(max)	J[27]	-26.260	0.000	96.280	0.00E+00	125.130	0.000
27	USD(min)	I[26]	-183.150	0.000	0.000	0.00E+00	84.340	0.000
27	USD(min)	1/4	-183.150	0.000	18.230	0.00E+00	82.880	0.000
27	USD(min)	2/4	-183.150	0.000	36.460	0.00E+00	80.310	0.000
27	USD(min)	3/4	-183.150	0.000	54.680	0.00E+00	76.620	0.000
27	USD(min)	J[27]	-183.150	0.000	72.910	0.00E+00	71.830	0.000
27	WSD(max)	I[26]	-17.660	0.000	12.020	0.00E+00	87.460	0.000
27	WSD(max)	1/4	-17.660	0.000	24.060	0.00E+00	86.310	0.000
27	WSD(max)	2/4	-17.660	0.000	36.100	0.00E+00	84.340	0.000
27	WSD(max)	3/4	-17.660	0.000	48.140	0.00E+00	81.550	0.000
27	WSD(max)	J[27]	-17.660	0.000	60.180	0.00E+00	77.940	0.000
27	WSD(min)	I[26]	-114.230	0.000	0.000	0.00E+00	62.550	0.000
27	WSD(min)	1/4	-114.230	0.000	12.040	0.00E+00	62.130	0.000
27	WSD(min)	2/4	-114.230	0.000	24.080	0.00E+00	60.900	0.000
27	WSD(min)	3/4	-114.230	0.000	36.120	0.00E+00	58.850	0.000
27	WSD(min)	J[27]	-114.230	0.000	48.160	0.00E+00	55.970	0.000
28	USD(max)	I[27]	-26.260	0.000	96.280	0.00E+00	125.130	0.000
28	USD(max)	1/4	-26.260	0.000	115.610	0.00E+00	118.040	0.000
28	USD(max)	2/4	-26.260	0.000	134.950	0.00E+00	109.630	0.000
28	USD(max)	3/4	-26.260	0.000	154.280	0.00E+00	99.890	0.000
28	USD(max)	J[28]	-26.260	0.000	173.610	0.00E+00	88.830	0.000
28	USD(min)	I[27]	-183.150	0.000	72.910	0.00E+00	71.830	0.000
28	USD(min)	1/4	-183.150	0.000	91.210	0.00E+00	65.900	0.000
28	USD(min)	2/4	-183.150	0.000	109.500	0.00E+00	58.830	0.000
28	USD(min)	3/4	-183.150	0.000	127.280	0.00E+00	50.370	0.000
28	USD(min)	J[28]	-183.150	0.000	143.600	0.00E+00	40.800	0.000
28	WSD(max)	I[27]	-17.660	0.000	60.180	0.00E+00	77.940	0.000
28	WSD(max)	1/4	-17.660	0.000	72.260	0.00E+00	73.490	0.000
28	WSD(max)	2/4	-17.660	0.000	84.340	0.00E+00	68.210	0.000
28	WSD(max)	3/4	-17.660	0.000	96.430	0.00E+00	62.110	0.000

Elem	Load	Part	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN·m)	Moment-y (kN·m)	Moment-z (kN·m)
28	WSD(max)	J[28]	-17.660	0.000	108.510	0.00E+00	55.170	0.000
28	WSD(min)	I[27]	-114.230	0.000	48.160	0.00E+00	55.970	0.000
28	WSD(min)	1/4	-114.230	0.000	60.240	0.00E+00	52.260	0.000
28	WSD(min)	2/4	-114.230	0.000	72.320	0.00E+00	47.640	0.000
28	WSD(min)	3/4	-114.230	0.000	84.410	0.00E+00	41.620	0.000
28	WSD(min)	J[28]	-114.230	0.000	96.490	0.00E+00	34.770	0.000
29	USD(max)	I[28]	-26.260	0.000	173.610	0.00E+00	88.830	0.000
29	USD(max)	1/4	-26.260	0.000	194.000	0.00E+00	76.500	0.000
29	USD(max)	2/4	-26.260	0.000	215.520	0.00E+00	62.850	0.000
29	USD(max)	3/4	-26.260	0.000	237.040	0.00E+00	47.880	0.000
29	USD(max)	J[29]	-26.260	0.000	258.560	0.00E+00	31.600	0.000
29	USD(min)	I[28]	-183.150	0.000	143.600	0.00E+00	40.800	0.000
29	USD(min)	1/4	-183.150	0.000	159.850	0.00E+00	30.150	0.000
29	USD(min)	2/4	-183.150	0.000	176.100	0.00E+00	18.400	0.000
29	USD(min)	3/4	-183.150	0.000	192.350	0.00E+00	5.530	0.000
29	USD(min)	J[29]	-183.150	0.000	208.610	0.00E+00	-8.440	0.000
29	WSD(max)	I[28]	-17.660	0.000	108.510	0.00E+00	55.170	0.000
29	WSD(max)	1/4	-17.660	0.000	121.250	0.00E+00	47.440	0.000
29	WSD(max)	2/4	-17.660	0.000	134.700	0.00E+00	38.890	0.000
29	WSD(max)	3/4	-17.660	0.000	148.150	0.00E+00	29.520	0.000
29	WSD(max)	J[29]	-17.660	0.000	161.600	0.00E+00	19.320	0.000
29	WSD(min)	I[28]	-114.230	0.000	96.490	0.00E+00	34.770	0.000
29	WSD(min)	1/4	-114.230	0.000	108.530	0.00E+00	27.120	0.000
29	WSD(min)	2/4	-114.230	0.000	120.570	0.00E+00	18.660	0.000
29	WSD(min)	3/4	-114.230	0.000	132.610	0.00E+00	9.370	0.000
29	WSD(min)	J[29]	-114.230	0.000	144.650	0.00E+00	-0.740	0.000
30	USD(max)	I[29]	-26.260	0.000	258.560	0.00E+00	31.600	0.000
30	USD(max)	1/4	-26.260	0.000	274.320	0.00E+00	18.840	0.000
30	USD(max)	2/4	-26.260	0.000	290.090	0.00E+00	5.370	0.000
30	USD(max)	3/4	-26.260	0.000	305.850	0.00E+00	-8.800	0.000
30	USD(max)	J[30]	-26.260	0.000	321.620	0.00E+00	-23.290	0.000
30	USD(min)	I[29]	-183.150	0.000	208.610	0.00E+00	-8.440	0.000
30	USD(min)	1/4	-183.150	0.000	220.510	0.00E+00	-19.390	0.000
30	USD(min)	2/4	-183.150	0.000	232.420	0.00E+00	-30.920	0.000
30	USD(min)	3/4	-183.150	0.000	244.330	0.00E+00	-43.060	0.000
30	USD(min)	J[30]	-183.150	0.000	256.230	0.00E+00	-56.170	0.000
30	WSD(max)	I[29]	-17.660	0.000	161.600	0.00E+00	19.320	0.000
30	WSD(max)	1/4	-17.660	0.000	171.450	0.00E+00	11.330	0.000
30	WSD(max)	2/4	-17.660	0.000	181.300	0.00E+00	2.900	0.000
30	WSD(max)	3/4	-17.660	0.000	191.160	0.00E+00	-5.980	0.000
30	WSD(max)	J[30]	-17.660	0.000	201.010	0.00E+00	-15.290	0.000
30	WSD(min)	I[29]	-114.230	0.000	144.650	0.00E+00	-0.740	0.000
30	WSD(min)	1/4	-114.230	0.000	153.470	0.00E+00	-8.670	0.000
30	WSD(min)	2/4	-114.230	0.000	162.290	0.00E+00	-17.040	0.000
30	WSD(min)	3/4	-114.230	0.000	171.110	0.00E+00	-25.850	0.000
30	WSD(min)	J[30]	-114.230	0.000	179.930	0.00E+00	-35.110	0.000

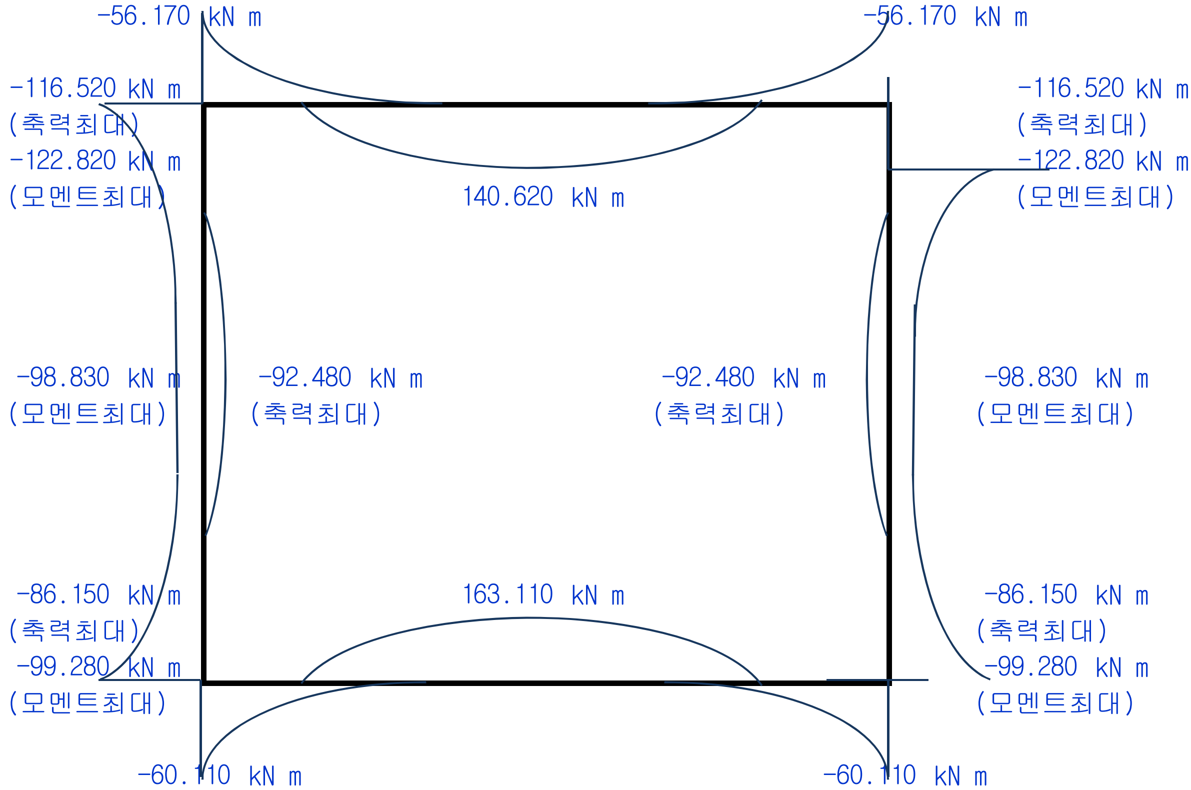
9. 단면력 요약

구분	위치	부재	극한하중			사용하중	비고
			Mu(kN · m)	Su(kN)	Pu(kN)	M(kN · m)	
상부슬래브	단부	23,30	-56.17	321.62	-	-35.11	
	중앙부	26,27	140.62	96.28	-	87.73	
벽체	상단부 (축력최대)	21,22	-116.52	143.30	-424.05	-48.75	
	상단부 (모멘트최대)	21,22	-122.82	143.30	-424.05	-76.77	
	중앙부 (축력최대)	15,16	-92.48	42.80	-437.22	-21.75	
	중앙부 (모멘트최대)	15,16	-98.83	-15.87	-437.22	-61.72	
	하단부 (축력최대)	9,10	-86.15	-133.69	-454.24	-34.51	
	하단부 (모멘트최대)	9,10	-99.28	-133.69	-454.24	-62.05	
하부슬래브	단부	1,8	-60.11	314.38	-	-36.03	
	중앙부	4,5	163.11	65.50	-	101.66	

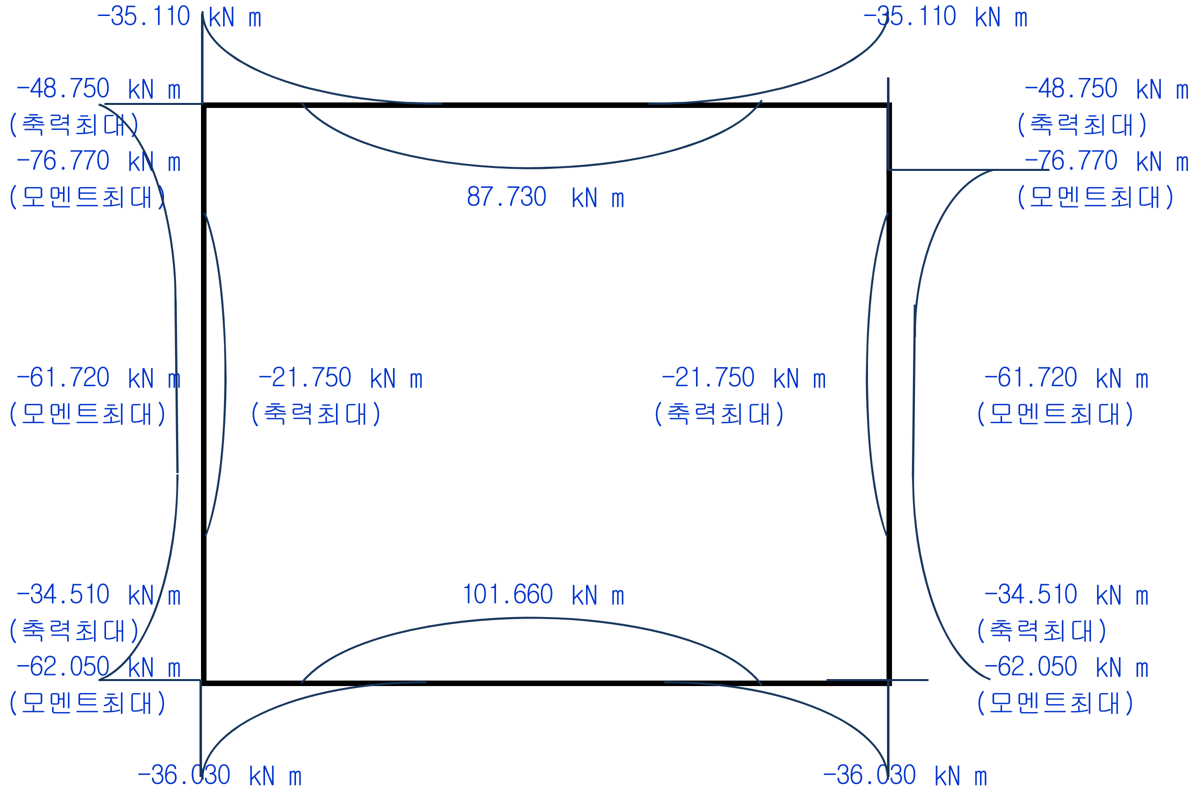
-'전단력과 사용하중 상태에서의 모멘트는 선택 부재에서의 최대값 적용

10. 단면력도

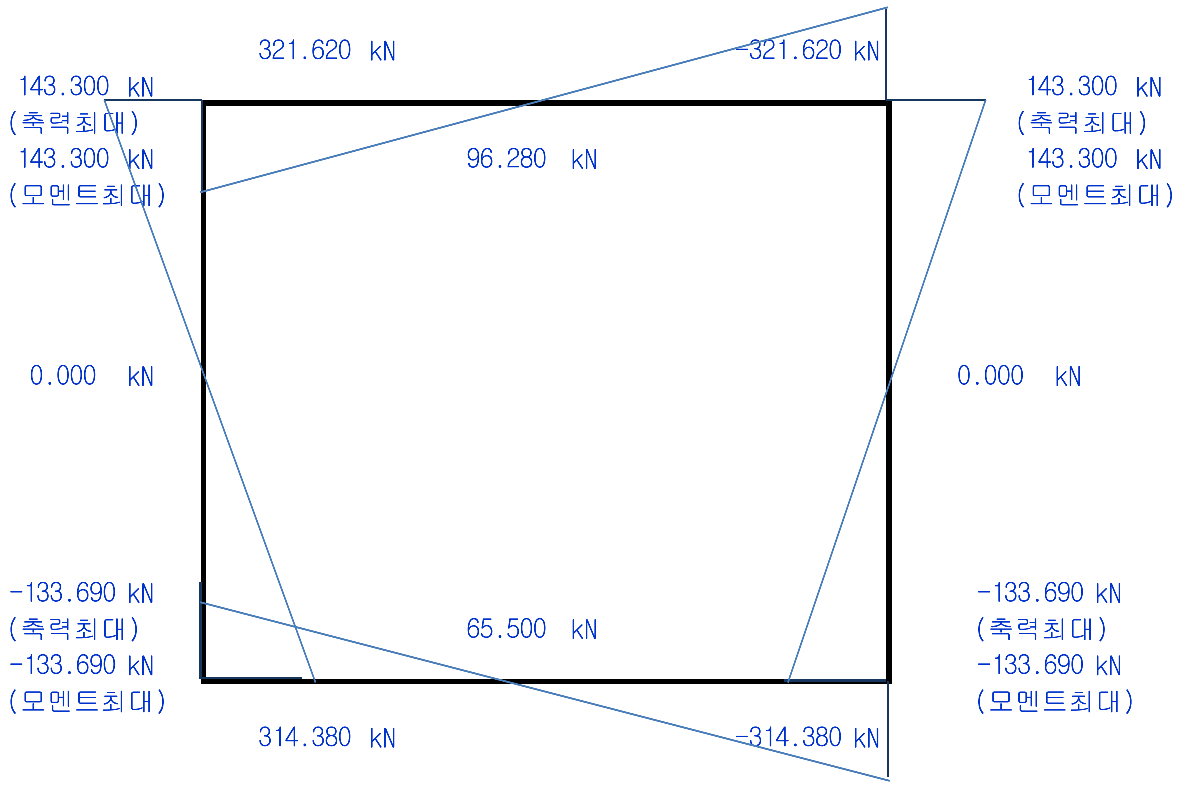
(1) 극한하중 모멘트도



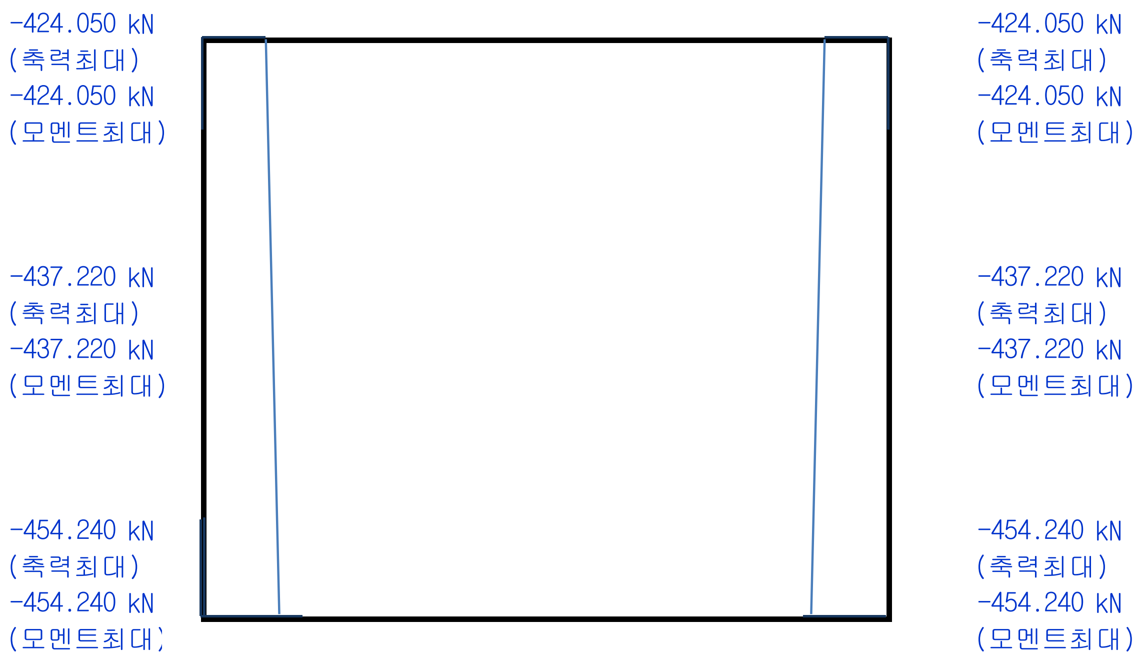
(2) 사용하중 모멘트도



(3) 극한하중 전단력도



(4) 극한하중 축력도



11. 단면검토

① 상부 슬래브 단부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	$\beta 1$	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	35.110
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Su (kN)
1000.0	500.0	400.0	100.0	56.170	321.620

전단설계시 단면의 유효높이 ds = 300.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.03725 ≥ 0.004 ----- ∴ O.K.

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2 \times 0.85 \times fck \times b} \times As^2 - fy \times d \times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 393.099 mm^2

사용철근비 : Puse = As / (B × D) = 0.00397

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req) = 393.099 mm^2 As(min) = 1400.000 mm^2

4/3×As(req) = 524.133 mm^2 ※ As = 524.133 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ O.K.

ⓑ 휨강도 검토

ØMn = Øf · fy · As · (D - a/2) = 221659907 N · mm

= 221.660 kN · m > 56.170 kN · m ----- ∴ O.K. [사용률 = 3.946]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d \ / \ 1000 =$ 232.782 kN

$Vu > \Phi v \times Vc$ ∴ 전단철근보강이 필요하다.

Req Av $= (321.6 - 232.782) \times 10^3 \times 150.0 \ / \ (400.0 \times 300.0 \times 0.85)$

= 130.644 ㎟

Use Av = 253.400 ㎟ (H13×2 EA, 간격 s = 150.0 mm)

$\varnothing v \cdot As = \varnothing v \cdot Av \cdot fy \cdot D \ / \ (s \times B) =$ 172.312 kN

s = 150.0 mm ≥ Min (600, 0.5D) = 150.0 mm ----- ∴ O.K.

$\varnothing v \cdot Vn = \varnothing v \cdot Vc + \varnothing v \cdot Vs =$ 405.094 kN > Vu ----- ∴ O.K.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M \ / \ [As \times (d - \chi/3)]$

$= 35.11 \times 1000000 \ / \ [1588.8 \times (400 - 83.857/3)]$

= 59.397 Mpa

$\chi = -nAs/b + nAs/b \sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 400/(7 \times 1588.8)]}$

= 83.857 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 \ / \ fs) - 2.5 \times Cc$ $= 375 \times (210 \ / \ 59.397) - 2.5 \times 92$ = 1095.829 mm

$300 \times (210 \ / \ fs)$ $= 300 \times (210 \ / \ 59.397)$ = 1060.663 mm

Sa는 작은 값인 1060.663 mm 를 적용

S = 125.00 ≤ Sa (= 1060.663 mm) ----- ∴ O.K.

② 상부슬래브 중앙부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	$\beta 1$	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	87.730
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	400.0	300.0	100.0	140.620	96.280

전단설계시 단면의 유효높이 ds = 300.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.02719 ≥ 0.004 ----- ∴ O.K.

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2\times 0.85\times fck\times b}\times As^2 - fy\times d\times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 1349.660 mm^2

사용철근비 : Puse = As / (B × D) = 0.00530

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req)	=	1349.660 mm^2	As(min)	=	1050.000 mm^2
4/3×As(req)	=	1799.547 mm^2	※ As	=	1349.660 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ O.K.

ⓑ 휨강도 검토

∅Mn = ∅f · fy · As · (D - a/2) = 164463107 N · mm

= 164.463 kN · m > 140.620 kN · m ----- ∴ O.K. [사용률 = 1.170]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d \ / \ 1000 =$ 232.782 kN

∴ 전단철근이 필요없음.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M \ / \ [As \times (d - \chi/3)]$

$= 87.73 \times 1000000 \ / \ [1588.8 \times (300 - 71.32/3)]$

$=$ 199.900 Mpa

$\chi = -nAs/b + nAs/b\sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 300/(7 \times 1588.8)]}$

$=$ 71.320 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

$Cc = 100 - 16 \ / \ 2 = 92.00$ mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 \ / \ fs) - 2.5 \times Cc = 375 \times (210 \ / \ 199.9) - 2.5 \times 92 =$ 163.946 mm

$300 \times (210 \ / \ fs) = 300 \times (210 \ / \ 199.9) =$ 315.157 mm

Sa는 작은 값인 163.946 mm 를 적용

$S = 125.00 \leq Sa$ (= 163.946 mm) ----- ∴ O.K.

③ 하부슬래브 단부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	β1	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	36.030
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	550.0	450.0	100.0	60.110	314.380

전단설계시 단면의 유효높이 ds = 350.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.04229 ≥ 0.004 ----- ∴ 0.K

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a/2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2\times 0.85\times fck\times b}\times As^2 - fy\times d\times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 373.481 mm^2

사용철근비 : Puse = As / (B × D) = 0.00353

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req) = 373.481 mm^2 As(min) = 1575.000 mm^2

4/3×As(req) = 497.974 mm^2 ※ As = 497.974 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ 0.K.

ⓑ 휨강도 검토

∅Mn = ∅f · fy · As · (D - a/2) = 250258307 N · mm

= 250.258 kN · m > 60.110 kN · m ----- ∴ 0.K [사용률 = 4.163]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d$ / 1000 = 271.579 kN

Vu > Φv x Vc ∴ 전단철근보강이 필요하다.

Req Av = $(314.4 - 271.579) \times 10^3 \times 150.0 / (400.0 \times 350.0 \times 0.85)$

= 53.951 ㎟

Use Av = 253.400 ㎟ (H13×2 EA, 간격 s = 150.0 mm)

$\varnothing v \cdot As = \varnothing v \cdot Av \cdot fy \cdot D / (s \times B)$ = 201.031 kN

s = 150.0 mm < Min (600, 0.5D) = 175.0 mm ----- ∴ O.K

$\varnothing v \cdot Vn = \varnothing v \cdot Vc + \varnothing v \cdot Vs$ = 472.610 kN > Vu ----- ∴ O.K

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M / [As \times (d - \chi/3)]$

$= 36.03 \times 1000000 / [1588.8 \times (450 - 89.542/3)]$

= 53.974 Mpa

$\chi = -nAs/b + nAs/b \sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 450/(7 \times 1588.8)]}$

= 89.542 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x Cc = 375 × (210 / 53.974) - 2.5 × 92 = 1229.025 mm

300 x (210 / fs) = 300 × (210 / 53.974) = 1167.220 mm

Sa는 작은 값인 1167.220 mm 를 적용

S = 125.00 ≤ Sa (= 1167.22 mm) ----- ∴ O.K.

④ 하부슬래브 중앙부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	β1	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	101.660
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	450.0	350.0	100.0	163.110	65.500

전단설계시 단면의 유효높이 ds = 350.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.03222 ≥ 0.004 ----- ∴ 0.K

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2\times0.85\times fck\times b}\times As^2 - fy\times d\times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 1334.427 mm^2

사용철근비 : Puse = As / (B × D) = 0.00454

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req) = 1334.427 mm^2 As(min) = 1225.000 mm^2

4/3×As(req) = 1779.236 mm^2 ※ As = 1334.427 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ 0.K.

ⓑ 휨강도 검토

ØMn = Øf · fy · As · (D - a/2) = 193061507 N · mm

= 193.062 kN · m > 163.110 kN · m ----- ∴ 0.K [사용률 = 1.184]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d \,/\, 1000 =$ 271.579 kN

∴ 전단철근이 필요없음.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M / [As \times (d - \chi/3)]$

$= 101.66 \times 1000000 / [1588.8 \times (350 - 77.81/3)]$

$=$ 197.447 Mpa

$\chi = -nAs/b + nAs/b \sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 350/(7 \times 1588.8)]}$

$=$ 77.810 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

$Cc = 100 - 16 / 2 = 92.00$ mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 / fs) - 2.5 \times Cc = 375 \times (210 / 197.447) - 2.5 \times 92 =$ 168.841 mm

$300 \times (210 / fs) = 300 \times (210 / 197.447) =$ 319.073 mm

Sa는 작은 값인 168.841 mm 를 적용

$S = 125.00 \le Sa$ (= 168.841 mm) ----- ∴ O.K.

⑤ 벽체 상단부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	β1	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	76.770
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	500.0	400.0	100.0	122.820	143.300

전단설계시 단면의 유효높이 ds = 300.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.03725 ≥ 0.004 ----- ∴ 0.K

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2 \times 0.85 \times fck \times b} \times As^2 - fy \times d \times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 867.679 mm^2

사용철근비 : Puse = As / (B × D) = 0.00397

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req) = 867.679 mm^2 As(min) = 1400.000 mm^2

4/3×As(req) = 1156.905 mm^2 ※ As = 1156.905 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ 0.K.

ⓑ 휨강도 검토

∅Mn = ∅f · fy · As · (D - a/2) = 221659907 N · mm

= 221.660 kN · m > 122.820 kN · m ----- ∴ 0.K [사용률 = 1.805]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d \ / \ 1000 =$ 232.782 kN

$1/2 \times \Phi v \times Vc = 116.391$ kN $\le Vu \le \Phi v \times Vc$ ∴ 슬래브이므로 최소전단 철근 배근이 필요없음.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M \ / \ [As \times (d - \chi/3)]$

$= 76.77 \times 1000000 \ / \ [1588.8 \times (400 - 83.857/3)]$

$=$ 129.874 Mpa

$\chi = -nAs/b + nAs/b\sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 400/(7 \times 1588.8)]}$

$=$ 83.857 ㎜

사용철근량 = 1588.800 mm^2 (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

$Cc = 100 - 16 \ / \ 2 = 92.00$ ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 \ / \ fs) - 2.5 \times Cc = 375 \times (210 \ / \ 129.874) - 2.5 \times 92 =$ 376.355 ㎜

$300 \times (210 \ / \ fs) = 300 \times (210 \ / \ 129.874) =$ 485.084 ㎜

Sa는 작은 값인 376.355 ㎜ 를 적용

$S = 125.00 \le Sa$ (= 376.355 mm) ----- ∴ O.K.

⑥ 벽체 중앙부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	β1	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	61.720
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	400.0	300.0	100.0	98.830	15.870

전단설계시 단면의 유효높이 ds = 300.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.02719 ≥ 0.004 ----- ∴ 0.K

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ----------------- (1)

a = As×fy / (0.85×fck×b) ----------------- (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2\times0.85\times fck\times b}\times As^2 - fy\times d\times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 938.100 mm^2

사용철근비 : Puse = As / (B × D) = 0.00530

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req) = 938.100 mm^2 As(min) = 1050.000 mm^2

4/3×As(req) = 1250.800 mm^2 ※ As = 1050.000 mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ 0.K.

ⓑ 휨강도 검토

ØMn = Øf · fy · As · (D - a/2) = 164463107 N · mm

= 164.463 kN · m > 98.830 kN · m ----- ∴ 0.K [사용률 = 1.664]

* 전단력 검토

$\varnothing v \cdot Vc = \varnothing v \cdot 1/6 \cdot \sqrt{fck} \cdot B \cdot d$ / 1000 = 232.782 kN

∴ 전단철근이 필요없음.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$fs = M / [As \times (d - \chi/3)]$

$= 61.72 \times 1000000 / [1588.8 \times (300 - 71.32/3)]$

$= 140.634$ Mpa

$\chi = -nAs/b + nAs/b \sqrt{[1 + 2bd/(nAs)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 300/(7 \times 1588.8)]}$

$= 71.320$ mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

$Cc = 100 - 16 / 2 = 92.00$ mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 / fs) - 2.5 \times Cc = 375 \times (210 / 140.634) - 2.5 \times 92 = 329.963$ mm

$300 \times (210 / fs) = 300 \times (210 / 140.634) = 447.970$ mm

Sa는 작은 값인 329.963 mm 를 적용

$S = 125.00 \leq Sa$ (= 329.963 mm) ----- ∴ O.K.

⑦ 벽체 하단부

* 검토 조건

pmin = max(0.25√fck / fy, 1.4/fy) = 0.00350

fck (Mpa)	fy (Mpa)	β1	Φf	Φv	Mo (kN.m)
30.0	400.0	0.836	0.900	0.850	62.050
B (mm)	H (mm)	d (mm)	피복 (mm)	Mu (kN.m)	Vu (kN)
1000.0	500.0	400.0	100.0	99.280	133.690

전단설계시 단면의 유효높이 ds = 300.0 d1' = 100 d2' = 0

* 휨모멘트 검토

공칭강도시 등가응력깊이 a = (As×fy) / (0.85×fck×B) = 24.922 mm

최외단 인장철근 변형률 εt = 0.03725 ≥ 0.004 ----- ∴ O.K

여기서 εt = 0.003×(H - a/β1 - Dc_min) / (a/β1)

0.005 ≤ εt 이므로 인장지배단면, Φf = 0.900 를 적용한다.

ⓐ 필요철근량 및 철근비 검토

Mu / Φ = As×fy / (d - a / 2) ------------------ (1)

a = As×fy / (0.85×fck×b) ------------------ (2)

式(2)를 式(1)에 대입하여 2차방정식으로 As를 구한다.

$$\frac{fy^2}{2\times 0.85\times fck\times b}\times As^2 - fy\times d\times As + \frac{Mu}{\Phi} = 0$$

필요 철근량 req.As = 699.026 mm^2

사용철근비 : Puse = As / (B × D) = 0.00397

최대철근비 : Pmax = 0.714 x k1 x Φx (fck / fy) x {600 / (600 + fy)} = 0.02283

최소철근비 : Pmin = 1.4 / fy = 0.00350

Pmin = 0.25 √fck / fy = 0.00342

적용 = 0.0035

As(req)	=	699.026	mm^2	As(min)	=	1400.000	mm^2
4/3×As(req)	=	932.034	mm^2	※ As	=	932.034	mm^2

사용철근량 = H16 × 8ea = 1588.800 ㎟ ----- ∴ O.K.

ⓑ 휨강도 검토

∅Mn = ∅f · fy · As · (D - a/2) = 221659907 N · mm

= 221.660 kN · m > 99.280 kN · m ----- ∴ O.K [사용률 = 2.233]

* 전단력 검토

$\phi_v \cdot V_c = \phi_v \cdot 1/6 \cdot \sqrt{f_{ck}} \cdot B \cdot d \ / \ 1000 =$ 232.782 kN

$1/2 \times \Phi_v \times V_c = 116.391$ kN $\leq V_u \leq \Phi_v \times V_c$ ∴ 슬래브이므로 최소전단 철근 배근이 필요없음.

▣ 사용성 검토 (균열방지 인장철근 간격검토)

ⓐ 응력산정

$f_s = M \ / \ [A_s \times (d - \chi/3)]$

$= 62.05 \times 1000000 \ / \ [1588.8 \times (400 - 83.857/3)]$

$=$ 104.972 Mpa

$\chi = -nA_s/b + nA_s/b\sqrt{[1 + 2bd/(nA_s)]}$

$= -7 \times 1588.8/1000 + 7 \times 1588.8/1000 \times \sqrt{[(1 + 2 \times 1000 \times 400/(7 \times 1588.8)]}$

$=$ 83.857 ㎜

사용철근량 = 1588.800 mm^2 (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

ⓑ 철근의 최대 중심간격

$C_c = 100 - 16 \ / \ 2 = 92.00$ ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : $375 \times (210 \ / \ f_s) - 2.5 \times C_c = 375 \times (210 \ / \ 104.972) - 2.5 \times 92 =$ 520.199 ㎜

$300 \times (210 \ / \ f_s) = 300 \times (210 \ / \ 104.972) =$ 600.159 ㎜

Sa는 작은 값인 520.199 ㎜ 를 적용

$S = 125.00 \leq S_a$ (= 520.199 mm) ----- ∴ O.K.

⑤ 벽체 상단부 - Pmax 일 경우

설계축력	:	Pu	=	424.05	(kN) ,	설계모멘트	:	Mu	=	116.52	(kN · m)				
설계전단력	:	Vu	=	143.30	(kN) ,	사용 모멘트	:	M	=	48.75	(kN.m)				
부 재 높 이	:	H	=	500.0	(mm) ,	f_y	=	400	MPa		f_{ck}	=	30	MPa	
폭	:	b	=	1000	(mm) ,	ϕ_f	=	0.90	,		ϕ_v	=	0.85		
유효높이(휨)	:	d	=	400.0	(mm) ,	d'	=	100	(mm),		ϕc	=	0.70		
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1	=	0.836	,		e	=	274.78	(mm)	

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0048) < Pmax (0.0800) ----------- O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 500.00 x 1000.00 = 12540000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (12540000.00 x 250.0 + 635520.0 x 100.0 + 317760.0 x 400.0) / 13493280.000

= 246.468 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 240.000 mm, ab = k1 x cb = 200.640 mm

fs' = 600 x (240.000 - 100.0) / 240.0 = 350 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 200.640 x 1000 = 5032051.200 N

Cs = As' x (fs' - 0.9 x fck) = 256591.200 N

Ts = As x fs = 635520.000 N

Pb = 5032051.200 + 256591.200 - 635520.000 = 4653122.400 N

Mb = 5032051.200x(400.0-200.640/2)+256591.200x(400.0-100.0)-4653122.400x(246.468-100.0)

= 903450909.646 N.mm

eb < e → 인장파괴 (eb = Mb / Pb = 194.160 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 146.000 mm , a = k1 x c = 122.056 mm 사용

fs = 600 x (400.000 - 146.000) / 146.000 = 400 Mpa (fs ≤ fy)

fs' = 600 x (146.000 - 100.000) / 146.000 = 189.0411 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 122.056 x 1000 = 3061164.480 N

Cs = 794.400 x (189.041 - 0.836 x 30) = 130250.695 N

Ts = 1588.800 x 400.000 = 635520.000 N

Pn = 3061164.480 + 130250.695 - 635520.000 = 2555895.175 N

Mn' = 3061164.480 x (400.0 - 122.056 / 2) + 130250.695 x (400.0 - 100.0)

= 1076724254.487 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.024 > 0.001 (e' = e + Xp - dc = 421.246493 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 10914006.720 N ≥ Pn = 2555895.175

ΦPn = 1789126.622 > ΦPb = 3257185.680 , 0.1 x fck x Ag = 1500000.000

∴ Φ = 0.700

ΦPn = 0.70 x 2555895.175 = 1789.127 kN ≥ Pu = 424.050 kN ------ 0.K!!!

ΦMn = ΦPn x 274.779 = 491.614 kN.m ≥ Mu = 116.520 kN.m ------ 0.K!!!

○ 전단력 검토

ϕVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 246883.693 N

여기서 , 1/2ϕVc< Vu ≤ ϕVc 이므로 최소 전단 철근 필요

- 계산된 스터럽 : min.Av = 0.35x(bo x s / fy)= 1.31㎠

- 배근할 스터럽 : H 13 다리수: N = 2 스터럽 간격 : s = 15 cm 0.K !!!

Av(req)= 1.31㎠ < Av = 2.534 cm² 0.K !!!

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 48.75 × 1000000 / [1588.8×(400 - 83.857/3)]

= 82.472 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×400/(7×1588.8)]

= 83.857 ㎜

사용철근량 = 1588.800 mm^2 (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x Cc = 375 × (210 / 82.472) - 2.5 × 92 = 724.869 ㎜

300 x (210 / fs) = 300 × (210 / 82.472) = 763.895 ㎜

Sa는 작은 값인 724.869 ㎜ 를 적용

S = 125.00 ≤ Sa (= 724.869 mm) ----- ∴ 0.K.

⑥ 벽체 상단부 - Mmax 일 경우

설계축력	:	Pu	=	424.05	(kN) ,	설계모멘트		:	Mu	=	122.82	(kN · m)				
설계전단력	:	Vu	=	143.30	(kN) ,	사용 모멘트		:	M	=	76.77	(kN.m)				
부 재 높 이	:	H	=	500.0	(mm) ,	f_y	=	400	MPa			f_{ck}	=	30	MPa	
폭	:	b	=	1000	(mm) ,	ϕ_f	=	0.90	,			ϕ_v	=	0.85		
유효높이(휨)	:	d	=	400.0	(mm) ,	d'	=	100	(mm),			ϕc	=	0.70		
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1	=	0.836	,			e	=	289.64	(mm)	

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0048) < Pmax (0.0800) ------------ O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 500.00 x 1000.00 = 12540000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (12540000.00 x 250.0 + 635520.0 x 100.0 + 317760.0 x 400.0) / 13493280.000

= 246.468 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 240.000 mm, ab = k1 x cb = 200.640 mm

fs' = 600 x (240.000 - 100.0) / 240.0 = 350 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 200.640 x 1000 = 5032051.200 N

Cs = As' x (fs' - 0.9 x fck) = 256591.200 N

Ts = As x fs = 635520.000 N

Pb = 5032051.200 + 256591.200 - 635520.000 = 4653122.400 N

Mb = 5032051.200x(400.0-200.640/2)+256591.200x(400.0-100.0)-4653122.400x(246.468-100.0)

= 903450909.646 N.mm

eb < e → 인장파괴 (eb = Mb / Pb = 194.160 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 135.000 mm , a = k1 x c = 112.860 mm 사용

fs = 600 x (400.000 - 135.000) / 135.000 = 400 Mpa (fs ≤ fy)

fs' = 600 x (135.000 - 100.000) / 135.000 = 155.5556 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 112.860 x 1000 = 2830528.800 N

Cs = 794.400 x (155.556 - 0.836 x 30) = 103649.781 N

Ts = 1588.800 x 400.000 = 635520.000 N

Pn = 2830528.800 + 103649.781 - 635520.000 = 2298658.581 N

Mn' = 2830528.800 x (400.0 - 112.860 / 2) + 103649.781 x (400.0 - 100.0)

= 1003579714.216 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.490 > 0.001 (e' = e + Xp - dc = 436.103231 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 10914006.720 N ≥ Pn = 2298658.581

ΦPn = 1609061.007 > ΦPb = 3257185.680 , 0.1 x fck x Ag = 1500000.000

∴ Φ = 0.700

ΦPn = 0.70 x 2298658.581 = 1609.061 kN ≥ Pu = 424.050 kN ------ O.K!!!

ΦMn = ΦPn x 289.636 = 466.041 kN.m ≥ Mu = 122.820 kN.m ------ O.K!!!

○ 전단력 검토

ϕVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 246883.693 N

여기서 , 1/2ϕVc< Vu ≤ ϕVc 이므로 최소 전단 철근 필요

- 계산된 스터럽 : min.Av = 0.35x(bo x s / fy)= 1.31㎠

- 배근할 스터럽 : H 13 다리수: N = 2 스터럽 간격 : s = 15 cm O.K !!!

Av(req)= 1.31㎠ < Av = 2.534 cm² O.K !!!

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 76.77 × 1000000 / [1588.8×(400 - 83.857/3)]

= 129.874 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×400/(7×1588.8)]

= 83.857 ㎜

사용철근량 = 1588.800 ㎜^{2} (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x Cc = 375 × (210 / 129.874) - 2.5 × 92 = 376.355 ㎜

300 x (210 / fs) = 300 × (210 / 129.874) = 485.084 ㎜

Sa는 작은 값인 376.355 ㎜ 를 적용

S = 125.00 ≤ Sa (= 376.355 mm) ----- ∴ O.K.

⑦ 벽체 중앙부 - Pmax 일 경우

설계축력	:	Pu	=	437.22	(kN) ,	설계모멘트	:	Mu	=	92.48	(kN · m)
설계전단력	:	Vu	=	42.80	(kN) ,	사용 모멘트	:	M	=	21.75	(kN.m)
부 재 높 이	:	H	=	400.0	(mm) ,	f_y = 400 MPa		f_{ck}	=	30	MPa
폭	:	b	=	1000	(mm) ,	ϕ_f = 0.90 ,		ϕ_v	=	0.85	
유효높이(휨)	:	d	=	300.0	(mm) ,	d' = 100 (mm),		ϕc	=	0.70	
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1 = 0.836 ,		e	=	211.52	(mm)

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0060) < Pmax (0.0800) ----------- O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 400.00 x 1000.00 = 10032000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (10032000.00 x 200.0 + 635520.0 x 100.0 + 317760.0 x 300.0) / 10985280.000

= 197.107 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 180.000 mm, ab = k1 x cb = 150.480 mm

fs' = 600 x (180.000 - 100.0) / 180.0 = 266.6667 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 150.480 x 1000 = 3774038.400 N

Cs = As' x (fs' - 0.9 x fck) = 190391.200 N

Ts = As x fs = 635520.000 N

Pb = 3774038.400 + 190391.200 - 635520.000 = 3328909.600 N

Mb = 3774038.400x(300.0-150.480/2)+190391.200x(300.0-100.0)-3328909.600x(197.107-100.0)

= 563069348.351 N.mm

eb < e → 인장파괴 (eb = Mb / Pb = 169.145 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 135.000 mm , a = k1 x c = 112.860 mm 사용

fs = 600 x (300.000 - 135.000) / 135.000 = 400 Mpa (fs ≤ fy)

fs' = 600 x (135.000 - 100.000) / 135.000 = 155.5556 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 112.860 x 1000 = 2830528.800 N

Cs = 794.400 x (155.556 - 0.836 x 30) = 103649.781 N

Ts = 1588.800 x 400.000 = 635520.000 N

Pn = 2830528.800 + 103649.781 - 635520.000 = 2298658.581 N

Mn' = 2830528.800 x (300.0 - 112.860 / 2) + 103649.781 x (300.0 - 100.0)

= 710161856.083 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.321 > 0.001 (e' = e + Xp - dc = 308.625631 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 8874006.720 N ≥ Pn = 2298658.581

ΦPn = 1609061.007 > ΦPb = 2330236.720 , 0.1 x fck x Ag = 1200000.000

∴ Φ = 0.700

ΦPn = 0.70 x 2298658.581 = 1609.061 kN ≥ Pu = 437.220 kN ------ 0.K!!!

ΦMn = ΦPn x 211.518 = 340.346 kN.m ≥ Mu = 92.480 kN.m ------ 0.K!!!

○ 전단력 검토

ϕVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 250956.548 N

여기서 , Vu < 1/2ϕVc 이므로 전단철근보강 불필요

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 21.75 × 1000000 / [1588.8×(300 - 71.32/3)]

= 49.559 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×300/(7×1588.8)]

= 71.320 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x Cc = 375 × (210 / 49.559) - 2.5 × 92 = 1359.01 mm

300 x (210 / fs) = 300 × (210 / 49.559) = 1271.21 mm

Sa는 작은 값인 1271.21 mm 를 적용

S = 125.00 ≤ Sa (= 1271.206 mm) ----- ∴ O.K.

⑧ 벽체 중앙부 - Mmax 일 경우

설계축력	:	Pu	=	437.22	(kN) ,	설계모멘트	:	Mu	=	98.83	(kN·m)				
설계전단력	:	Vu	=	15.87	(kN) ,	사용 모멘트	:	M	=	61.72	(kN.m)				
부 재 높 이	:	H	=	400.0	(mm) ,	f_y	=	400	MPa		f_{ck}	=	30	MPa	
폭	:	b	=	1000	(mm) ,	ϕ_f	=	0.90	,		ϕ_v	=	0.85		
유효높이(휨)	:	d	=	300.0	(mm) ,	d'	=	100	(mm),		ϕc	=	0.70		
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1	=	0.836	,		e	=	226.04	(mm)	

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0060) < Pmax (0.0800) ------------ O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 400.00 x 1000.00 = 10032000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (10032000.00 x 200.0 + 635520.0 x 100.0 + 317760.0 x 300.0) / 10985280.000

= 197.107 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 180.000 mm, ab = k1 x cb = 150.480 mm

fs' = 600 x (180.000 - 100.0) / 180.0 = 266.6667 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 150.480 x 1000 = 3774038.400 N

Cs = As' x (fs' - 0.9 x fck) = 190391.200 N

Ts = As x fs = 635520.000 N

Pb = 3774038.400 + 190391.200 - 635520.000 = 3328909.600 N

Mb = 3774038.400x(300.0-150.480/2)+190391.200x(300.0-100.0)-3328909.600x(197.107-100.0)

= 563069348.351 N.mm

eb < e → 인장파괴 (eb = Mb / Pb = 169.145 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 125.000 mm , a = k1 x c = 104.500 mm 사용

fs = 600 x (300.000 - 125.000) / 125.000 = 400 Mpa (fs ≤ fy)

fs' = 600 x (125.000 - 100.000) / 125.000 = 120 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 104.500 x 1000 = 2620860.000 N

Cs = 794.400 x (120.000 - 0.836 x 30) = 75404.448 N

Ts = 1588.800 x 400.000 = 635520.000 N

Pn = 2620860.000 + 75404.448 - 635520.000 = 2060744.448 N

Mn' = 2620860.000 x (300.0 - 104.500 / 2) + 75404.448 x (300.0 - 100.0)

= 664398954.600 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.742 > 0.001 (e' = e + Xp - dc = 323.149212 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 8874006.720 N ≥ Pn = 2060744.448

ΦPn = 1442521.114 > ΦPb = 2330236.720 , 0.1 x fck x Ag = 1200000.000

∴ Φ = 0.700

ΦPn = 0.70 x 2060744.448 = 1442.521 kN ≥ Pu = 437.220 kN ------ 0.K!!!

ΦMn = ΦPn x 226.042 = 326.070 kN.m ≥ Mu = 98.830 kN.m ------ 0.K!!!

○ 전단력 검토

ϕVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 250956.548 N

여기서 , Vu < 1/2ϕVc 이므로 전단철근보강 불필요

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 61.72 × 1000000 / [1588.8×(300 - 71.32/3)]

= 140.634 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×300/(7×1588.8)]

= 71.320 ㎜

사용철근량 = 1588.800 mm^2 (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x C = 375 × (210 / 140.634) - 2.5 × 92 = 329.96 ㎜

300 x (210 / fs) = 300 × (210 / 140.634) = 447.97 ㎜

Sa는 작은 값인 329.96 ㎜ 를 적용

S = 125.00 ≤ Sa (= 329.963 mm) ----- ∴ 0.K.

⑨ 벽체 하단부 - Pmax 일 경우

설계축력	:	Pu	=	454.24	(kN) ,	설계모멘트	:	Mu	=	86.15	(kN · m)			
설계전단력	:	Vu	=	133.69	(kN) ,	사용 모멘트	:	M	=	34.51	(kN.m)			
부 재 높 이	:	H	=	500.0	(mm) ,	f_y	=	400	MPa		f_{ck}	=	30	MPa
폭	:	b	=	1000	(mm) ,	ϕ_f	=	0.90	,		ϕ_v	=	0.85	
유효높이(휨)	:	d	=	400.0	(mm) ,	d'	=	100	(mm),		ϕc	=	0.70	
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1	=	0.836	,		e	=	189.66	(mm)

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0048) < Pmax (0.0800) ----------- O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 500.00 x 1000.00 = 12540000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (12540000.00 x 250.0 + 635520.0 x 100.0 + 317760.0 x 400.0) / 13493280.000

= 246.468 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 240.000 mm, ab = k1 x cb = 200.640 mm

fs' = 600 x (240.000 - 100.0) / 240.0 = 350 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 200.640 x 1000 = 5032051.200 N

Cs = As' x (fs' - 0.9 x fck) = 256591.200 N

Ts = As x fs = 635520.000 N

Pb = 5032051.200 + 256591.200 - 635520.000 = 4653122.400 N

Mb = 5032051.200x(400.0-200.640/2)+256591.200x(400.0-100.0)-4653122.400x(246.468-100.0)

= 903450909.646 N.mm

eb > e → 압축파괴 (eb = Mb / Pb = 194.160 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 244.000 mm , a = k1 x c = 203.984 mm 사용

fs = 600 x (400.000 - 244.000) / 244.000 = 383.6066 Mpa (fs ≤ fy)

fs' = 600 x (244.000 - 100.000) / 244.000 = 354.0984 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 203.984 x 1000 = 5115918.720 N

Cs = 794.400 x (354.098 - 0.836 x 30) = 261372.186 N

Ts = 1588.800 x 383.607 = 609474.098 N

Pn = 5115918.720 + 261372.186 - 609474.098 = 4767816.807 N

Mn' = 5115918.720 x (400.0 - 203.984 / 2) + 261372.186 x (400.0 - 100.0)

= 1602996361.621 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.087 > 0.001 (e' = e + Xp - dc = 336.125025 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 10914006.720 N ≥ Pn = 4767816.807

ΦPn = 3337471.765 > ΦPb = 3257185.680 , 0.1 x fck x Ag = 1500000.000

∴ Φ = 0.700

ΦPn = 0.70 x 4767816.807 = 3337.472 kN ≥ Pu = 454.240 kN ------ 0.K!!!

ΦMn = ΦPn x 189.657 = 632.976 kN.m ≥ Mu = 86.150 kN.m ------ 0.K!!!

○ 전단력 검토

φVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 247887.649 N

여기서 , 1/2φVc< Vu ≤ φVc 이므로 최소 전단 철근 필요

- 계산된 스터럽 : min.Av = 0.35x(bo x s / fy)= 1.31㎠

- 배근할 스터럽 : H 13 다리수: N = 2 스터럽 간격 : s = 15 cm 0.K !!!

Av(req)= 1.31㎠ < Av = 2.534 cm² 0.K !!!

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 34.51 × 1000000 / [1588.8×(400 - 83.857/3)]

= 58.382 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×400/(7×1588.8)]

= 83.857 mm

사용철근량 = 1588.800 mm^2 (철근도심 : 100 mm)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 mm

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x Cc = 375 × (210 / 58.382) - 2.5 × 92 = 1118.88 mm

300 x (210 / fs) = 300 × (210 / 58.382) = 1079.10 mm

Sa는 작은 값인 1079.10 mm 를 적용

S = 125.00 ≤ Sa (= 1079.104 mm) ----- ∴ 0.K.

⑩ 벽체 하단부 - Mmax 일 경우

설계축력	:	Pu	=	454.24	(kN) ,	설계모멘트	:	Mu	=	99.28	(kN · m)	
설계전단력	:	Vu	=	133.69	(kN) ,	사용 모멘트	:	M	=	62.05	(kN.m)	
부 재 높 이	:	H	=	500.0	(mm) ,	f_y =	400	MPa	f_{ck}	=	30	MPa
폭	:	b	=	1000	(mm) ,	ϕ_f =	0.90	,	ϕ_v	=	0.85	
유효높이(휨)	:	d	=	400.0	(mm) ,	d' =	100	(mm),	ϕc	=	0.70	
유효높이(전단)	:	d	=	300.0	(mm) ,	k_1 =	0.836	,	e	=	218.56	(mm)

○ 휨철근량 계산

- 철근비검토

As = H 16 @ 8 EA = 15.888 cm^2 , d1 = 10.0 cm

Σ = 15.888 cm^2

Asf = H 16 @ 4 EA = 7.944 cm^2 , d'1 = 10.0 cm

Pmin (0.0012) < Puse (0.0048) < Pmax (0.0800) ----------- O.K!!!

- 평형하중 산정

소성중심 계산

Cc = 0.84 x 30 x 500.00 x 1000.00 = 12540000.000 N

Cs = As x fy = 635520.00 N , Cs' = Asf x fy = 317760.000 N

Xp = (12540000.00 x 250.0 + 635520.0 x 100.0 + 317760.0 x 400.0) / 13493280.000

= 246.468 mm

- 평형상태 계산

Cb = 600 / (600 + fy) x d = 240.000 mm, ab = k1 x cb = 200.640 mm

fs' = 600 x (240.000 - 100.0) / 240.0 = 350 Mpa (fs' ≤ fy)

Cc = 0.836 x 30 x 200.640 x 1000 = 5032051.200 N

Cs = As' x (fs' - 0.9 x fck) = 256591.200 N

Ts = As x fs = 635520.000 N

Pb = 5032051.200 + 256591.200 - 635520.000 = 4653122.400 N

Mb = 5032051.200x(400.0-200.640/2)+256591.200x(400.0-100.0)-4653122.400x(246.468-100.0)

= 903450909.646 N.mm

eb < e → 인장파괴 (eb = Mb / Pb = 194.160 mm)

- 공칭강도 계산 : 반복계산을 통해 c = 205.000 mm , a = k1 x c = 171.380 mm 사용

fs = 600 x (400.000 - 205.000) / 205.000 = 400 Mpa (fs ≤ fy)

fs' = 600 x (205.000 - 100.000) / 205.000 = 307.3171 Mpa (fs ≤ fy)

Cc = 0.836 x 30 x 171.380 x 1000 = 4298210.400 N

Cs = 794.400 x (307.317 - 0.836 x 30) = 224209.131 N

Ts = 1588.800 x 400.000 = 635520.000 N

Pn = 4298210.400 + 224209.131 - 635520.000 = 3886899.531 N

Mn' = 4298210.400 x (400.0 - 171.380 / 2) + 224209.131 x (400.0 - 100.0)

= 1418233250.102 N.mm

- 편심오차 : | e' - Mn'/Pn | = 0.155 > 0.001 (e' = e + Xp - dc = 365.030449 mm)

- 축방향력 및 휨에대한 검토

(max) Pn = 0.80 x { 0.85 fck x (Ag - Ast) + fy x Ast} 10914006.720 N ≥ Pn = 3886899.531

ΦPn = 2720829.672 > ΦPb = 3257185.680 , 0.1 x fck x Ag = 1500000.000

∴ Φ = 0.700

ΦPn = 0.70 x 3886899.531 = 2720.830 kN ≥ Pu = 454.240 kN ------ 0.K!!!

ΦMn = ΦPn x 218.563 = 594.672 kN.m ≥ Mu = 99.280 kN.m ------ 0.K!!!

○ 전단력 검토

φVc = 0.85 × 1/6 × (1 + Pu / (14 x Ag)) x √fck × bo × d = 247887.649 N

여기서 , 1/2φVc< Vu ≤ φVc 이므로 최소 전단 철근 필요

- 계산된 스터럽 : min.Av = 0.35x(bo x s / fy)= 1.31㎠

- 배근할 스터럽 : H 13 다리수: N = 2 스터럽 간격 : s = 15 cm 0.K !!!

Av(req)= 1.31㎠ < Av = 2.534 cm^2 0.K !!!

○ 사용성 검토 (균열방지 인장철근 간격검토)

- 응력산정

fs = M / [As x (d - χ/3)]

= 62.05 × 1000000 / [1588.8×(400 - 83.857/3)]

= 104.972 Mpa

χ = -nAs/b + nAs/b√[1 + 2bd/(nAs)]

= -7 × 1588.8/1000 + 7 × 1588.8/1000 × √[(1 + 2×1000×400/(7×1588.8)]

= 83.857 ㎜

사용철근량 = 1588.800 mm^2 (철근도심 : 100 ㎜)

1단 H16 @ 125 (= 1588.8 ㎟)

- 철근의 최대 중심간격

Cc = 100 - 16 / 2 = 92.00 ㎜

여기서 Cc ; 인장철근이나 긴장재의 표면과 콘크리트 표면사이의 두께(mm)

Smin : 375 x (210 / fs) - 2.5 x C = 375 × (210 / 104.972) - 2.5 × 92 = 520.199 ㎜

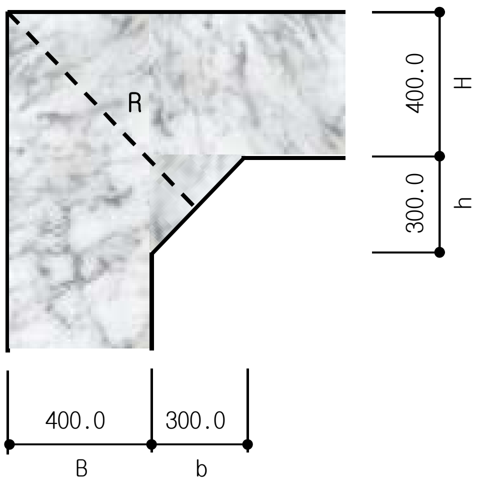
300 x (210 / fs) = 300 × (210 / 104.972) = 600.159 ㎜

Sa는 작은 값인 520.199 ㎜ 를 적용

S = 125.00 ≤ Sa (= 520.199 mm) ----- ∴ 0.K.

12. 우각부 검토

1) 상부 슬래브



CON'C 설계기준강도(fck)	:	30.0	MPa
CON'C 허용휨인장응력(fta) = 0.13√fck Mpa			
	:	0.712	MPa
철근 항복강도(fy)	:	400.0	MPa
철근 허용응력	:	180.0	MPa
사용 모멘트 Mo	:	76.77	kN · m

$R = \sqrt{2} \times b \;/\; (b+h) \times [B + h + (H \times h/b)]$

$= 777.82$ mm

① 보강철근 필요여부 판정

$$f_{tmax} = \frac{5 \times Mo}{R^2 \times W} = \frac{5 \times 76770000}{777.82^2 \times 1000.0} = 0.634 \text{ MPa}$$

$f_{tmax} = 0.634$ MPa $<$ $f_{ta} = 0.712$ MPa 이므로 보강철근 필요없음.

② 소요 철근량

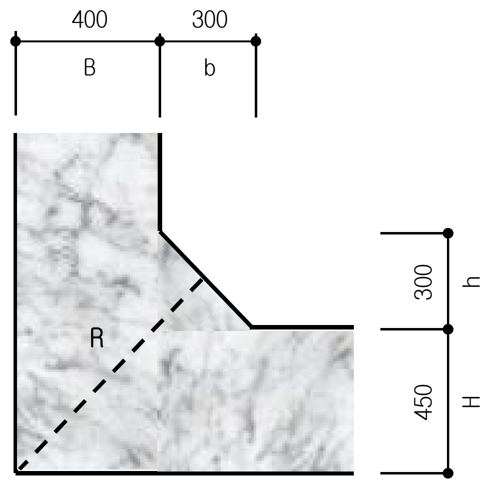
외측인장에 대한 보강 필요철근량(Asf)

$$Asf = \frac{2 \times Mo}{R \times fsa} = \frac{2 \times 0}{777.82 \times 180.0} = 0.00 \text{ ㎟}$$

▶ 보강 As = Asf - 주철근사용 As = 0.00 - 794.4 = -794.40 ㎟

----- 우각부 보강철근 필요없음

2) 하부 슬래브



CON'C 설계기준강도(fck)	:	30.0	MPa
CON'C 허용휨인장응력(fta) = 0.13√fck Mpa			
	:	0.712	MPa
철근 항복강도(fy)	:	400.0	MPa
철근 허용응력	:	180.0	MPa
사용 모멘트 Mo	:	62.05	kN · m

$R = \sqrt{(B2 + H2)} = 602.08$ mm

① 보강철근 필요여부 판정

$$f_{tmax} = \frac{5 \times Mo}{R^2 \times W} = \frac{5 \times 62050000}{602.08^2 \times 1000.0} = 0.856 \text{ MPa}$$

$f_{tmax} =$ 0.856 MPa > $f_{ta} =$ 0.712 MPa 이므로 보강철근 필요.

② 소요 철근량

외측인장에 대한 보강 필요철근량(Asf)

$$Asf = \frac{2 \times Mo}{R \times fsa} = \frac{2 \times 62050000}{602.08 \times 180.0} = 1145.10 \text{ ㎟}$$

▶ 보강 As = Asf - 주철근사용 As = 1145.10 - 794.4 = 350.70 ㎟

----- 우각부 보강철근 필요

외측인장에 대한 보강 사용철근량(As)

As = H16 @ 250 - 1 단 = 794.40 ㎟

Asf = 350.70 ㎟ < As = 794.40 ㎟ ----- ∴ O.K.

13. 배력철근 검토

1) 구조상세 - 철도설계기준(노반편) p.5-19

다. 종방향으로 영향을 받는 경우로써 상부슬래브, 하부슬래브와 벽체에 있어서 종방향 최소철근량은 복토의 깊이가 3.0 m 이하인 경우에는 콘크리트 단면적의 0.4%, 복토가 3.0m 이상인 경우에는 복토 깊이가 30m인 경우의 사용량인 1.0%를 기준으로 선형변화량을 사용하되, 종방향 철근량은 주철근량 미만이어야 한다.

라. 단단한 지반 위에 놓인 경우와 말뚝기초로 지지되는 경우로써 종방향 부등침하가 우려되지 않은 경우에는 다.항의 기준을 적용하지 않는다. 따라서 어느 정도 견고한 지반위에 놓이는 일반적인 구교로써 적적한 신축이음매가 설치되어 상세한 종방향해석이 필요하지 않다고 판단되는 경우 종방향 철근량은 슬래브의 경우 콘크리트단면적의 0.25%, 벽체의 경우 0.3%를 적용한다.

마. 배력철근은 단위폭주변의 인장 주철근량의 1/5 이상으로 하여 종단방향에 계산외의 응력이 발생하는 것을 고려하여야 한다.

2) 배력철근의 최소 철근비

슬래브의 배력 철근비 = 0.25 %　　　　벽체의 배력 철근비 = 0.30 %

① 상부슬래브

배력철근 최소 철근비 (Asreq)	= 1,000.0 × 400.0 × 0.25			=	1000.00	㎟
사용철근 인장측 ;	H 13	@	150	=	844.67	㎟
압축측 ;	H 13	@	300	=	422.33	㎟
				∑ As =	1267.00	㎟

따라서, As = 1267.00 ㎟ > Asreq = 1000.00 ㎟ ----- ∴ O.K.

사용 주철근량의 1/5 = 317.760 ㎟

따라서, As = 844.67 ㎟ > Asreq = 317.76 ㎟ ----- ∴ O.K.

② 하부슬래브

배력철근 최소 철근비 (Asreq)	= 1,000.0 × 450.0 × 0.25			=	1125.00	㎟
사용철근 인장측 ;	H 13	@	150	=	844.67	㎟
압축측 ;	H 13	@	300	=	422.33	㎟
				∑ As =	1267.00	㎟

따라서, As = 1267.00 ㎟ > Asreq = 1125.00 ㎟ ----- ∴ O.K.

사용 주철근량의 1/5 = 317.760 ㎟

따라서, As = 844.67 ㎟ > Asreq = 317.76 ㎟ ----- ∴ O.K.

③ 벽체

배력철근 최소 철근비 (Asreq) = 1,000.0 × 400.0 × 0.30 = 1200.00 ㎟

사용철근 인장측 ; H 13 @ 150 = 844.67 ㎟

압축측 ; H 13 @ 300 = 422.33 ㎟

∑ As = 1267.00 ㎟

따라서, As = 1267.00 ㎟ > Asreq = 1200.00 ㎟ ----- ∴ O.K.

사용 주철근량의 1/5 = 317.760 ㎟

따라서, As = 844.67 ㎟ > Asreq = 317.76 ㎟ ----- ∴ O.K.

14. 사용성검토

1) 상부슬래브 최소 두께

1.2(L+3)/30 = 1.2 × (2.640 + 3) / 30 = 225.6 mm

상부슬래브 두께 = 400.0 mm

Tuse = 400.0 mm > Tmin = 225.6 mm ----- ∴ O.K.

2) 철근 피로검토

피로응력한계 : 사용하중하에서 활하중과 충격에 의해 야기되는 직선철근의 최대 인장응력과 최소 응력 상이의 범위는 다음 값을 초과할수 없다.

f_f = 165 - 0.33fmin

여기서, f_f : 응력범위(Mpa)

fmin : 최소응력 수준으로 인장은 정(+), 압축은 부(-)의 값(Mpa)

① 상부슬래브 단부

2.64 m 지간에서의 모멘트 변화에 의한 피로검토

Mmax = 2.829 kN · m   Mmin = -1.038 kN · m

- 사용철근량 = H16 × 8ea = 1588.800 ㎟

n = 7

p = As / bd = 1588.80 / (1000 × 400) = 0.00397

$K = -np + \sqrt{(np)^2 + 2np}$

$= -7 \times 0.00397 + \sqrt{(7 \times 0.00397)^2 + 2 \times 7 \times 0.00397}$

= 0.210

J = 1 - K / 3 = 1 - 0.210 / 3 = 0.930

$$fs\ max = \frac{M}{As \times j \times d} = \frac{2.829 \times 10^{6}}{1588.80 \times 0.930 \times 400} = 4.790 \text{ Mpa}$$

$$fs\ min = \frac{-1.038 \times 10^{6}}{1588.80 \times 0.930 \times 400} = -2 \text{ Mpa}$$

Δf = fs max - fs min = 6.546 Mpa

f_f = 165 - 0.33fmin

= 165 - 0.33 × -1.76 = 165.579 Mpa > 6.546 Mpa O.K.

② 상부슬래브 중앙부

2.64 m 지간에서의 모멘트 변화에 의한 피로검토

Mmax = 9.896 kN · m   Mmin = 9.112 kN · m

- 사용철근량 = H16 × 8ea = 1588.800 ㎟

n = 7

p = As / bd = 1588.80 / (1000 × 300) = 0.00530

K = -np + √{(np)^2 + 2np}

= -7 × 0.0053 + √{ (7 × 0.0053) ^{2} + 2 × 7 × 0.0053 }

= 0.238

J = 1 - K / 3 = 1 - 0.238 / 3 = 0.921

$$fs\ max = \frac{M}{As \times j \times d} = \frac{9.896 \times 10^{6}}{1588.80 \times 0.921 \times 300} = 22.550 \text{ Mpa}$$

$$fs\ min = \frac{9.112 \times 10^{6}}{1588.80 \times 0.921 \times 300} = 21 \text{ Mpa}$$

Δf = fs max - fs min = 1.788 Mpa

f_f = 165 - 0.33fmin

= 165 - 0.33 × 20.76 = 158.148 Mpa > 1.788 Mpa O.K.

15. 기초지지력 검토

1) 구체자중

-	2.800	x	0.450	x	24.500					=	30.870	kN/m
-	2.800	x	0.400	x	24.500					=	27.440	kN/m
-	2.000	x	0.400	x	24.500					=	19.600	kN/m
-	2.000	x	0.400	x	24.500					=	19.600	kN/m
-	0.300	x	0.300	x	1/2	x	4	x	24.500	=	4.410	kN/m
								∴	Total	=	101.920	kN/m

101.920 / 2.800 = 36.400 kN/m²

2) 상부연직하중

= 166.598 kN/m²

3) 활하중

= 20.672 kN/m²

4) 최대 지반 반력

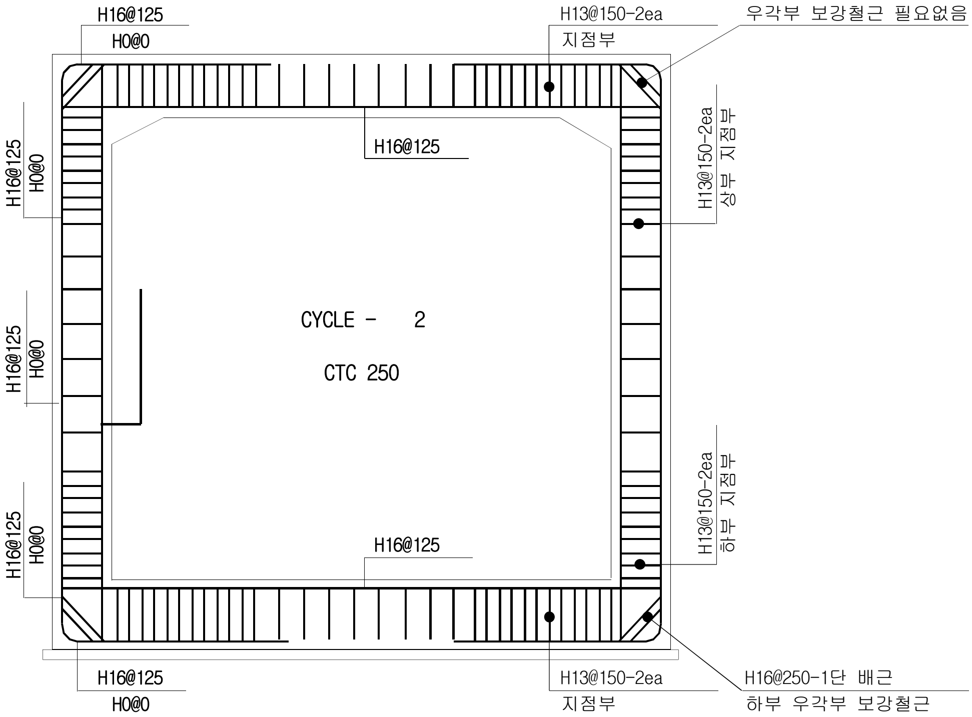
Q_{max} = 36.400 + 166.598 + 20.672

= 223.670 kN/m²

5) 지지력 검토

Q_{max} = 223.670 kN/m² < 300.00 kN/m² --- ∴ O.K.

16. 주철근 조립도



▶ 배력철근

- 상부 지점부 : H13, 인장측:150, 압축측:300
- 하부 지점부 : H13, 인장측:150, 압축측:300
- 벽체부 : H13, 인장측:150, 압축측:300

U-Type 개거 2.0x2.4

구조계산서

목 차

■ 해석결과 요약

* 기초 검토

구분	q_{max}(kN/m^2)	Qu(kN)	qa(kN/m^2)	비 고
직접기초	34.840	2240.680	287.267	∴ O.K

* 휨 검토

구분		B(m)	H(m)	dc(m)	헌치(m)	Mu(kn.m)	ΦMn(kn.m)	BENDING Bar(mm)	휨검토		최소철근비 검토	주철근간격 검토
									판정	S.F		
하부슬래브	단부	1.000	0.300	0.100	0.300	78.129	138.042	H 16 @ 150	O.K	1.767	O.K	O.K
	중앙부	1.000	0.300	0.100		59.672	90.378	H 16 @ 150	O.K	1.515	O.K	O.K
좌측 벽체	단부	1.000	0.300	0.100	0.300	78.129	138.042	H 16 @ 150	O.K	1.767	O.K	O.K
	중앙부	1.000	0.300	0.100		26.358	46.427	H 16 @ 300	O.K	1.761	O.K	O.K
우측 벽체	단부	1.000	0.300	0.100	0.300	78.129	138.042	H 16 @ 150	O.K	1.767	O.K	O.K
	중앙부	1.000	0.300	0.100		26.383	46.427	H 16 @ 300	O.K	1.760	O.K	O.K

* 전단검토

구분		ΦVc	Vu	판정	전단철근			전단검토	
					사용철근	다리수	간격	판정	S.F
하부슬래브	단부	155.188	30.052	전단철근 불필요	H 0	× 0	@ 0	O.K	-
	중앙부	-	-	-	H -	× -	@ -	-	-
좌측 벽체	단부	155.188	84.550	전단철근 불필요	H 0	× 0	@ 0	O.K	-
	중앙부	-	-	-	H -	× -	@ -	-	-
우측 벽체	단부	155.188	84.550	전단철근 불필요	H 0	× 0	@ 0	O.K	-
	중앙부	-	-	-	H -	× -	@ -	-	-

1. 설계개요 및 조건

1.1 설계개요

• 구조해석 프로그램 : MIDAS CIVIL

1.2 설계방법

(1) 부재의 단면검토 : 강도설계법

(2) 사용성 검토 : 허용응력설계법

1.3 일반사항

(1) 사용재료의 강도

① 콘크리트의 압축강도 : f_{ck} = 30 MPa

② 철근의 항복강도 : f_y = 400 MPa

(2) 사용재료의 탄성계수

① 콘크리트의 탄성계수 : E_c = 27,537 MPa

② 철근의 탄성계수 : E_s = 200,000 MPa

(3) 토사 내부마찰각 : Φ = 35°

(4) 토압 : K_o = 0.426

(5) 사 각(Skew) : 71.00 °

1.4 설계하중

(1) 고정하중

재 료	단위중량(kN/m^3)	재 료	단위중량(kN/m^3)
철근콘크리트	24.5	아스팔트 포장	23.0
흙(지하수위 이상)	20.0	흙(지하수위 이하)	11.0
지하수	10.0		

(2) 상재 하중(kN/m^2)

q_L = 15.0 kN/m^2

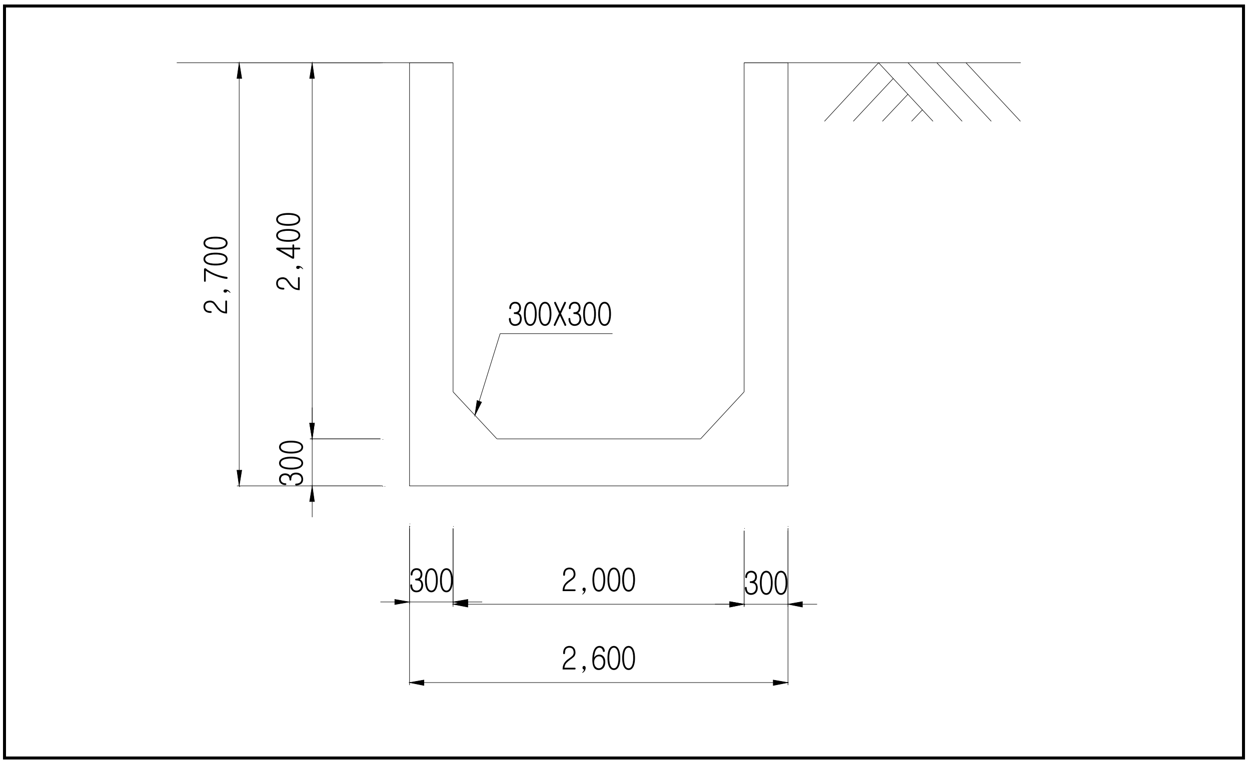
1.5 참고문헌

(1) 콘크리트 구조설계기준 (한국콘크리트 학회, 2012)

(2) 도로교 설계기준 (건설교통부, 2010)

(3) 철도설계기준 (노반편) (건설교통부 2013)

2. 단면가정

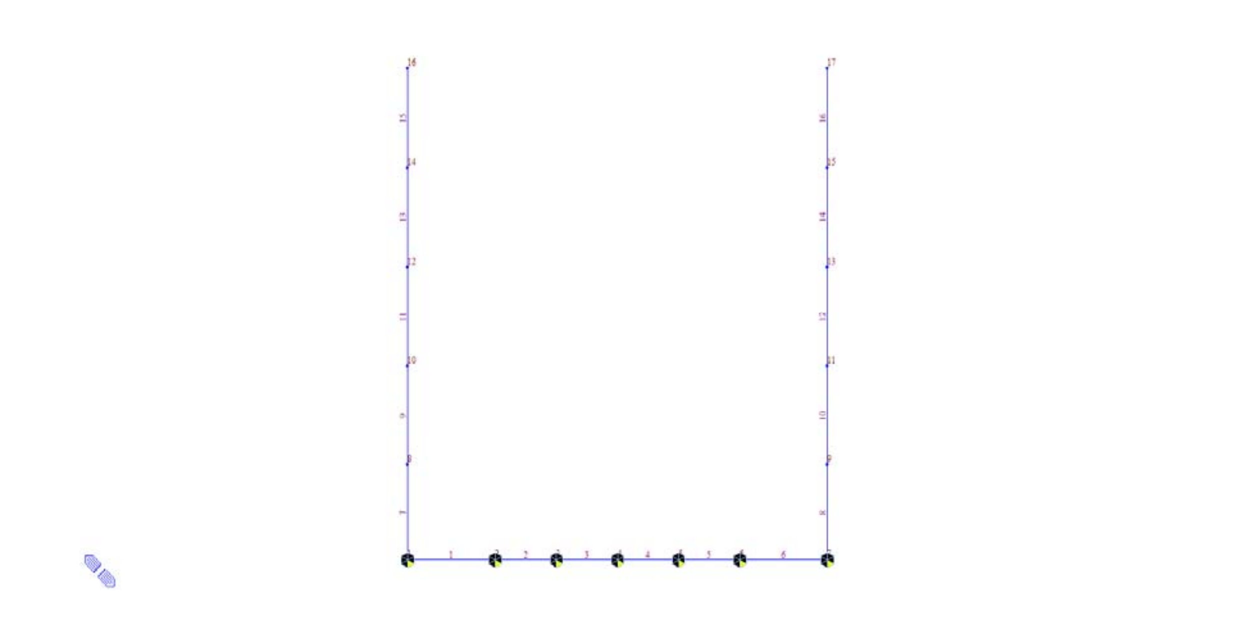


* N=15, $\Phi=35^{\circ}$ 적용 함.

* 지하수위는 극한 지하수위(GL -1.0m) 로 적용 함.

3. 구조해석 모델링

3.1 해석모형



3.2 단면제원

구 분	폭(m)	높이(m)	C.T.C(m)	단면적(m²)	단면2차모멘트(m⁴)	비 고
1	1.00	0.30	-	0.300	0.002250	하부슬래브
2	1.00	0.30	-	0.300	0.002250	좌측벽체
3	1.00	0.30	-	0.300	0.002250	우측벽체

3.3 지반반력계수

$Kv = Kvo \times (Bv / 30)^{n}$ (n = -3/4)

$Kvo = 1/30 \times \alpha \times Eo$

Bv : 구조물의 환산재하폭

Eo = 2800 x N = 2800 × 9 = 25,200.000 kN/m²

α = 1

Kvo = 1 × 1 / 0.3 × 25200 = 84000.000 kN/m^3

Bv = √ (2.600 × 1.000) = 1.612 m

∴ Kv = 84000.000 × (1.6 / 0.3) ^ (-3/4) = 23796.069 kN/m^3

절점번호	지반반력계수	비고
1,7	23796.069 × 0.3750 = 8923.526 kN/m^2	
2,6	23796.069 × 0.3750 = 8923.526 kN/m^2	
3,4,5	23796.069 × 0.4000 = 9518.428 kN/m^2	

※ 지반스프링은 압축력에만 저항하는 비선형 경계조건을 사용하여 부반력 발생시 자동제거함.

4. 하중산정

4.1 고정하중

(1) 구조물 자중 : 프로그램내 자동입력 --------------- LC 1

4.2 활하중에 의한 측압 --------------- LC 2

$Lh = 0.426 \times 15.000 = 6.390$ kN/m^2

4.3 토압

(1) 지하수위 유 --------------- LC 3

$Q_{N1L} = 1.000 \times 20.000 \times 0.426 = 8.520$ kN/m^2

$Q_{N1R} = 8.520 + 1.700 \times 11.000 \times 0.426 = 16.486$ kN/m^2

(2) 지하수위 무 --------------- LC 4

$Q_{N1L} = 2.700 \times 20.000 \times 0.426 = 23.004$ kN/m^2

4.4 수압

(1) 측벽 수압 --------------- LC 5

1) 벽체

$Q_{W1L} = 10.000 \times 1.700 = 17.000$ kN/m^2

4.5 온도하중 --------------- LC 6, LC 7

- 계절적 온도변화 ± 10°C를 프로그램내 SYSTEM TEMPERRATURE 기능을 이용하여 이상화 한다.

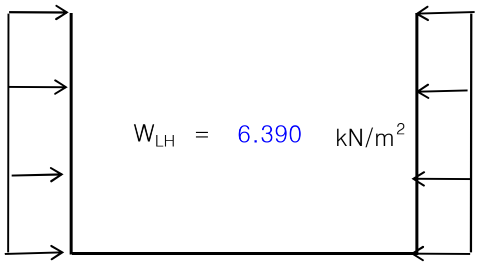
4.6 건조수축 --------------- LC 8

- 15^{0}c 의 온도하중으로 변환 설계
  - 건조수축계수 : $\varepsilon = 15 \times 10^{-5}$
  - 온도선팽창 계수 : $\alpha = 1.0 \times 10^{-5}$
  - $\triangle T = \dfrac{\varepsilon}{\alpha} = 15$ °C

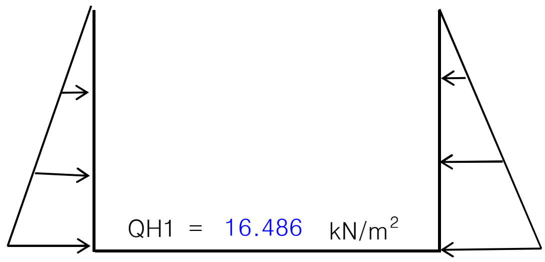
4.7 하중재하도

(1) LC 1 : 프로그램내 자동재하

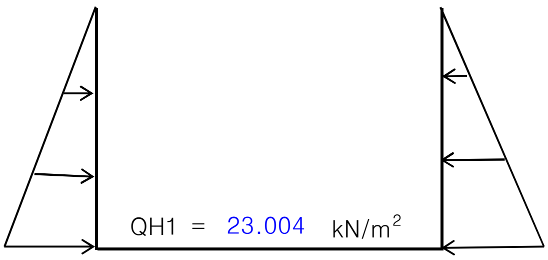
(2) LC 2



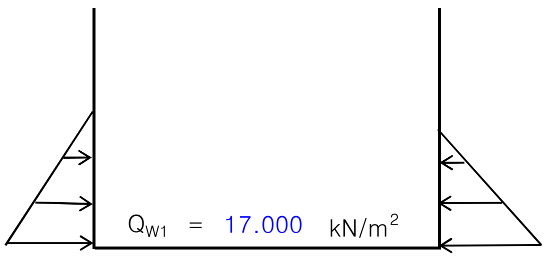
(3) LC 3



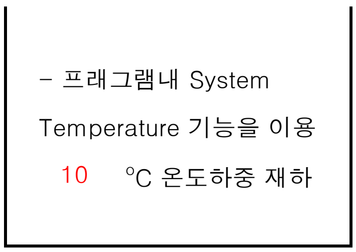
(4) LC 4



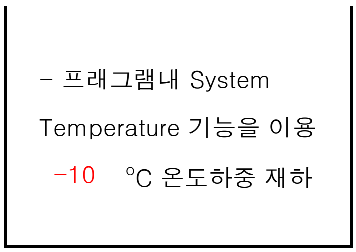
(5) LC 5



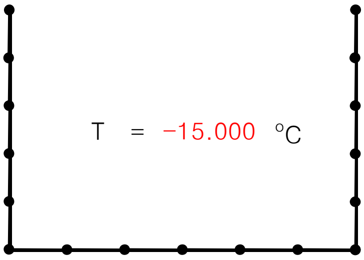
(6) LC 6



(7) LC 7



(8) LC 8



5. 하중조합

5.1 계수하중

구분	고정하중 (D	활하중측압	수평토압		수압	온도하중		건조 수축	비고
	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	
CB1	1.35	1.85	1.60		1.60				계수하중
CB2	1.35	1.85		1.60					
CB3	1.35	1.85	0.80		0.80				
CB4	1.35	1.85		0.80					
CB5	1.60	1.60	1.60		1.60				
CB6	1.60	1.60		1.60					
CB7	1.60	1.60	0.80		0.80				
CB8	1.60	1.60		0.80					
CB9	1.35	1.40	1.60		1.60	1.35			
CB10	1.35	1.40		1.60		1.35			
CB11	1.35	1.40	0.80		0.80	1.35			
CB12	1.35	1.40		0.80		1.35			
CB13	1.35	1.40	1.60		1.60		1.35	1.35	
CB14	1.35	1.40		1.60			1.35	1.35	
CB15	1.35	1.40	0.80		0.80		1.35	1.35	
CB16	1.35	1.40		0.80			1.35	1.35	

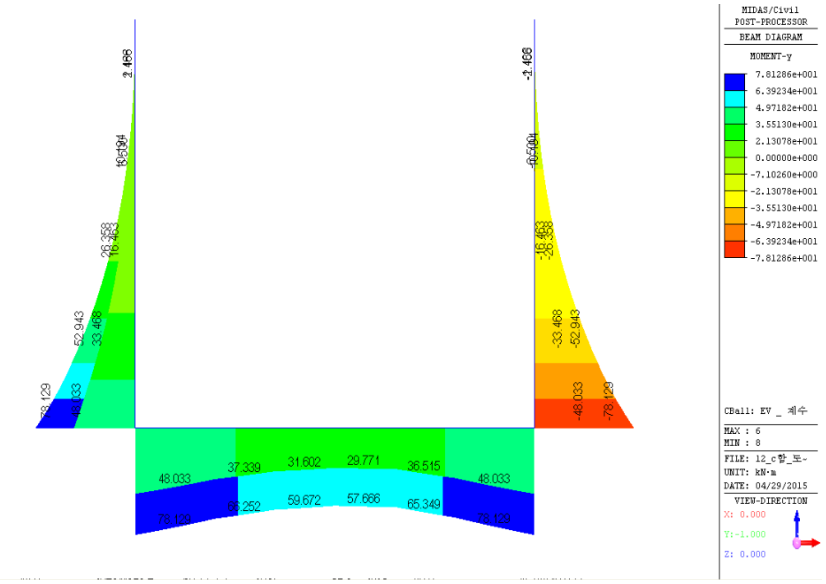
5.2 사용하중

구분	고정하중 (D	활하중측압	수평토압		수압	온도하중		건조 수축	비고
	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	
CB17	1.00	1.00	1.00		1.00				사용하중
CB18	1.00	1.00		1.00					
CB19	1.00	1.00	0.50		0.50				
CB20	1.00	1.00		0.50					
CB21	1.00	1.00	1.00		1.00	1.00			
CB22	1.00	1.00		1.00		1.00			
CB23	1.00	1.00	0.50		0.50	1.00			
CB24	1.00	1.00		0.50		1.00			
CB25	1.00	1.00	1.00		1.00		1.00	1.00	
CB26	1.00	1.00		1.00			1.00	1.00	
CB27	1.00	1.00	0.50		0.50		1.00	1.00	
CB28	1.00	1.00		0.50			1.00	1.00	
CB29	1.00		1.00		1.00				
CB30	1.00			1.00					

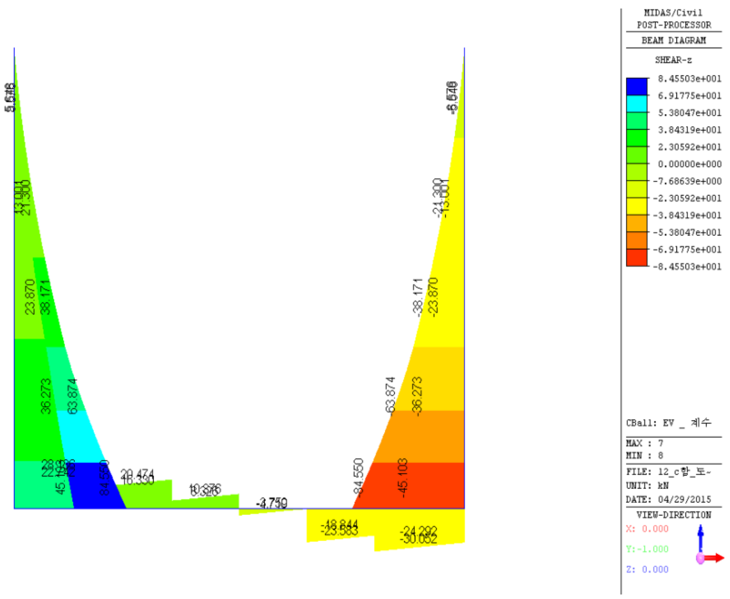
6. 구조해석 결과

6.1 부재력도 (ENVELOPE)

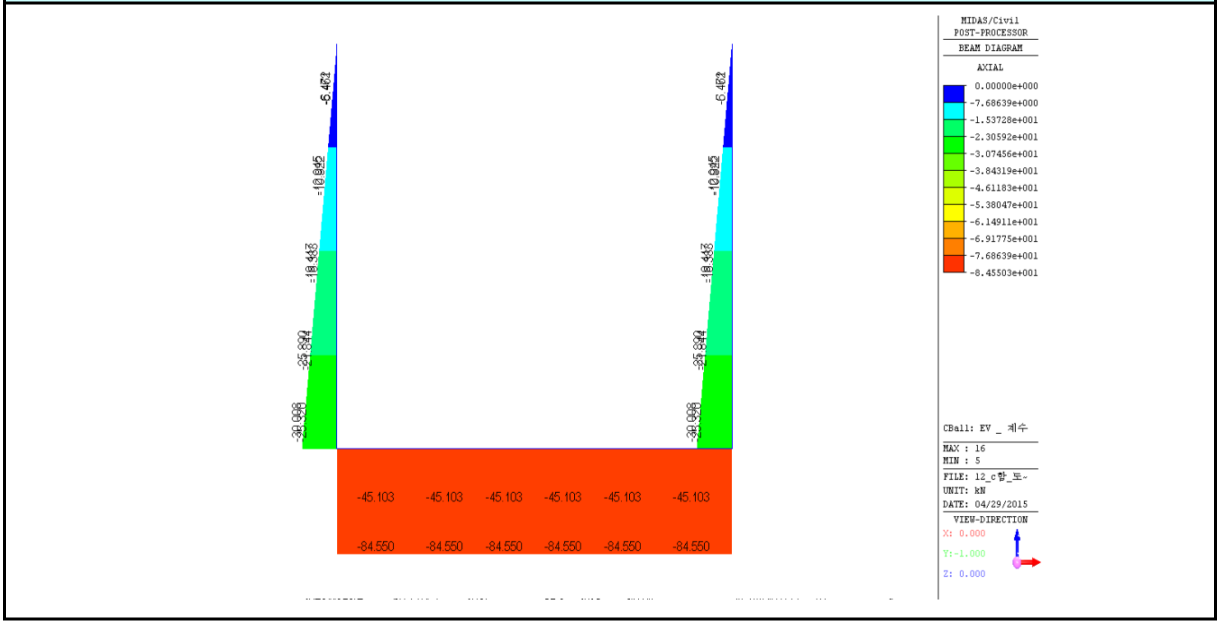
B.M.D (계수하중) [kN·m]



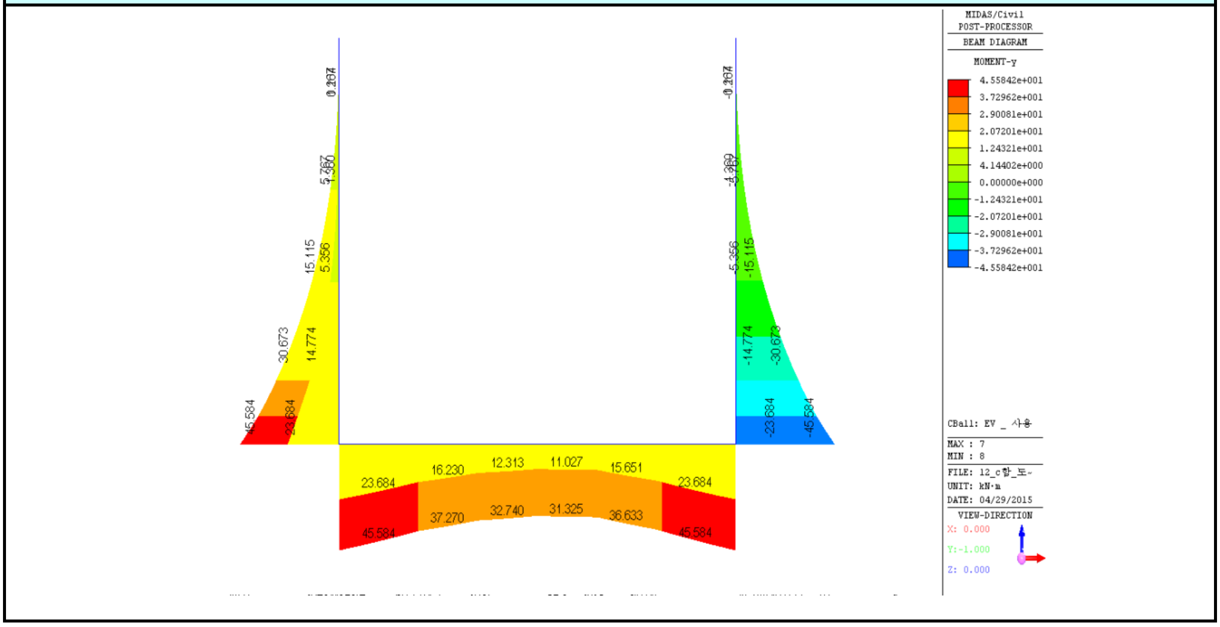
S.F.D (계수하중) [kN]



A.F.D (계수하중) [kN]



B.M.D (사용하중) [kN·m]



6.2 부재력 집계

구 분		단면제원		Mu[kN·m]	Vu[kN]	Mo[kN·m]	비고
		B (mm)	H (mm)				
하 부 슬래브	단 부	1,000	400	78.129	30.052	45.584	
	중앙부	1,000	300	59.672		32.740	
좌측벽체	단 부	1,000	400	78.129	84.550	45.584	
	중앙부	1,000	300	26.358		15.115	
우측벽체	단 부	1,000	400	78.129	84.550	45.584	
	중앙부	1,000	300	26.383		15.115	

7. 단면설계

7.1 하부슬래브

7.1.1 하부슬래브 단부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	78.129 kN.m	Vu	=	30.052 kN	Mo	=	45.584 kN.m	d (휨)	=	300 mm
B	=	1000 mm	H	=	400 mm	헌치	=	300 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	300 mm

Use A_s =	1단	1CYCL	H	16 @	150	,	2CYCLI	H	0 @	0	=	1,324	mm^2
	2단	1CYCL	H	@		,	2CYCLI	H	0 @	0	=	–	mm^2
			TOTAL	=	1,324	mm^2	[	p =	0.00441	]			

▶ 주철근이 항복한다고 가정하면, T = $A_s f_y$ = 529,600 N

C = $0.85 f_{ck}$ a b = 25,500 a C = T ; a = 21 mm

▶ c = a / β_1 = 21 / 0.84 = 25 mm

ε_t = 0.003 (d_t−c) / c = 0.003 × (300 − 25) / 25

= 0.03323 > 0.002 ⇒ **가정만족 (주철근 항복)**

> 0.005 ⇒ **인장지배 단면**

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.03323 - 0.002)$$

= 3.252 > 0.9 ⇒ ϕ_f = **0.900**

▶ 최소 철근비 검토

P_{min1} = 1.4 / f_y = 0.00350 ⇒ $A_{s1,\ min}$ = 1,050 mm^2

P_{min2} = 0.25 $\sqrt{f_{ck}}$ / f_y = 0.00342 ⇒ $A_{s2,\ min}$ = 1,027 mm^2

$A_{s,\ min}$ = 1,050 mm^2

$A_{s,\ Req}$ = 738 mm^2 , 4/3 $A_{s,\ Req}$ = 984 mm^2

MIN ($A_{s,min}$, 4/3 $A_{s,req}$) = 984 mm^2 < Use As = 1,324 mm^2 ∴ O.K

◈ Md = ϕ_f T (d − a / 2) = 0.900 × 529,600 × (300 − 21 / 2) / 10 E+6

= 138.042 kN.m > Mu = 78.129 kN.m ∴ O.K [S.F = 1.767]

(2) 전단 검토

▶ ϕ_v Vc = ϕ_v 1/6 $\sqrt{f_{ck}}$ B d = 155.188 kN > Vu = 30.052 kN

∴ 전단철근 불필요

(3) 주철근 간격 검토

▶ 주철근 응력, fs

A = 2 × 100 × 1,000 / 7 = 30,000 mm^2

k = − np + √ [2 np + (np)^2] [∵ n = Es / Ec = 7.3 ≒ 7]

= − 7 × 0.00441 + √ [2 × 0.031 + 0.031 ^{2}] = 0.22

j = 1 − k / 3 = 1 − 0.22 / 3 = 0.927

fs = M / [As·d·j] = 123.801 MPa

◈ 주철근 간격

Cc = d_t' − d_b/2 = 100 − 15.9 / 2 = 92 mm (여기서, d_b = 16)

s_1 = 375 (210 / fs) − 2.5 Cc = 406 mm

s_2 = 300 (210 / fs) = 509 mm

> s = 150 mm ∴ O.K

7.1.2 하부슬래브 중앙부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	59.672 kN.m	Vu	=	0.000 kN	Mo	=	32.740 kN.m	d (휨)	=	200 mm
B	=	1000 mm	H	=	300 mm	헌치	=	0 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	200 mm

Use A_s =	1단	1CYCL H	16 @ 150	,	2CYCLI H	0 @	0	=	1,324	mm^2
	2단	1CYCL H	@	,	2CYCLI H	0 @	0	=	−	mm^2
		TOTAL	=	1,324 mm^2	[p =	0.00662]				

▶ 주철근이 항복한다고 가정하면, $T = A_s f_y = 529{,}600$ N

$C = 0.85 f_{ck}\ a\ b = 25{,}500$ a $\qquad C = T$; $a = 21$ mm

▶ $c = a / \beta_1 = 21 / 0.84 = 25$ mm

$\varepsilon_t = 0.003\ (d_t - c) / c = 0.003 \times (200 - 25) / 25$

$= 0.02115$ > 0.002 ⇒ 가정만족 (주철근 항복)

$\quad\quad\quad\quad$ > 0.005 ⇒ 인장지배 단면

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.02115 - 0.002)$$

$= 2.246 > 0.9 \Rightarrow \phi_f = 0.900$

▶ 최소 철근비 검토

$P_{min1} = 1.4 / f_y = 0.00350 \Rightarrow A_{s1,\ min} = 700\ mm^2$

$P_{min2} = 0.25 \sqrt{f_{ck}} / f_y = 0.00342 \Rightarrow A_{s2,\ min} = 685\ mm^2$ $\qquad A_{s,\ min} = 700\ mm^2$

$A_{s,\ Req} = 858\ mm^2$, $4/3\ A_{s,\ Req} = 1{,}143\ mm^2$

MIN ($A_{s,min}$, $4/3\ A_{s,req}$) = 700 mm^2 < Use As = 1,324 mm^2 ∴ O.K

◈ $Md = \phi_f\ T\ (d - a / 2) = 0.900 \times 529{,}600 \times (200 - 21 / 2) / 10\ E+6$

$= 90.378$ kN.m > Mu = 59.672 kN.m ∴ O.K [S.F = 1.515]

(2) 주철근 간격 검토

▶ 주철근 응력, fs

$A = 2 \times 100 \times 1{,}000 / 7 = 30{,}000\ mm^2$

$k = -np + \sqrt{[2\ np + (np)^2]}$ [∵ n = Es / Ec = 7.3 ≒ 7]

$= -7 \times 0.00662 + \sqrt{[2 \times 0.046 + 0.046^2]} = 0.261$

$j = 1 - k / 3 = 1 - 0.261 / 3 = 0.913$

$fs = M / [As \cdot d \cdot j] = 135.422$ MPa

◈ 주철근 간격

$Cc = d_t' - d_b/2 = 100 - 15.9 / 2 = 92$ mm (여기서, d_b = 16)

$s_1 = 375\ (210 / fs) - 2.5\ Cc = 351$ mm

$s_2 = 300\ (210 / fs) = 465$ mm

> s = 150 mm ∴ O.K

7.2 좌측벽체

7.2.1 좌측벽체 단부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	78.129 kN.m	Vu	=	84.550 kN	Mo	=	45.584 kN.m	d (휨)	=	300 mm
B	=	1000 mm	H	=	400 mm	헌치	=	300 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	300 mm

Use A_s =	1단	1CYCLE	H 16 @ 150	,	2CYCLE	H 0 @ 0	=	1,324	mm^2
	2단	1CYCLE	H 0 @	,	2CYCLE	H 0 @ 0	=	−	mm^2
		TOTAL =	1,324 mm^2		[p = 0.00441]				

▶ 주철근이 항복한다고 가정하면, $T = A_s f_y$ = 529,600 N

C = $0.85 f_{ck}$ a b = 25,500 a C = T ; a = 21 mm

▶ c = a / β_1 = 21 / 0.84 = 25 mm

ε_t = 0.003 (d_t−c) / c = 0.003 × (300 − 25) / 25

= 0.03323 > 0.002 ⇒ 가정만족 (주철근 항복)

 > 0.005 ⇒ 인장지배 단면

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.03323 - 0.002)$$

= 3.252 > 0.9 ⇒ ϕ_f = 0.900

▶ 최소 철근비 검토

P_{min1} = 1.4 / f_y = 0.00350 ⇒ $A_{s1,\ min}$ = 1,050 mm^2

P_{min2} = 0.25 $\sqrt{f_{ck}}$ / f_y = 0.00342 ⇒ $A_{s2,\ min}$ = 1,027 mm^2 $A_{s,\ min}$ = 1,050 mm^2

$A_{s,\ Req}$ = 738 mm^2 , 4/3 $A_{s,\ Req}$ = 984 mm^2

MIN ($A_{s,min}$, 4/3 $A_{s,req}$) = 984 mm^2 < Use As = 1,324 mm^2 ∴ O.K

◈ Md = ϕ_f T (d − a / 2) = 0.900 × 529,600 × (300 − 21 / 2) / 10 E+6

= 138.042 kN.m > Mu = 78.129 kN.m ∴ O.K [S.F = 1.767]

(2) 전단 검토

▶ ϕ_v Vc = ϕ_v 1/6 $\sqrt{f_{ck}}$ B d = 155.188 kN > Vu = 84.550 kN

∴ 전단철근 불필요

(3) 주철근 간격 검토

▶ 주철근 응력, fs

A = 2 × 100 × 1,000 / 7 = 30,000 mm^2

k = − np + √ [2 np + (np)^2] [∵ n = Es / Ec = 7.3 ≒ 7]

= − 7 × 0.00441 + √ [2 × 0.031 + 0.0309 ^{2}] = 0.22

j = 1 − k / 3 = 1 − 0.22 / 3 = 0.927

fs = M / [As·d·j] = 123.801 MPa

◈ 주철근 간격

Cc = d_t' − d_b/2 = 100 − 15.9 / 2 = 92 mm (여기서, d_b = 16)

s_1 = 375 (210 / fs) − 2.5 Cc = 406 mm

s_2 = 300 (210 / fs) = 509 mm > s = 150 mm ∴ O.K

7.2.2 좌측벽체 중앙부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	26.358 kN.m	Vu	=	0.000 kN	Mo	=	15.115 kN.m	d (휨)	=	200 mm
B	=	1000 mm	H	=	300 mm	헌치	=	0 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	200 mm

Use A_s =	1단	1CYCL H	16 @ 300 ,	2CYCLI H	0 @ 0	=	662 mm^2
	2단	1CYCL H	@ ,	2CYCLI H	0 @ 0	=	− mm^2
		TOTAL =	662 mm^2	[p =	0.00331]		

▶ 주철근이 항복한다고 가정하면, T = $A_s f_y$ = 264,800 N

C = 0.85 f_{ck} a b = 25,500 a　　C = T ; a = 10 mm

▶ c = a / β_1 = 10 / 0.84 = 12 mm

ε_t = 0.003 (d_t−c) / c = 0.003 × (200 − 12) / 12

= 0.04530　> 0.002 ⇒ **가정만족 (주철근 항복)**

　　　　　> 0.005 ⇒ **인장지배 단면**

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.04530 - 0.002)$$

= 4.259 > 0.9 ⇒ **ϕ_f = 0.900**

▶ 최소 철근비 검토

P_{min1} = 1.4 / f_y = 0.00350 ⇒ $A_{s1,\ min}$ = 700 mm^2

P_{min2} = 0.25 $\sqrt{f_{ck}}$ / f_y = 0.00342 ⇒ $A_{s2,\ min}$ = 685 mm^2　　$A_{s,\ min}$ = 700 mm^2

$A_{s,\ Req}$ = 371 mm^2 , 4/3 $A_{s,\ Req}$ = 495 mm^2

MIN ($A_{s,min}$, 4/3 $A_{s,req}$) = 495 mm^2 < Use As = 662 mm^2 ∴ O.K

◈ Md = ϕ_f T (d − a / 2) = 0.900 × 264,800 × (200 − 10 / 2) / 10 E+6

= **46.427** kN.m > Mu = **26.358** kN.m ∴ O.K [S.F = 1.761]

(2) 주철근 간격 검토

▶ 주철근 응력, fs

A = 2 × 100 × 1,000 / 3 = 60,000 mm^2

k = − np + √ [2 np + (np)^2] [∵ n = Es / Ec = 7.3 ≒ 7]

= − 7 × 0.00331 + √ [2 × 0.023 + 0.0232 ^{2}] = 0.193

j = 1 − k / 3 = 1 − 0.193 / 3 = 0.936

fs = M / [As·d·j] = 121.966 MPa

◈ 주철근 간격

Cc = d_t' − d_b/2 = 100 − 15.9 / 2 = 92 mm (여기서, d_b = 16)

s_1 = 375 (210 / fs) − 2.5 Cc = 416 mm

s_2 = 300 (210 / fs) = 517 mm　　> s = 300 mm ∴ O.K

7.3 우측벽체

7.3.1 우측벽체 단부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	78.129 kN.m	Vu	=	84.550 kN	Mo	=	45.584 kN.m	d (휨)	=	300 mm
B	=	1000 mm	H	=	400 mm	헌치	=	300 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	300 mm

Use A_s =	1단	1CYCl H	16 @	150	,	2CYCLI H	0 @	0	=	1,324 mm^2
	2단	1CYCl H	@		,	2CYCLI H	0 @	0	=	− mm^2
		TOTAL	=	1,324 mm^2		[p = 0.00441]				

▶ 주철근이 항복한다고 가정하면, T = $A_s f_y$ = 529,600 N

C = $0.85 f_{ck}$ a b = 25,500 a C = T ; a = 21 mm

▶ c = a / β_1 = 21 / 0.84 = 25 mm

ε_t = 0.003 (d_t−c) / c = 0.003 × (300 − 25) / 25

= 0.03323 > 0.002 ⇒ **가정만족 (주철근 항복)**

> 0.005 ⇒ **인장지배 단면**

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.03323 - 0.002)$$

= 3.252 > 0.9 ⇒ **ϕ_f = 0.900**

▶ 최소 철근비 검토

P_{min1} = 1.4 / f_y = 0.00350 ⇒ $A_{s1, min}$ = 1,050 mm^2

P_{min2} = 0.25 $\sqrt{f_{ck}}$ / f_y = 0.00342 ⇒ $A_{s2, min}$ = 1,027 mm^2] $A_{s, min}$ = 1,050 mm^2

$A_{s, Req}$ = 738 mm^2 , 4/3 $A_{s, Req}$ = 984 mm^2

MIN ($A_{s,min}$, 4/3 $A_{s,req}$) = 984 mm^2 < Use As = 1,324 mm^2 ∴ O.K

◈ Md = ϕ_f T (d − a / 2) = 0.900 × 529,600 × (300 − 21 / 2) / 10 E+6

= 138.042 kN.m > Mu = 78.129 kN.m ∴ O.K [S.F = 1.767]

(2) 전단 검토

▶ ϕ_v Vc = ϕ_v 1/6 $\sqrt{f_{ck}}$ B d = 155.188 kN > Vu = 84.550 kN

∴ 전단철근 불필요

(3) 주철근 간격 검토

▶ 주철근 응력, fs

A = 2 × 100 × 1,000 / 7 = 30,000 mm^2

k = − np + √ [2 np + (np)^2] [∵ n = Es / Ec = 7.3 ≒ 7]

= − 7 × 0.00441 + √ [2 × 0.031 + 0.0309 ^{2}] = 0.22

j = 1 − k / 3 = 1 − 0.22 / 3 = 0.927

fs = M / [As·d·j] = 123.801 MPa

◈ 주철근 간격

Cc = d_t' − d_b/2 = 100 − 15.9 / 2 = 92 mm (여기서, d_b = 16)

s_1 = 375 (210 / fs) − 2.5 Cc = 406 mm

s_2 = 300 (210 / fs) = 509 mm > s = 150 mm ∴ O.K

7.3.2 우측벽체 중앙부

(1) 휨 검토

f_{ck}	=	30 MPa	f_y	=	400 MPa	d_t'	=	100 mm	d'	=	100 mm
Mu	=	26.383 kN.m	Vu	=	0.000 kN	Mo	=	15.115 kN.m	d (휨)	=	200 mm
B	=	1000 mm	H	=	300 mm	헌치	=	0 mm	d (전단)	=	200 mm
ϕ_f	=	0.900	ϕ_v	=	0.85	주철근	1	단 배치	d_t	=	200 mm

Use A_s =	1단	1CYCl	H	16 @	300	,	2CYCLI	H	0 @	0	=	662 mm^2
	2단	1CYCl	H	@		,	2CYCLI	H	0 @	0	=	– mm^2
			TOTAL	=	662 mm^2		[p =	0.00331]				

▶ 주철근이 항복한다고 가정하면, T = A_s f_y = 264,800 N

C = 0.85 f_{ck} a b = 25,500 a   C = T ; a = 10 mm

▶ c = a / β_1 = 10 / 0.84 = 12 mm

ε_t = 0.003 (d_t–c) / c = 0.003 × (200 – 12) / 12

= 0.04530   > 0.002 ⇒ **가정만족 (주철근 항복)**

  > 0.005 ⇒ **인장지배 단면**

$$\phi_f = \phi_c + \frac{(\phi_t - \phi_c)}{(0.005 - \varepsilon_y)}(\varepsilon_t - \varepsilon_y) = 0.65 + \frac{(0.900 - 0.65)}{(0.005 - 0.002)}(0.04530 - 0.002)$$

= 4.259 > 0.9 ⇒ ϕ_f = 0.900

▶ 최소 철근비 검토

P_{min1} = 1.4 / f_y = 0.00350 ⇒ $A_{s1,\,min}$ = 700 mm^2

P_{min2} = 0.25 $\sqrt{f_{ck}}$ / f_y = 0.00342 ⇒ $A_{s2,\,min}$ = 685 mm^2] $A_{s,\,min}$ = 700 mm^2

$A_{s,\,Req}$ = 372 mm^2 , 4/3 $A_{s,\,Req}$ = 496 mm^2

MIN ($A_{s,min}$, 4/3 $A_{s,req}$) = 496 mm^2 < Use As = 662 mm^2 ∴ O.K

◈ Md = ϕ_f T (d – a / 2) = 0.900 × 264,800 × (200 – 10 / 2) / 10 E+6

= **46.427** kN.m > Mu = **26.383** kN.m ∴ O.K [S.F = 1.760]

(2) 주철근 간격 검토

▶ 주철근 응력, fs

A = 2 × 100 × 1,000 / 3 = 60,000 mm^2

k = – np + √ [2 np + (np)^2]   [∵ n = Es / Ec = 7.3 ≒ 7]

= – 7 × 0.00331 + √ [2 × 0.023 + 0.0232 ^{2}] = 0.193

j = 1 – k / 3 = 1 – 0.193 / 3 = 0.936

fs = M / [As·d·j] = 121.966 MPa

◈ 주철근 간격

Cc = d_t' – d_b/2 = 100 – 15.9 / 2 = 92 mm (여기서, d_b = 16)

s_1 = 375 (210 / fs) – 2.5 Cc = 416 mm

s_2 = 300 (210 / fs) = 517 mm   > s = 300 mm ∴ O.K

7.4 온도 및 배력철근 설계

<table>
<tr><th colspan="2" rowspan="2">구 분</th><th colspan="2">단면제원</th><th rowspan="2">사 용
주철근</th><th colspan="2">소요 배력철근량 (mm^2)</th><th colspan="3" rowspan="2">사용 배력철근량 (mm^2)</th><th rowspan="2">판정</th></tr>
<tr><th>B (mm)</th><th>H (mm)</th><th>주철근 20%</th><th>온도철근</th></tr>
<tr><td rowspan="4">하 부
슬래브</td><td rowspan="2">단 부</td><td rowspan="2">1,000</td><td rowspan="2">300</td><td>H16@150</td><td rowspan="2">265</td><td rowspan="2">600</td><td>인장</td><td>H13@200=634</td><td rowspan="2">950</td><td rowspan="2">O.K</td></tr>
<tr><td></td><td>압축</td><td>H13@400=317</td></tr>
<tr><td rowspan="2">중앙부</td><td rowspan="2">1,000</td><td rowspan="2">300</td><td>H16@150</td><td rowspan="2">265</td><td rowspan="2">600</td><td>인장</td><td>H13@200=634</td><td rowspan="2">950</td><td rowspan="2">O.K</td></tr>
<tr><td></td><td>압축</td><td>H13@400=317</td></tr>
<tr><td rowspan="4">좌측
벽체</td><td rowspan="2">단부</td><td rowspan="2">1,000</td><td rowspan="2">300</td><td>H16@150</td><td rowspan="2">265</td><td rowspan="2">750</td><td>인장</td><td>H13@200=634</td><td rowspan="2">950</td><td rowspan="2">O.K</td></tr>
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<tr><td rowspan="2">중앙부</td><td rowspan="2">1,000</td><td rowspan="2">300</td><td>H16@300</td><td rowspan="2">132</td><td rowspan="2">750</td><td>인장</td><td>H13@200=634</td><td rowspan="2">950</td><td rowspan="2">O.K</td></tr>
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<tr><td rowspan="4">우측
벽체</td><td rowspan="2">단부</td><td rowspan="2">1,000</td><td rowspan="2">300</td><td>H16@150</td><td rowspan="2">265</td><td rowspan="2">750</td><td>인장</td><td>H13@200=634</td><td rowspan="2">950</td><td rowspan="2">O.K</td></tr>
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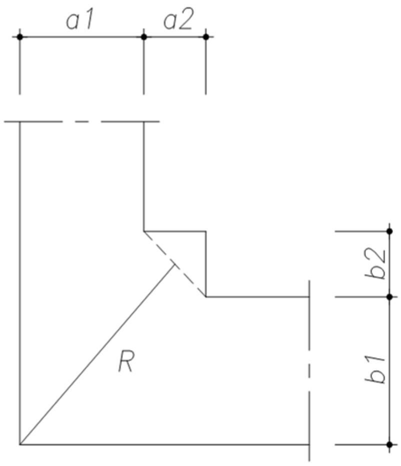
<tr><td></td><td>압축</td><td>H13@400=317</td></tr>
</table>

7.5 우각부 보강 설계

(1) 하부슬래브 (좌,우측)



f_{ck}	=	30	MPa
f_y	=	400	MPa
Mo	=	45.584	kN·m
a1	=	300	mm
a2	=	300	mm
b1	=	300	mm
b2	=	300	mm

R = {(a2 × b1 + b2 × a1 + a2 × b2) / (a2 + b2)} × √2

= 636 mm

– 주철근 (절곡하여 보강철근으로 고려할 수 있는 주철근량)

As' = H 16 @ 300 = 662 mm^2

H 0 @ 0 = 0 mm^2

1) 최대인장응력, f_{tmax}

$$f_{tmax} = \frac{5 \times Mo}{R^2 \times W} = \frac{5 \times 45.584 \times 10^6}{(636.396)^2 \times 1{,}000}$$

= 0.563 Mpa > 0.08 √ fck = 0.438 Mpa **보강철근 필요 !**

2) 보강철근량 산정, As

– 필요철근량

$$As = \frac{2 \times Mo}{R \times f_{sa}} - As'$$

$$= \frac{2 \times 45.584 \times 10^6}{636 \times 180} - 662$$

= 796 mm^2 – 662 mm^2

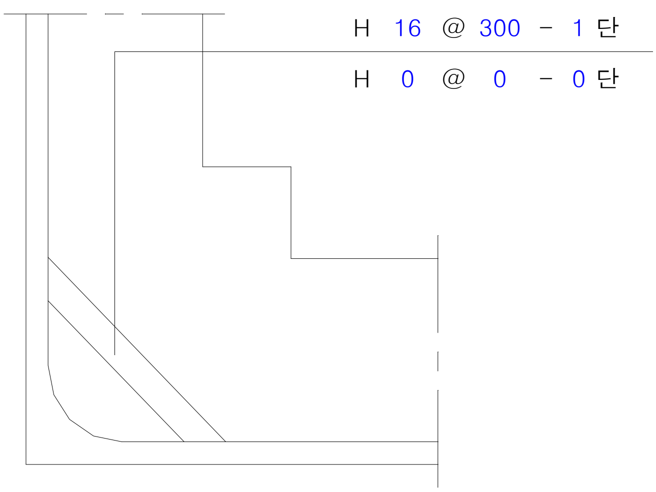
= 134 mm^2

– 사용철근량

As = H 16 @ 300 – 1 단

\+ H 0 @ 0 – 0 단

= 662 mm^2 O.K



8. 사용성 검토

8.1 처짐 검토

1) 하부 슬래브 : 최소부재두께 적용

철근의 항복응력 f_y = 400 MPa

부재의 지간길이 L = 2.300 m

슬래브 양단 연속의 경우

$T = L / 28 = 0.082$ m

(보정계수) FACTOR = 1.000

처짐계산을 하지않는 경우의 최소두께

$T = 0.082\text{m} < H = 0.300\text{m}$ (처짐검토 생략)

9. 기초지반 지지력 검토

9.1 작용하중

9.1.1 구 체 자 중 (W1) :

(0.3 × 2.4 + 0.3 × 2 × 1 EA + 0.3 × 0.3 / 2 × 2 EA + 0.3 × 2.4) × 24.500 / 2.6 = 20.071 kN/m^2

9.1.2 상재고정하중(W2)

(1) 상재토

되메움토 : 0.000 × 20.000 = 0.000 kN/m^2

아스콘 : 0.000 × 23.000 = 0.000 kN/m^2

9.1.3 활 하 중(L)

(1) 활 하 중 :

= 0.000 kN/m^2

9.1.4 수로개거내의 물의자중(w3)

2.000 × 1.920 × 1.000 / 2.600 / 1.000 × 10.000 = 14.769 kN/m^2

9.2 지지력 검토

9.2.1 최대지반반력 (qmax) : W1 + W2 + L + W3

20.071 + 0.000 + 0.000 + 14.769 = 34.840 kN/m^2

9.2.2 허용지반반력 (qa)

- 정역학 공식에 의한 지반의 허용 연직 지지력 (Terzaghi식 사용)

$Qu = A \cdot [\ c \cdot Nc + q \cdot Nq + 0.5 \cdot r1 \cdot B \cdot Nr\ ]$

Qu : 하중의 편심 및 경사가 없는 연속기초지반의 극한지지력 (kN)

· A : 재하면적 = 2.600 m^2

· c : 지반의 점착력 = 0.000 kN/m^2

· q : 상재하중, (q = r2 · Df) = 48.000 kN/m^2

· r1 : 지지 지반의 흙의 단위중량 = 20.000 kN/m^3

· r2 : 근입 지반의 흙의 단위중량 = 20.000 kN/m^3

· B : 확대기초의 폭 = 2.600 m

· L : 확대기초의 길이 = 1.000 m

· Φ : 지반의 전단 저항각

$\Phi = \sqrt{(12 \times N)} + 15 = \sqrt{(12 \times 15)} + 15$ = 28 °

· Nc = 25.100 , Nq = 12.700 , Nr = 9.700 (전단 저항각 28 ° 일때)

· Qu = 2.600 × [0.000 × 25.100 + 48.000 × 12.700 + 0.500 × 20.000 × 2.600 × 9.700] = 2240.680 kN

· Qa = 2240.680 / 3 = 746.893 kN

· qa = 746.893 / 2.600 = 287.267 kN/m^2

* 평상시 최대 지반반력 (도.설.해 2008) = 400.000 kN/m^2

∴ q_{max} = 34.840 kN/m^2	<	q_a = 287.267 kN/m^2		∴ O.K