Final Submittal

Structural Calculations

FY 98 MCON Project P-501
Two Torpedo Magazines

Naval Weapons Station
Yorktown, VA

Construction Contract Number:
N62470-94-B-4155
MHE Project: 2446

February 1999

Mason & Hanger Engineering, Inc.
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(606) 223-4773 Fax (606) 223-1846

Mason & Hanger ENGINEERING INC.

Encl (3)

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Project P-501 Location NAS Yorktown, VA

Subject

MCON Project P-501

Construct Two Torpedo Magazines

STRUCTURAL CALCULATIONS

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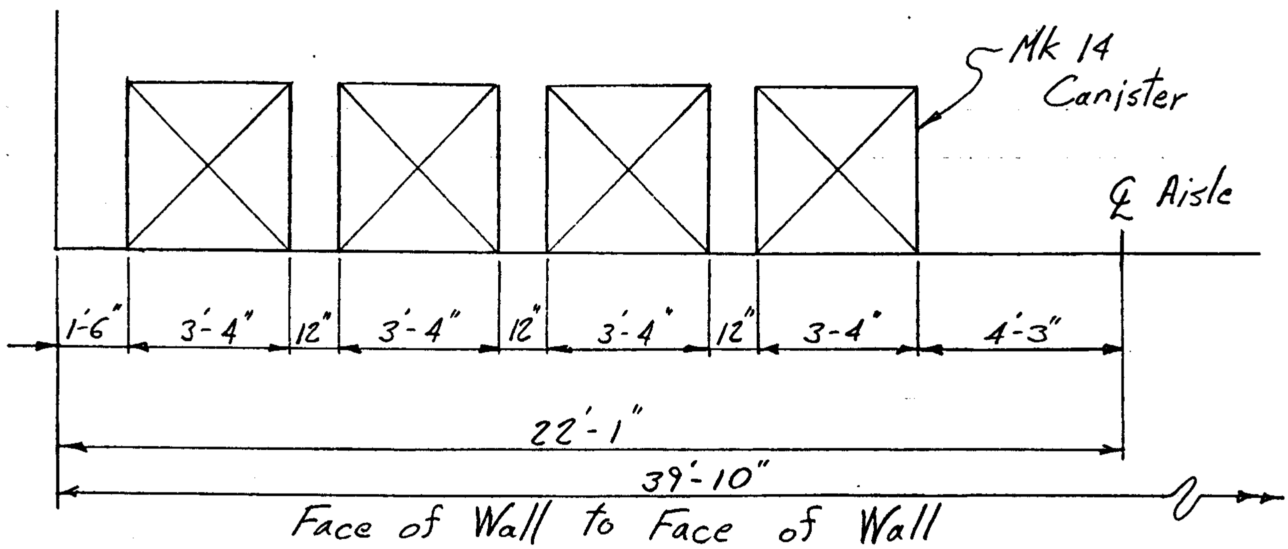
Required Width of Magazine

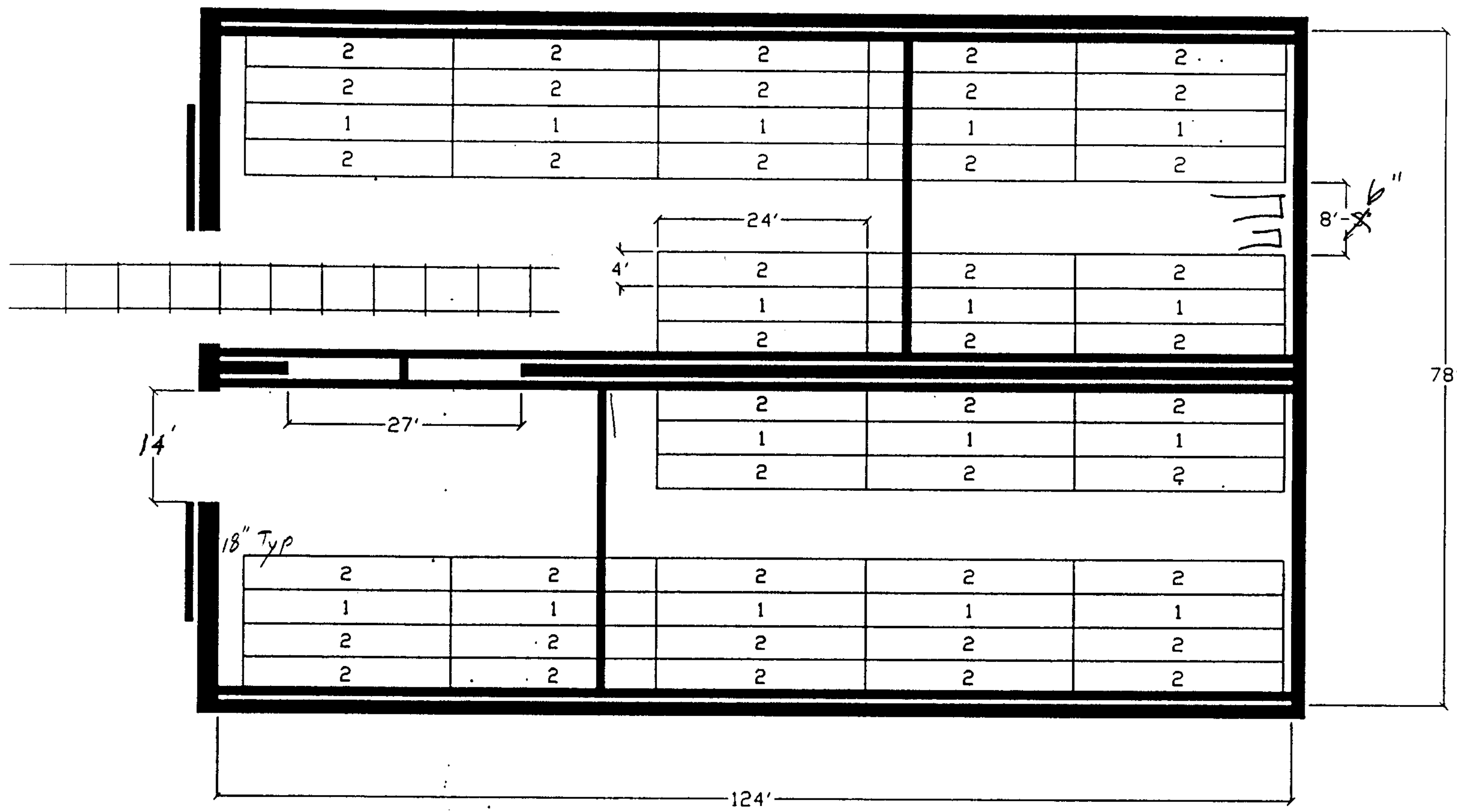
Using Mk 14 Canister Size: 40" Wide × 280" Long (23'-4")
For Crane & Strongback Connection, provide 18" Clearance between wall and canister. Provide 12" Space between canisters.



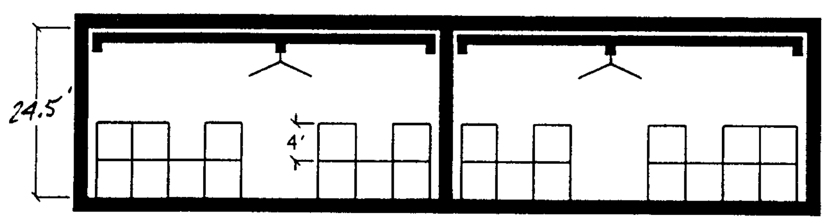
Actual Container Length: 280" = 23'-4"

Must Leave 12" between Containers

TOP



FRONT



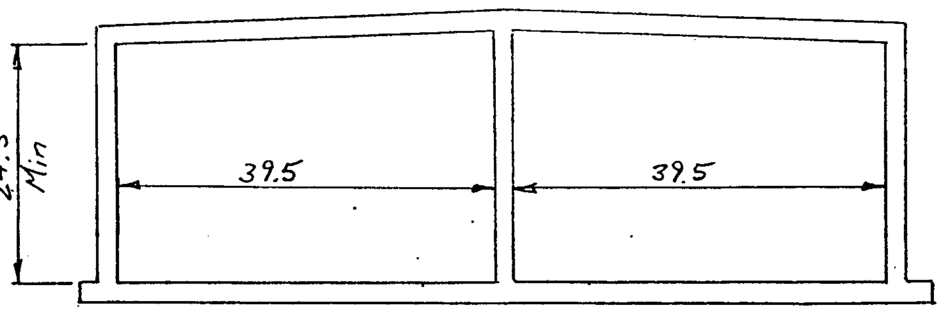
COMNAVSEASYSCOM

TYPE M MAGAZINE (VE)

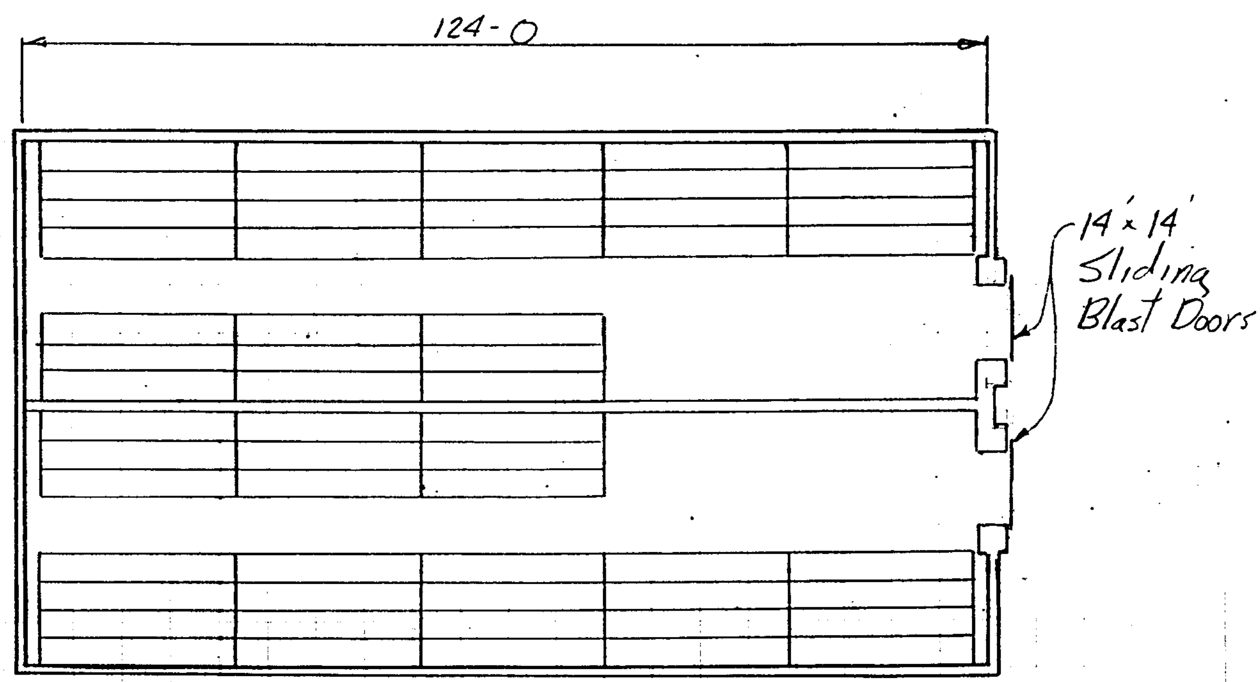
BY: NWS SEAL BEACH

27 NOV 1991

Magazine Design

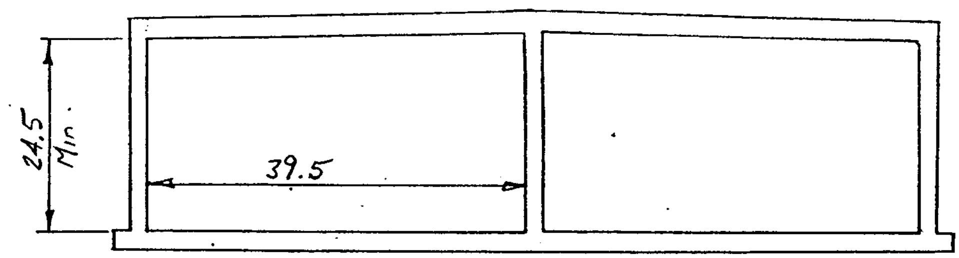


Typical Section



Floor Plan

Concrete Structural Design

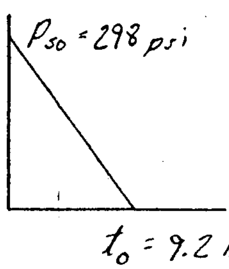


Design Blast Load on Roof

i_s = 1370 psi-ms
P_{so} = 298 psi
t_o = 9.2 ms

From: "Basis of Design for Explosives Safety of Standard Missile Magazine Milcon P-137"
NCEL TM #51-91-11

By Phil Wager & Bill Keenan



Materials:

Concrete Strength: $f'c$ = 4000 psi
$f'dc$ = 1.19 × 4000 = 4,760 psi

Reinforcing Steel f_y = 66,000 psi per P-397

f_{dy} = 1.17 × 66,000 = 77,220 psi

Design Reference: Structures to Resist the Effects of Accidental Explosions
P-397, Nov 1990.

Explosive Safety Objective: Prevent sympathetic detonation

Allowable Deflections:
from NCEL TM#51-91-11
(Max Support Rotations)

Roof: Slabs, $\theta_u = 4.0°$
Beams, $\theta_u = 2.0°$

Headwall: Slabs, $\theta_u = 4.0°$
Beams, $\theta_u = 2.0°$

Steel Doors: $12°$, $\frac{X_u}{X_E} = 20$

For ESQD Design:

NEW = 350,000 lbs for Donor Magazine (Type F)
NEW = 100,000 lbs for Exceptor Magazine (Type M)

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ROOF

SLAB

Mason & Hanger Engineering, Inc.
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Project P-501 Location Yorktown, VA

Subject Roof Slab Analysis for 7.5 Ton Crane

Crane Loads for 5-Ton Crane on P-298 @ Port Hadlock, WA
⇒ From Navy Crane Center, Design for 12 Kip Load on Each Support Plate.

Per Telecon w/ Vipin Shah from Navy Crane Center 8/7/96
Use 150 % of 5 Ton Crane Load for 7.5 Ton Crane plus slight increase in Dead Load.

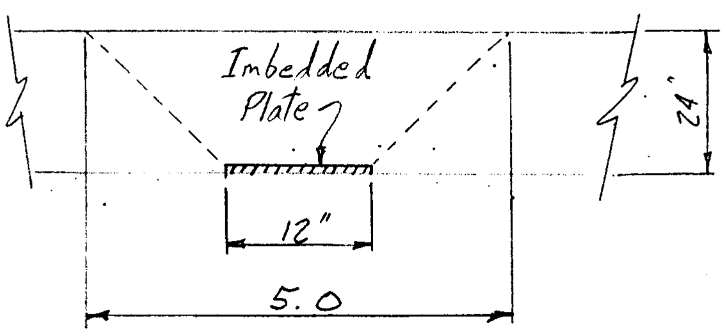
$P = 12{,}000 \times 1.5 + 500$ (Increase in W_{DL})
$= 18.5$ Kips on Any One Support ℞
Assume Support ℞ on Opposite Side of Bay would support 1/4 of this Max. Load.

Therefore, Maximum Support Plate Reaction (Including Impact)

$P = 1.1(18.5 \times .8) + (18.5 \times .2) = 16.28^{K} + 3.7^{K} = 20$ Kips

Max Factored Crane Load, $P_u = 1.7(20) = 34$ Kips

Min Factored Crane Load, $P_u = 1.7(\frac{20}{4}) = 8.5$ Kips



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Project P-501, ADCAP Magazines Location NWS Yorktown, VA

Subject

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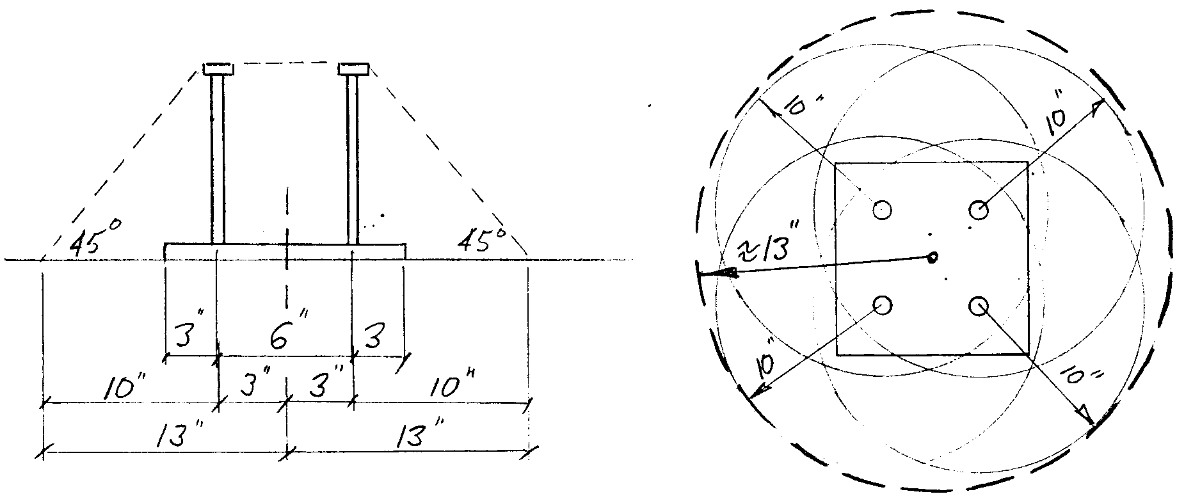
Capacity of Embedded Crane Support Plate



For Headed Anchor Bolt Design

Effective Projected Stress Area, $A_e = \pi r^2 = \pi(13)^2 = 531 \text{ in}^2$

Pullout Strength of Concrete, $U_p = 4\beta\sqrt{f'c}\,A_e$
(Tensile Capacity of Failure Cone)

$U_p = 4(.85)\sqrt{4000}\,(531) = 114{,}184$ lbs

Check design resistance of bolt tensile strength, $F_u A_t$

$F_u A_t = 4(19{,}370) = 77{,}480 \text{ lbs} < 114{,}184 \text{ lbs}$ OK
(Table 3)

Total effective Design Tension Load, $T = \frac{T_F}{\phi} = \frac{34{,}000 \text{ (Previous sht)}}{.9} = 38^K$
From Table 2A, Basic Design Values
for ¾ A307 = 12.02 K/Ea × 4 = 48 Kips > 38 K Actual Load
OK
Include Add'l 1" × 18" Bolt @ Center of ℘

From: "Design of Headed Anchor Bolts" by Shipp & Haninger

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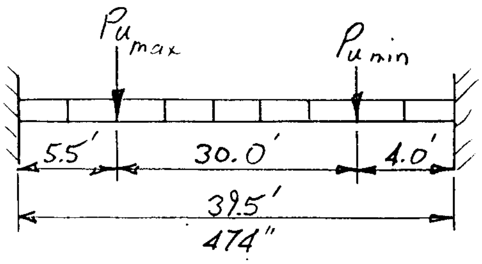
Project P-501 Location Yorktown, VA

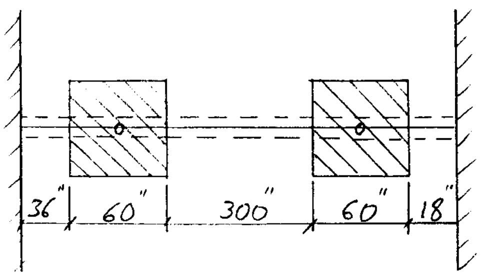
Subject Roof Slab Analysis for 7.5 Ton Crane

Design One Foot Wide Strip of Roof



Uniform Load on Roof, W
= Live Load + Soil + Conc Slab

$$W_u = 1.4(\overset{Conc}{2 \times 150} + \overset{Soil}{2 \times 120}) + 1.7(200)$$
$$= 1096 \text{ psf} / 12 = 91.33 \text{ lbs/In}$$

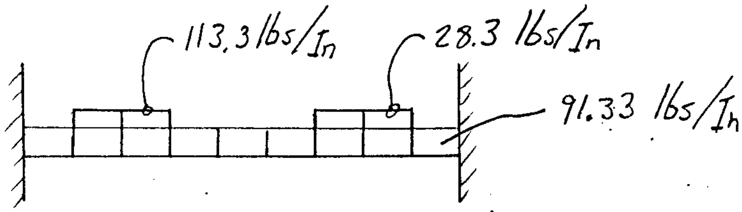


Add in Crane Loads
See Page 1 (Two Sheets Back)

Design Roof Strip Supporting 7.5 Ton Crane Loads

Uniform Load from $P_{u\,max} = \frac{34}{5 \times 5} = 1{,}360 \text{ psf}/12 = 113.3 \text{ lbs/In.}$

Uniform Load from $P_{u\,min} \; \frac{8.5}{5 \times 5} = 340 \text{ psf}/12 = 28.3 \text{ lbs/In.}$



From "CAST" Utility, Maximum Static Load,
Based on DL + LL + Crane Load:

$M_u = 2{,}043{,}751$ in-lbs

Check Capacity of Roof Slab for Static Loads

Check Shear (from CAST Output)

V_u @ Support = 28.1 Kips $\quad d = 21.365''$

$V_u @ d = 28.1 - 21.365\,(91.33)\frac{1}{1000} = 26.15$ Kips

$v_u = \dfrac{V_u @ d}{\phi b d} = \dfrac{26.15}{.85 \times 12 \times 21.37} = 120$ psi

$v_c = 2\sqrt{f'_c} = 2\sqrt{4000} = 126$ psi

$v_c > v_u$ No Shear Reinf Required

Check 24" Slab for Maximum Moment

$M_{u_{max}} = 2{,}043{,}751$ in-lbs $\quad d = 24'' - \underbrace{1\tfrac{1}{2}}_{\text{Clr}} - \underbrace{\tfrac{1}{2}}_{\text{Stirrup}} - \underbrace{\tfrac{1}{2}(1.27)}_{\#10\ \text{Bar}} = 21.365''$

$\dfrac{M_u}{\phi b d^2} = \dfrac{2{,}043{,}751}{.9(12)(21.365)^2} = 415 \qquad \rho_{Req'd} = .00738$

$A_{s\,Req'd} = .00738 \times 12 \times 21.365 = 1.89\ in^2$

Reinf Provided for Blast Design #10 @ 8", $A_s = 1.91\ in^2$

Therefore, $A_{s\,Prov'd} = 1.91 > A_{s\,Req'd} = 1.89\ in^2$

24" Slab w/ #10 @ 8" is Adequate

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**********************************************************************
*       CAST-UTILITY  BY  CAST INC.                                  *
*                                                                    *
*        M A S O N    &    H A N G E R        ENGINEERS              *
*                                                                    *
*                       TIME: 9/11/97 16:12:58        PAGE:    1     *
**********************************************************************


************************************
* SUMMARY OF THE INPUT INFORMATION *
************************************

TYPE OF THE PROBLEM : CONTINUOUS BEAM CALCULATION

* Types and the locations of the supports are:

   Fixed support at X=     0.0000
   Fixed support at X=   474.0000

* Total number of different materials     :     1

  From X=      0.0000   to X=   474.0000   E= 3830000.0000

* Total number of different sections      :     1

  From X=      0.0000 to X=   474.0000 IX=  1000.0000

* Total number of distributed loads           :     3
  At X=      0.0000  W1=     91.3300   At X=   474.0000  W2=     91.3300
  At X=     36.0000  W1=    113.3000   At X=    96.0000  W2=    113.3000
  At X=    396.0000  W1=     28.3000   At X=   456.0000  W2=     28.3000


*************************
* SUMMARY OF THE RESULTS *
*************************

* MAXIMUM VALUES

  * Max. displacement is    3.35674  at   X=   237.0000
  * Min. displacement is    0.00000  at   X=     0.0000

  * Max. shear force is 28119.00389  at   X=     0.0000
  * Min. shear force is -23667.41611  at   X=   474.0000

  * Max. moment is 2043750.96895  at  X=     0.0000
  * Min. moment is -893037.56715  at  X=   237.0000

* TOTAL APPLIED LOADS

  * Total applied concentrated load:     0.0000
  * Total applied concentrated moment:     0.0000
  * Total applied distributed load: 51786.4200

* SUPPORT REACTIONS

  * Reaction at X=     0.0000 : Force= -28119.0039 Moment= -2043750.9689
  * Reaction at X=   474.0000 : Force= -23667.4161 Moment= 1830260.6668
```

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****************************
* DETAILS OF THE ANALYSIS *
****************************
```

* DISPLACEMENT AT OUTPUT POINTS :

At X=	0.0000	Displacement=	0.00000
At X=	36.0000	Displacement=	0.29036
At X=	51.0000	Displacement=	0.53844
At X=	66.0000	Displacement=	0.83028
At X=	81.0000	Displacement=	1.14807
At X=	96.0000	Displacement=	1.47668
At X=	118.5000	Displacement=	1.96316
At X=	237.0000	Displacement=	3.35674
At X=	355.5000	Displacement=	1.86793
At X=	396.0000	Displacement=	1.00572
At X=	411.0000	Displacement=	0.70773
At X=	426.0000	Displacement=	0.44214
At X=	441.0000	Displacement=	0.22438
At X=	456.0000	Displacement=	0.07151
At X=	474.0000	Displacement=	0.00000

* SHEAR FORCES :

	Coordinate	Shear (LT side)	Shear (RT side)
X=	0.0000		28119.0039
X=	36.0000	24831.1239	24831.1239
X=	51.0000	21761.6739	21761.6739
X=	66.0000	18692.2239	18692.2239
X=	81.0000	15622.7739	15622.7739
X=	96.0000	12553.3239	12553.3239
X=	118.5000	10498.3989	10498.3989
X=	237.0000	-324.2061	-324.2061
X=	355.5000	-11146.8111	-11146.8111
X=	396.0000	-14845.6761	-14845.6761
X=	411.0000	-16640.1261	-16640.1261
X=	426.0000	-18434.5761	-18434.5761
X=	441.0000	-20229.0261	-20229.0261
X=	456.0000	-22023.4761	-22023.4761
X=	474.0000	-23667.4161	

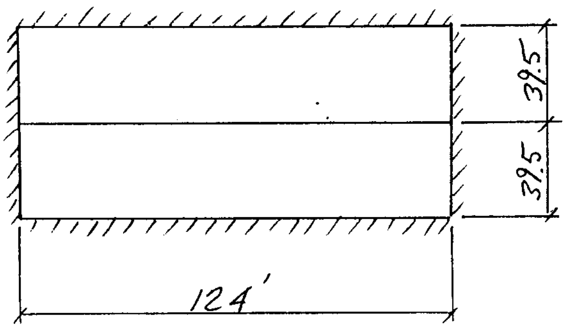
* MOMENTS :

	Coordinate	Moment (LT side)	Moment (RT side)
X=	0.0000		2043750.9689
X=	36.0000	1090648.6690	1090648.6690
X=	51.0000	741202.6857	741202.6857
X=	66.0000	437798.4524	437798.4524
X=	81.0000	180435.9691	180435.9691
X=	96.0000	-30884.7642	-30884.7642

X=	118.5000	-290216.6454	-290216.6454
X=	237.0000	-893037.5672	-893037.5672
X=	355.5000	-213379.7965	-213379.7965
X=	396.0000	312968.0699	312968.0699
X=	411.0000	549111.5866	549111.5866
X=	426.0000	812171.8533	812171.8533
X=	441.0000	1102148.8700	1102148.8700
X=	456.0000	1419042.6367	1419042.6367
X=	474.0000	1830260.6668	

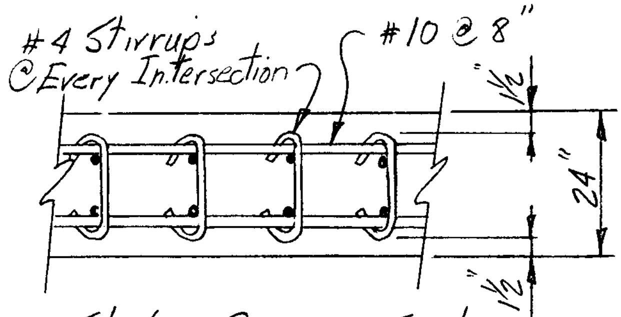
Roof Slab Design - For Blast Loads

Design Roof Slab as Flat Slab, Fixed Four Sides Using Outside and Center Walls for Support



← Clear Spans

Analyze 24" Thick Roof Slab



Typ. Slab Cross-Section

Clear Distances to ℄ Reinf

Inside or Outside Face of Slab

Outside #10) $2 + \frac{1.27}{2} = 2.635"$

Inside #10: $2 + 1.27 + \frac{1.27}{2} = 3.905"$

Use #10 @ 8" EWEF

$A_s = 1.27 \times \frac{12}{8} = 1.91 \text{ in}^2/\text{ft}.$

Outside Bars Run Perpendicular to Center Wall.

Calculate Mass of System, Using 24" Earth Cover + 23" Slab

$$\text{Mass, } M = \frac{(24 \times 150 + 24 \times 120) \times 10^6}{12 \times 32.2 \times 1728} = 9{,}705 \text{ psi}$$

Provide #4 Stirrups @ Every Bar Intersection in Accordance (Para 4-22) with TM5-1300 (P397) Since the scaled distance is greater than 1, $W^{1/3} = (350{,}000)^{1/3} = 70.5$ ft, Actual $d = 88$ ft. Therefore, Scaled Distance, $d_s = 88/70.5 = 1.25 > 1.00$

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Lexington, Kentucky

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Project P-501 Location Yorktown, VA

Subject Roof Slab Design

Roof Design (Cont'd)

Run CBARCS, To Determine Design Properties

Initial & Rebound Response P501A.OUT

$K_{LM} = .6935$

$r_u = 23.13$ psi

$K_E = 16.50$ psi

Resistance to Blast Loads must be reduced by the dead loads of the earth cover and the weight of the concrete slab.

$$r_{avail} = r_u - r_{DL}\left(\frac{f_{ds}}{f_y}\right) \qquad \text{For } \Delta_x \le 2^\circ \; f_{ds} = f_{dy} = 1.17 f_y$$

$$r_{DL} = \frac{24 \times 150 + 24 \times 120}{12 \times 144} = 3.75 \text{ psi}$$

$$r_{avail} = 23.13 - 3.75\left(\frac{1.17 \times 66{,}000}{66{,}000}\right) = 18.74 \text{ psi}$$

Run SOLVER to determine Dynamic Responses & Deflection Using the following Properties:

Mass = 9,705 $K_{LM} = .6935$

Initial $r_u = 18.74$ Initial $K_E = 16.50$

Rebound $r_u = -23.13$ Rebound $K_E = 16.50$

From SOLVER Output: RF501

Max Defl, $\Delta_x = 7.85$ Inches

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Lexington, Kentucky

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Project P-501 Location Yorktown, VA

Subject Roof Slab Design

Dead Load Deflection, Assume both ends are fixed

$$\Delta_x = \frac{wl^4}{384EI} \qquad \text{Avg } I = 757.33 \text{ in}^4 \text{ (from CBARCS)}$$

$$W = \frac{2 \times 150 + 2 \times 120}{12 \times 12}$$

$$W = 3.75 \text{ psi}$$

$$\Delta_x = \frac{3.75\,(39.5 \times 12)^4\,(12)}{384\,(29{,}000{,}000)(757.33)}$$

$\Delta_x = 0.27$ Inches

$$\operatorname{Tan} \phi = \frac{\Delta_x}{L/2} = \frac{.27}{39.5/2 \times 12} = .001139$$

$$\phi = .065^\circ$$

Total Slab Deflection: $= Defl_{(DL)} + Defl_{(Blast)}$

Total $\Delta_x = .27 + 7.85 = 8.12$ Inches

$$\operatorname{Tan} \phi = \frac{\Delta_x}{L/2} = \frac{8.12}{39.5/2 \times 12} = .0343$$

Maximum $\phi = 1.96^\circ < 2.0^\circ$ OK

Use Type I Cross-Section

24" Thick Roof Slab

Reinforcement = #10's @ 8" EWEF

#4 Single Leg Stirrups @ Every Bar Intersection

File : P501A.OUT

CBARCS Output

P501 TORPEDO MAGAZINES - 39.5 FT ROOF SPAN, 2 DEG ROTATION

BLAST WALL HEIGHT	39.50 FT
BLAST WALL LENGTH	124.00 FT
DURATION OF LOAD	9.20000 MSEC
FICTITIOUS PEAK PRESSURE	298.00000 PSI
EFFECTIVE IMPULSE	1370.00 PSI MS

HEIGHT 474.00 IN LENGTH 1488.00 IN

DYNAMIC CONCRETE STRENGTH	4760.00
DYNAMIC STEEL STRESS	77220.00
THICKNESS CONCRETE INCHES	24.0000
THICKNESS OF SAND INCHES	.0000
THETA ALLOWABLE DEGREES	2.0000

AREA VERT TOP STEEL/FT	1.9100	COVER	2.6350
AREA VERT BOT STEEL/FT	1.9100	COVER	2.6350
AREA HORIZ TOP STEEL/FT	1.9100	COVER	3.9050
AREA HORIZ BOT STEEL/FT	1.9100	COVER	3.9050

TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI	3555611.
RATIO MOD STEEL/CONCRETE	8.16
GROSS MOMENT INERTIA	1152.00
AVE CRACKED MOM INERTIA	362.67
AVE MOMENT INERTIA	757.33
AVERAGE PERCENT STEEL	.0077
D FACTOR MU=1/6	2769785725.
D FACTOR MU= 0.3	2959104988.

ALLOW SHEAR UNREINFORCED WEB 115.99 PSI 2404.50 LBS/IN WIDTH

ALLOW SHEAR AT SUPPORT 753.98 PSI 15630.09 LBS/IN WIDTH

UNREINFORCED CONCRETE THETA LE 2 DEG

POSITIVE VERTICAL MOMENT	243925.57
NEGATIVE VERTICAL MOMENT	243925.57
POSITIVE HORIZONTAL MOMENT	228316.19
NEGATIVE HORIZONTAL MOMENT	228316.19

SUPPORT ON 4 SIDES

YIELD LINE X FROM SIDE

LOCATION YIELD LINE LENGTH 314.19

LOCATION YIELD LINE HEIGHT	237.00	
ULTIMATE LOAD CAPACITY RU	23.1294	
SHEAR LOAD AT VERTICAL SUPPORT	4360.16	LB/IN WIDTH
SHEAR LOAD AT HORIZONTAL SUPPORT	4651.63	LB/IN WIDTH
SHEAR AT DISTANCE FROM VERTICAL SUPPORT	193.72	PSI
SHEAR AT DISTANCE FROM HORIZONTAL SUPPORT	205.41	PSI
ALLOWABLE MAX DEFLECTION	8.2901	

SHEAR CAPACITY(VC) EXCEEDED

LOAD MASS FACTOR	.6935
MASS CONCRETE ONLY	3739.48

FIRST YIELD POINT AT PT 3

ELASTIC LIMIT RE PSI	13.05
ELASTIC DEFLECTION XE	.6109

SECOND YIELD AT PT 2

ELASTO PLASTIC LIMIT	16.29
ELASTO-PLASTIC DEFLECTION	.7547
ULTIMATE RESISTANCE	23.13
PLASTIC DEFLECTION	2.1736

ULTIMATE RESISTANCE RU	23.13
ELASTIC DEFLECTION LIMIT XE	1.4015
STIFFNESS KE	16.50

MASS	3739.481
LOAD	298.000
DURATION	9.200
RESISTANCE	23.129
STIFFNESS	16.503

GAS PRESSURE .00 DURATION .00

NATURAL PERIOD	94.580049
MAXIMUM DEFLECTION	11.284880
TIME TO MAXIMUM DEFLECTION	63.336560
DURATION/NATURAL PERIOD	.097272
LOAD/RESISTANCE	12.884016
ELASTIC DEFLECTION LIMIT	1.401502

MAX FRAGMENT SPALL VELOCITY FT/SEC 28.168024

WALL COLLAPSES(THETA EXCEEDED)

AVERAGE SCAB VELOCITY	16.05
MAX SCAB VELOCITY	80.23

SOLVER INPUT RF501

```
SOLVE    RF501: ANALYSIS USING 24" SLAB W/ #10 @ 8"
    1     1        .05        200                                             -1

    2     2       9705       0.01
    1      16.50      18.74                        .6935       1.0
    3        0.0                          10       .6935       0.0
    1      16.50     -23.13                        .6935       1.0
    3        0.0                         -10       .6935       0.0
    1     3
       0.0        298        9.2        0.0         30        0.0
STOP
```

SOLVER OUTPUT RF501

1ONE DEGREE OF FREEDOM SOLVER INPUT
VERSION 2.2 FEB 1989

PROBLEM DESCRIPTION
SOLVE RF501: ANALYSIS USING 24" SLAB W/ #10 @ 8"

ANALYSIS CONTROL CARD

```
TYPE OF SOLUTION......................     1
  EQ. 0,  DEFAULTS TO 1
  EQ. 1,  NEWMARK-BETA METHOD
  EQ. 2,  WILSON-THETA METHOD
NUMBER OF LOAD CASES..................     1
TIME STEP.............................     0.0500
TIME LIMIT............................   200.0000
NEWMARK*S GAMMA.......................     0.0000
  EQ. 0.0, DEFAULTS TO 0.5
NEWMARK*S BETA........................     0.0000
  EQ. 0.0, DEFAULTS TO 0.25
WILSON*S THETA........................     0.0000
  EQ. 0.0, DEFAULTS TO 1.4
PRINT OPTION..........................    -1
  EQ. 0, DEFAULTS TO 1
  EQ. 1, PRINT EVERY STEP
  EQ. N, PRINT EVERY N-TH STEP
  EQ. -1, PRINT SIGNIFICANT CHANGES
```

INITIAL CONDITIONS

```
INITIAL DEFLECTION....................     0.0000
INITIAL VELOCITY......................     0.0000
INITIAL ACCELERATION..................     0.0000
```

STIFFNESS CONTROL CARD

```
NUMBER OF STIFFNESSES.................     2
NUMBER OF REBOUND STIFFNESSES.........     2
MASS..................................  9705.0000
PERCENT OF CRITICAL DAMPING...........     0.0100
```

INITIAL RESISTANCE-DEFLECTION CURVE

NUMBER	MODE	STIFFNESS	RESISTANCE	YLD DEFL	MASS FACT	DAMP FRAC
1	1	0.16500E+02	0.18740E+02	0.00000E+00	0.69	1.00
2	3	0.00000E+00	0.00000E+00	0.10000E+02	0.69	0.00

REBOUND RESISTANCE-DEFLECTION CURVE

NUMBER	MODE	STIFFNESS	RESISTANCE	YLD DEFL	MASS FACT	DAMP FRAC
3	1	0.16500E+02	-0.23130E+02	0.00000E+00	0.69	

```
1.00
      4      3 0.00000E+00 0.00000E+00-0.10000E+02        0.69
0.00


 LOAD DATA

 LOAD CASE NUMBER......................    1
 NUMBER OF LOAD POINTS.................    3


 POINT              TIME              LOAD
     1            0.0000          298.0000
     2            9.2000            0.0000
     3           30.0000            0.0000


 GENERATED RESISTANCE - DEFLECTION CURVES

 NATURAL PERIOD........................  126.8992

 NUMBER  MODE   STIFFNESS  RESISTANCE    YLD DEFL   MASS FACT   DAMP
FRAC
      1     1 0.16500E+02 0.18740E+02 0.11358E+01        0.69
1.00
      2     3 0.00000E+00 0.18740E+02 0.10000E+02        0.69
0.00
      3     1 0.16500E+02-0.23130E+02-0.14018E+01        0.69
1.00
      4     3 0.00000E+00-0.23130E+02-0.10000E+02        0.69
0.00
1* * * * NEWMARK-BETA SOLUTION * * * * *
 LOAD CASE............................    1
 TIME STEP............................    0.0500


 * * * * * SOLUTION RESULTS  * * * * * *
        STEP   TIME STIF DEFLECTION   VELOCITY        ACC RESISTANCE
LOAD

 ELAS    0    0.0000    1      0.000     0.0000     0.0443       0.00
298.00
 *CHG  176    8.7662    2      1.136     0.1929    -0.0009      18.74
14.05
 REB  1565   78.2500    1      7.851    -0.0001    -0.0028      18.74
0.00
 *CHG 2203  110.1456    3      6.715    -0.0554     0.0001       0.00
0.00
 RLOD 2834  141.7000    3      5.614     0.0001     0.0027     -18.16
0.00
 *CHG 3472  173.5984    1      6.715     0.0536    -0.0001       0.00
0.00


 ***** TIME LIMIT REACHED

 MAXIMUM DEFLECTION IN SOLUTION
 STEP NUMBER.......................... 1564
 TIME AT MAXIMUM......................      78.2000
 MAXIMUM DEFLECTION...................       7.851
```

```
 MINIMUM DEFLECTION IN SOLUTION
 STEP NUMBER............................ 2833
 TIME AT MINIMUM........................      141.6500
 MINIMUM DEFLECTION.....................        5.614


SOLUTION TIME LOG
   ASSEMBLY.......................          0.06
   SDOF SOLUTION.(LOAD CASE  1)...          0.11

   TOTAL TIME.....................          0.17
1ONE DEGREE OF FREEDOM SOLVER INPUT
 VERSION 2.2  FEB 1989

 PROBLEM DESCRIPTION
 STOP

 ***** END SOLVER RUNS *****
```

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

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SIDEWALL

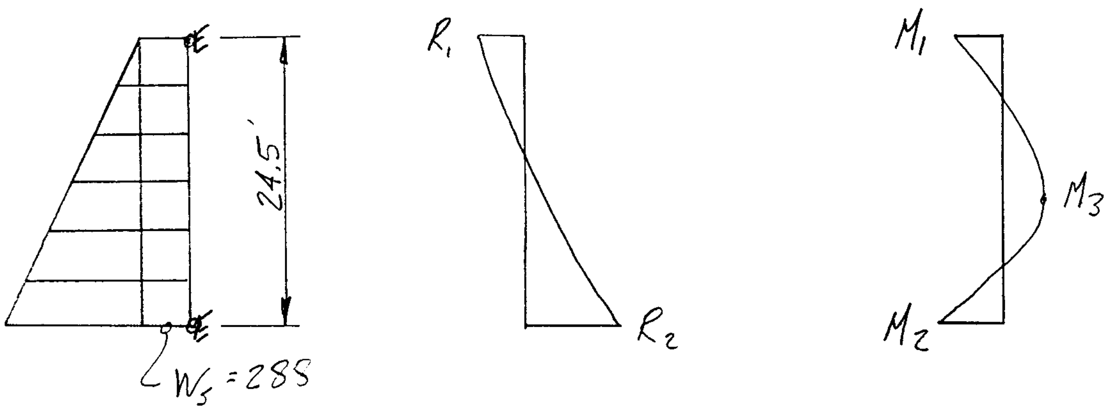
DESIGN

Sidewall Design - Using Wall Height = 24.5 feet

Design for Lateral Earth Pressure

From Soils Report, Design Walls for a Passive Earth Coeff $K = 0.6$
Therefore, Uniform Soil Pressure, $P_E = 0.6 W_E = .6(120) = 72$ pcf

Uniform Load, from Top of Wall to Top of Earth Cover
$W_S = 4 \text{ ft} \times 72 = 288$ lbs/ft.



Triangular Load, $P_T = 72 \times 24.5 \times \frac{24.5}{2} = 21,609$ lbs

From CAST Program: $M_{max} = 808,176 \times \frac{1}{12} = 67,348$ ft-lbs

$M_u = 1.7 \times 67,348 = 114,492$ ft-lbs

$d = 24 - 1\frac{1}{2} - \frac{1.27}{2} = 21.865''$

$\frac{M_u}{\phi b d^2} = \frac{114,492 \times 12}{.9(12)(21.865)^2} = 266$ $\rho_{Req'd} = .00461$

$A_{S\,Req'd} = .00461(12)(21.865) = 1.21 \text{ in}^2$

$A_{S\,Prov'd} = 1.27 \times \frac{12}{8} = 1.91 \text{ in}^2 > 1.21 \text{ in}^2$ OK

A_S Prov'd (#10 @ 8") = 1.91 in² is Adequate

Sidewall Design

Check Shear

From CAST Program, Max Shear = 18,654 lbs

Design for Shear @ "d" distance from Support

Therefore, design Shear, $V_n = 18{,}654 - 21.865\left(\frac{18{,}654 - 10{,}011}{294}\right)$
$= 18{,}654 - 643 = 18{,}011$ lbs

$V_u = 1.7 \times 18{,}011 = 30{,}619$ lbs

$v_u = \frac{V_u}{\phi b d} = \frac{30{,}619}{.85(12)(21.865)} = 137$ psi

$v_c = [1.9(f'dc)^{1/2} + 2500\rho] \leq 3.5(f'dc)^{1/2}$ $\qquad f'dc = 4000 \times 1.0$

$= [1.9(4000)^{1/2} + 2500(.0073)]$ $\qquad \rho = \frac{1.91}{12 \times 21.865} = .0073$

$= [120.2 + 18.25] = 138.5\text{psi} > 137$ psi OK

No Shear Reinforcement Required

```
************************************************************************
*       CAST-UTILITY  BY  CAST INC.                                    *
*                                                                      *
*        M A S O N    &    H A N G E R        ENGINEERS                *
*                                                                      *
*                        TIME: 9/28/98 10:48:02         PAGE:    1     *
************************************************************************


************************************
* SUMMARY OF THE INPUT INFORMATION *
************************************

TYPE OF THE PROBLEM : CONTINUOUS BEAM CALCULATION

* Types and the locations of the supports are:

   Fixed support at X=     0.0000
   Fixed support at X=   294.0000

* Total number of different materials     :     1

  From X=      0.0000    to X=   294.0000   E= 3605000.0000

* Total number of different sections      :     1

  From X=      0.0000 to X=   294.0000 IX=  6358.0000

* Total number of distributed loads          :     1
  At X=      0.0000  W1=   171.0000   At X=   294.0000  W2=     24.0000


**************************
* SUMMARY OF THE RESULTS *
**************************

* MAXIMUM VALUES

  * Max. displacement is    0.08276  at  X=   147.0000
  * Min. displacement is    0.00000  at  X=     0.0000

  * Max. shear force is 18654.30000  at  X=     0.0000
  * Min. shear force is -10010.70000  at  X=   294.0000

  * Max. moment is 808176.60000  at  X=     0.0000
  * Min. moment is -351146.25000  at  X=   147.0000

* TOTAL APPLIED LOADS

  * Total applied concentrated load:      0.0000
  * Total applied concentrated moment:     0.0000
  * Total applied distributed load: 28665.0000

* SUPPORT REACTIONS

  * Reaction at X=     0.0000 : Force= -18654.3000 Moment= -808176.6000
  * Reaction at X=   294.0000 : Force= -10010.7000 Moment= 596408.4000
```

```
****************************
* DETAILS OF THE ANALYSIS *
****************************
```

* DISPLACEMENT AT OUTPUT POINTS :

```
At X=     0.0000 Displacement=    0.00000
At X=    73.5000 Displacement=    0.05006
At X=   147.0000 Displacement=    0.08276
At X=   220.5000 Displacement=    0.04304
At X=   294.0000 Displacement=    0.00000
```

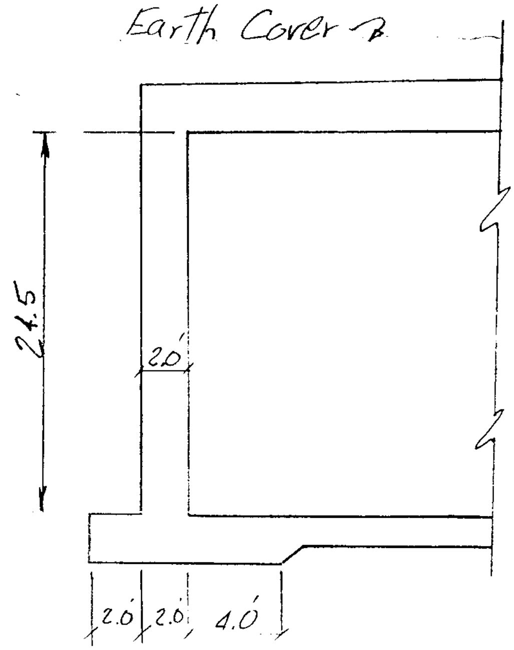
* SHEAR FORCES :

Coordinate		Shear (LT side)	Shear (RT side)
X=	0.0000		18654.3000
X=	73.5000	7436.3625	7436.3625
X=	147.0000	-1080.4500	-1080.4500
X=	220.5000	-6896.1375	-6896.1375
X=	294.0000	-10010.7000	

* MOMENTS :

Coordinate		Moment (LT side)	Moment (RT side)
X=	0.0000		808176.6000
X=	73.5000	-134110.8563	-134110.8563
X=	147.0000	-351146.2500	-351146.2500
X=	220.5000	-41462.2687	-41462.2687
X=	294.0000	596408.4000	

Check Foundation Slab for Vertical Loads



Use ½ of Roof Slab to be Supported by Ext Walls.

$$W_{DL} = \left(\frac{39.5}{2} + 2\right)\left(2 \times [120 + 150]\right) + 24.5 \times 2 \times 150 + 8 \times 2 \times 150$$

$$W_{DL} = 21{,}495 \text{ lbs}/8 = 2687 \text{ psf}$$

Allowable Bearing Pressure = 4000 psf

4000 > 2687 OK

From TM5-1300, 4-66.2.4 "Floor Slabs" pg 4-227

"Slabs poured on grade usually do not require .... shear reinforcement."

"Soil Strata having enough bearing capacity to support the dead load of the structure can be considered to provide the support required by the slab."

$$d = 24 - 3 - \frac{1.27}{2} = 20.365''$$

Check Shear Stress on Wide Side of Ftg

$$V_u = 1.4 \times 2687 \times \left(4 - \frac{20.365}{12}\right) = 8663 \text{ lbs/ft.}$$

$$v_u = \frac{V_u}{\phi b d} = \frac{8663}{.85(12)(20.365)} = 41.7 \text{ psi}$$

$$v_c = 2\sqrt{4000} = 126 \text{ psi} > 41.7 \text{ psi} \quad OK$$

SEISMIC CALC.

REF. NAVFAC P-355

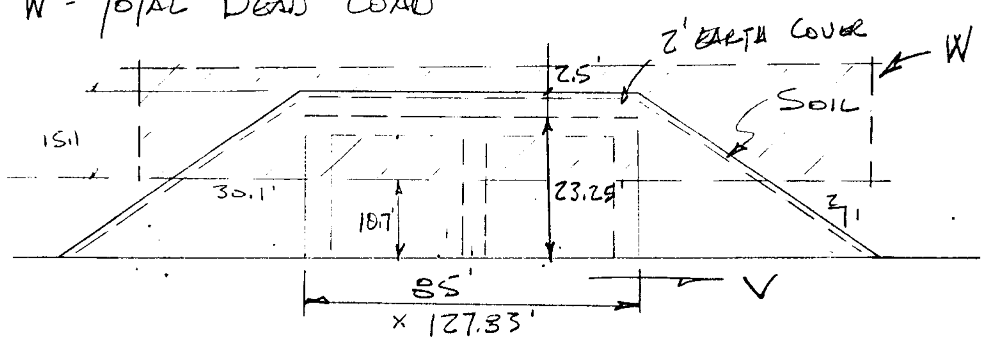
SEISMIC ZONE 3 PER FIG 3-1 REF.
SOIL PROFILE S_2, SITE FACTOR 1.2 PER GEOTECH. INVEST. BY DAMES + MOORE, MAY '94

BASE SHEAR, $V = ZIKCSW = 0.75 \times 1.5 \times 1.33 \times .14 \times W = \underline{.21\,W}$

WHERE: $Z = 0.75$ TABLE 3-1 REF.
$I = 1.5$ ESSENTIAL FAC. TABLE 3-2
$K = 1.33$ BOX SYSTEM TABLE 3-3
$S = 1.2$ SEE ABOVE
$C = 1/15\sqrt{T}$, $T = \dfrac{0.05\,h_n}{\sqrt{D}} = \dfrac{0.05 \times 23'}{\sqrt{85'}} = .12$

$CS = 1.2 \times .12 = .14$

W = TOTAL DEAD LOAD



ROOF:	$1.92 \times 85' \times 125.9 \times .15 =$	3,082.0
SIDE WALL:	$1.92 \times 10.7 \times 124.0 \times 2 \times .15 =$	764.2
MID WALL:	$2' \times 6.33' \times 124.0 \times .15 =$	235.5
"	$+ 1.5 \times 4.37' \times 70.0' \times .15 =$	68.8
REAR W.:	$1.92 \times 10.7' \times 85 \times .15 =$	261.9
FRONT W:	" " " $\times .15 =$	261.9
	$1.92 \times 2' \times 85' \times .15 =$	49.0
		4,723.3^K +

CONTINUE NEXT SHT.

S-29

WING WALL: $1.0 \times \frac{15.1' \times 30.1}{2} \times 2 \times .15 = 68.2$

$1.0 \times 1.55' \times \frac{15.1' \times 30.1'}{2} \times 2 \times .15 = 10.5$

SOIL ON ROOF $2' \times .120 \times 85' \times 125.9 = 2,568.4$

ON SLOPE $.120 \times \frac{14.6 \times 29.2}{2} \times 125.9 = 330.9$

$2978^K + 4723.3$ ✓ FROM SHT 1

$W = 7,701.3^K$ ✓

BASE SHEAR, $V = .21 \times 7,701.3 = 1,617.3^K$ ✓ SHORT DIRECTION

SHEAR CAPACITY OF CONC. WALLS:

$V_c = 2\sqrt{f'_c}\, b\, d = 2\sqrt{4000} \times 23 \times d \times .85 / 1000$

$V_c = 2.47^K d$

TOTAL LOAD

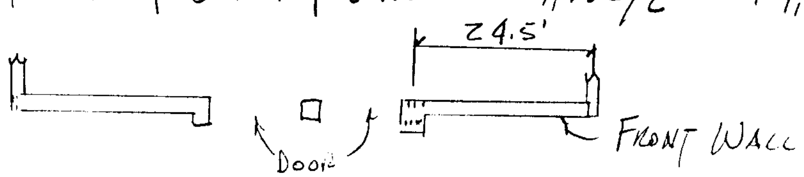
$\text{MIN. } d = \frac{1,617.3^K}{2.47^K/1''} = 654.8'' \text{ OR } 54.6'$ ✓ FOR TOTAL SHEAR

EA. WALL CAN RESIST TOTAL LOAD → { FRONT WALL + WING WALLS: $27' \times 2 + 51.5' \times 2 = 157' >> 54.6'$ ✓
REAR WALL ": $85' >> 54.6'$ ✓ }

SHEAR OK IN SHORT DIRECTION, LONG DIRECTION OK BY INSPECTION ✓

MOMENT: $1,617.3^K \times (23.25' + .96') = 39,152^{'-K}$

SAY TOT. MOMENT @ FRONT WALL $39,152/2 = 19,576^{'-K}$ ✓



24.5' T

$T = \frac{19,576^{'-K}}{24.5} = 799.^K$ ✓

$T = 799^K$, $\frac{799^K}{20\ KSI} = 39.95\ IN^2$ REQ'D

20 KSI ← ALLOW. TENSILE STRESS IN 60KSI REBAR

@ DOOR 14-#11 , $A_S = 5.313 \times 14 = 74.4\ IN^2 >> 39.95\ IN^2$

@ WALL #10 @ 8" EA. FACE OR $\frac{39.95\ IN^2}{4.303\ IN^2/BAR} = 9.3$ BARS OK

MOMENT OK IN SHORT DIRECTION, LONG DIRECTION OK BY INSPECTION

$\boxed{\text{WALLS MORE THAN ADEQUATE TO RESIST SEISMIC LOADS IN ZONE 3. - BOTH DIRECTION}}$

CENTER

WALL

Mason & Hanger Engineering, Inc.
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Project P-501 ADCAP Magazines Location NWS Yorktown, VA
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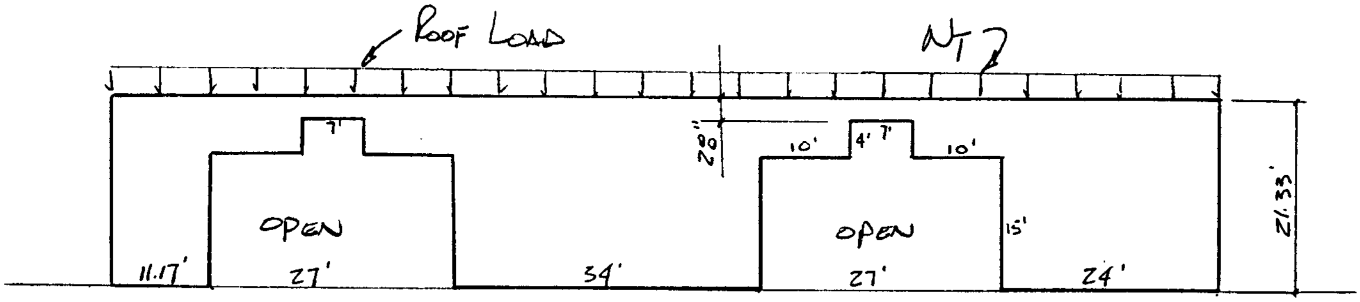
CENTER WALL - ROOF SUPPORT



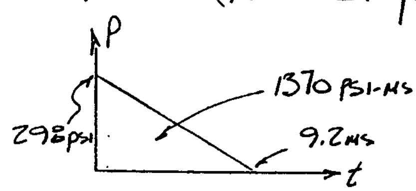
ROOF LOAD TO WALL =

DEAD:

SLAB	$.150 \times 23/12 =$	.288
SOIL	$2' \times .120$ =	.240
GRAVEL (6")	$.5 \times .120$ =	.060
M&E		.010
		.598 KSF D.L.
LIVE:		.200 KSF (PER CRITERIA)

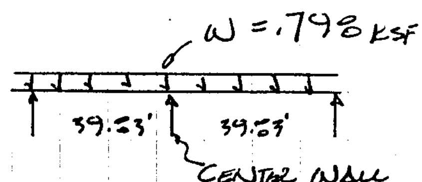
} .798 KSF

BLAST LOAD (PER CRITERIA)



DEAD + LIVE

$.200 + .598 = .798 \text{ KSF}$



LOAD TO CENTER WALL $= .798 \times 1.125 \times 39.83'$
$= 35.76$ KLF $= W_T$ (UNFACTORED)

ULT. $W_{uT} = (.20 \times 1.7 + .598 \times 1.4) \times 1.125 \times 39.83' = 52.75$ KLF (FACTORED)

FACTORED DL $= .20 \times 1.7 \times 1.125 \times 39.83' = 15.2$ KLF

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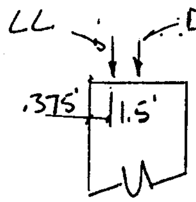
MATERIALS - $f'_c = 4000\,psi$ REFERENCES
$f_y = 60{,}000\,psi$ ACI 318

THICKNESS - TRY $1.5' > 1/25 \times Ht = .9'$, OK ($Ht$ = 21.3') SECT 14.5.3

RESULTANT OF VERT. LOAD - FULL LOAD @ MIDDLE OF WALL
- FULL DEAD + LIVE LOAD ON 1 SPAN



LIVE $.20 \times 1.7 \times .375' = .128^{\prime\text{-}k}$
DEAD $\underset{1.177^k}{.598 \times 1.4} \times .75' = \underline{.628}$
$.756^{\prime\text{-}k}$

RESULTANT: $\dfrac{.756^{\prime k}}{1.177^k} = .642'$ OK

RESULTANT WITHIN MID 1/3

DESIGN CAP. (EMPIRICAL METHOD) ACI 318 SECT. 14.5.2

$$\phi P_{nw} = 0.55\,\phi f'_c A_g\left[1 - \left(\frac{k\,l_c}{32h}\right)^2\right],$$

$l_c = 21.33' \times 12 = 256"$
$h = 18"$
$k = 0.8$ (FIXED ENDS)
$\phi = 0.7$
$A_g = 12" \times 18" = 216\,in^2$

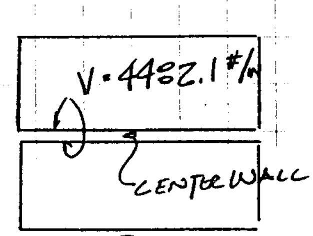
$$0.55 \times 0.7 \times 4{,}000 \times 216\left[1 - \left(\frac{0.8 \times 256}{32 \times 18}\right)^2\right] =$$

$\phi P_{nw} = 290.6$ kip/f $> \{ 52.8^{k/\prime}$ (SHT 1) OK
(+ WALL WT, $.15 \times 1.5' \times 1.4 \times 21.33 = 6.7^{k/\prime}$

L + D LOADING OK

CHECK BLAST LOADING:

FROM ROOF DESIGN - CBARC OUTPUT:



$$P_{TO\ WALL} = \frac{4482.1^{\#/"} \times 12" \times 2}{1000} = 107.6^{k/f}$$

BLAST + DEAD $= 107.6^{k/f} + 15.2\,klf = 122.8\,klf < 290.6\,klf$ OK

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REBAR (MIN.)

REF. ACI 318
SECT. 14.3

VERT. $A_s = .0012 \times 18 \times 12 = .26 in^2$
HORIZ. " $= .0020 \times 18 \times 12 = .43 in^2$

$\boxed{\text{USE 18" THK W/ \#5@12" EWEF}}$ CENTER WALL

CHECK WALL EACH SIDE OF OPENING

MOMT SHER DISP VIEW REPT RETN

Moment diagram

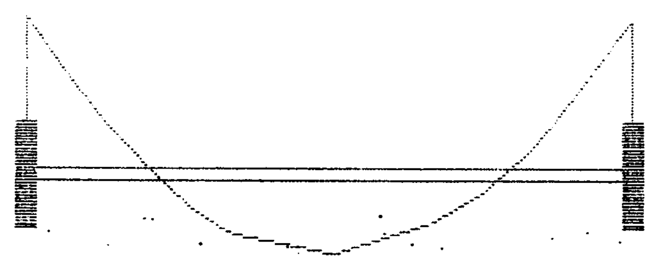
SUPPORT REACTIONS:

AT X= 0.0000 FORCE= -48409.526250 MOMENT= -2197638.5050

AT X= 256.00 FORCE= -49242.485050 MOMENT= 2140433.25

MAXIMUM MOMENT = 2197638.5000 AT X= 0.0000

MINIMUM MOMENT = -1093061.6250 AT X= 128.00



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*        CAST-UTILITY  BY  CAST INC.                                   *
*                                                                      *
*        M A S O N    &    H A N G E R        ENGINEERS               *
*                                                                      *
*                     TIME: 7/17/92 9:51:09          PAGE:    1        *
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```

```
************************************
* SUMMARY OF THE INPUT INFORMATION *
************************************

TYPE OF THE PROBLEM : CONTINUOUS BEAM CALCULATION

* Types and the locations of the supports are:

   Fixed support at X=     0.0000
   Fixed support at X=   256.0000

* Total number of different materials    :     1

  From X=      0.0000    to X=    256.0000    E= 3605000.0000

* Total number of different sections     :     1

  From X=      0.0000 to X=    256.0000 IX= 316982.0000

* Total number of distributed loads             :     3
  At X=      0.0000  W1=    284.6000   At X=     64.0000  W2=    284.6000
  At X=     64.0000  W1=    687.0000   At X=     88.0000  W2=    687.0000
  At X=     88.0000  W1=    374.7000   At X=    256.0000  W2=    374.7000

**************************
* SUMMARY OF THE RESULTS *
**************************

* MAXIMUM VALUES

  * Max. displacement is    0.00395  at  X=   128.0000
  * Min. displacement is    0.00000  at  X=     0.0000

  * Max. shear force is 48409.52035  at  X=     0.0000
  * Min. shear force is -49242.47965  at  X=   256.0000

  * Max. moment is 2197638.57136  at  X=     0.0000
  * Min. moment is -1093061.63334  at  X=   128.0000

* TOTAL APPLIED LOADS

  * Total applied concentrated load:      0.0000
  * Total applied concentrated moment:      0.0000
  * Total applied distributed load: 97652.0000

* SUPPORT REACTIONS

  * Reaction at X=      0.0000 : Force= -48409.5203 Moment= -2197638.5714
  * Reaction at X=    256.0000 : Force= -49242.4797 Moment= 2140433.3620
```

```
****************************
* DETAILS OF THE ANALYSIS *
****************************
```

* DISPLACEMENT AT OUTPUT POINTS :

```
At X=     0.0000 Displacement=    0.00000
At X=    16.0000 Displacement=    0.00022
At X=    32.0000 Displacement=    0.00076
At X=    48.0000 Displacement=    0.00149
At X=    64.0000 Displacement=    0.00226
At X=    70.0000 Displacement=    0.00254
At X=    76.0000 Displacement=    0.00280
At X=    82.0000 Displacement=    0.00304
At X=    88.0000 Displacement=    0.00326
At X=   128.0000 Displacement=    0.00395
At X=   130.0000 Displacement=    0.00395
At X=   172.0000 Displacement=    0.00303
At X=   192.0000 Displacement=    0.00218
At X=   214.0000 Displacement=    0.00116
At X=   256.0000 Displacement=    0.00000
```

* SHEAR FORCES :

Coordinate	Shear (LT side)	Shear (RT side)
X= 0.0000		48409.5203
X= 16.0000	43855.9203	43855.9203
X= 32.0000	39302.3203	39302.3203
X= 48.0000	34748.7203	34748.7203
X= 64.0000	30195.1203	30195.1203
X= 70.0000	26073.1203	26073.1203
X= 76.0000	21951.1203	21951.1203
X= 82.0000	17829.1203	17829.1203
X= 88.0000	13707.1203	13707.1203
X= 128.0000	-1280.8797	-1280.8797
X= 130.0000	-2030.2797	-2030.2797
X= 172.0000	-17767.6797	-17767.6797
X= 192.0000	-25261.6797	-25261.6797
X= 214.0000	-33505.0797	-33505.0797
X= 256.0000	-49242.4797	

* MOMENTS :

Coordinate	Moment (LT side)	Moment (RT side)
X= 0.0000		2197638.5714
X= 16.0000	1459515.0458	1459515.0458
X= 32.0000	794249.1202	794249.1202
X= 48.0000	201840.7946	201840.7946
X= 64.0000	-317709.9310	-317709.9310
X= 70.0000	-486514.6531	-486514.6531

X=	76.0000	-630587.3752	-630587.3752
X=	82.0000	-749928.0973	-749928.0973
X=	88.0000	-844536.8194	-844536.8194
X=	128.0000	-1093061.6333	-1093061.6333
X=	130.0000	-1089750.4740	-1089750.4740
X=	172.0000	-673993.3287	-673993.3287
X=	192.0000	-243699.7357	-243699.7357
X=	214.0000	402734.6166	402734.6166
X=	256.0000	2140433.3620	

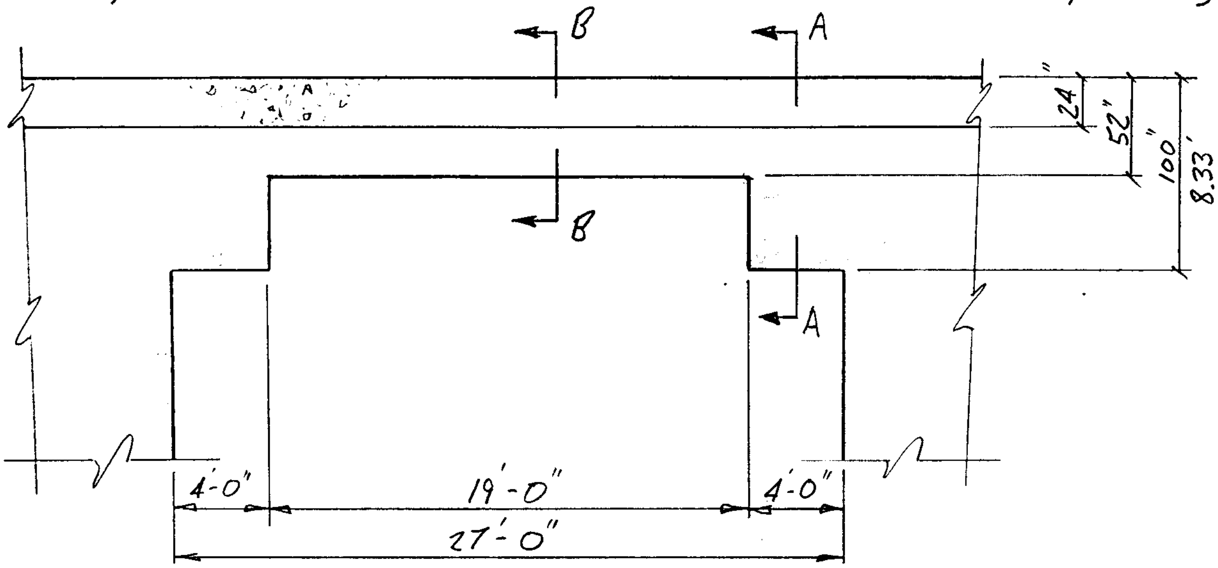
CENTER

WALL

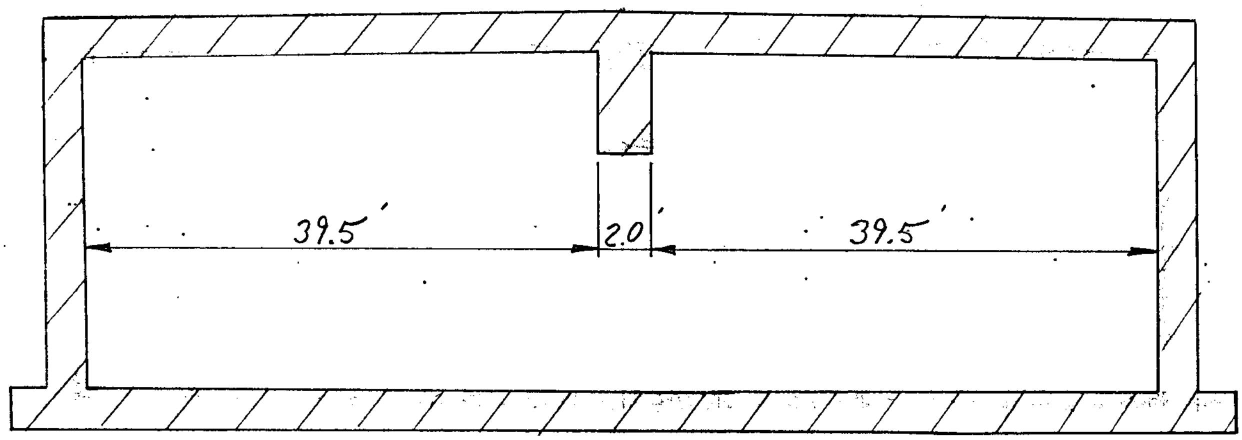
BEAM

Mason & Hanger Engineering, Inc. Lexington, Kentucky	
Project P-501 ADCAP Magazines Location NWS Yorktown, VA	File 2446 Page 1
	Sheet of Sheets
	Date 10/98
	Computed By TA
Subject Center Wall Beams	Checked By

Design Beam Sections Over Center Wall Openings



Elevation
Center Wall Opening



Check Center Beam for T-Beam Analysis

Check Allowable Width of Flange

$$\text{Width} \le \tfrac{1}{4} L = \tfrac{1}{4}(39.5) = 9.88$$

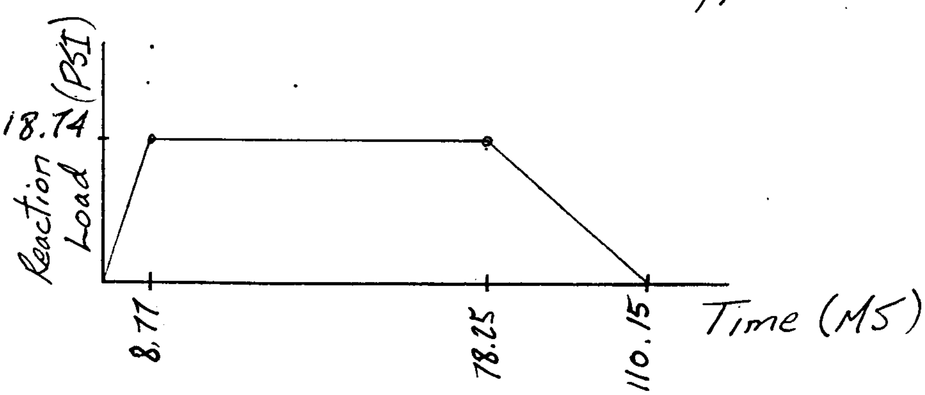
$$\le 4 W_w = 4(2.0) = 8.00 \Leftarrow \text{Use } 8.0 \text{ ft.}$$

$$\le 8 W_s + W_w = 8\left(\tfrac{24}{12}\right) + 2 = 18.00$$

Load Analysis for Beam Design

Design Beam to Carry DL + Dynamic Blast Loads

Use Output from Slab Analysis to determine Slab Reactions on Center Support Beam:



Blast Reactions, Per Square Inch of Roof Slab

Total Reaction on Center Support

$R = 18.74 \times \frac{39.83 \times 12}{2} \times 2 = 8957.0$ lbs/Inch of Beam

Uniform Load on Beam (from Slab) for Analysis

Using 8 ft Wide Beam, $W_S = \frac{8957}{8 \times 12} = 93.30$ psi

Direct Blast Load on Beam: $P = 298$ psi, $t = 9.2$ ms

Acts only on Width of Web

Remaining Blast Load is Included in Slab Reactions

Uniform Direct Blast Load on Beam:

$$W_B = \frac{298 \times 24}{8 \times 12} = 74.5 \text{ psi}$$

Project P-501 ADCAP Magazines Location NWS Yorktown, VA

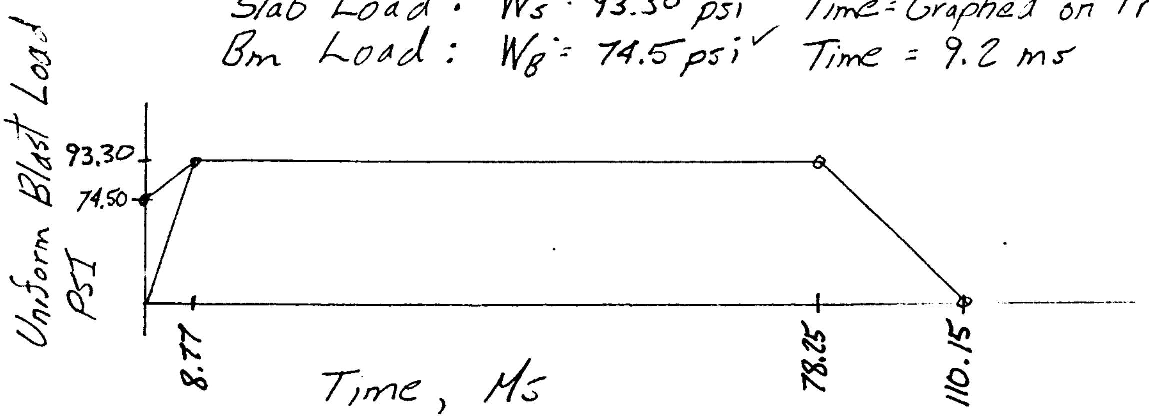
Subject Beam Design

Load Analysis

Combine Dynamic Loads on Beam

Slab Load : $W_S = 93.30$ psi ✓ Time = Graphed on Previous sht.

Bm Load : $W_B = 74.5$ psi ✓ Time = 9.2 ms



Total Dynamic Load Diagram

Calculate Equivalent Mass for Beam Sections

Beam Slab, $M_S = \frac{[24 \times 150 + 24 \times 120] \times 10^6}{12 \times 32.2 \times 1728} = 9{,}705$ ✓ lbs-ms²/in² (6480)

Add 20% of Mass from Remaining Area of Supported Slab & Earth Cover per P-397, Section 4-43.1

Roof Slab, $M_R = \frac{[24 \times 150 + 24 \times 120] \times \left(\frac{478}{2} - 36\right) \times 2 \times 10^6}{12 \times 32.2 \times 1728} \times 0.2 = 788{,}043$ ✓ lbs-ms²/In² (6330; 203)

A) Center Beam, $M_C = \frac{[52 \times 150 + 24 \times 120] \times 10^6}{12 \times 32.2 \times 1728} = 15{,}995$ ✓ lbs-ms²/In²

B) Deepened Beam, $M_D = \frac{[100 \times 150 + 24 \times 120] \times 10^6}{12 \times 32.2 \times 1728} = 26{,}779$ ✓ lbs-ms²/In²

Equivalent Mass of Beams

A) $M_E = K_{LM} \times (M_S + M_R + M_C) = .72 \left(\frac{9{,}705 \times 72 + 788{,}043 + 15{,}995 \times 24}{96} \right)$
$= 14{,}030$ ✓ lbs-ms/In²

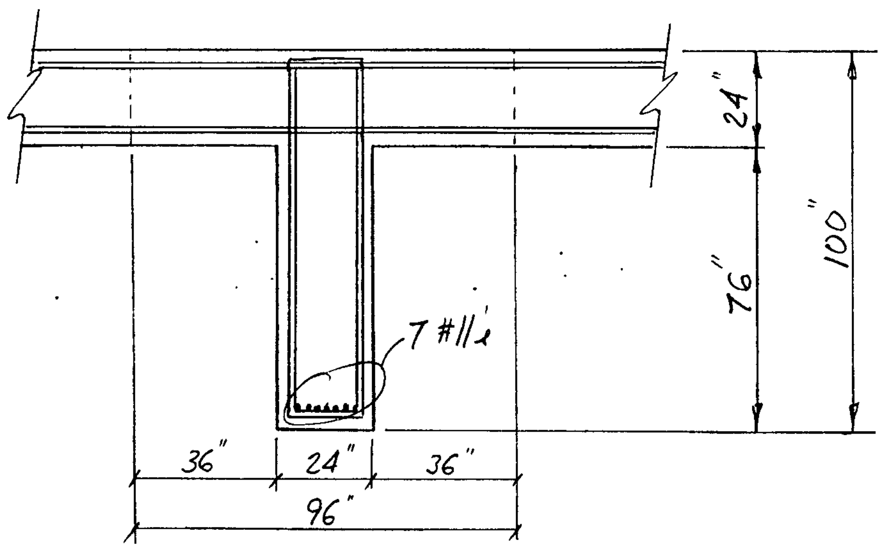
B) $M_E = K_{LM} \times (M_S + M_R + M_D) = .66 \left(\frac{9705 \times 72 + 788{,}043 + 26{,}779 \times 24}{96} \right)$
$= 14{,}640$ lbs-ms/In²

4'-0" × 100"

Center Wall

Beam

Design Center Wall Beams (Section A-A)



Section A-A

Check as T-Beam

Check location of Neutral Axis

Using 7 #11's in Bottom of Web: $d = 100 - 1\frac{1}{2} - \frac{1}{2} - \frac{1.41}{2} = 97.3''$

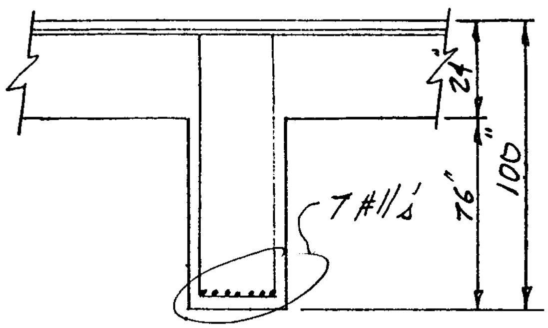
$\rho = \frac{A_s}{bd} = \frac{10.92}{96 \times 97.3} = .0012$ $\quad A_s = 7 \times 1.56\,in^2 = 10.92\,in^2$

$a = \frac{\rho f_{dy} d}{.85 f'_{dc}} = \frac{.0012(77,220)(97.3)}{.85(4760)} = 2.23'' <$ Flange Thickness of 24"

Beam can be designed as T-Beam

Analyze Beam as full depth section, over entire span and Calculate Deflections - Add to Additional Deflection produced by Center Bm (52") Section

Beam Analysis (Cont'd)



Top of Beam
Use #10 @ 8" (Each Direction)
(Std Slab Reinf)
Bottom of Beam
7 #11's
#4 Stirrups @ 8"

for CBARCS Clearances

Vert	Top of Beam, Parallel, $2 + 1.27 + \frac{1}{2}(1.27) = 3.905$	$A_s = 1.27 \times \frac{12}{8} = 1.91\ in^2$
Horiz	Transverse, $2 + \frac{1}{2}(1.27) = 2.635$	$A_s = 1.27 \times \frac{12}{8} = 1.91\ in^2$
Vert	Bott of Beam, Parallel, $1\frac{1}{2} + \frac{1}{2} + \frac{1}{2}(1.41) = 2.705$	$A_s = 10.92 \times \frac{1}{8} = 1.37\ in^2$
Horiz	Transverse, $1\frac{1}{2} + \frac{1}{2}(.5) = 1.75$	$= .5 \times \frac{12}{8} = 0.75\ in^2$

Run CBARCS to determine Initial Design Parameters from CBARCS: Roof2.in Output:

$r_u = 153.22$

$K_E = 5,320$

Reduce Ultimate Resistance of Beam, r_u, by the dead loads from concrete and earth cover to determine $r_{available}$ for Blast Load Resistance.

$r_{avail} = r_u - r_{DL}\left(\frac{f_{ds}}{f_y}\right)$ For $\Delta_x \le 2^\circ$ $f_{dy} = 1.17 f_y$ P-397 Table 4.1

$f_{ds} = f_{dy}$ Table 4.2

$$W_{DL\ from\ Slab} = \frac{\overset{540}{\left[\frac{24}{12} \times 150 + 2 \times 120\right]}}{144} = 3.75\ lbs/In^2$$

$$W_{DL\ from\ Beam} = \frac{\overset{1477.5}{\left[\frac{99}{12} \times 150 + 2 \times 120\right]}}{144} = 10.26\ lbs/In^2$$

$$r_{DL} = \frac{3.75 \times 478 + 10.26 \times 24}{96} = 21.24\ psi\ \text{on Beam}$$

$$r_{avail} = 153.22 - 21.24\left[\frac{1.17(66,000)}{66,000}\right]\ 128.37\ psi$$

Check Direct Shear Capacity for 100" Deep Beam

Max Support Shear

52" Deep Beam, $V_u = 153.51 \times 96 \times \frac{19 \times 12}{2} = 1,680,013$ lbs

100" Deep Beam, $V_u = 153.22 \times 96 \times 48 = 706,038$ lbs

Total $V_u = 1,680,013 + 706,038 = 2,386,051$ lbs.

Concrete Capacity, $V_c = 0.18 f'_{dc} b d$

For Flange, $V_F = .18(4400)(72)(21.865) = 1,246,830$ lbs

For Web, $V_W = .18(4400)(24)(97.295) = 1,849,383$

Total $V_c = 1,246,830 + 1,849,383 = 3,096,213$ lbs

Since $V_c = 3,096,213 < V_u = 2,386,051$ lbs O.K

Diagonal Shear Analysis

Since "d" distance is greater than length of deepened section, diagonal shear analysis is not required for thickened section of beam.

Diagonal shear Analysis of 52" Deep Beam will control

- See Attached Analysis. (Pg 21)

Root 2.in

CBARCS Program

```
100  0,0,0,0,1.17
110  P501 MAGAZINE - 24" x 100" x 27' ROOF BEAM
120  0,1,0,0,0,0
130  350000,1,0,0,0,0,0,0
140  8735.3,27,8,88.53,116.4,0,0,0,0,0,0
150  4760,66000,100,2,6,0,0,0
160  1.91,1.37,1.91,0.75,3.905,2.70,2.635,1.75
```

27 ft long   100" Deep Beam

Fixed Each End

$M_E = 16,176$

$K_E = 5,319.81$

$r_u = 153.22$

```
P501 MAGAZINE - 24" x 100" x 27' ROOF BEAM

        BLAST WALL HEIGHT          27.00 FT
        BLAST WALL LENGTH           8.00  FT
        DURATION OF LOAD               116.40000 MSEC
        FICTITIOUS PEAK PRESSURE        88.53000 PSI
        EFFECTIVE IMPULSE                8735.30 PSI MS



      HEIGHT     324.00 IN    LENGTH       96.00 IN

     DYNAMIC CONCRETE STRENGTH      4760.00
     DYNAMIC STEEL STRESS          77220.00
     THICKNESS CONCRETE   INCHES   100.0000
     THICKNESS OF SAND    INCHES      .0000
     THETA ALLOWABLE     DEGREES     2.0000

     AREA VERT TOP STEEL/FT          1.9100   COVER     3.9050
     AREA VERT BOT STEEL/FT          1.3700   COVER     2.7000
     AREA HORIZ TOP STEEL/FT         1.9100   COVER     2.6350
     AREA HORIZ BOT STEEL/FT          .7500   COVER     1.7500

          TYPE 1 CONSTRUCTION

     CONCRETE MODULUS PSI                 3555611.
     RATIO MOD STEEL/CONCRETE                8.16
     GROSS MOMENT INERTIA                 83333.33
     AVE CRACKED MOM INERTIA               8596.89
     AVE MOMENT INERTIA                   45965.11
     AVERAGE PERCENT STEEL                   .0014
     D FACTOR MU=1/6                 168107423499.
     D FACTOR  MU= 0.3               179597833468.

     ALLOW SHEAR UNREINFORCED WEB    102.67   PSI         9865.74 LBS/IN
WIDTH
     ALLOW SHEAR AT SUPPORT          753.98   PSI        72454.10 LBS/IN
WIDTH
     UNREINFORCED CONCRETE  THETA LE 2 DEG

     POSITIVE VERTICAL MOMENT      848187.27
     NEGATIVE VERTICAL MOMENT     1162420.79
     POSITIVE HORIZONTAL MOMENT    471300.58
     NEGATIVE HORIZONTAL MOMENT   1178030.17
     FIXED  END  BEAM

     LOCATION YIELD LINE LENGTH                           .00
     LOCATION YIELD LINE HEIGHT                        162.00
     ULTIMATE LOAD CAPACITY  RU                      153.2242
     SHEAR LOAD AT VERTICAL SUPPORT                       .00 LB/IN WIDTH
     SHEAR LOAD AT HORIZONTAL SUPPORT                24822.32 LB/IN WIDTH
```

SHEAR AT DISTANCE FROM VERTICAL SUPPORT	.00 PSI
SHEAR AT DISTANCE FROM HORIZONTAL SUPPORT	105.09 PSI
ALLOWABLE MAX DEFLECTION	5.6666

LOAD MASS FACTOR	.7200
MASS CONCRETE ONLY	16176.00

SHEAR CAPACITY(VC) EXCEEDED

ELASTIC LIMIT RE PSI	132.88
ELASTIC DEFLECTION XE	.0233
ULTIMATE RESISTANCE	153.22
PLASTIC DEFLECTION	.0412

ULTIMATE RESISTANCE RU	153.22
ELASTIC DEFLECTION LIMIT XE	.0288
STIFFNESS KE	5319.81

MASS	16176.001
LOAD	88.530
DURATION	116.400
RESISTANCE	153.224
STIFFNESS	5319.812

GAS PRESSURE	.00	DURATION	.00
NATURAL PERIOD			10.956377
MAXIMUM DEFLECTION			.032770
TIME TO MAXIMUM DEFLECTION			5.631161
DURATION/NATURAL PERIOD			10.623950
LOAD/RESISTANCE			.577781
ELASTIC DEFLECTION LIMIT			.028803

MAX FRAGMENT SPALL VELOCITY FT/SEC	.778161

SOLVER Program
Roof 2


```
SOLVE    ROOF BEAM: ANALYSIS OF 24" X 100" X 27' LONG HEADER BEAM
    1     1          .1          200                                    -1

    2     2      14640        0.01
    1    5319.81     128.37                        1.0        1.0
    3       0.0                        20          1.0        0.0
    1    5319.81    -153.22                        1.0        1.0
    3       0.0                       -20          1.0        0.0
    1     5
       0.00       74.50       8.77       93.30      78.25      93.30     110.15
0.0
        300        0.0
STOP
```

Use Equivalent M_E from Page 3 ⇒ $M_E = 14640$

Use r_{avail} from Page 6 ⇒ $r_{avail} = 128.37$

Use Dynamic Loading from Graph on Page 3

SOLVER Output
Roof 2. out Page 12

```
1ONE DEGREE OF FREEDOM SOLVER INPUT
 VERSION 2.2   FEB 1989

 PROBLEM DESCRIPTION
 SOLVE    ROOF BEAM: ANALYSIS OF 24" X 100" X 27' LONG HEADER BEAM


 ANALYSIS CONTROL CARD

 TYPE OF SOLUTION.......................       1
   EQ. 0,   DEFAULTS TO 1
   EQ. 1,   NEWMARK-BETA METHOD
   EQ. 2,   WILSON-THETA METHOD
 NUMBER OF LOAD CASES...................       1
 TIME STEP..............................     0.1000
 TIME LIMIT.............................   200.0000
 NEWMARK*S GAMMA........................     0.0000
   EQ. 0.0, DEFAULTS TO 0.5
 NEWMARK*S BETA.........................     0.0000
   EQ. 0.0, DEFAULTS TO 0.25
 WILSON*S THETA.........................     0.0000
   EQ. 0.0, DEFAULTS TO 1.4
 PRINT OPTION...........................      -1
   EQ. 0, DEFAULTS TO 1
   EQ. 1, PRINT EVERY STEP
   EQ. N, PRINT EVERY N-TH STEP
   EQ. -1, PRINT SIGNIFICANT CHANGES


 INITIAL CONDITIONS

 INITIAL DEFLECTION.....................     0.0000
 INITIAL VELOCITY.......................     0.0000
 INITIAL ACCELERATION...................     0.0000


 STIFFNESS CONTROL CARD

 NUMBER OF STIFFNESSES..................       2
 NUMBER OF REBOUND STIFFNESSES..........       2
 MASS...................................  14640.0000
 PERCENT OF CRITICAL DAMPING............      0.0100


 INITIAL RESISTANCE-DEFLECTION CURVE

 NUMBER  MODE    STIFFNESS  RESISTANCE    YLD DEFL     MASS FACT    DAMP
FRAC

      1       1 0.53198E+04 0.12837E+03 0.00000E+00          1.00
1.00
      2       3 0.00000E+00 0.00000E+00 0.20000E+02          1.00
0.00


 REBOUND RESISTANCE-DEFLECTION CURVE

 NUMBER  MODE    STIFFNESS  RESISTANCE    YLD DEFL     MASS FACT    DAMP
FRAC

      3       1 0.53198E+04-0.15322E+03 0.00000E+00          1.00
```

```
1.00
     4      3 0.00000E+00 0.00000E+00-0.20000E+02        1.00
0.00


 LOAD DATA

 LOAD CASE NUMBER......................     1
 NUMBER OF LOAD POINTS.................     5


 POINT              TIME            LOAD
     1            0.0000         74.5000
     2            8.7700         93.3000
     3           78.2500         93.3000
     4          110.1500          0.0000
     5          300.0000          0.0000


 GENERATED RESISTANCE - DEFLECTION CURVES

 NATURAL PERIOD........................    10.4232

 NUMBER  MODE   STIFFNESS  RESISTANCE   YLD DEFL   MASS FACT   DAMP
FRAC
     1      1 0.53198E+04 0.12837E+03 0.24131E-01        1.00
1.00
     2      3 0.00000E+00 0.12837E+03 0.20000E+02        1.00
0.00
     3      1 0.53198E+04-0.15322E+03-0.28802E-01        1.00
1.00
     4      3 0.00000E+00-0.15322E+03-0.20000E+02        1.00
0.00
1* * * * NEWMARK-BETA SOLUTION * * * * *
 LOAD CASE.............................     1
 TIME STEP.............................     0.1000


 * * * * * SOLUTION RESULTS  * * * * * *
         STEP   TIME STIF DEFLECTION   VELOCITY       ACC RESISTANCE
LOAD

 ELAS     0    0.0000    1      0.000     0.0000     0.0051      0.00
74.50
 *CHG    39    3.8292    2      0.024     0.0068    -0.0032    128.39
82.71
 REB     62    6.2000    1      0.032    -0.0002    -0.0028    128.35
87.79
 RLOD   112   11.2000    1      0.018     0.0001     0.0025     57.05
93.30
 *CHG   164   16.3060    1      0.032     0.0002    -0.0024    128.39
93.30
 REB    164   16.4000    1      0.032     0.0000    -0.0024    128.43
93.30
 RLOD   216   21.6000    1      0.019     0.0000     0.0023     59.25
93.30
 REB    269   26.9000    1      0.031    -0.0002    -0.0022    126.25
93.30
 RLOD   321   32.1000    1      0.019     0.0002     0.0022     61.36
93.30
 REB    373   37.3000    1      0.031    -0.0001    -0.0021    124.27
```

93.30

RLOD	425	42.5000	1	0.020	0.0001	0.0020	63.28	93.30
REB	477	47.7000	1	0.031	-0.0001	-0.0020	122.39	93.30
RLOD	529	52.9000	1	0.020	0.0000	0.0019	65.10	93.30
REB	581	58.1000	1	0.030	0.0000	-0.0019	120.63	93.30
RLOD	634	63.4000	1	0.020	0.0002	0.0018	66.86	93.30
REB	686	68.6000	1	0.030	-0.0001	-0.0017	118.94	93.30
RLOD	738	73.8000	1	0.020	0.0001	0.0017	68.45	93.30
REB	790	79.0000	1	0.030	-0.0001	-0.0018	117.32	91.11
RLOD	848	84.8000	1	0.017	0.0001	0.0017	49.29	74.14
REB	894	89.4000	1	0.024	-0.0001	-0.0017	85.25	60.69
RLOD	953	95.3000	1	0.011	0.0001	0.0016	20.35	43.43
REB	998	99.8000	1	0.018	-0.0001	-0.0016	53.28	30.27
*CHG	1042	104.1757	3	0.008	-0.0022	0.0012	-0.01	17.47
RLOD	1057	105.7000	3	0.006	0.0001	0.0015	-8.75	13.02
*CHG	1073	107.2650	1	0.008	0.0018	0.0006	0.00	8.44
REB	1102	110.2000	1	0.012	-0.0001	-0.0015	21.41	0.00
*CHG	1128	112.7607	3	0.008	-0.0024	0.0000	0.00	0.00
RLOD	1154	115.4000	3	0.004	0.0001	0.0014	-20.75	0.00
*CHG	1180	117.9742	1	0.008	0.0023	0.0000	0.00	0.00
REB	1206	120.6000	1	0.011	0.0000	-0.0014	20.12	0.00
*CHG	1232	123.1876	3	0.008	-0.0022	0.0000	0.00	0.00
RLOD	1258	125.8000	3	0.004	0.0000	0.0013	-19.50	0.00
*CHG	1285	128.4010	1	0.008	0.0022	0.0000	0.00	0.00
REB	1310	131.0000	1	0.011	0.0000	-0.0013	18.90	0.00
*CHG	1337	133.6145	3	0.008	-0.0021	0.0000	0.00	0.00
RLOD	1363	136.3000	3	0.004	0.0001	0.0012	-18.28	0.00
*CHG	1389	138.8279	1	0.008	0.0020	0.0000	0.00	0.00
REB	1415	141.5000	1	0.011	-0.0001	-0.0012	17.72	0.00
*CHG	1441	144.0414	3	0.008	-0.0020	0.0000	0.00	0.00
RLOD	1467	146.7000	3	0.004	0.0001	0.0012	-17.18	0.00

*CHG	1493	149.2548	1	0.008	0.0019	0.0000	0.00	0.00
REB	1519	151.9000	1	0.011	-0.0001	-0.0011	16.66	0.00
*CHG	1545	154.4682	3	0.008	-0.0019	0.0000	0.00	0.00
RLOD	1571	157.1000	3	0.005	0.0000	0.0011	-16.15	0.00
*CHG	1597	159.6817	1	0.008	0.0018	0.0000	0.00	0.00
REB	1623	162.3000	1	0.011	0.0000	-0.0011	15.65	0.00
*CHG	1649	164.8951	3	0.008	-0.0017	0.0000	0.00	0.00
RLOD	1675	167.5000	3	0.005	0.0000	0.0010	-15.17	0.00
*CHG	1702	170.1086	1	0.008	0.0017	0.0000	0.00	0.00
REB	1727	172.7000	1	0.010	0.0000	-0.0010	14.70	0.00
*CHG	1754	175.3220	3	0.008	-0.0016	0.0000	0.00	0.00
RLOD	1780	178.0000	3	0.005	0.0001	0.0010	-14.22	0.00
*CHG	1806	180.5354	1	0.008	0.0016	0.0000	0.00	0.00
REB	1832	183.2000	1	0.010	-0.0001	-0.0009	13.79	0.00
*CHG	1858	185.7489	3	0.008	-0.0015	0.0000	0.00	0.00
RLOD	1884	188.4000	3	0.005	0.0001	0.0009	-13.37	0.00
*CHG	1910	190.9623	1	0.008	0.0015	0.0000	0.00	0.00
REB	1936	193.6000	1	0.010	0.0000	-0.0009	12.96	0.00
*CHG	1962	196.1757	3	0.008	-0.0014	0.0000	0.00	0.00
RLOD	1988	198.8000	3	0.005	0.0000	0.0009	-12.56	0.00

```
 ***** TIME LIMIT REACHED

 MAXIMUM DEFLECTION IN SOLUTION
 STEP NUMBER..........................   61
 TIME AT MAXIMUM......................       6.1000
 MAXIMUM DEFLECTION...................       0.032

 MINIMUM DEFLECTION IN SOLUTION
 STEP NUMBER.......................... 1153
 TIME AT MINIMUM......................     115.3000
 MINIMUM DEFLECTION...................       0.004


SOLUTION TIME LOG
   ASSEMBLY......................          0.17
   SDOF SOLUTION.(LOAD CASE  1)...         0.16

   TOTAL TIME....................          0.33
1ONE DEGREE OF FREEDOM SOLVER INPUT
```

VERSION 2.2 FEB 1989

PROBLEM DESCRIPTION
STOP

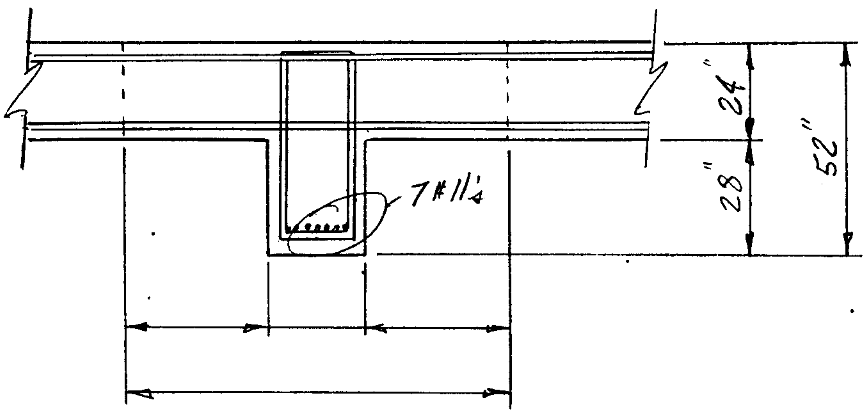
***** END SOLVER RUNS *****

19'-0" × 52"

Center Wall

Beam

Design Center Wall Beam (Section B-B)



Section B-B

Check as T-Beam

Check location of Neutral Axis

Using 7 # 11's in Bott of Web $\quad d = 52 - 1\tfrac{1}{2} - \tfrac{1}{2} - \tfrac{1.41}{2} = 49.3''$

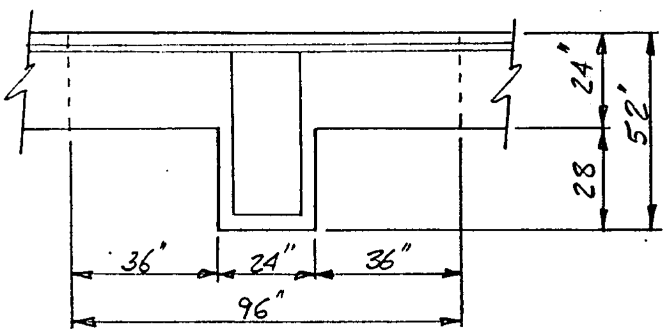
$$\rho = \frac{A_s}{bd} = \frac{7 \times 1.56}{96 \times 49.3} = .0023$$

$$a = \frac{\rho f_{dy} d}{.85 f'_{dc}} = \frac{.0023(77{,}220)(49.3)}{.85(4760)} = 2.16'' < \text{Flange Thickness of } 24''$$

Beam can be Designed as T-Beam

Calculate Deflections of Center Bm Section and Add to Deflections of Deep Bm (100")

Design Center Section of Roof Support Beam



Top of Beam
Use #10 @ 8" (Each Direction)

Bottom of Beam
7 #11's $A_s = 1.56 \times 7 = 10.92$
Dbl #4 Stirrups @ 8"

Use Same Clearances as for Deepened Beam Section

Run CBARCS To determine Initial Design Parameters
from CBARCS Output:

$r_u = 153.51$ ✓
$K_E = 3220.37$ ✓

Reduce Ultimate Resistance of Beam, r_u, by the dead loads from concrete and earth cover to determine $r_{available}$ for Blast Load Resistance.

$r_{avail} = r_u - r_{DL}\left(\frac{f_{ds}}{f_y}\right)$ for $\Delta_x \leq 2°$ $f_{dy} = 1.17 f_y$ P-397 Table 4.1
$f_{ds} = f_{dy}$ Table 4.2

$W_{DL\ from\ Slab} = \dfrac{\frac{24}{12} \times 150 + 2 \times 120}{144} = 3.67\ lbs/In^2$

$W_{DL\ from\ Beam} = \dfrac{\frac{52}{12} \times 150 + 2 \times 120}{144} = 6.09\ lbs/In^2$

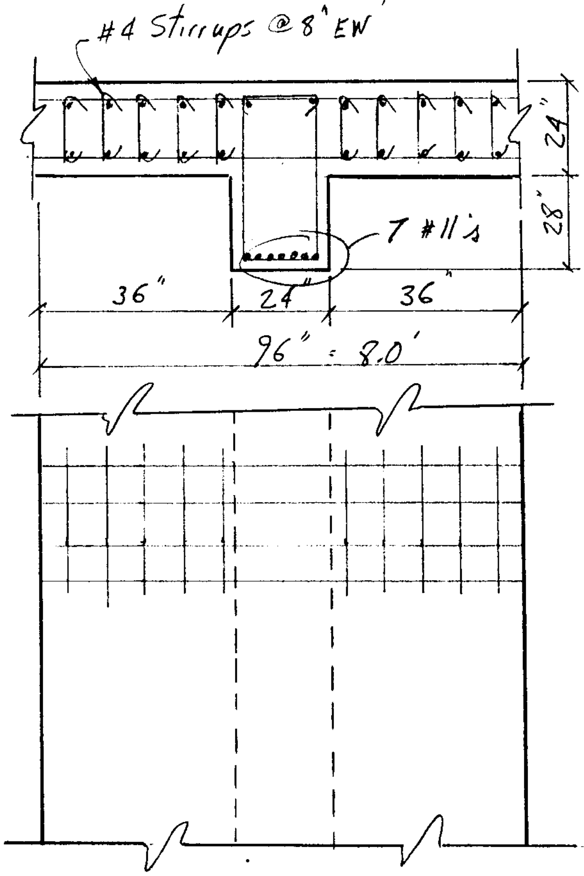
$r_{DL} = \dfrac{3.67 \times 478 + 6.09 \times 24}{96} = 19.80$ psi on Beam

$r_{avail} = 153.51 - 19.80\left[\dfrac{1.17(66{,}000)}{66{,}000}\right] = 130.34$

23.166

Diagonal Shear Design for 52" Deep Beam

Consider T-Bear Shear Strength as being made up of Two Components: Flange Shear Cap. + Web Shear Capacity



Roof Plan

Calculate Flange Shear Capacity

Width, Use $W = 96 - 24 = 72''$ (T-Flange-Web)

To determine A_s, Use $\frac{72}{8} = 9 + 1 = 10$ Bars (See Sketch)

$A_s = 1.27 \times 10 = 12.7\ in^2$ (#10's)

$d = 24 - 1\frac{1}{2} - \frac{1.27}{2} = 21.865''$

$\rho = \frac{A_s}{bd} = \frac{12.7}{72(21.865)} = .008$

$v_c = [1.9(4000)^{1/2} + 2500(.008)]$

$v_c = 140.2\ psi < 3.5(4000)^{1/2} = 221\ psi$ OK

Calculate Stirrup Capacity

P.397, Eq 4-26

$A_v = \frac{(v_u - v_c) b_s s_s}{\phi f_{ds}}$ Using #4 Stirrups @ 8" EW, Solve for v_u

$(v_u - v_c) = \frac{A_v \phi f_{ds}}{b_s s_s} = \frac{.2 \times 10 \times .85 \times 66{,}000}{72 \times 8} = 195\ psi$

$v_u = 195 + 140.2 = 335.2$

$V_u = v_u b d = 335.2(72)(21.865) = 527{,}700\ lbs$

Cont'd Diagonal Shear Design for 52" Deep Beam

Calculate Web Shear Capacity

Width = 24" Height = 52" $d = 52 - 1\frac{1}{2} - \frac{1}{2} - \frac{1.41}{2} = 49.295"$

$A_s = 7$ #11's, $7 \times 1.56\ in^2 = 10.92\ in^2$

$\rho = \frac{A_s}{bd} = \frac{10.92}{24(49.295)} = .00923$

$v_c = [1.9(4000)^{1/2} + 2500(.00923)] = 143.24\ psi < 221$ OK $3.5 f_{dc}^{1/2}$

Calculate Stirrup Capacity

$A_v = \frac{(v_u - v_c)\, b_s S_s}{\phi f_{ds}}$ Use Dbl #4 Stirrups

$(v_u - v_c) = \frac{A_v \phi f_{ds}}{b_s S_s} = \frac{.2 \times 4 \times .85 \times 66{,}000}{24 \times 8} = 233.75\ psi$

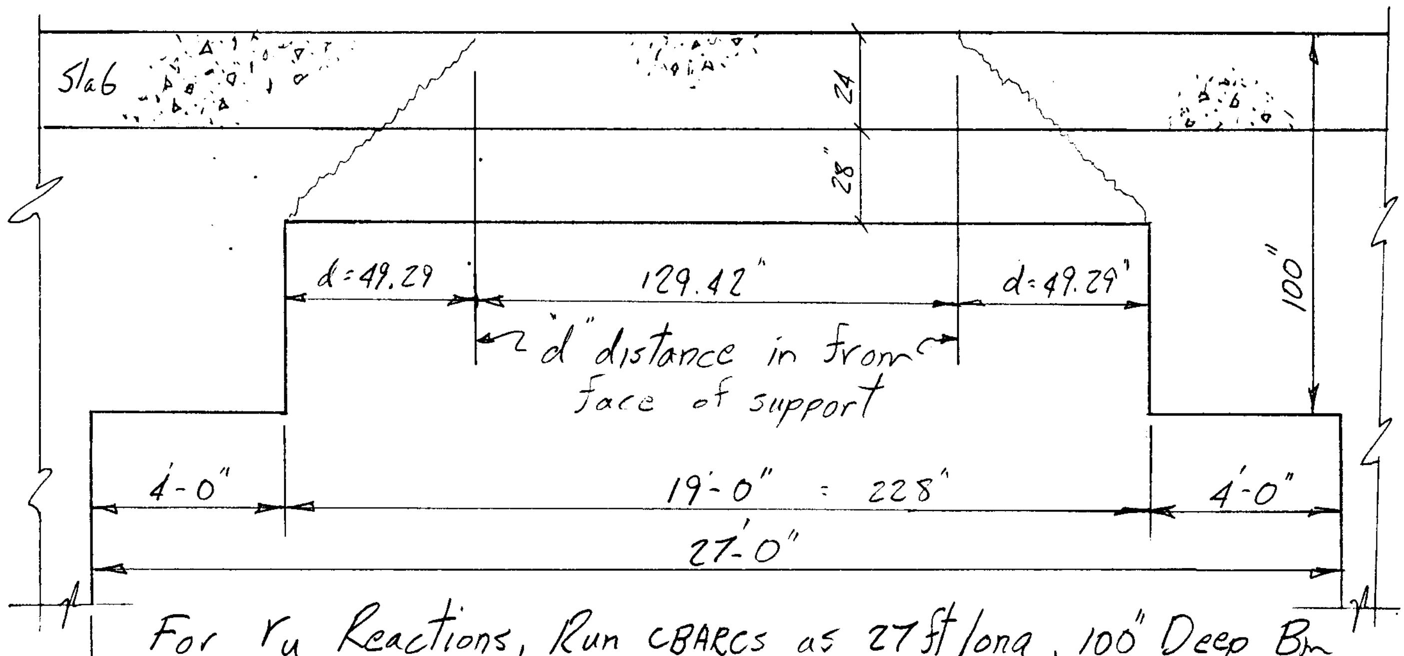
$v_u = 233.75 + 143.24 = 376.99\ psi$

$V_u = v_u b d = 376.99(24)(49.295) = 446{,}009\ lbs$

Total Shear Capacity of 52" Deep T-Beam

$V_u = \underset{(Flange)}{527{,}700} + \underset{(Web)}{446{,}009} = 973{,}709\ lbs$

Cont'd Diagonal Shear Design for 52" Deep Beam



For r_u Reactions, Run CBARCS as 27 ft long, 100" Deep Bm

From CBARCS Roof3.out (for 52" Bm) r_u = 153.51 psi

For 8 ft wide T-Beam - Uniform Load, $W = 96 \times 153.51$
$= 14{,}737$ lbs/In.

Calculate Shear Reactions @ d distance from face of Support

$$V_u = 14{,}737 \times \left(\overset{64.71}{\frac{228}{2}} - 49.29\right) = 953{,}629 \text{ lbs}$$

From Previous Sheet, $V_{u\,capacity}$ = 973,709 > 953,629 OK

Shear Capacity is Adequate for 52" Beam

Check Direct Shear Capacity for 52" Deep Beam

Max Support Shear, $V_u = 14,737 \times \frac{228}{2} = 1,680,013$ lbs

Concrete Capacity, $V_c = 0.18 f'_{dc} b d$

For Flange, $V_F = .18(4400)(72)(21.865) = 1,246,830$ lbs

For Web, $V_W = .18(4400)(24)(49.295) = 936,999$ lbs

Total, $V_c = V_F + V_W = 1,246,830 + 936,999$

$V_c = 2,183,829 > V_u = 1,680,013$ OK.

No Diagonal Shear Steel will be Required

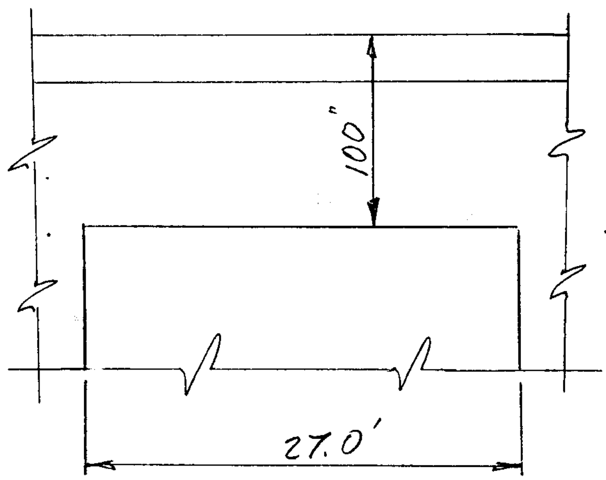
Mason & Hanger Engineering, Inc.
Lexington, Kentucky

File 2446	Page 24
Sheet	of Sheets
Date	10/98
Computed By	TA
Checked By	

Project P-501 ADCAP Magazines Location NWS Yorktown, VA

Subject

Beam Deflection Analysis



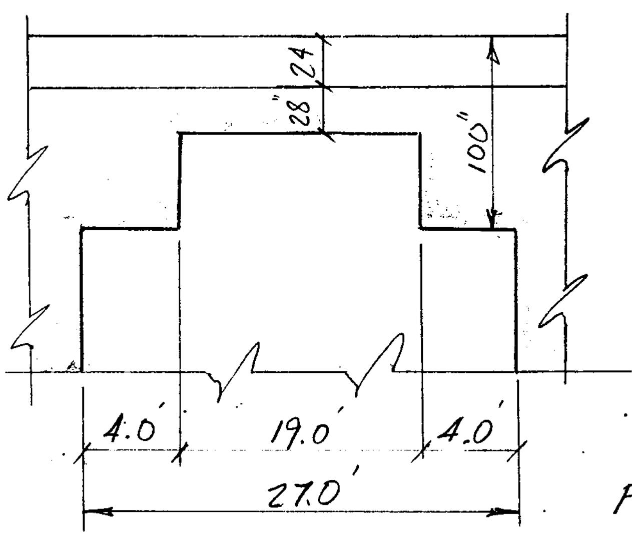
Analyze Beam Over Center Wall Opening as Full Depth

From CBARCS: Roof 2.out Page 9

$K_E = 5{,}319.81$ psi $r_{avail} = 128.37$

From SOLVER: Roof 2.out

$\Delta_x = .032''$



Analyze Beam with Minimum Cross-Section - Determine Appropriate Length required to provide similar r_u

From CBARCS: Roof 3.out

$K_E = 3220.37$ $r_{avail} = 130.34$

From SOLVER: Roof 3.out

$\Delta_x = .054''$

Total Deflection of Beam, $\Delta_x = .032 + .054$
$= .086$ inches

Δ_x Allow for $2^\circ = \frac{27 \times 12}{2} \tan 2^\circ = 5.66'' > .086$ OK

Roof3.in
CBARCS Program

```
100  0,0,0,0,1.17
110  P501 MAGAZINE - 24" x 52" x 19' ROOF BEAM
120  0,1,0,0,0,0
130  350000,1,0,0,0,0,0,0
140  8735.3,19,8,88.53,116.4,0,0,0,0,0,0
150  4760,66000,52,2,6,0,0,0
160  1.91,1.37,1.91,0.75,3.905,2.70,2.635,1.75
```

19 ft long   52" Deep Beam

Fixed Each End

M_E = 8411.52

K_E = 3220.37

r_u = 153.51

Roof 3. out Page 26
CBARCS Output

P501 MAGAZINE - 24" x 52" x 19' ROOF BEAM

BLAST WALL HEIGHT	19.00 FT
BLAST WALL LENGTH	8.00 FT
DURATION OF LOAD	116.40000 MSEC
FICTITIOUS PEAK PRESSURE	88.53000 PSI
EFFECTIVE IMPULSE	8735.30 PSI MS

HEIGHT 228.00 IN LENGTH 96.00 IN

DYNAMIC CONCRETE STRENGTH	4760.00
DYNAMIC STEEL STRESS	77220.00
THICKNESS CONCRETE INCHES	52.0000
THICKNESS OF SAND INCHES	.0000
THETA ALLOWABLE DEGREES	2.0000

AREA VERT TOP STEEL/FT	1.9100	COVER	3.9050
AREA VERT BOT STEEL/FT	1.3700	COVER	2.7000
AREA HORIZ TOP STEEL/FT	1.9100	COVER	2.6350
AREA HORIZ BOT STEEL/FT	.7500	COVER	1.7500

TYPE 1 CONSTRUCTION

CONCRETE MODULUS PSI	3555611.
RATIO MOD STEEL/CONCRETE	8.16
GROSS MOMENT INERTIA	11717.33
AVE CRACKED MOM INERTIA	2017.50
AVE MOMENT INERTIA	6867.42
AVERAGE PERCENT STEEL	.0028
D FACTOR MU=1/6	25116083298.
D FACTOR MU= 0.3	26832807568.

ALLOW SHEAR UNREINFORCED WEB 105.64 PSI 5080.60 LBS/IN
WIDTH
ALLOW SHEAR AT SUPPORT 753.98 PSI 36262.86 LBS/IN
WIDTH
UNREINFORCED CONCRETE THETA LE 2 DEG

POSITIVE VERTICAL MOMENT	425021.67
NEGATIVE VERTICAL MOMENT	572459.99
POSITIVE HORIZONTAL MOMENT	239640.58
NEGATIVE HORIZONTAL MOMENT	588069.37

FIXED END BEAM

LOCATION YIELD LINE LENGTH	.00
LOCATION YIELD LINE HEIGHT	114.00
ULTIMATE LOAD CAPACITY RU	153.5059
SHEAR LOAD AT VERTICAL SUPPORT	.00 LB/IN WIDTH
SHEAR LOAD AT HORIZONTAL SUPPORT	17499.68 LB/IN WIDTH

SHEAR AT DISTANCE FROM VERTICAL SUPPORT	.00 PSI
SHEAR AT DISTANCE FROM HORIZONTAL SUPPORT	210.35 PSI
ALLOWABLE MAX DEFLECTION	3.9876

LOAD MASS FACTOR	.7200
MASS CONCRETE ONLY	8411.52

SHEAR CAPACITY(VC) EXCEEDED

ELASTIC LIMIT RE PSI	132.15
ELASTIC DEFLECTION XE	.0381
ULTIMATE RESISTANCE	153.51
PLASTIC DEFLECTION	.0689

ULTIMATE RESISTANCE RU	153.51
ELASTIC DEFLECTION LIMIT XE	.0477
STIFFNESS KE	3220.37

MASS	8411.520
LOAD	88.530
DURATION	116.400
RESISTANCE	153.506
STIFFNESS	3220.367

GAS PRESSURE	.00	DURATION	.00
NATURAL PERIOD			10.154629
MAXIMUM DEFLECTION			.054228
TIME TO MAXIMUM DEFLECTION			5.219093
DURATION/NATURAL PERIOD			11.462753
LOAD/RESISTANCE			.576720
ELASTIC DEFLECTION LIMIT			.047667

MAX FRAGMENT SPALL VELOCITY FT/SEC	1.388463

SOLVER Program
Roof 3


```
SOLVE    ROOF BEAM: ANALYSIS OF 24" X 52" X 19' LONG HEADER BEAM
    1     1         .1        200                                  -1

    2     2      13868       0.01
    1    3220.37    130.34                   1.0        1.0
    3       0.0                     20       1.0        0.0
    1    3220.37   -153.51                   1.0        1.0
    3       0.0                    -20       1.0        0.0
    1     5
   0.00      74.50      8.77      93.30      78.25      93.30     110.15
0.0
     300        0.0
STOP
```

Use Equivalent M_E from Page 3 ⇒ $M_E = 13{,}868$

Use r_{avail} from Page 19 ⇒ $r_{avail} = 130.34$

Use Dynamic Loading from Graph on Page 3

SOLVER Output
Roof 3.out


```
1ONE DEGREE OF FREEDOM SOLVER INPUT
 VERSION 2.2  FEB 1989

 PROBLEM DESCRIPTION
 SOLVE   ROOF BEAM: ANALYSIS OF 24" X 52" X 19' LONG HEADER BEAM


 ANALYSIS CONTROL CARD

 TYPE OF SOLUTION......................     1
   EQ. 0,  DEFAULTS TO 1
   EQ. 1,  NEWMARK-BETA METHOD
   EQ. 2,  WILSON-THETA METHOD
 NUMBER OF LOAD CASES..................     1
 TIME STEP.............................     0.1000
 TIME LIMIT............................   200.0000
 NEWMARK*S GAMMA.......................     0.0000
   EQ. 0.0, DEFAULTS TO 0.5
 NEWMARK*S BETA........................     0.0000
   EQ. 0.0, DEFAULTS TO 0.25
 WILSON*S THETA........................     0.0000
   EQ. 0.0, DEFAULTS TO 1.4
 PRINT OPTION..........................    -1
   EQ. 0, DEFAULTS TO 1
   EQ. 1, PRINT EVERY STEP
   EQ. N, PRINT EVERY N-TH STEP
   EQ. -1, PRINT SIGNIFICANT CHANGES


 INITIAL CONDITIONS

 INITIAL DEFLECTION....................     0.0000
 INITIAL VELOCITY......................     0.0000
 INITIAL ACCELERATION..................     0.0000


 STIFFNESS CONTROL CARD

 NUMBER OF STIFFNESSES.................     2
 NUMBER OF REBOUND STIFFNESSES.........     2
 MASS..................................  13868.0000
 PERCENT OF CRITICAL DAMPING...........      0.0100


 INITIAL RESISTANCE-DEFLECTION CURVE

 NUMBER  MODE   STIFFNESS  RESISTANCE   YLD DEFL    MASS FACT    DAMP
FRAC

      1      1 0.32204E+04 0.13034E+03 0.00000E+00        1.00
1.00
      2      3 0.00000E+00 0.00000E+00 0.20000E+02        1.00
0.00


 REBOUND RESISTANCE-DEFLECTION CURVE

 NUMBER  MODE   STIFFNESS  RESISTANCE   YLD DEFL    MASS FACT    DAMP
FRAC

      3      1 0.32204E+04-0.15351E+03 0.00000E+00        1.00
```

```
1.00
     4     3 0.00000E+00 0.00000E+00-0.20000E+02        1.00
0.00


 LOAD DATA

 LOAD CASE NUMBER......................     1
 NUMBER OF LOAD POINTS.................     5


 POINT              TIME              LOAD
     1            0.0000           74.5000
     2            8.7700           93.3000
     3           78.2500           93.3000
     4          110.1500            0.0000
     5          300.0000            0.0000


 GENERATED RESISTANCE - DEFLECTION CURVES

 NATURAL PERIOD........................    13.0387

 NUMBER  MODE   STIFFNESS  RESISTANCE    YLD DEFL    MASS FACT    DAMP
FRAC
     1     1 0.32204E+04 0.13034E+03 0.40474E-01        1.00
1.00
     2     3 0.00000E+00 0.13034E+03 0.20000E+02        1.00
0.00
     3     1 0.32204E+04-0.15351E+03-0.47668E-01        1.00
1.00
     4     3 0.00000E+00-0.15351E+03-0.20000E+02        1.00
0.00
1* * * * NEWMARK-BETA SOLUTION * * * * *
 LOAD CASE.............................     1
 TIME STEP.............................     0.1000


 * * * * * SOLUTION RESULTS  * * * * * *
        STEP   TIME STIF DEFLECTION   VELOCITY        ACC RESISTANCE
LOAD

 ELAS    0    0.0000    1       0.000     0.0000     0.0054        0.00
74.50
 *CHG   49    4.8091    2       0.040     0.0091    -0.0034      130.34
84.81
 REB    78    7.8000    1       0.054     0.0000    -0.0028      130.38
91.22
 RLOD  143   14.3000    1       0.031     0.0000     0.0026       57.29
93.30
 REB   209   20.9000    1       0.053    -0.0002    -0.0025      128.16
93.30
 RLOD  274   27.4000    1       0.032     0.0002     0.0024       59.50
93.30
 REB   339   33.9000    1       0.052    -0.0001    -0.0024      126.06
93.30
 RLOD  404   40.4000    1       0.032     0.0001     0.0023       61.54
93.30
 REB   469   46.9000    1       0.052     0.0000    -0.0022      124.08
93.30
 RLOD  535   53.5000    1       0.033     0.0002     0.0021       63.50
```

93.30

REB	600	60.0000	1	0.051	-0.0001	-0.0021	122.19	93.30
RLOD	665	66.5000	1	0.033	0.0001	0.0020	65.30	93.30
REB	730	73.0000	1	0.051	-0.0001	-0.0020	120.44	93.30
RLOD	796	79.6000	1	0.034	0.0000	0.0016	66.76	89.35
REB	851	85.1000	1	0.043	-0.0001	-0.0016	95.64	73.27
RLOD	927	92.7000	1	0.023	0.0001	0.0015	30.00	51.04
REB	981	98.1000	1	0.031	-0.0001	-0.0015	56.16	35.24
*CHG	1041	104.0435	3	0.013	-0.0025	0.0013	-0.01	17.86
RLOD	1058	105.8000	3	0.011	0.0001	0.0014	-6.85	12.72
*CHG	1077	107.6362	1	0.013	0.0020	0.0005	0.00	7.35
REB	1111	111.1000	1	0.018	0.0000	-0.0012	16.87	0.00
*CHG	1144	114.3614	3	0.013	-0.0025	0.0000	0.00	0.00
RLOD	1177	117.7000	3	0.008	0.0001	0.0012	-16.33	0.00
*CHG	1209	120.8823	1	0.013	0.0024	0.0000	0.00	0.00
REB	1242	124.2000	1	0.018	-0.0001	-0.0011	15.83	0.00
*CHG	1275	127.4032	3	0.013	-0.0023	0.0000	0.00	0.00
RLOD	1307	130.7000	3	0.008	0.0001	0.0011	-15.35	0.00
*CHG	1340	133.9241	1	0.013	0.0023	0.0000	0.00	0.00
REB	1372	137.2000	1	0.018	0.0000	-0.0011	14.87	0.00
*CHG	1405	140.4451	3	0.013	-0.0022	0.0000	0.00	0.00
RLOD	1437	143.7000	3	0.009	0.0000	0.0010	-14.42	0.00
*CHG	1470	146.9660	1	0.013	0.0021	0.0000	0.00	0.00
REB	1503	150.3000	1	0.018	-0.0001	-0.0010	13.96	0.00
*CHG	1535	153.4869	3	0.013	-0.0021	0.0000	0.00	0.00
RLOD	1568	156.8000	3	0.009	0.0001	0.0010	-13.53	0.00
*CHG	1601	160.0078	1	0.013	0.0020	0.0000	0.00	0.00
REB	1633	163.3000	1	0.017	0.0000	-0.0009	13.12	0.00
*CHG	1666	166.5288	3	0.013	-0.0019	0.0000	0.00	0.00
RLOD	1698	169.8000	3	0.009	0.0000	0.0009	-12.71	0.00
*CHG	1731	173.0497	1	0.013	0.0019	0.0000	0.00	0.00

```
 REB   1763   176.3000     1        0.017       0.0000      -0.0009        12.32
0.00
 *CHG 1796   179.5706     3        0.013      -0.0018       0.0000         0.00
0.00
 RLOD 1829   182.9000     3        0.009       0.0001       0.0009       -11.93
0.00
 *CHG 1861   186.0915     1        0.013       0.0018       0.0000         0.00
0.00
 REB   1894   189.4000     1        0.017      -0.0001      -0.0008        11.56
0.00
 *CHG 1927   192.6124     3        0.013      -0.0017       0.0000         0.00
0.00
 RLOD 1959   195.9000     3        0.010       0.0000       0.0008       -11.21
0.00
 *CHG 1992   199.1334     1        0.013       0.0017       0.0000         0.00
0.00


 ***** TIME LIMIT REACHED

 MAXIMUM DEFLECTION IN SOLUTION
 STEP NUMBER...........................      77
 TIME AT MAXIMUM.......................          7.7000
 MAXIMUM DEFLECTION....................          0.054

 MINIMUM DEFLECTION IN SOLUTION
 STEP NUMBER........................... 1176
 TIME AT MINIMUM.......................        117.6000
 MINIMUM DEFLECTION....................          0.008


SOLUTION TIME LOG
   ASSEMBLY......................            0.16
   SDOF SOLUTION.(LOAD CASE  1)...           0.16

   TOTAL TIME....................            0.32
1ONE DEGREE OF FREEDOM SOLVER INPUT
 VERSION 2.2  FEB 1989

 PROBLEM DESCRIPTION
 STOP

 ***** END SOLVER RUNS *****
```

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

Project Location

Subject

File Page
Sheet of Sheets
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Checked By

FOUNDATION

AXIAL LOAD ON WALL FROM ROOF:

$r_u = r_{AVAIL}$ (BLAST LOAD) $+ r_{DL}$ (CONC. + EARTH)

$r_u = 17.85$ PSI $+ 3.66$ PSI $= 21.51$ PSI

$P_u = 5{,}490.8$ #/IN OF WALL $\times$ 12"/FT $= 65.9^k$/FT (BLAST + DL)

FROM ROOF → $P_{u\,DL} = 3.66\frac{(478)}{2} = 0.87^k/_{IN}$ $\times$ 12" → STRIP OF WALL $= 10.5^k$/FT (DL ONLY)

WALL WT $= 20.5' \times 1.92' \times 0.15 = 5.9^k$/FT

$10.5 + 5.9 = 16.4^k$/FT (DL) $\qquad 65.9 + 5.9 = 71.8$ (TOTAL)

TOTAL LOAD ON FOOTING $= \underline{71.8^k/FT}$ BLAST + DL

ASSUME HORIZ. LOAD IS TAKEN OUT BY SLAB –
DESIGN AS SIMPLE SPREAD FOOTING:

DL ONLY $\quad 16.4^k/6 \times 1 = 2.73$ KSF $<$ 4.0 KSF ALLOWABLE FOR STATIC LOADS ✓

<u>CK BEAM SHEAR</u> $\quad d = 24" - 3" - 0.5 = 20.5"$
@ WALL

$V_u = w_u (2' \times 1'\ \text{LENGTH}) = 2.73(2) = 5.46^k$

$v_u = \frac{V_u}{\phi b w d} \quad \frac{5460}{0.85(12)(20.5)} = 26.11$ PSI $\qquad v_c = 2\sqrt{4000} = 126.5 > 26.11$ ✓

SHEAR IS OK

<u>Mu</u>

$M_u = \frac{w l^2}{2} = \frac{2.73\,\text{KSF}\,(2)^2(1')}{2} = 5.46$ K-FT $\qquad M_t = 5.46/0.9 = 6.07$

$jd \approx 0.9d = 0.9(20.5) = 18.5" = 1.54'$

$T = M_t/jd = 6.07/1.54 = 3.94^k \qquad A_s = T/f_y = 3.94/60 = 0.07\ \text{IN}^2$

$\rho = A_s/bd = 0.07/12(20.5) = 0.0003 < \rho_{MIN} = 0.0033$

$A_s = 0.0033(12)(20.5) = 0.81\ \text{IN}^2$ → #7 @ 8" $\qquad$ WE HAVE #10 @ 8" ✓

<u>T & S</u> $= 0.0018(12)(24) = 0.52\ \text{IN}^2$ → #6 @ 8" $\qquad$ " ✓

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

File 2286 Page
Sheet of Sheets
Date
Computed By
Checked By W. S. FOY

Project P298 Location PT. HADLOCK, WA
Subject FOUNDATION DESIGN

CHECK FOUNDATION EXTENSION FOR BLAST LOADS:

TOTAL LOAD FROM WALL (INCLUDING BLAST LOAD) = $71.8^{K}/FT$

$71.8/6 \times 1'_{LENGTH} = 11.97$ KSF $= w$

ASSUME SOIL COULD ACTUALLY EXERT THIS FORCE BEFORE IT HAS TIME TO REACT IN A SPLIT-SECOND BLAST CONDITION.

$M = \frac{w\ell^2}{2}$ $\ell = 2'$ $M = \frac{11.97(2)^2}{2} = 23.94$ K-FT $M_u = 1.7(23.94) = 40.7$

$M_N = 40.7/0.9 = 45.2$ FT-K $= 542.7$ K-IN

$d = 24'' - 3 - 0.5 = 20.5''$ $a = A_s f_y / 0.85 f'_c b = 60 A_s / 0.85(4)(12) = 1.47 A_s$

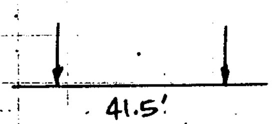
$M_N = A_s f_y (d - a/2) = 60 A_s (20.5 - 1.47/2) = 1230 A_s - 44.1 A_s^2$

$\frac{-1230 \pm \sqrt{1230^2 - 4(-44.1)(-542.7)}}{2(-44.1)} = \frac{-1230 \pm 1190.4}{-88.2} = 0.45$ IN^2/FT

$\rho = 0.45/12(20.5) = 0.0018 < \rho_{MIN} = 0.0033$

$A_s = 0.0033(12)(20.5) = 0.81$ $IN^2/FT \longrightarrow$ #7@8''
WE HAVE #10@8'' ✓

CK INTERIOR SLAB



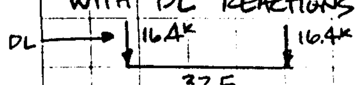
CK CAPACITY OF CURRENT DESIGN
$A_s = 1.91$ IN^2/FT $d = 12.5''$ $a = 1.47 A_s = 2.81''$

CLR SPAN = 41.5 − 3 − 6 = 32.5 $M_N = 60(1.91)\left(12.5 - \frac{2.81}{2}\right) = 1271.5$ K-IN $= 106$ K-FT

$106(0.9) = 95.4 = M_u$ $M = \frac{95.4}{1.7} = 56.1$ K-FT

$M = \frac{w\ell^2}{12} = \frac{w(32.5)^2}{12} = 56.1$ $\underline{w = 0.64^{K}/FT}$ THIS WILL INCREASE SINCE 'BEAM' IS ACTUALLY DOUBLY-REINFORCED.

FLOOR SLAB DESIGN IS FOR $1^{K}/FT$ FLOOR LOAD
THIS 15'' SLAB IS GOOD FOR A UNIFORM LOAD OF $\underline{1372\ \#/FT^2}$ (SEE P 3)
WITH DL REACTIONS AND NO INT. LOAD ⟶ EQUIV. UNIF. LOAD:



$\frac{16.4 \times 2}{32.5} = \frac{1.009\ K/FT}{1' \text{ WIDE STRIP}} = 1009^{K}/FT^2 < 1372$

W/NO INTERIOR LOAD; 1372 > 1.009 w/ INT. LOAD = $1^{K}/FT^2 < 1.37^{K}/FT^2$ ✓

CK SHEAR CAPACITY OF CURRENT SLAB:

$V_c = 2\sqrt{4000} = 126.5$ Psi $\qquad d = 11.5''$

SET $V_u = V_c = \dfrac{V_u}{\phi b w d}$ $\qquad 126.5 = \dfrac{V_u}{0.85(12)(11.5)}$

MAX $V_u = 14.84^k$

WE HAVE 16.4^k IN A STATIC CONDITION, BUT REINF. TOP & BOTTOM WILL TAKE OUT STATIC SHEAR LOADS.

IN A BLAST CONDITION, WE HAVE 71.8^K SHEAR LOAD @ FOOTING.
THIS WILL OBVIOUSLY CREATE A SHEAR FAILURE AT THE SLAB EDGE CREATING A FLOATING SLAB CONDITION.

CHECK SLAB ON GRADE FOR MAXIMUM ALLOWABLE FLOOR LOAD.

SOLVE FOR ALLOWABLE FLOOR LOAD:

ASSUME SOIL MODULUS = 1400 Psi

ASSUME CONCRETE ULTIMATE DESIGN FLEXURAL STRENGTH = $f_{CT} \cong 9\sqrt{f'_c} = 9\sqrt{3000} = 493$ SAY 500 Psi

" MODULUS OF ELASTICITY $E_c = 57000\sqrt{f'_c} = 57000\sqrt{3000} = 3.1 \times 10^6$ Psi

" $f_y = 60{,}000$ Psi

THICKNESS $h = 15''$

FLEXURAL REINF. = #10 @ 8" = $1.91 \text{ in}^2/\text{FT}$

S = % OF REINF. = $1.91/12 \times 15 = 0.0106 = 1.06\%$

FROM TABLE 3-1 IN ARMY TM 5-809-12:

ASSUME $f_{CT} \approx$ 550 Psi : FOR $f'_c = 3000$ $\quad f_{CT} = 493$
FOR $f'_c = 4000$ $\quad f_{CT} = 569$

FROM TABLE:

SLAB THICKNESS	ALLOW. LIVE LOAD
14"	1326 #/FT²
15"	1372 #/FT²
16"	1418 #/FT²

INTERPOLATE → $\dfrac{1}{2} = \dfrac{X}{92}$ $\quad X = 46$

SLAB IS DESIGNED FOR $1^k/\text{FT}^2 < 1.37^k/\text{FT}^2$ ✓

SLAB WILL SUPPORT DESIGN LIVE LOAD IN A FLOATING CONDITION

MIDDLE FOOTING

LOAD FROM ROOF → f_{DL} = 3.66 PSI

$P_{u\,DL}$ = 3.66(478") = 1.75 K/IN × 12" STRIP OF WALL = 21 K/FT

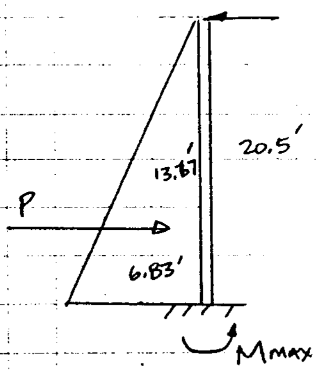
WALL WT = [(21.25 − 6.25) × 1.5 + (6.25 × 2)] 0.15 = 5.25 K/FT

TOTAL LOAD ON FOOTING (DL ONLY) = 26.25 K/FT

P/A = 26.25 / 12×1 = 2.19 < 4 ✓

SPECIAL CASE

CK EXTERIOR FOOTING W/ MOMENT FROM SOIL PRESSURE:



SOIL WT = 120 PCF

EQUIV. FLUID PRESS. = 30 PCF ±

P = 20.5 × 30 × ½ × 20.5 = 6.3 K

20.5/3 = 6.83'

$M_{MAX} = \frac{Pab}{2\ell^2}(a+\ell) = \frac{6.3(13.67)(6.83)}{2(20.5)^2}(13.67 + 20.5)$

M_{MAX} = 23.91 K-FT = M_O

FTG WT = 6 × 1 × 2 × 0.15 = 1.8 K

M_R = 23.1 × 3 = 69.4 FS = 69.4 / 23.9 = 2.9 > 1.5 ✓ SOIL = 2 × 20.5 × 1 × 0.12 = 4.92

P = 16.4 K + 1.8 + 4.92 = 23.1 K $\bar{x} = \frac{-23.91 + 69.4}{23.1}$ = 1.97 e = [6/2 − 1.97] = 1.03

$P_{MAX} = \frac{23.1}{6\times1}\left(1 + \frac{6(1.03)}{6}\right)$ = 7.82 > 4.0 KSF ALLOW

WIDEN FTG. AREA: TRY 8'

FTG WT = 2.4 K P = 23.72 K M_R = 23.72 × 5' = 118.6 $\bar{x} = \frac{118.6 - 23.91}{23.72}$ = 3.99

$P_{MAX} = \frac{23.72}{8\times1}(1.01)$ = 2.99 < 4 ✓ e = 4 − 3.99 = 0.01

MAX FOOTINGS 8' WIDE

CK STEEL IN WALL:

M = 23.91 M_u = 40.65 M_N = 45.2 K-FT = 542

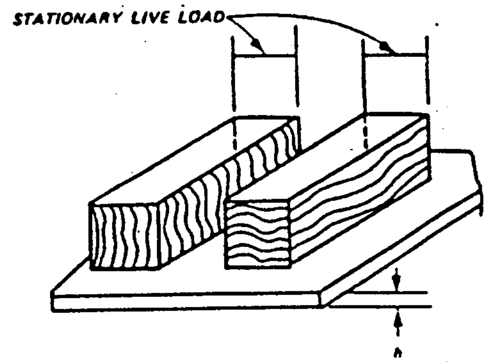
d = 23" − 2" − 0.625 = 20.4"

a = 1.47 As $M_N = 60A_s(20.4 - 1.47A_s/2)$ 1224 As − 44.1 A_s^2 − 542 = 0

$\frac{-1224 \pm \sqrt{1224^2 - 4(-44.1)(-542)}}{-88.2} = \frac{-1224 \pm 1184}{-88.2}$ As = 0.45 REQ'D

WE HAVE 1.91 ✓

Slab Thickness (inches) h	Stationary Live Load w in lb/ft^2 for These Flexural Strengths of Concrete			
	550 $\frac{lb}{in^2}$	600 $\frac{lb}{in^2}$	650 $\frac{lb}{in^2}$	700 $\frac{lb}{in^2}$
6	868	947	1,026	1,105
7	938	1,023	1,109	1,194
8	1,003	1,094	1,185	1,276
9	1,064	1,160	1,257	1,354
10	1,121	1,223	1,325	1,427
11	1,176	1,283	1,390	1,497
12	1,228	1,340	1,452	1,563
14	1,326	1,447	1,568	1,689
15	1372			
16	1,418	1,547	1,676	1,805
18	1,504	1,641	1,778	1,915
20	1,586	1,730	1,874	2,018



NOTE: Stationary live loads tabulated above are based on a modulus of subgrade reaction (k) of 100 lb/in^3. Maximum allowable stationary live loads for other moduli of subgrade reaction will be computed by multiplying the above-tabulated loads by a constant factor. Constants for other subgrade moduli are tabulated below.

Modulus of Subgrade reaction	25	50	100	200	300
Constant factor	0.5	0.7	1.0	1.4	1.7

For other modulus of subgrade reaction values, the constant values may be found from the expression $\sqrt{k/100}$.

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FSCM NO.
53711

DEPARTMENT OF THE NAVY
NAVAL SEA SYSTEMS COMMAND

OR-68/143A
ISSUE DATE: [illegible]
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SUPERSEDING OR-68/143
18 JUNE 1987

ORDNANCE REQUIREMENT, PACKING

INSTALLATION AND REMOVAL OF PACKAGING, HANDLING, STORAGE AND TRANSPORTATION (PHS&T) EQUIPMENT FOR MK 13, MK 14, AND MK 15 CANISTERS

NOTES:	MK 13 CANISTER	MK 14 CANISTER	MK 15 CANISTER
LENGTH (SKID EXTENSION IN DOWN POSITION)..	240.60 IN	279.12 IN	240.60 IN
WIDTH....................................	39.40 IN	40.12 IN	39.40 IN
HEIGHT...................................	41.25 IN	43.00 IN	41.25 IN
NET WIEGHT...............................	2270 LBS	3235 LBS	3015 LBS
GROSS WEIGHT (MAXIMUM)...................	4286 LBS	7335 LBS	4381 LBS
CUBE.....................................	226.19 CU FT	278.66 CU FT	226.19 CU FT

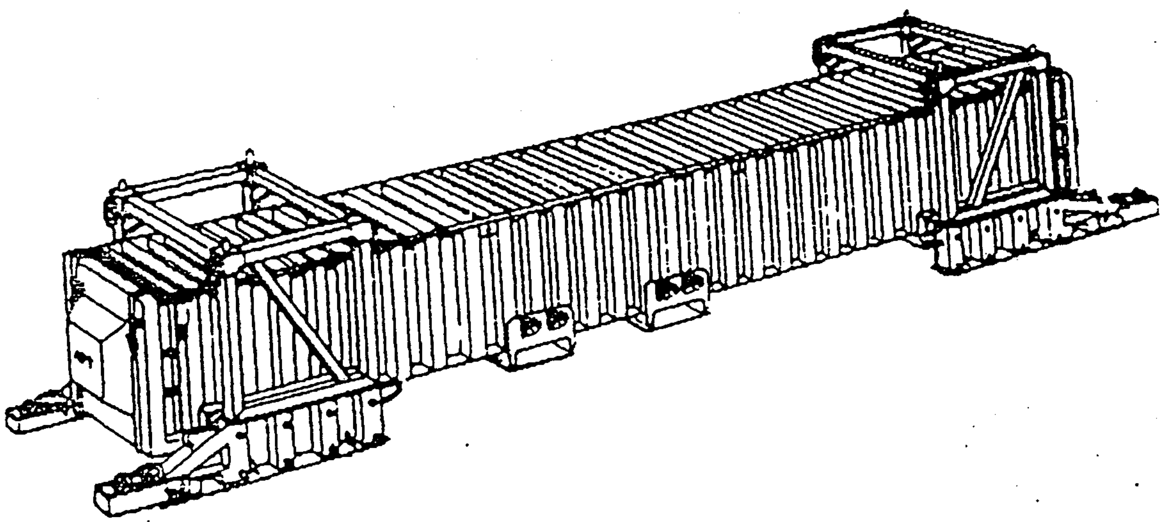


FIGURE NO. 2

REV	DESCRIPTION	TDA	SYSCMD	DATE
A	IADO PROCEDURES	[illegible]	[illegible]	[illegible]

REVISION APPROVAL

TECHNICAL DIRECTING ACTIVITY APPROVAL (Signature and date)

NAVAL SEA SYSTEMS COMMAND HEADQUARTERS APPROVAL

Signature and date [illegible]

3-3. Stationary live loads.

Floor slabs on grade should have adequate structural live loads. Since floor slabs are designed for moving live loads, the design should be checked for stationary live loading conditions. Table 3-1 lists values for maximum stationary live loads on floor slabs. For very heavy stationary live loads, the floor slab thicknesses listed in table 3-1 will control the design. Table 3-1 was prepared using the equation

$$w = 257.876s \sqrt{\frac{kh}{E}} \qquad \text{(eq 3-1)}$$

where

w = the maximum allowable distributed stationary live load, pounds per square foot

s = the allowable extreme fiber stress in tension excluding shrinkage stress and is assumed to be equal to one-half the normal 28-day concrete flexural strength, pounds per square inch

k = the modulus of subgrade reaction, pounds per cubic inch

h = the slab thickness, inches

E = the modulus of elasticity for the slab (assumed to equal 4.0 x 106 pounds per square inch)

The above equation may be used to find allowable loads for combinations of values of s, h, and k not given in table 3-1. Further safety may be obtained by reducing allowable extreme fiber stress to a smaller percentage of the concrete flexural strength have been presented by Grieb and Werner, Waddell, and Hammitt (see Biblio). The selection of the modulus of subgrade reaction for use in table 3-1 is discussed in paragraph 4-2*d*. The design should be examined for the possibility of differential settlements which could result from nonuniform subgrade support. Also, consideration of the effects of long-term overall settlement for stationary live loads may be necessary for compressible soils (see TM 5-818-1/AFM 88-3, Chap. 7).

SLAB UNDER RAILROAD TRACK:

ACTUAL COOPER E-80 RAILROAD LOADING = 8.0 K/FT OF RAIL
(THIS LOADING INCLUDES ENGINE & STD. CAR WTS.)

OUR SITUATION IS UNIQUE SINCE IT WILL INVOLVE FLAT CARS LOADED W/ MISSILE STORAGE BOXES. IT IS ASSUMED, HOWEVER, SINCE STANDARD ENGINES WILL NOT BE INSIDE MAGAZINE, THAT E-80 LOADINGS WILL ADEQUATELY REPRESENT THESE HEAVILY-LOADED FLAT CAR WEIGHTS.

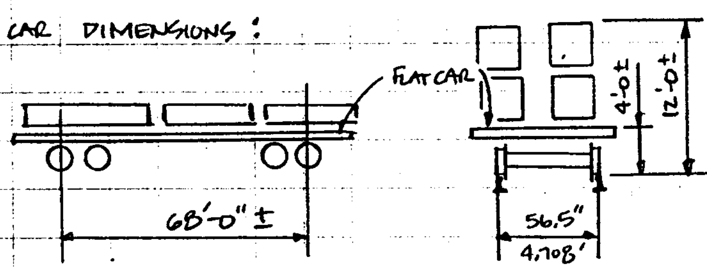
ASSUME CANISTER WTS. ARE < 5 TONS (10^K) SINCE THIS IS THE CRANE CAPACITY

CANISTER MK14 DIMENSIONS:

LENGTH = 279.12" = 23.26'
WIDTH = 40.12" = 3.34'
HEIGHT = 43.00" = 3.58'

CANISTER WT = 3235#
CANISTER W/MISSILE = 7335#

FLAT CAR DIMENSIONS:



ASSUME FLAT CAR CAN CARRY (6) CANISTERS PER LEVEL. ON (2) LEVELS

(12) CANISTERS × 7.34 K/CAN. = 88.1 K + FLAT CAR WT < E80 LOADING
= 88.1 K + 60 K ± = 148.1 K

148.1 ÷ 8 WHEELS = 18.5 K/WHEEL < 52 K/WHEEL (E80)
148.1 K ÷ 68' = 2.2 K/FT < 8 K/FT (E80)

USE 8.0 K/FT PER RAIL OR MAX. AXIAL WHEEL LOAD = 30 K (CONS.)

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

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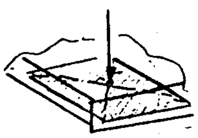
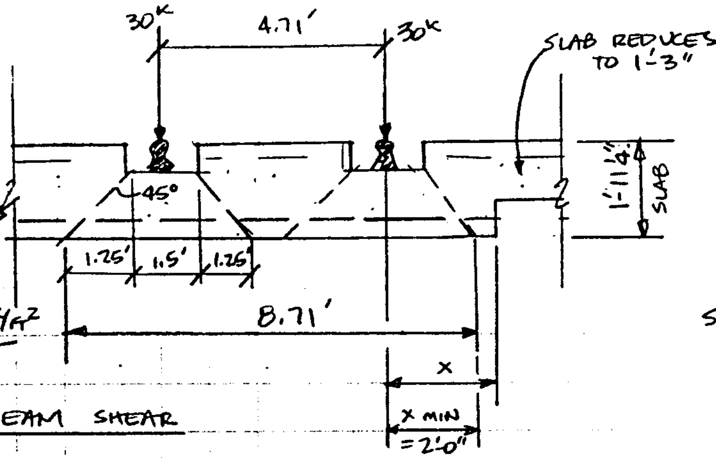
Checked By W.S. Foy	

Project P-501 Location Yorktown, VA

Subject Slab Under RR Track

(RAILS EXTEND IN AND OUT OF PAGE)





30k LOAD WILL DISTRIBUTE IN (2) DIRECTIONS TO AN AREA OF 4'x 4' = 16FT²

$30/16 = \underline{1.875^k/ft^2}$

$60^k / 8.71' = 6.89^k/ft - w_u$

$d = 15 - 3 - \frac{1.27}{2} = \underline{11.37''}$ (#10)

SOIL PRESSURE $= \frac{6.89^k/ft \times 8.71'}{8.71' \times 1'}$

$= 6.89^k/ft^2$

CHECK BEAM SHEAR

$v_c = 2\sqrt{4000} = 126.5\,psi$ SET $v_u = v_c = 126.5$

$v_u = 126.5 = \frac{V_u}{\phi b_w d} = \frac{V_u}{0.85(12)(11.37)}$ MAX $V_u = 14.67^k$

$V_u = w_u\,(x \times 1'_{LENGTH})$ $14.67^k = 6.89^{k/ft}(x)^{ft}$

MAX $x = 2.13'$

MIN $x = 1.25'$ DUE TO 45° DISTRIBUTION OF LOAD

Mu

$M_u = \frac{w_u l^2}{12} = \frac{6.89\,(8.71)^2}{12} = 43.6\,k\text{-}ft$ (CONSERVATIVE)

$d' = 2.5''$ $A_s = \#10 @ 8'' = 1.91\,in^2 = A_s'$ $f'_c = 4000\,psi$ $f_y = 60{,}000\,psi$

$b = 12''$

TOTAL NOMINAL MOMENT CAPACITY OF DOUBLY REINFORCED BEAM:

$$M_N = A_s' f_y\,(d - d') + \underbrace{(A_s - A_s')}_{0}\,f_y\,(d - a/2)$$

$A_s = A_s' = 1.91$ $A_s - A_s' = 0$

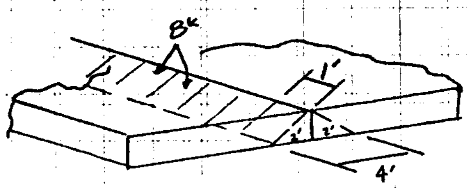
$M_N = 1.91^{in^2}(60{,}000\,psi)(11.37 - 2.5) = \frac{1016502\ \#\text{-}in}{12000} = 84.7\,k\text{-}ft$

$84.7 > 43.6$ ✓

CK STATIONARY LIVE LOAD!

(PER ARMY TM-5-809-12)

USING 8^k/LIN FT E-80 LOADING:



$8^k \div 1 \times 4 = 2^k/ft^2 = 2000\,\#/ft^2$

USING 30^k MAX WHEEL LOAD = 1875#/ft²

$W = 257.876\,S\,\sqrt{\frac{kh}{E}}$

$W = 257.876\,(275)\sqrt{\frac{100\,(15)}{4\times10^6}}$

$W = 1373\ \#/ft^2$

W = MAX ALLOW. DIST. LOAD

$S = \frac{\text{FLEX. STRENGTH}}{2} = \frac{550}{2} = 275\,\#/in^2$

h = THICKNESS = 15"

$E = 4\times10^6\,psi$

$k = 100\,\#/in^3$

WE HAVE 2000#/FT² BY ONE METHOD & 1875#/FT² BY ANOTHER

HOWEVER, 1373 #/FT² IS BASED ON AN UNREINFORCED SLAB.

$\boxed{\text{USE 24" THICK SLAB UNDER TRACKS}}$

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

Project Location

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BLAST DOORS

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

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Date 4/6/94
Computed By TA
Checked By W. S. Foy

Project P-298 Location Port Hadlock, WA
Subject Blast Door Design

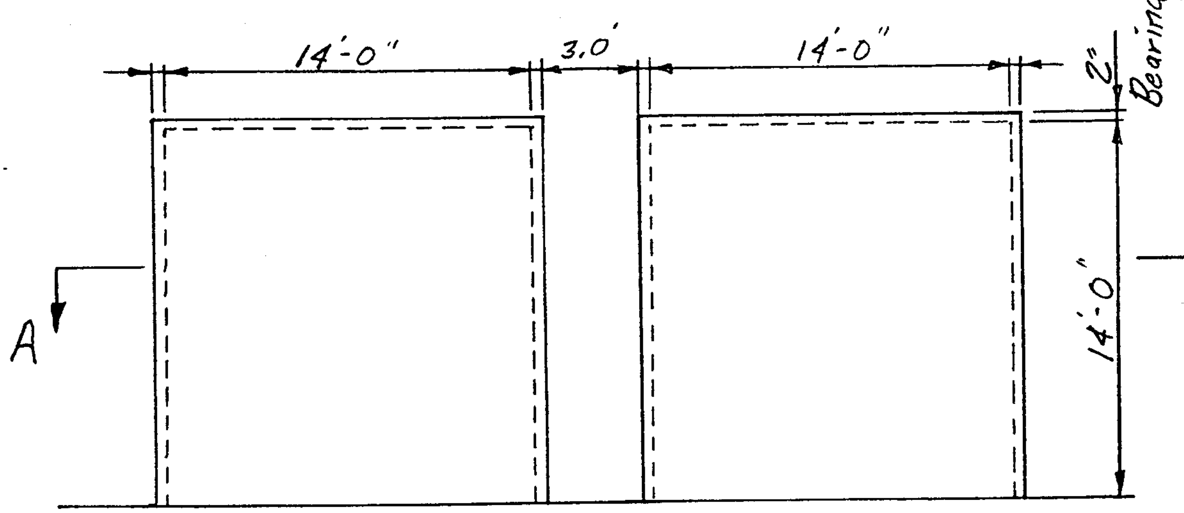
Design Blast Loads for Blast Doors + Front Headwall

$i_{r\alpha} = 1872$ psi-msec

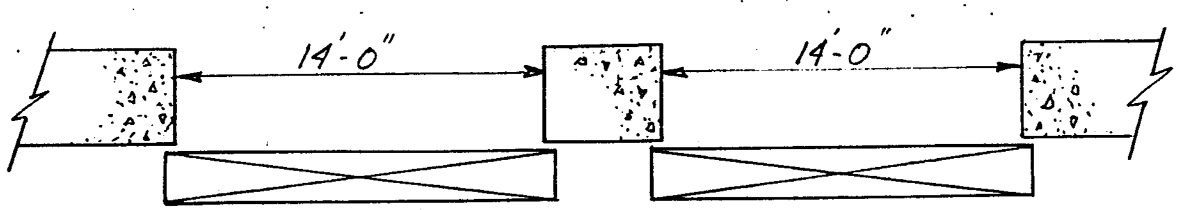
$P_{r\alpha} = 152$ psi

$t_o = 24.6$ msec

Blast Door Layout



Front Elevation



Section A-A

Try Using:

1/2" Exterior Plate
1/4" Interior Plate
S10 x 35 Stiffeners
@ 15" Centers

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Lexington, Kentucky

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Checked By L.I.S. Fox

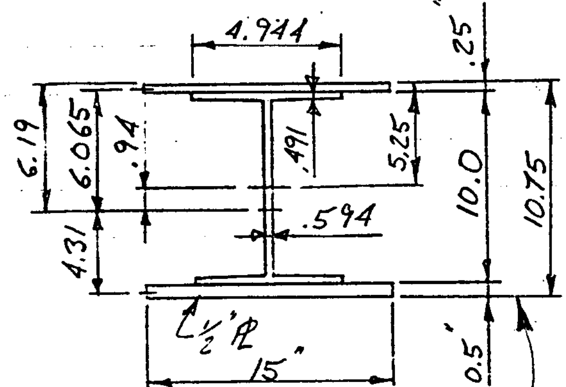
Project P-298 Location Port Hadlock, WA
Subject Blast Door Design

Blast Door Design
Door Cross-Section Properties

Stiffeners: S10 × 35
Spaced @ 15" c.c

Calculate the elastic and plastic section moduli of the combined section.

Initial Elastic Moment of Inertia Using S10 × 35
¼" Plate In Tension (Fully Effective)



Outside Face of Door

	Area		L		
S10 × 35	10.3	×	5.25	=	54.08
½ × 15" PL	7.5	×	10.5	=	78.75
¼ × 15" PL	3.75	×	.125	=	.47
	21.55 ✓				133.30 ✓

$$\text{Centroid} = \frac{133.30}{21.55} = 6.19 \text{ inches}$$

Inertia

S10 × 35		147
½ × 15	=	0.16 ✓
¼ × 15	=	0.02 ✓
		147.18 in⁴ ✓

Ad^2:	A		d^2		
S10 × 35 =	10.3	×	$(0.94)^2$	=	9.10 ✓
½ × 15" PL	7.5	×	$(4.31)^2$	=	139.32 ✓
¼ × 15" PL	3.75	×	$(6.065)^2$		137.94 ✓
					286.36 in⁴

Combined $I_x = 147.18 + 286.36 = 433.54 \text{ in}^4$

$S_T = \frac{433.54}{4.56} = 95.07 \text{ in}^3$ $\quad S_B = \frac{433.54}{6.19} = 70.04 \text{ in}^3$

Check Effectiveness of Plates In Compression (½" PL In Compression)

AISC 1.9.1.2 for A36 Steel; Allow $\frac{W}{T} \le \frac{95}{\sqrt{F_y}} = \frac{95}{6} = 15.83$

Assume PL welded Each side of S10 Flange,

Therefore, Effective Flange width, $W = 15.83(\frac{1}{2}) = 7.92$" Ea Side

Beam Spacing = 15" c.c.

Check $\frac{b_f}{2t_f} \le 8.5$ $\quad b_f = \frac{15 - 4.944}{2} = 5.03$"

$\frac{b_f}{2t_f} = \frac{5.03}{2(0.5)} = 5.03 < 8.5$ OK, ½" PL is fully Effective ✓

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Nov 90

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Mason & Hanger Engineering, Inc.
Lexington, Kentucky

Project P-298 Location Port Hadlock, WA
Subject Door Design

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Checked By W. S. Foy

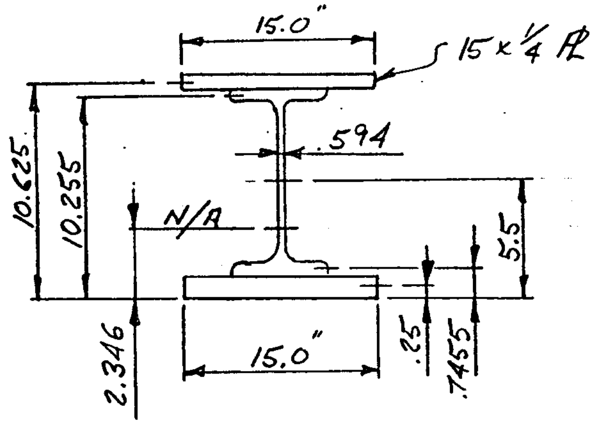
Door Cross-Section Properties

$\boxed{S10 \times 35}$

Calculate Plastic Section Moduli of the combined section.

Initial Plastic Section Modulus, Z

Using 15" Wide 1/4" + 1/2" PL's



S10 × 35

$t_w = 0.594"$
$b_f = 4.944"$
$t_f = 0.491"$
$d = 10.00"$

Total Area of Section

15 × .25	=	3.75
4.944 × .491	=	2.43
7.663 × .594	=	4.55
1.355 × .594		0.81
4.944 × .491	=	2.43
15 × 0.50	=	7.50
		21.47 in^2 × ½ = 10.735 in^2

Centroid of Section

Equal Area Above & Below Neutral Axis

Area Above = 10.735 − (7.50 + 2.43) + .594T

$= T = \frac{.805}{.594} = 1.355"$

Neutral Axis = .5 + .491 + 1.355
= 2.346" from Bottom.

Plastic Section Modulus, Z:

3.75 × (10.625 − 2.346)	=	31.05
2.43 × (10.255 − 2.346)	=	19.22
4.55 × (7.663 × ½)	=	17.43
0.81 × (1.355 × ½)	=	0.55
2.43 × (2.346 − .7455)		3.89
7.50 × (2.346 − .25)	=	15.72
		87.86 in^3

$Z_{Initial} = 87.86\ in^3$ ✓

MOMENT CAPACITY, Mp

$M_p = f_{ds} \times \frac{1}{2} \times (S+Z)$ ASSUME $1 < \mu < 3$

$f_{ds} = f_{dy}$

$f_{dy} = a \times c \times f_y$

$a = 1.1$ $f_y < 50$ Ksi

$c = 1.36$ A36 STEEL, PRESSURE

$f_y = 36$ Ksi

$f_{dy} = 1.1 \times 1.36 \times 36 = 53.8$ Ksi

$f_{du} = c \times f_u$

$c = 1.10$ TABLE 5-3, NAVFAC P-397

$f_u = 58$ Ksi

$f_{du} = 1.1 \times 58 = 63.8$

$f_{ds} = 53.8$ Ksi

$M_p = 53.8 \text{ Ksi} \times (70.04 + 87.86) \times \frac{1}{2}$

$= 4247.5$ in·Kip

$= 4.248 \cdot 10^6$ in·lb

ULTIMATE UNIT RESISTANCE, r_u

$r_u = \frac{8 M_p}{L^2}$ TABLE 3-1, NAVFAC P-397

$M_p = 4.248 \times 10^6$ in·lb

$L = 14' \times 12'' = 168$ in

$r_u = \frac{8 \times 4.771 \times 10^6}{(168)^2}$

$= 1204$ psi

Mason & Hanger Engineering, Inc.
Lexington, Kentucky

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Date 4/6/94
Computed By TA
Checked By K.S. FOY

Project P-298 Location Port Hadlock, WA

Subject Door Design

Blast Door - Shear Design

End Reactions from Door, $R = \frac{r_u \ell}{2}$
(For 15" Wide Section)

Use r_u from SOLVER Analysis,
$r_u = 80.3$ PSI

$R = \frac{80.3 \times (14 \times 12)}{2} = 6{,}745$ lbs/In of Jamb

Web Section Effective in Shear will be taken as the distance from the Inside Face of Cover Plate to Inside Face of Opposite Cover Plate. $d = 10$ inches.

For $S10 \times 35$, $v = \frac{6{,}745 \times 15}{10(.594)} = 17{,}033$ psi

Allowable $f_{dr} = 0.55 f_{ds} = 0.55(53{,}800) = 29{,}590$ psi

Since $v = 17{,}033 < 29{,}590$ No Shear Plates Are Required