Fermilab

Particle Physics Division
Mechanical Department Engineering Note

Number:	MD-ENG-03-026
Date:	November 11, 2003
Project Reference:	Install Blue Block Shielding, WBS 1.1.2.3.3.3.3.2, UID 73211
Project:	NuMI
Title:	Rails between target hall and access shaft
Author(s):	Andy Stefanik A. Stefanik
Reviewer(s):	Bob Woods R J Woods
Key Words:	target pile, target pile shielding, transporter, rails, track
Abstract Summary:	An existing TSB transporter is used to carry Duratek green shielding blocks, Duratek blue shielding blocks, and other loads from the access shaft to the target hall. The transporter runs on A.S.C.E. 40 lb rails in the TSB. The same type of rail is installed between the access shaft and the target hall for the transporter to run on. Structural installation details are checked in this note.
Applicable Codes:	For concrete bearing strength: Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02). For steel and weld allowable stresses: AISC Manual of Steel Construction, Allowable Stress Design, 9th Edition.

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SUBJECT: Rails between target hall and access shaft

NAME: A. M. Stefanik

DATE: Nov 6, 2003

REVISION DATE:

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SUBJECT: Rails between target hall & access shaft

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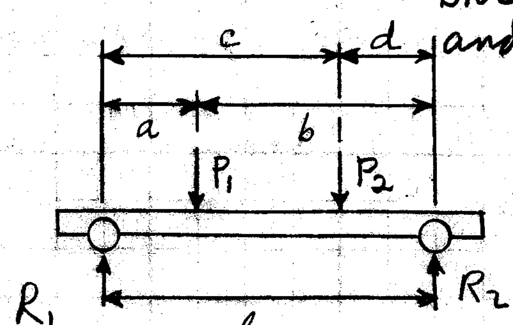
1. A.S.C.E. 40 Lb rail is used. Dimensions are on page 2.
2. A TSB transporter will run on the rails. Transporter weight ≐ 4,000 Lbs. Transporter drawing ME-30673 is in Appendix A.
3. Wheel diameter is 14". Wheel drawing MC-30673 is in Appendix A.
4. The transporter/rail/wheel combination has been used successfully for 3 decades in the neutrino beam line.
5. Rough check of wheel/rail load capacity: Follow procedure on page 3.

$A = 1.875"$ $\quad B = 1.875 - 2(5/16") = 1.25"$ $\quad D = 14"$

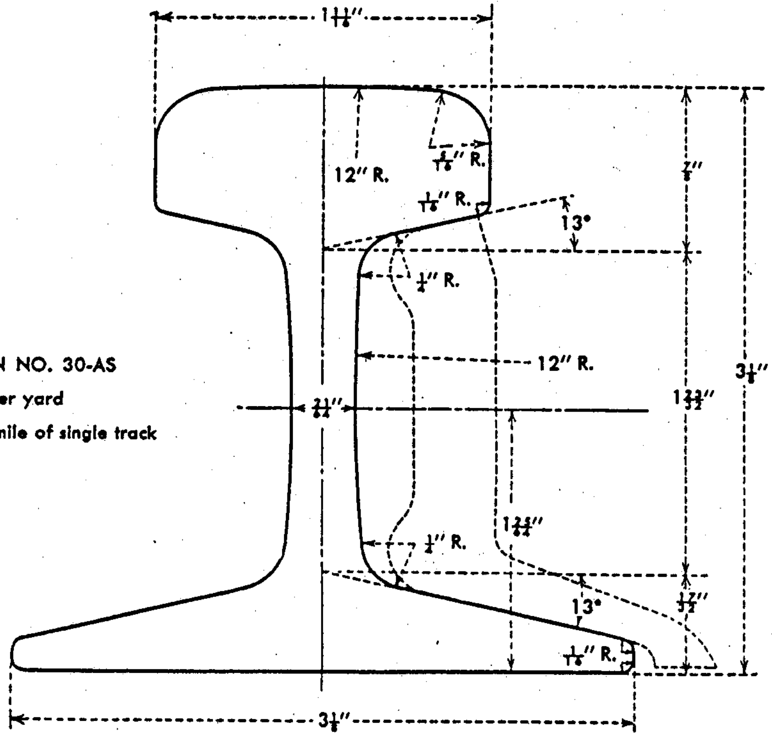
Load distribution: Maximum wheel loading case: 2 Duratek blocks (10 tons each) in a lifting basket (P_1) and an empty lifting basket (P_2)



$\ell = 144"$
$a = 31"$
$b = 144 - 31 = 113"$
$d = 44"$
$c = 144 - 44" = 100"$

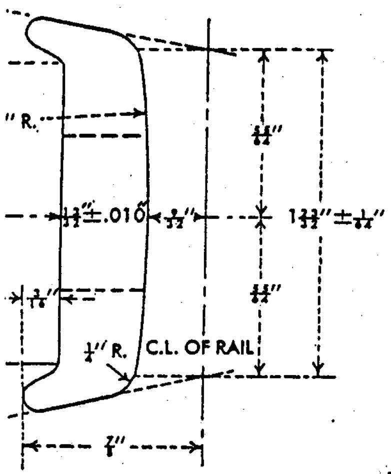
RAI SECTION NO. 30-AS

AMERICAN SOCIETY OF CIVIL ENGINEERS' TYPE

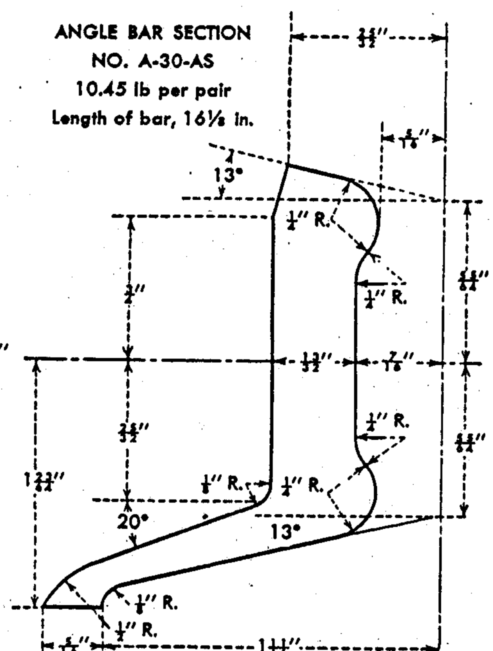


IL SECTION NO. 30-AS
30 lb per yard
t tons per mile of single track

BAR SECTION
NO. J-30
8 lb per pair
of bar, 16⅛ in.

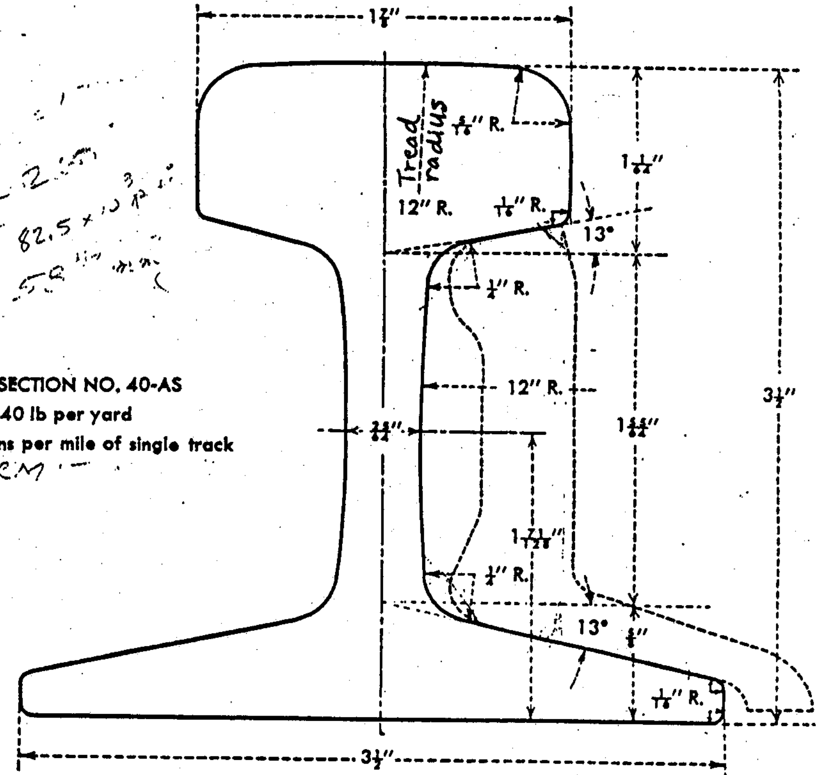


ANGLE BAR SECTION
NO. A-30-AS
10.45 lb per pair
Length of bar, 16⅛ in.



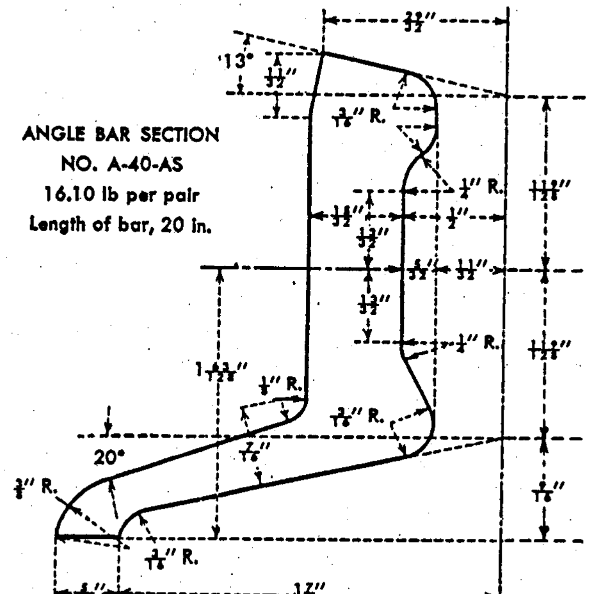
RAIL SECTION NO. 40-AS

AMERICAN SOCIETY OF CIVIL ENGINEERS' TYPE

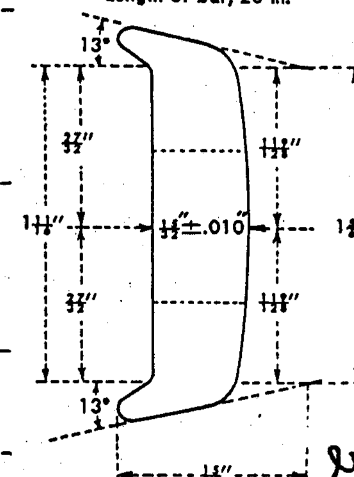


RAIL SECTION NO. 40-AS
40 lb per yard
70.4 net tons per mile of single track

ANGLE BAR SECTION
NO. A-40-AS
16.10 lb per pair
Length of bar, 20 in.



SPLICE BAR SECTION NO. J-40
11.07 lb per pair
Length of bar, 20 in.



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MISCELLANEOUS

Page 3
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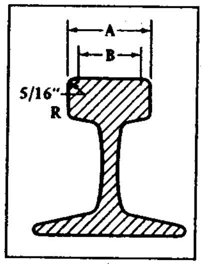
Size of Rail Necessary to Carry a Given Load. — The following formulas may be employed for determining the size of rail and wheel suitable for carrying a given load. Let, A = the width of the head of the rail in inches; B = width of the tread of the rail in inches; C = the wheel-load in pounds; D = the diameter of the wheel in inches.



Then the width of the tread of the rail in inches is found from the formula:

$$B = \frac{C}{1250D} \qquad (1)$$

The width A of the head equals $B + \frac{5}{8}$ inch. The diameter D of the smallest track wheel that will safely carry the load is found from the formula:

$$D = \frac{C}{A \times K} \qquad (2)$$

in which $K = 600$ to 800 for steel castings; $K = 300$ to 400 for cast iron.

As an example, assume that the wheel-load in a given case is 10,000 pounds; the diameter of the wheel is 20 inches; and the material steel casting. Determine the size of rail necessary to carry this load. From Formula (1):

$$B = \frac{10{,}000}{1250 \times 20} = 0.4 \text{ inch.}$$

Hence the width of the rail required equals $0.4 + \frac{5}{8}$ inch = 1.025 inch. Determine also whether a wheel 20 inches in diameter is large enough to safely carry the load. From Formula (2):

$$D = \frac{10{,}000}{1.025 \times 600} = 16\tfrac{1}{4} \text{ inches.}$$

This is the smallest diameter of track wheel that will safely carry the load; hence a 20-inch wheel is ample.

American Railway Engineering Association Formulas. — The American Railway Engineering Association recommends for safe operation of steel cylinders rolling on steel plates that the allowable load p in pounds per inch of length of the cylinder should not exceed the value calculated from the formula

$$p = \frac{\text{y.s.} - 13{,}000}{20{,}000} 600\, d \text{ for diameter } d \text{ less than 25 inches.}$$

This formula is based on steel having a yield strength, y.s., of 32,000 pounds per square inch. For roller or wheel diameters of up to 25 inches, the Hertz stress (contact stress) resulting from the calculated load p will be approximately 76,000 pounds per square inch.

Example: For a 10-inch diameter roller the safe load per inch of roller length is

$$p = \frac{32{,}000 - 13{,}000}{20{,}000} 600 \times 10 = 5700 \text{ lbs per inch of length.}$$

Therefore, to support a 10,000 pound load the roller or wheel would need to be 10,000/5700 = 1.75 inches wide.

Lifting basket weight = 5,000 Lbs maximum

$P_1 = 2(20{,}000 \text{ Lbs}) + 5{,}000 \text{ Lbs} = 45{,}000 \text{ Lbs}$

$P_2 = 5{,}000 \text{ Lbs}$

$R_1|_{P_1} = \dfrac{P_1 b}{l} = 45{,}000 \text{ Lbs} \left(\dfrac{113''}{144''}\right) = 35{,}315 \text{ Lbs}$

$R_1|_{P_2} = \dfrac{P_2 d}{l} = 5{,}000 \text{ Lbs} \left(\dfrac{44''}{144''}\right) = 1{,}530 \text{ Lbs}$

$R_1 = R_1|_{P_1} + R_1|_{P_2} = 36{,}845 \text{ Lbs}$

The 2 wheels at R_1 share R_1 equally,

$\therefore$ Maximum load on 1 wheel $= \dfrac{36{,}845}{2} + \dfrac{4{,}000}{4}$ ← Transporter weight

$= 19{,}425$ Lbs / wheel

Rail capacity with 14"ϕ wheel $\equiv C_{rail} = 1{,}250$ BD

$\therefore C_{rail} = 1{,}250(1.25)(14) = 21{,}875 \text{ Lbs} > 19{,}425 \text{ Lbs}$

OK

Wheel capacity:

K is not known. Calculate a minimum value for K using the design load information on dwg ME-30673.

Minimum wheel loading $= \dfrac{88{,}000 \text{ Lbs}}{4 \text{ wheels}}$

$= 22{,}000$ Lbs / wheel

Back calculate a K-value and compare it to the 600 to 800 range given on page 3.

Wheel capacity = DAK

$$K = \frac{\text{Wheel capacity}}{DA} = \frac{22{,}000}{14(1.875)} \doteq 840$$

$$\frac{840-800}{800} \times 100 = 5\%$$

∴ A minimum wheel capacity of 22,000 Lbs is reasonable.

So, $22{,}000 \frac{\text{Lbs}}{\text{wheel}} > 19{,}425 \frac{\text{Lbs}}{\text{wheel}}$ $\underline{\underline{\text{ok}}}$ ✓

The transporter was load tested with 40 tons.

∴ The minimum test load per wheel was 40/4 = 10 tons = 20,000 Lbs, which is the design wheel loading.

From: "John Voirin" <voirin@fnal.gov>
To: "Andy Stefanik" <stefanik@fnal.gov>
Sent: Tuesday, November 04, 2003 6:36 AM
Subject: Re: Memory Test

A little over 40 ton.

> ----- Original Message -----
> **From:** Andy Stefanik
> **To:** John Voirin
> **Sent:** Monday, November 03, 2003 5:59 PM
> **Subject:** Memory Test
>
> John,
>
> How much weight did you put on the transporter when you tested it for us?
>
> Andy

6. The rails and transporter will be load tested after installation. The test load will be two baskets plus four Duratek blocks plus 4,100 Lbs at axle R_2.

For the load test: $P_1 = 45{,}000$ Lbs (Same as in § 5)

$P_2 = 45{,}000$ Lbs

Geometry is same as in § 5.

- R_1 – Follow § 5.

$R_1|_{P_1} = 35{,}315$ Lbs

$R_1|_{P_2} = 45{,}000 \text{ Lbs} \left(\frac{44}{144}\right) = 13{,}750$ Lbs

$R_1 = 35{,}315 + 13{,}750 = 49{,}065$ Lbs

$\therefore$ Wheel test load $= \frac{49{,}065}{2} + \frac{4{,}000}{4} = \underline{25{,}530 \text{ Lbs}}$

Test load % $= \frac{25{,}530 - 19{,}425}{19{,}425} * 100 \doteq 30\% > 25\%$ $\underline{\underline{\text{OK}}}$

- R_2

$R_2|_{P_1} = \frac{P_1 a}{l} = 45{,}000 \text{ Lbs} \frac{(31'')}{(144'')} = 9{,}690$ Lbs

$R_2|_{P_2} = \frac{P_2 c}{l} = 45{,}000 \text{ Lbs} \left(\frac{100''}{144''}\right) = 31{,}250$ Lbs

$R_2 = 9{,}690 + 31{,}250 = 40{,}940$ Lbs

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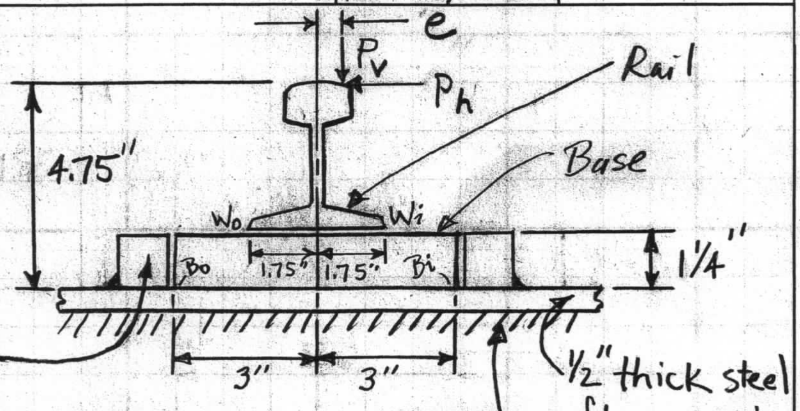
Wheel load = $\frac{40,940 \text{ Lbs}}{2} + \frac{4,000}{4} = 21,470$ Lbs

Must add 4,100 Lbs over the axle at R_2.

$\therefore$ Wheel test load = 21,470 + 4,100 = 25,570 Lbs

Size the track components for the wheel test load of 25,570 Lbs.

7. Base



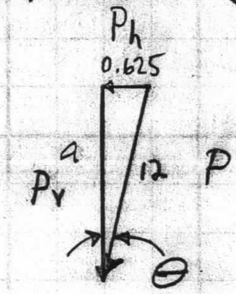
$P_v = 26{,}000$ Lbs (From §6.)

Tread radius = 12"

Offset "e" = $1.875/2 - 5/16 = 0.625$"

o ≡ outboard
i ≡ inboard

- Find the developed side load Ph:



$a = \sqrt{12^2 - 0.625^2} = 11.983713$"

$\tan\theta = \frac{P_h}{P_v}$

$\frac{0.625}{11.983713} = \frac{P_h}{P_v}$

$P_h = 0.052\,P_v = 0.052(26{,}000)$

$= 1{,}352$ Lbs

- Check overturning moment at pivot point "B_o":

$P_v\left(\frac{6}{2}+e\right) \overset{?}{>} P_h\,(\text{height})$

$26{,}000\text{ Lbs}\,(3+0.625)\text{ in} \overset{?}{>} 1{,}352\text{ Lbs}\,(4.75\text{ in})$

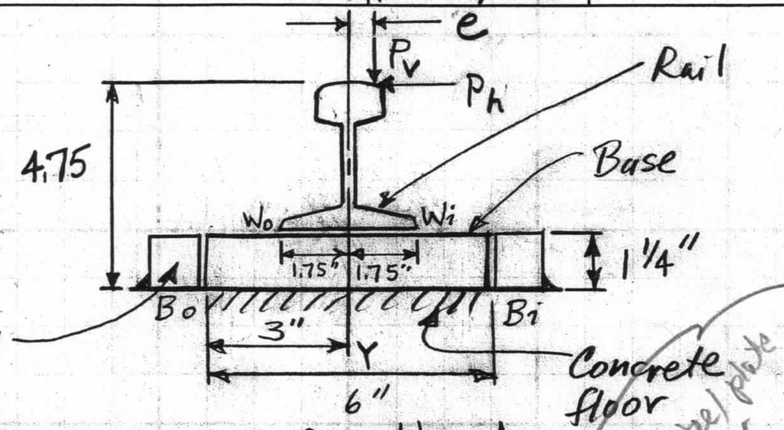
$94{,}250$ in-Lbs $> 6{,}422$ in Lbs <u>OK</u>

- Check overturning moment at pivot point "Wo":

$26{,}000\,(3.5/2 + 0.625) \overset{?}{>} 1{,}352\,(3.5)$

$61{,}750$ in-Lbs $> 4{,}732$ in-Lbs <u>OK</u>

7. Base



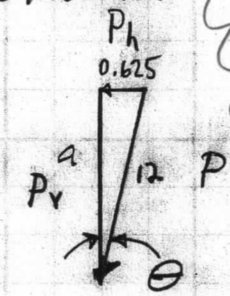
$P_v = 26{,}000$ Lbs (From §6.)

Tread radius = 12"

Offset "e" = $1.875/2 - 5/16 = 0.625$"

O ≡ outboard
i ≡ inboard

- Find the developed side load Ph:



$$a = \sqrt{12^2 - 0.625^2} = 11.983713''$$

$$\tan\theta = \frac{P_h}{P_v}$$

$$\frac{0.625}{11.983713} = \frac{P_h}{P_v}$$

$$P_h = 0.052\,P_v = 0.052(26{,}000)$$

$$= 1{,}352 \text{ Lbs}$$

- Check overturning moment at pivot point "B_o":

$$P_v\left(\tfrac{6}{2}+e\right) \overset{?}{>} P_h\,(\text{height})$$

$$26{,}000 \text{ Lbs } (3+0.625)\text{in} \overset{?}{>} 1{,}352 \text{ Lbs } (4.75 \text{ in})$$

$$94{,}250 \text{ in-Lbs} > 6{,}422 \text{ in Lbs} \quad \underline{\underline{OK}}$$

- Check overturning moment at pivot point "W_o":

$$26{,}000\,(3.5/2 + 0.625) \overset{?}{>} 1{,}352(3.5)$$

$$61{,}750 \text{ in-Lbs} > 4{,}732 \text{ in-Lbs} \quad \underline{\underline{OK}}$$

SUBJECT: shore/pylon/for andy pictures

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- $\Sigma F_{vertical} = 0$

$$-P_V + R_V = 0$$

$$R_V = P_V$$

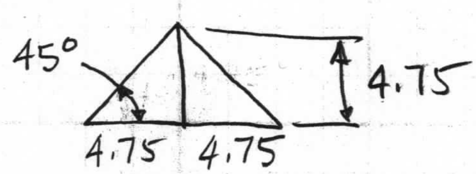
R_V is distributed along the base plate.

- Base plate bearing pressure on the concrete floor

Assume P_V spreads out longitudinally (along the rail) at a 45° angle starting at the top of the rail and laterally at a 45° angle starting at the outer edges of the 3½" wide rail.

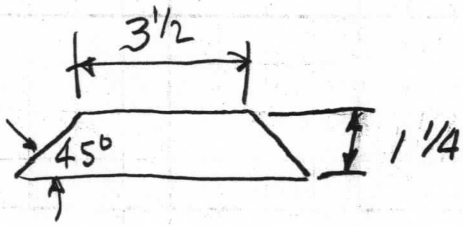
Bearing area dimension along the rail:

$= 2(4.75) = 9.5''$



Bearing area dimension lateral to the rail:

$= 3\frac{1}{2} + 2(1\frac{1}{4}) = 6''$



Bearing area $= 9.5 \times 6$

$= 57\ in^2$

Applied concrete bearing pressure $= \dfrac{26{,}000 \text{ Lbs}}{57 \text{ in}^2} = 456 \text{ psi}$

Multiply by ACI load factor: $456(1.4) = 640$ psi ← Load Factor Applied

Concrete design bearing strength ≡ CDBS:

$f'_c = 4{,}000$ psi per Elaine McCluskey in FESS.

$\text{CDBS} = 0.85\,\phi\, f'_c\, A_1 \sqrt{\dfrac{A_2}{A_1}} \rightarrow$ Limit $\sqrt{\dfrac{A_2}{A_1}}$ to 2 max. See App. B.

$\phi = 0.65$

$A_1 = 57 \text{ in}^2$

$\sqrt{\dfrac{A_2}{A_1}} \rightarrow$ Concrete depth is much > than 2".

The bearing load spreads out 2:1 in the concrete. 2" of depth adds 8" to each dimension of the surface bearing area.

$A_2 = (9.5+8)(6+8) = 245 \text{ in}^2$

$\sqrt{\dfrac{A_2}{A_1}} = \sqrt{\dfrac{245}{57}} = 2$

$\therefore \text{CDBS} = 0.85(0.65)(4{,}000)(57)(2) = 251{,}940 \text{ Lbs}$

Allowable concrete bearing pressure $= \dfrac{251{,}940 \text{ Lbs}}{57 \text{ in}^2}$

$= 4{,}420 \text{ psi} > 640 \text{ psi}$

OK

8. Weld between rail and base; Weld pitch = Key pitch = 24"

The rail will be welded on both the inboard and outboard sides at each weld location.

• Lateral load = P_h = 1,352 Lbs

As seen in § 7, there is no vertical load.

Weld spec: 1/4" equal leg fillet, E7018

$$\text{Weld load capacity} = \frac{\frac{1}{4}(0.707)\,\cancel{in} \times 1\,\cancel{in}}{\text{inch of weld}} \times 21{,}000\ \frac{\text{Lbs}}{\cancel{in^2}}$$

$$= 3{,}700\ \text{Lbs/in of weld}$$

Use 2" of weld on each side of the rail at each weld location.

2 (2") = 4 inches of weld

$$4\ \text{in}\ \cancel{\text{of weld}} \times 3{,}700\ \frac{\text{Lbs}}{\cancel{\text{in of weld}}} = 14{,}800\ \text{Lbs} > 1{,}352\ \text{Lbs}$$

Note: This is only a check check of the weld. Weld is sized in the next calc.

• Weld pitch

Weld pitch = 24" = Key pitch

+ Check deflection between the welds, Model as a simply supported beam.

$$\underset{@\text{center}}{\Delta_{max}} = \frac{P l^3}{48 E I}$$

$$I_{yy} = \cancel{I_{yy\,rail}}^{\text{neglect}} + I_{yy\,base} = \text{thickness}(\text{length})^3/12$$

$$= 1.25\,in\,(6\,in)^3/12 = 22.5\,in^4$$

$$\underset{@center}{\Delta max} = \frac{1{,}352\,\cancel{Lbs}\,(24)^3\,in^{\cancel{3}}}{48\,(29{,}000{,}000\,\frac{\cancel{Lbs}}{\cancel{in^2}})(22.5\,\cancel{in^4})} = 0.0006'' \text{ ok}$$

\+ Check the stress. Model as a simply supported beam.

$$M_{max} = \frac{Pl}{4} = \frac{1{,}352\,Lbs\,(24\,in)}{4} = 8{,}112\ in\text{-}Lbs$$

$$f_b = \frac{Mc}{I} = \frac{8{,}112\,\cancel{in}\text{-}lbs\,(6/2)\,\cancel{in}}{22.5\,in^{\cancel{4}\,2}} = 1082\ psi$$

Base material is M1020. $F_y = 30{,}000$ psi

$F_b \equiv$ allowable bending stress $= 0.6\,(30{,}000) = 18{,}000$ psi
$> 1{,}082$ psi ok

$F_v \equiv$ allowable shear stress $= 0.4\,(30{,}000) = 12{,}000$ psi

$$f_v = \frac{1{,}352\,Lbs}{(1.25)(6)\,in^2} = 180\ psi < 12{,}000\ psi \text{ ok}$$

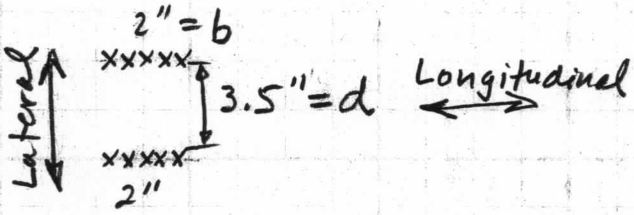
+ A resistive moment will develop in the welds. Model the rail as a beam with fixed ends to calculate a maximum stress value.

$$M_{max} = \frac{Pl}{8} = \frac{1{,}352 \text{ Lbs}(24 \text{ in})}{8} = 4{,}056 \text{ in-Lbs}$$

(@ center & ends)

+ Treat the weld as a line.

The weld group is subjected to twisting.



$$J_w = \frac{b^3 + 3bd^2}{6} = \frac{2^3 \text{ in}^3 + 3(2)(3.5)^2 \text{ in}^3}{6} = 13.6 \text{ in}^3$$

$$f_{t_{long}} = \frac{Tc_{long}}{J_w} = \frac{4{,}056 \cancel{\text{in}}\text{-Lbs}\,(3.5/2)\,\cancel{\text{in}}}{13.6 \text{ in}^3} = 525 \text{ Lbs/in of weld}$$

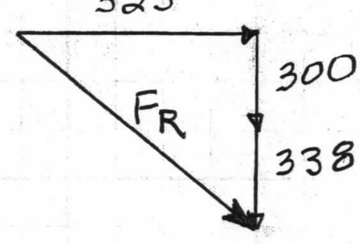
$$f_{t_{lateral}} = \frac{Tc_{lateral}}{J_w} = \frac{4{,}056(2/2)}{13.6} = 300 \text{ Lbs/in of weld}$$

f_v ≡ lateral shear due to 1,352 Lb side load.

$$f_v = \frac{1{,}352 \text{ Lbs}}{2(2 \text{ in})} = 338 \text{ Lbs/in of weld}$$

Combine the loads:

Resultant force on the weld ≡ F_R



$$F_R = \sqrt{(525)^2 + (300 + 338)^2}$$

$$= 826 \frac{\text{Lbs}}{\text{in of weld}} < 3{,}700 \frac{\text{Lbs}}{\text{in of weld}}$$ (page 11) OK

+ The keys are 3" long with 2" of weld.

$b = 2''$ $d = 6 + 1\frac{1}{4} + 1\frac{1}{4} = 8.5''$

$$J_w = \frac{2^3 + 3(2)(8.5)^2}{6} = 73.6 \text{ in}^3$$

Since $73.6 \text{ in}^3 > 13.6 \text{ in}^3$ (J_w for the rail welds), the key welds are <u>OK</u>.

9. What if key spacing is 72" instead of 24"?

Weld pitch = 24". Key pitch = 72". Follow § 7.

+ Check deflection between the keys. Model as a simply supported beam.

$$\Delta_{max\ @center} = \frac{Pl^3}{48EI} = \frac{1,352(72)^3}{48(29,000,000)(22.5\ in^4)} = 0.016'' \ \underline{\underline{OK}}$$

+ Check the stress. Model as a simply supported beam.

$$M_{max} = \frac{Pl}{4} = \frac{1,352(72)}{4} = 24,336 \text{ in-Lbs}$$

$$f_b = \frac{Mc}{I} = \frac{24,336(6/2)}{22.5} = 3,245 \text{ psi} < 18,000 \text{ psi} \ \underline{\underline{OK}}$$

$$f_v = \frac{P_h}{A_c} = \frac{1,352 \text{ Lbs}}{1.25(6) \text{ in}^2} = 180 \text{ psi} < 12,000 \text{ psi} \ \underline{\underline{OK}}$$

+ Resistive moment

$$M_{max} = \frac{Pl}{8} = \frac{1,352(72)}{8} = 12,168 \text{ in-Lbs}$$

SUBJECT

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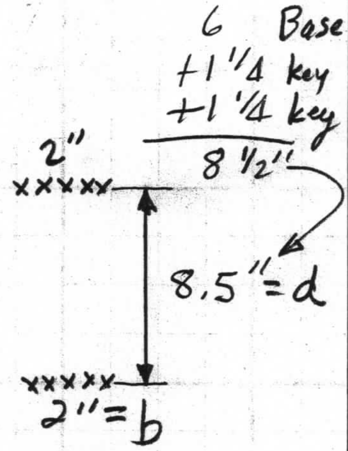
+ Treat the weld as a line.

The weld group is subjected to twisting.



$$J_W = \frac{b^3 + 3bd^2}{6}$$

Longitudinal

Lateral

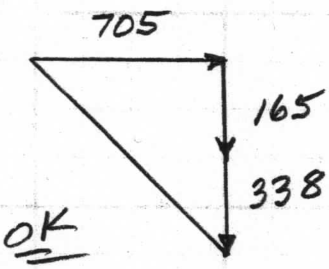
$$= \frac{2^3 + 3(2)(8.5)^2}{6} = 73.6 \text{ in}^3$$

$$f_{t\,long} = \frac{T\,c_{long}}{J_W} = \frac{12{,}168 \text{ in-Lbs } (8.5/2) \text{ in}}{73.6 \text{ in}^3} = 705 \frac{\text{Lbs}}{\text{in of weld}}$$

$$f_{t\,lateral} = \frac{T\,c_{lateral}}{J_W} = \frac{12{,}168\,(2/2)}{73.6} = 165 \frac{\text{Lbs}}{\text{in of weld}}$$

$$f_v = \frac{1{,}352 \text{ Lbs}}{2(2 \text{ in})} = 338 \frac{\text{Lbs}}{\text{in of weld}}$$

Combine the loads

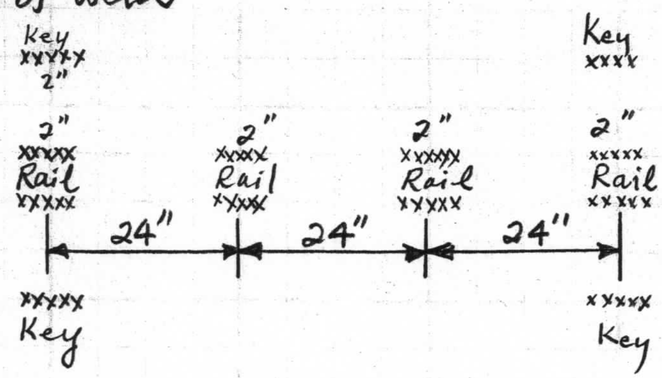


$$F_R = \sqrt{(705)^2 + (165 + 338)^2}$$

$$= 870 \frac{\text{Lbs}}{\text{in of weld}} < 3{,}700 \frac{\text{Lbs}}{\text{in of weld}} \quad \underline{\underline{OK}}$$

+ Horizontal shear force

Check the horizontal shear force that developes on the two weld groups between the keys.



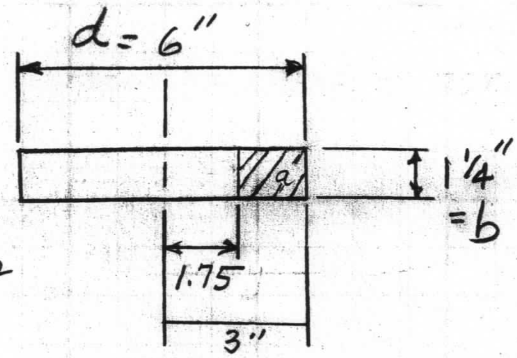
FERMILAB
ENGINEERING NOTE

SECTION	PROJECT	SERIAL-CATEGORY	PAGE 17

SUBJECT

NAME Ams

DATE Nov 6, 2003 | REVISION DATE



$F_h \equiv$ horizontal shear force

$F_h = \dfrac{VQ}{I} \qquad Q = ay$

$a = 1.25\,in\,(3-1.75)\,in = 1.5625\,in^2$

$y = (3+1.75)/2 = 2.375\,in$

$Q = ay = (1.5625\,in^2)(2.375\,in) = 3.71\,in^3$

$I = \dfrac{bd^3}{12} = \dfrac{1.25\,in\,(6)^3\,in^3}{12} = 22.5\,in^4$

$V = P_h = 1{,}352$ Lbs

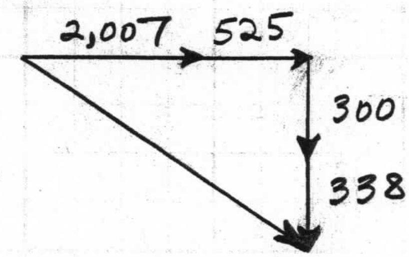
$F_h = \dfrac{1{,}352\,Lbs\,(3.71\,\cancel{in^3})}{22.5\,in^{\cancel{4}}} = 223\,\dfrac{Lbs}{in}$

Force per inch of weld $= \dfrac{223\,\frac{Lbs}{\cancel{in}} \times 72\,\cancel{in}}{4\,\cancel{welds} \times 2\,\frac{in}{\cancel{weld}}}$

$= 2{,}007$ Lbs / in of weld

Recalculate the resultant force on the rail weld.

From the bottom of page 13:



$F_R = \sqrt{(2{,}007+525)^2 + (300+338)^2}$

$= 2{,}611\,\dfrac{Lbs}{\text{in of weld}} < 3{,}700\,\dfrac{Lbs}{\text{in of weld}}$ $\underline{\underline{OK}}$

FERMILAB
ENGINEERING NOTE

SECTION | PROJECT | SERIAL-CATEGORY | PAGE 18

SUBJECT

NAME Ams

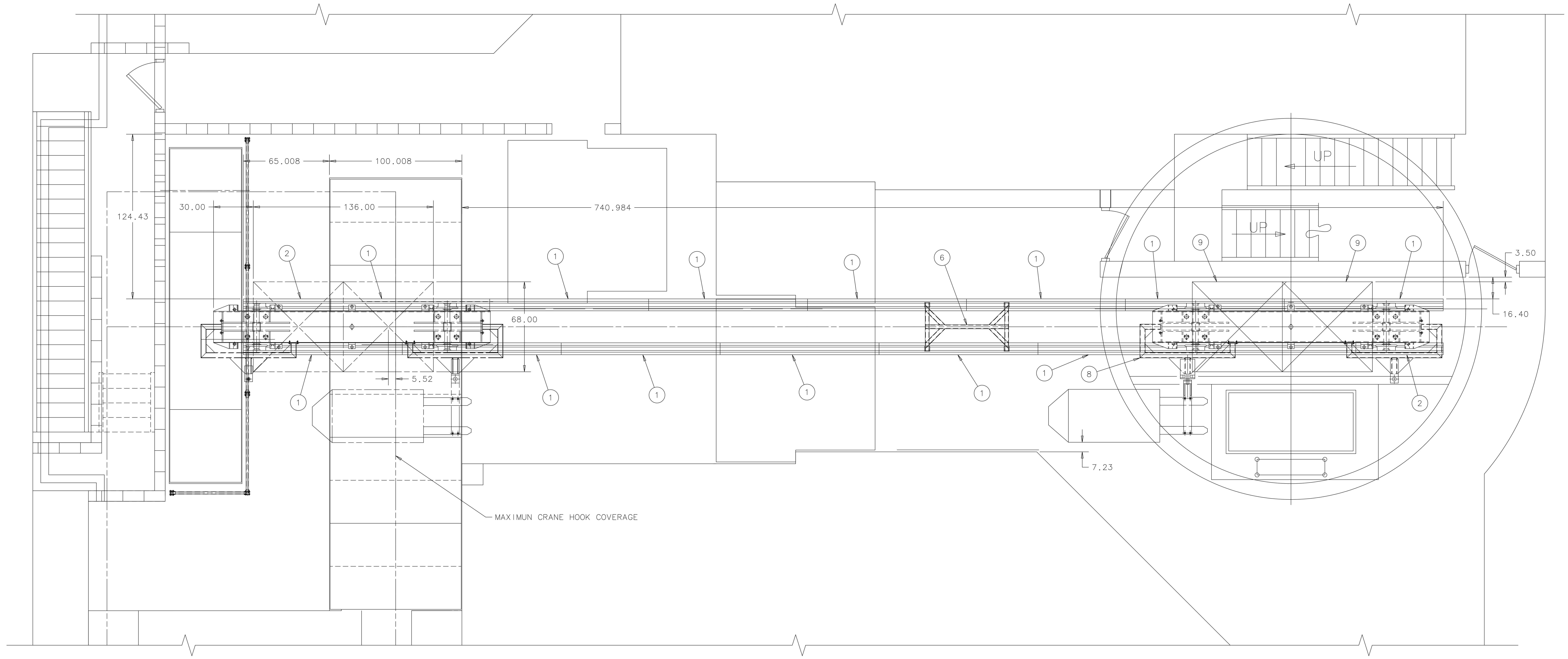
DATE Nov 6, 2003 REVISION DATE

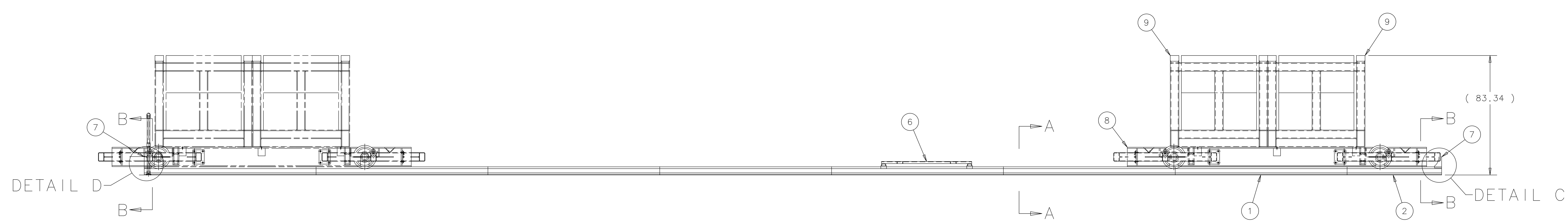
10. Conclusions regarding welds:

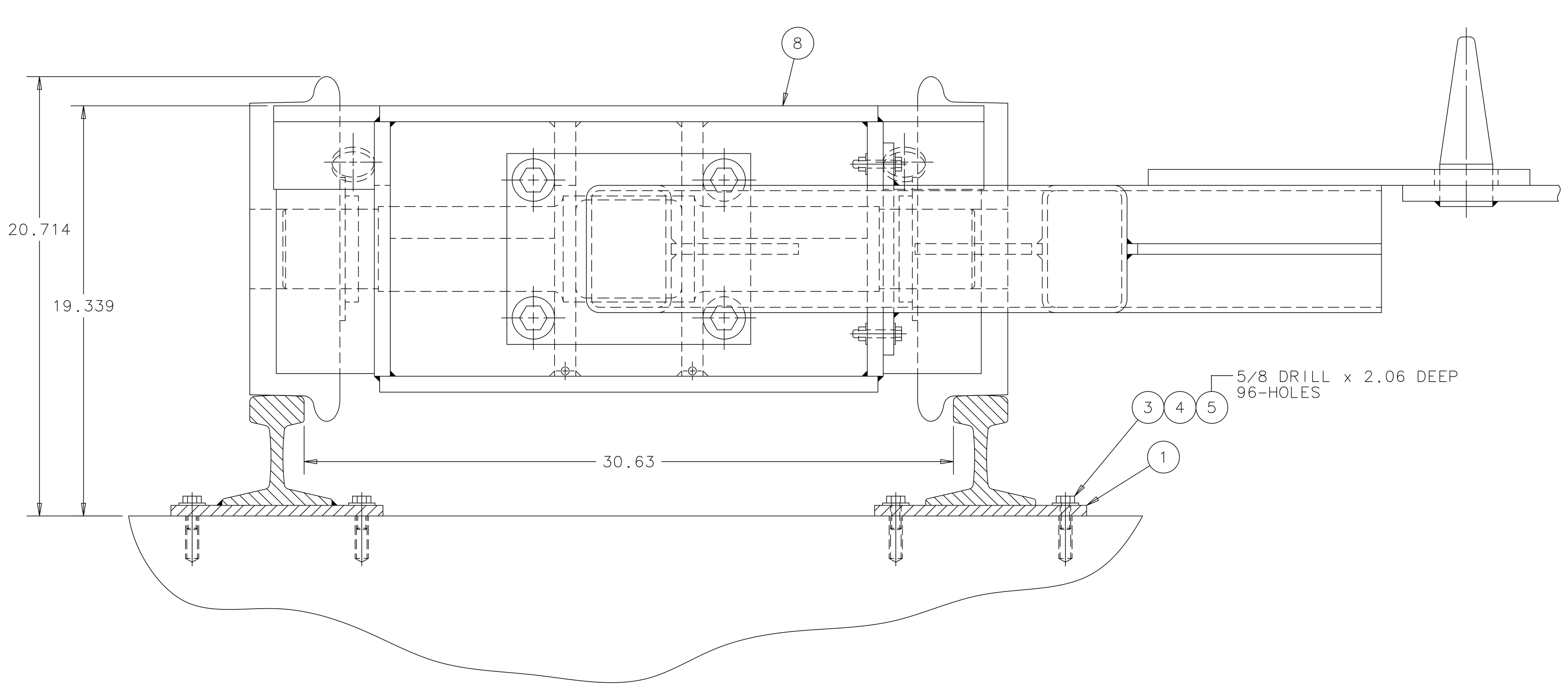
Rail weld: 1/4" equal leg fillet, 2" minimum length, 24" pitch maximum, E7018.

Key: Length = 3".
1/4" equal leg fillet, 2" minimum length, 72" pitch maximum, E7018, wrap weld around ends of key @ 1/4".

Appendix A

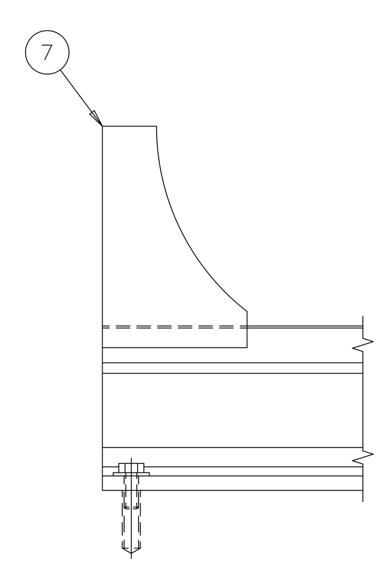






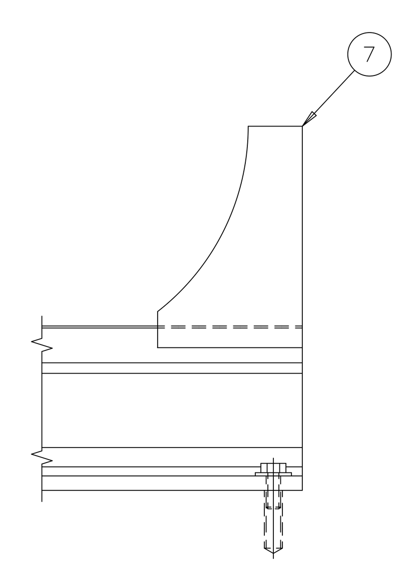
SECTION A-A

SCALE: 1/4



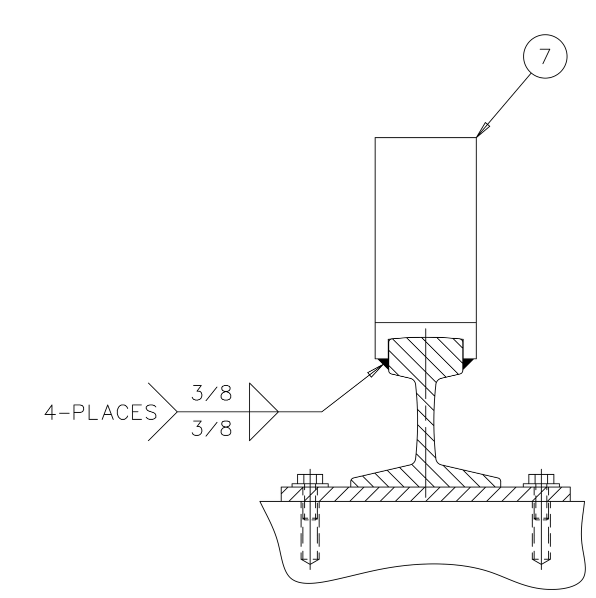
DETAIL D

SCALE: 1/2



DETAIL C

SCALE: 1/2



SECTION B-B

SCALE: 1/2

NOTES:

1. WHEN INSTALLING TRANSPORT RAIL ASSEMBLIES ITEMS 1 AND 2, USE TRANSPORT RAIL GAUGE ITEM 6 TO MAINTAIN PROPER DISTANCE BETWEEN RAILS.

ITEM	PART NO.	DESCRIPTION OR SIZE	QTY.
9	MD-4277	SHIELDING BLOCK BASKET	2
8	MD-427682	TRANSPORT & PUSH-BAR ASSEMBLY	1
7	MC-427728	RAIL WHEEL STOP	4
6	MD-427718	TRANSPORT RAIL GAUGE	1
5	COML	HDI-L 1/2" FLUSH DROP-IN ANCHOR HILTI ITEM NO. 00247816	96
4	COML	STD. FLAT WASHER, 1/2 NOM., S.S. 304	96
3	COML	HHCS, 1/2-13 UNC x 1.25 LG., S.S. 304	96
2	MC-427716	TRANSPORT RAIL ASSEMBLY, SHORT	2
1	MC-427715	TRANSPORT RAIL ASSEMBLY, LONG	14

PARTS LIST

REV	DESCRIPTION	DRAWN / APPROVED	DATE / DATE

UNLESS OTHERWISE SPECIFIED

.XX	.XXX	ANGLES
±	±	± 1°

1. BREAK ALL SHARP EDGES .015 MAX.
2. DO NOT SCALE DRAWING.
3. DIMENSIONS BASED UPON ANSI Y14.5M-1982
4. ALL DIMENSIONS ARE INCHES
5. MAX. ALL MACH. SURFACES 125

ORIGINATOR	A.STEFANIK	19-FEB-2003
DRAWN	W.CYKO	19-FEB-2003
CHECKED		
APPROVED		

USED ON

MATERIAL: SEE ABOVE PARTS LIST

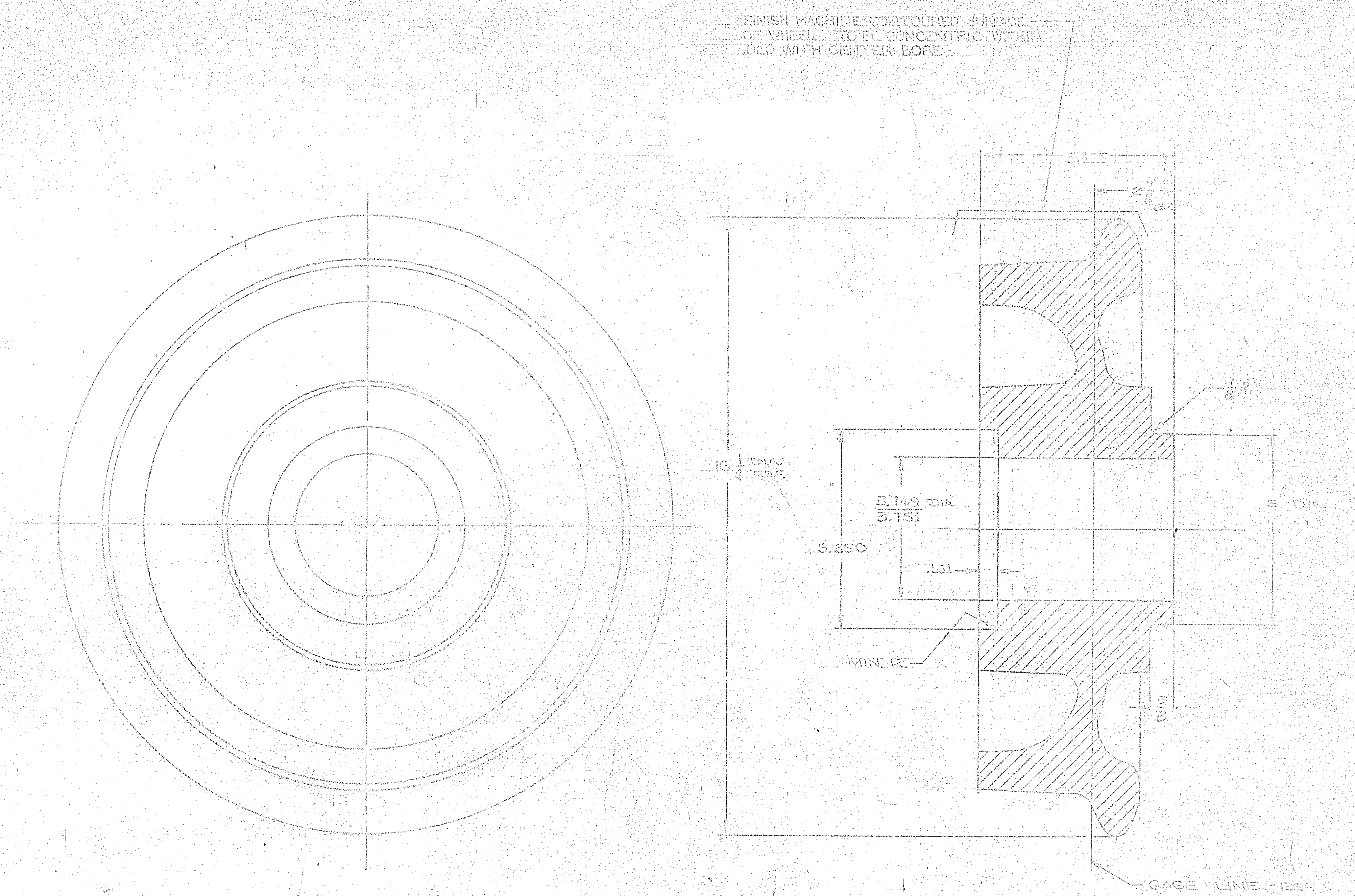
FERMI NATIONAL ACCELERATOR LABORATORY
UNITED STATES DEPARTMENT OF ENERGY

PPD/MECHANICAL DEPARTMENT
NUMI TARGET HALL
TRANSPORT AND RAIL LAYOUT

SCALE	DRAWING NUMBER	SHEET	REV
1/32 & AS NOTED	8875.126-ME-427729	1 OF 1	

CREATED WITH : Ideas9m3 GROUP: PPD/MECHANICAL DEPARTMENT

Created: 14:31:36 on 19-05-11 (D-M-Y) By: friend State: 1-INITIAL



△A 1. FINISH : PRIME WITH RUSTOLEUM-FINAL 2 COATS WITH #H-5 BLACK
COLORS ARE FROM RUSTOLEUM'S 'NEW COLOR HORIZONS SYSTEM - DO NOT PAINT HOLES
NOTE

A	ADDED NOTE 1	RJL	3-7-78

UNLESS OTHERWISE SPECIFIED
TOLERANCES
FRACTIONS ± 1/16 | DECIMALS ± .030 | ANGLES ± 1/2°
1. BREAK ALL SHARP EDGES 1/64 MAX.
2. DIMENSIONING IN ACCORD WITH USASI Y14.5-1966
125/ MAX. ALL MACHINED SURFACES

1210 ME-30673

ORIGINATED		DATE
DRAWN	R.L.S.	
CHECKED	R. HAAS	
PROJ. ENGINEER		
APPROVED		

	41965	16" WHEEL – RAIL CASTING	
ITEM NO.	QTY. REQ'D	IDENT. No.	DESCRIPTION

LIST OF MATERIALS OR PARTS

NATIONAL ACCELERATOR LABORATORY
U.S. ATOMIC ENERGY COMMISSION

TITLE: RAILROAD WHEEL
CARRIAGE TRANSPORT

SCALE 1/4 | SHEET | DRAWING No. 1210-MC-30679 | REV. A

FA 8/77

WT. OF TRANSPORTER 3896.65
WT. OF BED PLATE 3823.05
7719.70 Lbs.
WT. OF PAYLOAD 80000.00
TOTAL WT. 87719.70 Lbs.
NOTE: WT. OF UNDER CARRIAGE NOT REPRESENTED

NATIONAL RUBBER CUSHIONING STACK:
FREE HEIGHT 3.719
COMP. HT. W./TRANS. & BED 3.245
COMP. HT. W./ PAYLOAD (40 TON) 2.555
COMP. HT. TRANSPORTER ONLY 3.419

REV.	DESCRIPTION	DRAWN / APPD.	DATE
A	[illegible]		
B	[illegible]		
C	ADDED NOTES ADDED DIMENSIONS	J.B.S.	
D	ADDED TABLE A	RWR	2-25-80

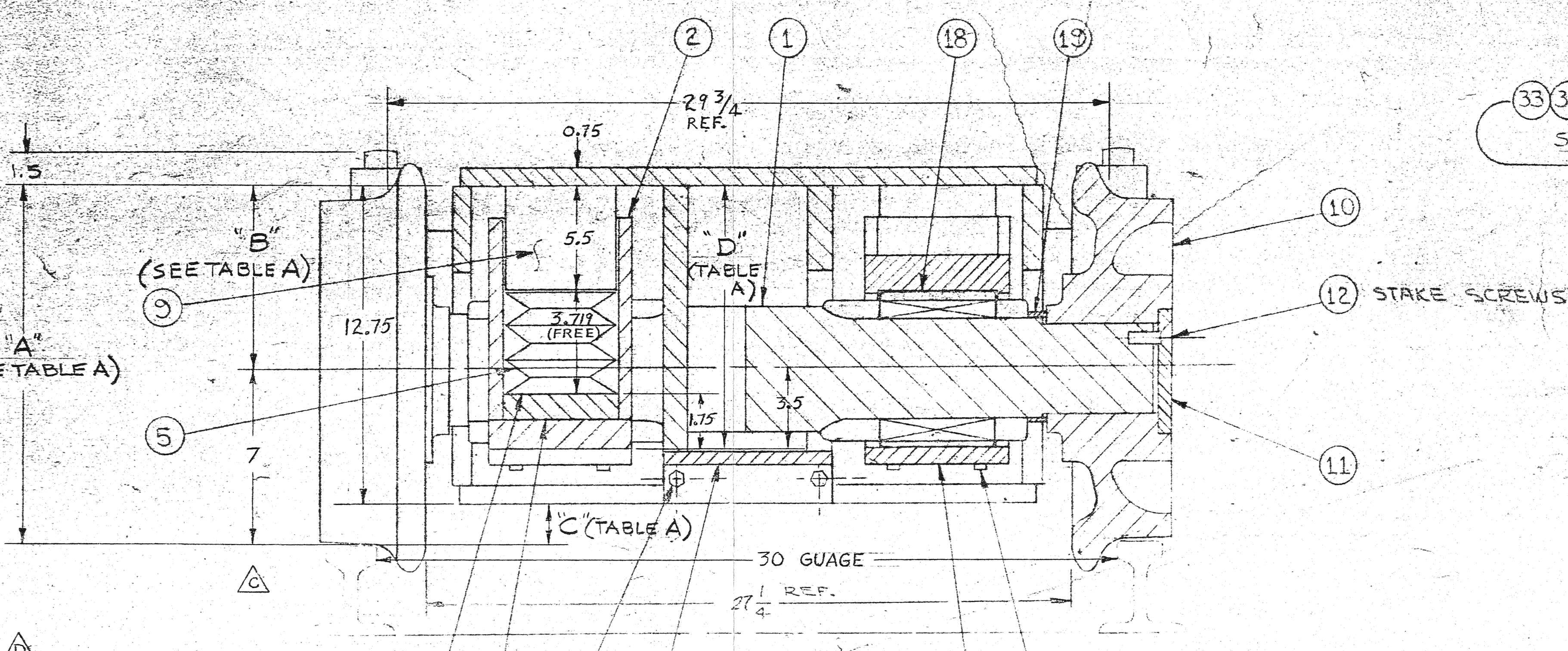
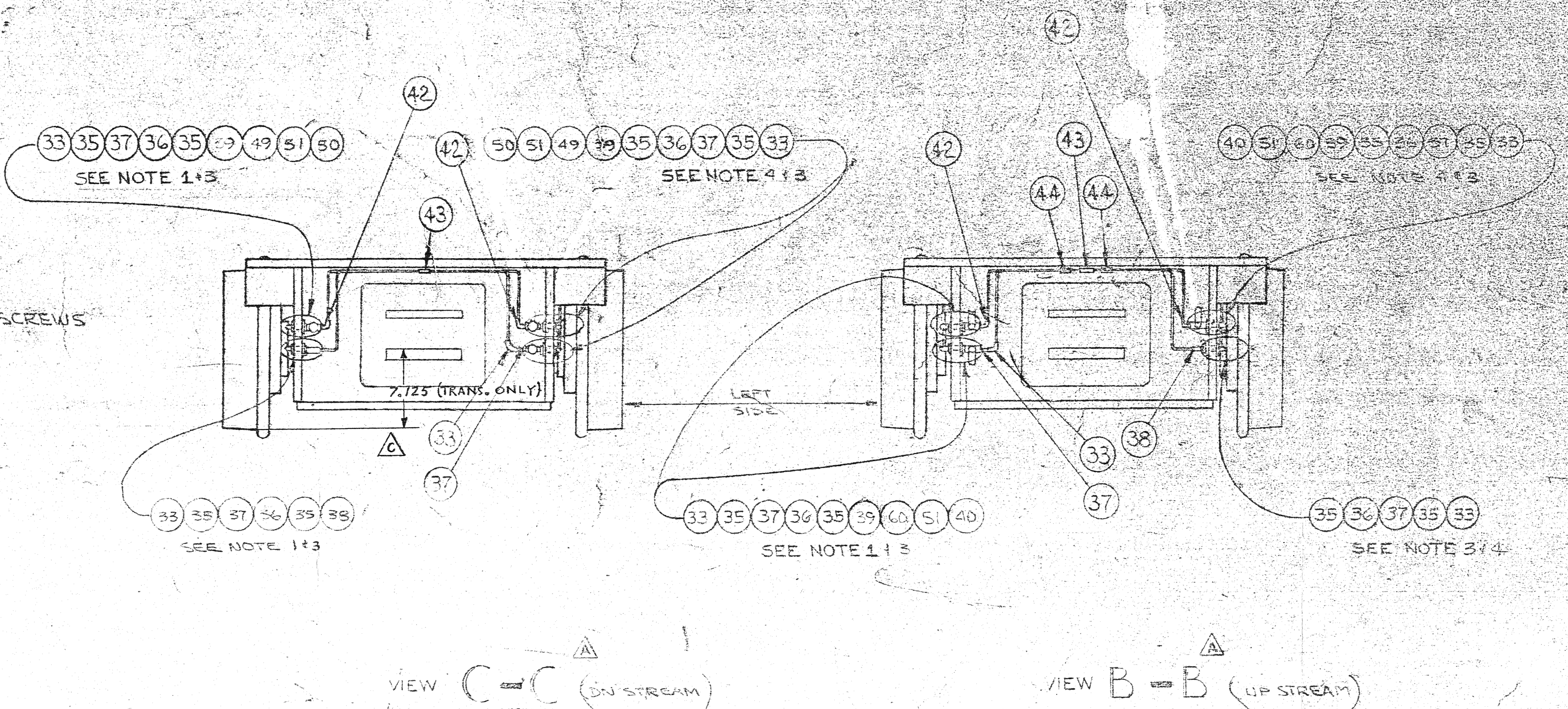
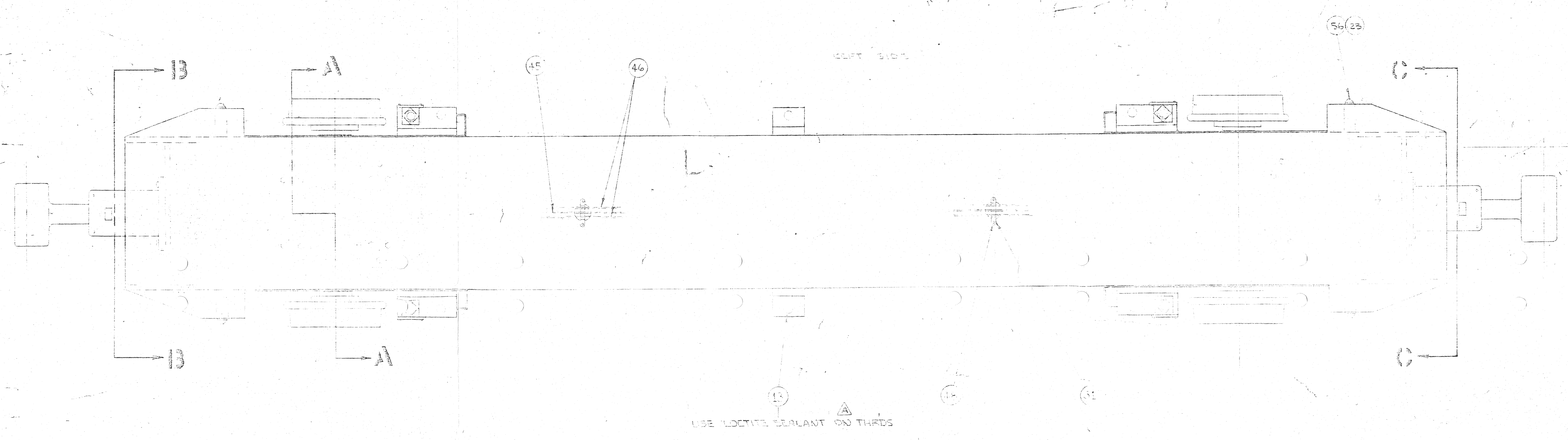


TABLE A

DIM "A"	DIM "B"	DIM "C"	DIM "D"	
14.169	7.169	1.419	10.669	COMPR HGT-TRANSPORTER ONLY
13.995	6.995	1.245	10.495	" " -TRANSPORTER & BPLT
13.305	6.305	.555	9.805	" " -40 TON PAYLOAD





ITEM NO.	PART NO.	DESCRIPTION	QTY. REQ.
62		SCREW 3/8-16 x 3/4 HEX. HD. STL.	8
61	1210-MC-32455	AXLE SUSPENSION RETAINER	[illegible]
60		PIPE NIPPLE PARKER-HANNIFIN # 3/8 FF-SS	[illegible]
59		SWAGELOK PLUG #600-P [illegible]	[illegible]
56	1210-MC-31314	YAW CYL. STOP MOD. ASS'Y	2
55	1210-MC-31313	VERT. CYL. MOD. ASS'Y	4
54		1/4-20 UNC x 1/2 LG. FERRY CAP C'BORE SCW - STL.	4
53		LOCKWASHER 1 1/4 SPLIT STL.	8
52		CLAMP-LOOP TYPE TA MFG. CORP. #TA120-R6HR	[illegible]
51		PIPE CONN. PARKER-HANNIFIN #3/8 [illegible]-SS	3
50		HOSE ENERPAC # HC921	3
49		3/8 PIPE NIPPLE 3" LG. 316 ST. STL.	3
48		SCREW 1/4-20UNC PAN HD. x 1/2 LG.	4
47		TYPE 304, 3/8 O.D. x .065 WALL ST. STL. TUBING SEAMLESS ANNEALED CUT TO FIT	AS REQ'D
46		TYPE 304, 3/8 O.D. x .065 WALL ST. STL. TUBING SEAMLESS ANNEALED 208 IN LG.	2
45		TYPE 304, 3/8 O.D. x .065 WALL ST. STL. TUBING SEAMLESS ANNEALED 200 IN LG.	1
44		UNION SWAGELOK #600-6-316	2
43		UNION TEE SWAGELOK #600-3-316	2
42		MALE ELBOW SWAGELOK #600-2-6-316	4
41		3/8 HI-FLOW MALE COUPLER ENERPAC #CH-604	4
40		3/8 HI-FLOW FEMALE COUPLER ENERPAC # CR-400	3
39		FEMALE PIPE TEE PARKER-HANNIFIN #3/8 MMO-SS	6
38		FEMALE CONN. SWAGELOK #600-7-6-316	2
37		3/8 PIPE NIPPLE 2" LG. 316 ST. STL.	10
36	1210-MB-[illegible]	WASHER	8
35		LOCKNUT 3/8 RIGID CONDUIT	16
34		MALE CONN. SWAGELOK #600-1-6-316	8
33		FEMALE ELBOW SWAGELOK #600-8-6-316	10
[illegible]	[illegible]	[illegible]	
[illegible]	1210-M[illegible]	COUPLING PIN	[illegible]
28		STOP 1/2 DIA. STOCK [illegible] STL.	4
27		COUPLING-WILLISON #37036A	2
26		BOLT 3/4-10UNC x 2 LG. HEX. HD. STL.	8
25	1210-MD-3068[illegible]	COUPLING HOUSING	2
23	1210-MB-30686	CYLINDER HEAD-"YAW"	4
22	1210-MC-30685	CYLINDER HEAD	4
21		ENERPAC 10 TON HYD. CYL. #HC102 ORDER WITHOUT COUPLER	2
[illegible]			
19	1210-MB-30684	AXLE SPACER	4
18	1210-MD-32454	TIMKIN BEARING CORP. CLASS B #HM120848-90012	4
17		BOLT 1/2-20UNF x 1 1/2 LG. HEX. HD. STL.	24
16	1210-ME-30683	SUPPORT BEAM	1
15		SCREW 1/4-20UNC x 3/4 LG. HEX. HD. STL.	16
14	1210-MC-30682	TIE BAR	8
13	1210-MB-30681	LOAD BUTTON	6
12		~~SCREW 5/8-11UNC x 1 1/4 FLAT HD STL.~~	12
11	1210-MB-30680	AXLE END CAP	[illegible]
10	1210-MC-30679	RAILROAD WHEEL	[illegible]
9	1210-MC-30678	SUSPENSION SUPPORT	8
8		LOCKWASHER 1/2 SPLIT STL.	16
7		BOLT 1/2-13UNC x 1 1/8 LG. HEX. HD. STL.	16
6	1210-MC-30677	SUSPENSION UNDERSILL	4
5		RUBBER CUSHIONING PAD NAT. CASTINGS DIV. MI-240 SERIES (#36541)	16
4		RUBBER CUSHIONING PAD NAT. CASTINGS DIV. MI-240 SERIES (#36542)	16
3	1210-MB-30676	SUSPENSION SHIM	8
2	1210-MD-30675	SUSPENSION HOUSING	4
1	1210-MC-30674	AXLE	2

PARTS LIST

UNLESS OTHERWISE SPECIFIED: FRACTIONS ± 1/64, DECIMALS ± .01, ANGLES ± 1/2°
1. BREAK ALL SHARP EDGES 1/64 MAX.
2. DO NOT SCALE DWG.
3. DIMENSIONING IN ACCORD WITH MIL-STD-8 (?)
4. MAX. ALL MACHINED SURFACES

ORIGINATOR
DRAWN KGL
CHECKED HJS
APPROVED J.R. LINDBERG
APPROVED
USED ON
MATERIAL

NATIONAL ACCELERATOR LABORATORY
U.S. ATOMIC ENERGY COMMISSION

TRANSPORTER ASSY

1210-ME-30673 D

EST. WEIGHT — ≈ 6000 lbs.

NOTE
1. PARTS ARE LISTED IN SEQUENCE OF ASSEMBLY. ORDER IS AS READ FROM LEFT TO RIGHT.
2. [illegible] WITH TEFLON TAPE [illegible]
3. [illegible]

1210-ME-30673 D

Appendix B

CODE

COMMENTARY

Appendix B

Source:
Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)

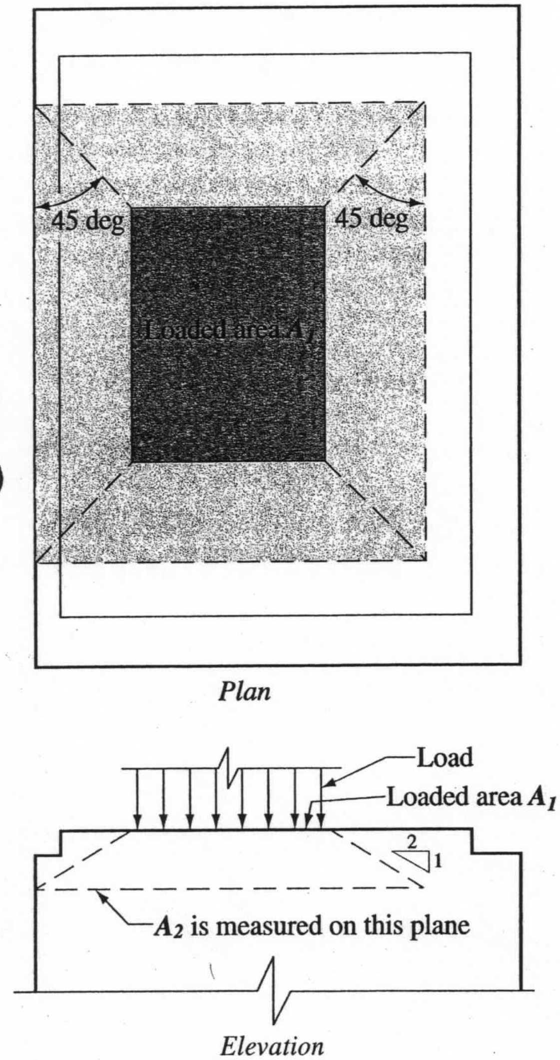


Fig. R10.17—Application of frustum to find A_2 *in stepped or sloped supports*

10.17 — Bearing strength

10.17.1 — Design bearing strength of concrete shall not exceed ϕ **(0.85$f_c'A_1$)**, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

R10.17 — Bearing strength

R10.17.1 — This section deals with bearing strength of concrete supports. The permissible bearing stress of **0.85f_c'** is based on tests reported in Reference 10.41. (See also 15.8).

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.12.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Fig. R10.17 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support.

2

CODE

1.2 — Members also shall meet all other requirements of this code to ensure adequate performance at service load levels.

9.1.3 — Design of structures and structural members using the load factor combinations and strength reduction factors of Appendix C shall be permitted. Use of load factor combinations from this chapter in conjunction with strength reduction factors of Appendix C shall not be permitted.

).2 — Required strength

9.2.1 — Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$$U = 1.4(D + \cancel{F}) \qquad (9\text{-}1)$$

$$U = 1.2(D + F + T) + 1.6(L + H) \qquad (9\text{-}2)$$
$$+ 0.5(L_r \text{ or } S \text{ or } R)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \qquad (9\text{-}3)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \qquad (9\text{-}4)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \qquad (9\text{-}5)$$

$$U = 0.9D + 1.6W + 1.6H \qquad (9\text{-}6)$$

$$U = 0.9D + 1.0E + 1.6H \qquad (9\text{-}7)$$

except as follows:

(a) The load factor on L in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load L is greater than 100 lb/ft^2.

COMMENTARY

The changes were made to further unify the design profession on one set of load factors and combinations, and to facilitate the proportioning of concrete building structures that include members of materials other than concrete. When used with the strength reduction factors in 9.3, the designs for gravity loads will be comparable to those obtained using the strength reduction and load factors of the 1999 and earlier codes. For combinations with lateral loads, some designs will be different, but the results of either set of load factors are considered acceptable.

Chapter 9 defines the basic strength and serviceability conditions for proportioning structural concrete members.

The basic requirement for strength design may be expressed as follows:

$$\text{Design Strength} \geq \text{Required Strength}$$

$$\phi\,(\text{Nominal Strength}) \geq U$$

In the strength design procedure, the margin of safety is provided by multiplying the service load by a load factor and the nominal strength by a strength reduction factor.

R9.2 — Required strength

The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the general building code multiplied by appropriate load factors.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to compute moments and shears.

The code gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with **0.9*D*** are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

CODE

(b) Where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in Eq. (9-4) and (9-6).

(c) Where earthquake load E is based on service-level seismic forces, $1.4E$ shall be used in place of $1.0E$ in Eq. (9-5) and (9-7).

(d) The load factor on H shall be set equal to zero in Eq. (9-6) and (9-7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

9.2.2 — If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.3 — Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.2.4 — For a structure in a flood zone, the flood load and load combinations of ASCE 7 shall be used.

9.2.5 — For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

COMMENTARY

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If special circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors U may be appropriate for such members.

The wind load equation in ASCE 7-98[9.1] and IBC 2000[9.2] includes a factor for wind directionality that is equal to 0.85 for buildings. The corresponding load factor for wind in the load combination equations was increased accordingly (1.3/0.85 = 1.53 rounded up to 1.6). The code allows use of the previous wind load factor of 1.3 when the design wind load is obtained from other sources that do not include the wind directionality factor.

Model building codes and design load references have converted earthquake forces to strength level, and reduced the earthquake load factor to 1.0 (ASCE 7-93[9.3]; BOCA/NBC 93[9.4]; SBC 94[9.5]; UBC 97[9.6]; and IBC 2000[9.2]). The code requires use of the previous load factor for earthquake loads, approximately 1.4, when service-level earthquake forces from earlier editions of these references are used.

R9.2.2 — If the live load is applied rapidly, as may be the case for parking structures, loading docks, warehouse floors, elevator shafts, etc., impact effects should be considered. In all equations, substitute (L + impact) for L when impact should be considered.

R9.2.3 — The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

R9.2.4 — Areas subject to flooding are defined by flood hazard maps, usually maintained by local governmental jurisdictions.

R9.2.5 — The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113 percent of the specified prestressing steel yield strength but not more than 96 percent of the nominal ultimate strength of the prestressing steel. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

4

CODE

9.3 — Design strength

9.3.1 — Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in 9.3.2, 9.3.4, and 9.3.5.

9.3.2 — Strength reduction factor ϕ shall be as follows:

9.3.2.1 — Tension-controlled sections as defined in 10.3.4 .... 0.90 (See also 9.3.2.7)

9.3.2.2 — Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement conforming to 10.9.3.. 0.70

(b) Other reinforced members 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as the net tensile strain in the extreme tension steel at nominal strength increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix B is used, for members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero. For

COMMENTARY

R9.3 — Design strength

R9.3.1 — The design strength of a member refers to the nominal strength calculated in accordance with the requirements stipulated in this code multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are (1) to allow for the probability of understrength members due to variations in material strengths and dimensions, (2) to allow for inaccuracies in the design equations, (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered, and (4) to reflect the importance of the member in the structure.[9.7,9.8]

In the 2002 code, the strength reduction factors were adjusted to be compatible with the ASCE 7-98[9.1] load combinations, which were the basis for the required factored load combinations in model building codes at that time. These factors are essentially the same as those published in Appendix C of the 1995 edition, except the factor for flexure/tension controlled limits is increased from 0.80 to 0.90. This change is based on past[9.7] and current reliability analyses,[9.9] statistical study of material properties, as well as the opinion of the committee that the historical performance of concrete structures supports $\phi = 0.90$.

R9.3.2.1 — In applying 9.3.2.1 and 9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

R9.3.2.2 — Before the 2002 edition, the code specified the magnitude of the ϕ-factor for cases of axial load or flexure, or both, in terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Members with spiral reinforcement are assigned a higher ϕ than tied columns since they have greater ductility or toughness.

For sections subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in 10.3.3 and 10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain

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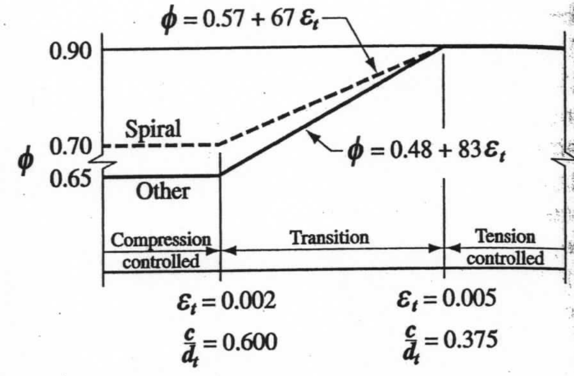

Interpolation on c/d_t: Spiral $\phi = 0.37 + 0.20/(c/d_t)$
Other $\phi = 0.23 + 0.25/(c/d_t)$

Fig. R9.3.2—Variation of ϕ with net tensile ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Fig. R9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.

Since the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 60 steel and to prestressed sections. Fig. R9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b as defined in the 1999 and earlier editions of the code. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 60 reinforcement. For a comparison of these provisions with the 1999 code Section 9.3, see Reference 9.10.

9.3.2.3 — Shear and torsion 0.75

9.3.2.4 — Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models).................................. 0.65

9.3.2.5 — Post-tensioned anchorage zones 0.85

R9.3.2.5 — The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Since 18.13.4.2 limits the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f_{ci}'$, the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f_{ci}' \approx 0.6\lambda f_{ci}'$.

9.3.2.6 — Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models 0.75

R9.3.2.6 — The ϕ factor used in strut-and-tie models is taken equal to the ϕ factor for shear. The value of ϕ for strut-and-tie models is applied to struts, ties, and bearing areas in such models.