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DISTRIBUTION			ABSTRACT		
*	NAME	MAIL ADDR	Load capacities of the vault roof beams, in terms of total LLTV upward dynamic thrust are determined as follows: 533 kips, faulted condition and 455 kips, emergency condition.		
	R. Durand (20)	LB11	DOE/NBM--2011058 DE82 011058 MASTER <div style="border: 1px solid black; padding: 5px; margin: 10px 0;"> <small>DISCLAIMER</small> This book was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness or usefulness of any information, apparatus, product, or process disclosed, or represents that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof. </div> DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED <i>JH</i>		
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The purpose of this investigation is to determine the upward thrust loads, due to LLTV dynamic reactions, which the vault roof beams of the LLTR (Bldg. 059) can sustain.

Figure 1 shows the arrangement of the roof beams arrangement. These are reinforced concrete beams which were poured as an integral part of the vault roof. Details of the reinforcement were taken from the original engineering drawings, reference 1, and are shown on figure 2

Figure 3 shows the arrangement of the supports for the LLTV and the points at which the reactions are transmitted to the vault roof. Obviously, other support arrangements will yield different results. However, with the limitations imposed by considerations such as near symmetrical restraint for the LLTV and maintaining a clearance for removal of the RPT, this arrangement does not permit too much modification, so the results of the investigation should not change significantly.

The load capacity of the roof is taken as that total LLTV reaction to the roof which will stress the critical element (tensile reinforcing) to 90 percent of yield. Loading was also computed for stressing the reinforcing to 75 percent. Load factors were taken as unity for beam dead load and LLTV reaction. The capacities so determined are consistent with the terms "faulted" and "emergency" conditions with the 1971 draft of the ASME Section III, Division 2 Code, reference 5.

Results

The static load capacity of the tensile reinforcing for Beam "A" is the limiting case for the total LLTV reaction. These LLTV loads are 533 kips, faulted, and 455 kips, emergency.

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PLAN OF HIGH BAY FLOOR - *2d*. VAULT ROOF

FIGURE 1

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BEAM A

BEAM B & C

MATL PROPERTIES

$f'_c = 3000$ psi conc ult. comp. strength

$F_y = 40000$ psi, AIS, A305 intermediate gr. reinf

$n = 9$

FIGURE 2

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BEAM A

Top face (Tension) Reinf.

$$A_s = 8.89 \text{ in}^2$$

$$\Sigma_o = 27.9 \text{ in}$$

Bottom face (Compr) Reinf

$$A_s' = 28.08 \text{ in}^2$$

$$\Sigma_o = 79.8 \text{ in}$$

$h = 49.5 \text{ in.}$ except at support where

$$h = 37.5 \text{ in.}$$

$$\text{Ave. width, } b = 27 \text{ in}$$

$$w_{DL} = 1.39 \text{ k/ft}$$

BEAMS B & C

Top face (Tension) Reinf.

$$A_s = 8.89 \text{ in}^2$$

$$\Sigma_o = 27.9 \text{ in}$$

Bottom face (Compr) Reinf

$$A_s' = 17.16 \text{ in}^2$$

$$\Sigma_o = 48.4 \text{ in}$$

$h = 49.5 \text{ in.}$ except at support where

$$h = 37.5 \text{ in}$$

$$\text{Ave. width, } b = 43.25 \text{ in}$$

$$w_{DL} = 2.23 \text{ k/ft}$$

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DETERMINE BENDING CAPACITY OF BEAMS AT YIELD.

1. Assume Tension Reinforcing critical

$$M_{YLD} = \frac{\phi A_s F_{sy}}{12000} \quad (\text{FT-KIPS})$$

2. Check concrete stress & Compression Reinf. stress to verify that these stresses are less than yield

$$f_c = \frac{F_{sy}}{n} \times \frac{k}{1-k} \leq 0.85 f'_c \quad \text{For Concrete}$$

$$\text{and } f'_s = F_{sy} \times \frac{k-d'/d}{1-k} \leq F_{sy} \quad \text{For Compressive Reinforcing}$$

3. So long as f_c & f'_s are less than yield, Ultimate Strength criteria is not appropriate, since this method assume all elements are at a yielded condition.

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Beam Load Investigation

Determine Allowable LLTV Reactions on Beams for specific Load points as envisioned in conceptual design arrangement, Fig. 3

Limiting values of beam capacity are

- 1) Max $M = .9 M_{YLD}$ For Faulted Cond.
- 2) Max $M = .75 M_{YLD}$ For Emergency Cond.

Ref 5, p. CB68

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Investigation of Bending Capacity of Beams (Example 10, p. 11, ref. 2)

GIVEN DATA

b = beam width (Av.) (in)

d = beam depth (in)

A_s = Tension Reinf. Area (in^2)

A'_s = Compressive Reinf. Area (in^2)

$$\eta = g = \frac{E_s}{E_c}$$

d' = Distance from Compr. Conc. face to Centroid of Compressive Reinf

PARAMETERS

$$m = \frac{\eta A_s}{bd} + \frac{(\eta-1) A'_s}{bd} = \eta p + (\eta-1) p'$$

$$g = \frac{\eta A_s}{bd} + \frac{(\eta-1) A'_s}{bd} \times \frac{d'}{d} = \eta p + (\eta-1) p' \left(\frac{d'}{d} \right)$$

DETERMINE DIMENSIONAL COEFFICIENTS

$$k = \sqrt{m^2 + 2g} - m$$

or Table 11

$$z = \frac{1/6 + \frac{(\eta-1) A'_s}{k bd} \times \frac{d'}{kd} \left(1 - \frac{d'}{kd} \right)}{1/2 + \frac{(\eta-1) A'_s}{k bd} \left(1 - \frac{d'}{kd} \right)}$$

or Table 12

$$j = 1 - zk$$

or Table 13

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Beam A Investigation

$$b_{av} = 27"$$

$$d = 49.5 - 2.1 = 47.4"$$

$$A_s = 8.89 \text{ in}^2$$

$$A_s' = 28.08 \text{ in}^2$$

$$d' = 3.6 \text{ in}$$

$$\eta p = \frac{9(8.89)}{27(47.4)} = .063$$

$$(\eta - 1) p' = \frac{8(28.08)}{27(47.4)} = .176$$

$$d'/d = \frac{3.6}{47.4} = .076$$

$$m = .063 + .176 = .239$$

$$g = .063 + .076(.176) = .076$$

$$k = .22 \quad (\text{Table 11})$$

$$1/k \times d'/d = .35$$

$$1/k \times (\eta - 1) p' = .80$$

$$z = .34 \quad (\text{Table 12})$$

$$j = .93 \quad (\text{Table 13})$$

$$M_{YLD} = \frac{.93(47.4)(8.89)40000}{12000} = 1306 \text{ 'K}$$

$$f_c = \frac{40000}{9} \times \frac{.22}{.78} = 1254 \text{ psi} < .85(3000) \quad \text{OK}$$

$$f_s' = 40000 \times \frac{.144}{.78} = 7385 \text{ psi} < F_{sy} \quad \text{OK}$$

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Beams B & C Investigation

$$b = 43.25 \text{ in (Av)}$$

$$d = 49.5 - 2.1 = 47.4''$$

$$A_s = 8.89 \text{ in}^2$$

$$A_s' = 17.16 \text{ in}^2$$

$$d' = 2.1$$

$$\eta p = \frac{9(8.89)}{43.25(47.4)} = .039$$

$$(\eta-1)p' = \frac{8(17.16)}{43.25(47.4)} = .067$$

$$d'/d = \frac{2.1}{47.4} = .044$$

$$\eta = .039 + .067 = .106$$

$$g = .039 + .044(.067) = .042$$

$$k = .20 \quad \text{Table 11}$$

$$1/k \times \frac{d'}{d} = .22$$

$$1/k \times (\eta-1)p' = .335$$

$$z = .29 \quad \text{Table 12}$$

$$j = .94 \quad \text{Table 13}$$

$$M_{YLD} = .94(47.4)(8.89) \frac{40000}{12000} = 1320 \text{ 'K}$$

$$f_c = \frac{40000}{9} \times \frac{.20}{.80} = 1111 \text{ psi} < .85(3000) \quad \text{OK}$$

$$f_s' = 40000 \times \frac{.156}{.80} = 7800 \text{ psi} < F_{sy} \quad \text{OK}$$

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FIGURE 3

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LLTV REACTION LOAD DISTRIBUTION TO VAULT ROOF
TOTAL LLTV = P OR $\frac{1}{2}P$ ON EA SUPPORT BEAM

$$FEM = \frac{.25P}{2(13)^2} \left[(3.25)(9.75)(3.25 + 13) + (3.25)(9.75)(9.75 + 13) \right]$$

$$= \frac{.25P}{2(13)^2} (3.25)(9.75) [3.25 + 13 + 9.75 + 13]$$

$$= .914P$$

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DETERMINE MAX ALLOWABLE LLTV REACTION WITH
BEAM "A" CRITICAL

$$\text{BM D.L.} = 1.39 \text{ K/FT}$$

$$\text{MOMENT CAP. @ YIELD, } M_{YLD} = 1306 \text{ K}$$

$$\text{MAX MOMENT} = M_a$$

$$\text{D.L. } M_a = 21.2(14.5) - \frac{1.39(14.5)^2}{2} = 161.3 \text{ K}$$

FAULTED COND.

$$\text{LLTV } M = .9(1306) + \text{D.L. } M_a = 1337 \text{ K}$$

DETERMINE P, LLTV LOADING (MAX)

$$.173 P(14.5) = 1337 \text{ K}$$

$$\underline{P = 533 \text{ K}}$$

MAX. FAULTED ALLOW LLTV
REACTION

EMERGENCY COND

$$\text{LLTV } M = .75(1306) + \text{DL } M_a = 1141 \text{ K}$$

DETERMINE P, LLTV LOADING (MAX)

$$.173 P(14.5) = 1141 \text{ K}$$

$$\underline{P = 455 \text{ K}}$$

MAX. EMERGENCY ALLOW
LLTV REACTION

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INVESTIGATE BEAMS B & C LOAD DISTRIBUTION

For simplicity $p = .356 P$

$$p_b + p_c = p$$

⑤

Bm C

③

Bm B

Equate deflections @ Intersection of bms B & C (Points a & c)

$$\text{Bm B} \quad \Delta_c \propto \frac{1}{EI} \left[\frac{11^2 \times 19^2}{3 \times 30} p_b \right] + \Delta_{DL}$$

$$\text{Bm C} \quad \Delta_c \propto \frac{1}{EI} \left[\left(\frac{11^2 \times 19^2}{3 \times 30} \right) p_c + \left\{ \frac{11.5 \times 11}{6 \times 30} (30^2 - 11.5^2 - 11^2) p \right\} \right] + \Delta_{DL}$$

$$\Delta_c = \Delta_a$$

$$485.3 p_b = 485.3 p_c + 454.5 p$$

substituting $p_b = p - p_c$ & solving

$$p_c = .032 p = .032 (.356) P = .011 P$$

$$p_b = .968 p = .968 (.356) P = .345 P$$

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BEAM B

$$D.L. M_C = 33.5(11) - \frac{2.23(11)^2}{2} = 233.6^{1k}$$

$$\text{FOR FAULTED COND, } M_U = .9(1320) = 1188^{1k}$$

$$LLTV \text{ MAX } M = 1188 + 233.6 = 11(.218)P$$

$$\text{OR } \underline{P = 523}^{1k} \quad \text{FAULTED}$$

$$\text{FOR EMERGENCY COND, } M_U = .75(1320) = 990$$

$$LLTV \text{ MAX } M = 990 + 233.6 = 11(.218)P$$

$$\underline{P = 512}^{1k} \quad \text{EMERGENCY}$$

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BEAM C

$$D.L. M_c = 33.5(11.5) - \frac{2.23(11.5)^2}{2} = 237.8^{ik}$$

FOR FAULTED COND, $M_0 = 1188^{ik}$

$$LLTV \text{ MAX } M = 1188 + 237.8 = 11.5(.224)P$$

$$\underline{P = 553^k} \quad \text{MAX FAULTED}$$

FOR EMERGENCY COND, $M_0 = 990^{ik}$

$$LLTV \text{ MAX } M = 990 + 237.8 = 11.5(.224)P$$

$$\underline{P = 477^k} \quad \text{MAX EMERGENCY}$$

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1. CHECK BOND AND SHEAR FOR 533^k Faulted Load.

$$V_u = .243(533) - 21.2 = 108^k$$

BOND $\mu_u = \phi(9.5) \frac{\sqrt{f_c'}}{D} = .85(9.5) \frac{\sqrt{3000}}{1.27} = 348 \text{ psi}$
#10 Re bar

$$\text{Calc. Bond} = \frac{V_u}{\sum_o j d}$$

$$\sum_o = 27.9''$$

$$j = .93$$

$$d = 35.4'' \text{ @ Support}$$

$$\mu_{calc} = \frac{108(1000)}{27.9(.93)(35.4)} = 118 \text{ psi} << 348 \text{ OK}$$

SHEAR $V_{uc} = 2\phi\sqrt{f_c'} = 2(.85)\sqrt{3000} = 93 \text{ psi}$

$$\text{calc shear} = \frac{V_u}{b_{eff} d}$$

$$d = 35.4'' \text{ @ Support}$$

$$b_{eff} \cong b_{bar} + 2d$$

$$= 27 + 2(35.4) = 97.8''$$

$$v_{calc} = \frac{108(1000)}{97.8(35.4)} = 31 \text{ psi} < 93 \text{ OK}$$

2. CHECK BEAM TO WALL ANCHORAGE THRU VERT WALL REINF.

Vert wall reinf: #6 @ 11" + #8 @ 11"

Assume eff. Length = $37.5 - 3 = 34.5''$ eff length

$$\text{Bond } \mu_{\#6} = \frac{.85(9.5)\sqrt{3000}}{.75} = 590 \text{ psi for \#6 bars}$$

$$\mu_{\#8} = \frac{.85(9.5)\sqrt{3000}}{1} = 442 \text{ psi for \#8 bars}$$

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DOWEL ANCHORS (CONT'D)

CAPACITY OF EACH PAIR OF VERT BARS (BOND)

$$= \sum \mu_u l \Sigma_o$$

$$\Sigma_o = 2.4" \text{ for } \#6$$

$$= 3.1" \quad \#8$$

$$= 590(33.5)(2.4) + 442(33.5)(3.1) = 93.3^k / \text{pair (BOND)}$$

CAPACITY OF EACH PAIR OF VERT BARS (TENSION)

$$= \sum \phi A_s f_y = .85 \times 40,000 (.79 + .44) = 41.8^k / \text{pr}$$

$$< 93.3^k / \text{pr}$$

TENSION GOVERNS :

$$\text{CAPACITY OF VERT BARS} / F_T = 41.8 \times \frac{12}{11} = 45.6^k / F_T$$

Assume eff width, b_{eff} , same as for shear

$$\text{TOTAL VERT BAR CAPACITY} = \frac{97.8}{12} \times 45.6$$

$$= 372^k > 108^k, V_u$$

OK

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