EE MEMORANDUM B2

Date: January, 2016
To: Office
From: Krishna Haridas
Re: Memo 2 – Engineering Evaluation of Existing Vaults 3 and 4
File: 12582B

Available Information
1. VMH-905 Vault 3 Shop Drawing
2. VMH-906 Vault 3 Shop Drawing
3. VMH-907 Vault 3 Shop Drawing
4. VMH-908 Vault 3 Shop Drawing
5. Design Calculations By Rotondo Precast dated 11-14-1996

Vaults 3 & 4

Review of the shop drawings VMH-905, VMH-906, VMH-907 and VMH-908 for the precast vaults supplied by Rotondo Precast and the design package submitted by Black & Veatch indicates Vaults 3 and 4 are designed to carry up to 5 feet of granular fill plus HS20 truck live load. Top of Vault 3 is at Elev. +13.6 and for Vault 4 is at Elev. +12.4 per data provided by CH2M Hill dated 10-16-2015. The construction of proposed Wills Street requires adding fill to the site resulting in approximately 13.4 feet of normal weight of fill over Vault 3 and 4.6 feet of normal weight of fill over Vault 4. A structural capacity evaluation of Vault 4 was not performed as the proposed loading does not exceed that permitted by the designer of vaults.

Structural Capacity Evaluation of Vault 3

Loads:
- HS-20 Truck Loading per AASHTO LRFD Bridge Design Specifications
- Construction surcharge equivalent to 2 feet of fill
- Weight of the proposed fill

Since this is a review of the vault design performed in 1996, the load factors and code provisions used by the original designers are adopted. The concrete side walls and roof and floor slabs are analyzed as continuous one way slabs ignoring the openings shown on the shop drawings. Wall corner moments are obtained using an equivalent frame analysis assuming fixed wall to slab connections similar to original design.
**Conclusion**

**Vault 3:** MRCE structural evaluation finds that the structural capacity of Vault 3 is inadequate to carry the proposed 13.4 feet of fill and will require structural modifications to handle the load. Structural modifications should follow the latest code provisions.

**Vault 4:** Increased height of fill above Vault 4 (4.6 feet) is within the permissible fill height of 5 feet per original design by Rotondo Precast dated 11-14-1996 and hence Vault 4 is capable of carrying the additional fill without structural modifications.
1. VAULT DRAWINGS
2'-0" x 11'-0" x 7'-0" PRECAST VAULT

ELEVATION
WALL 1 SHOWING LIFTING

PLAN VIEW TOP SLAB
REINFORCING DETAIL

PLAN VIEW BOTTOM SLAB
REINFORCING DETAIL

DETAIL "A"

SECTION A-A
WALL 4

SECTION B-B

CONCRETE MIXTURE
Cements: Ordinary Portland Cements - One Part by Weight of Water - One Part by Weight of Cement
 aggregates - Ordinary aggregates

ABBRIVATIONS

ADDITIONAL

NOTE: Heights shown above are based on a concrete unit weight of 145 lb/ft³.

ROTONDO PRECAST
A DIVISION OF OLCCASTLE PRECAST
MANUFACTURERS OF QUALITY PRECAST CONCRETE PRODUCTS

ALLEN'S WALLS
Baltimore, Maryland
2. VAULT 3 – ROOF SLAB EVALUATION
**Inside Dimensions of Vault**

- Slab Length 11 ft
- Slab Width 7 ft
- Headroom 7 ft

Roof slab thickness: \( r_1 = 10\text{in} \)

Floor slab thickness: \( t_2 = 7.875\text{in} \)

Wall thickness: \( s_1 = 6.875\text{in} \)

**Material Properties**

- Concrete: \( f_c = 5\text{ksi} \)
- Concrete: \( V_{\text{concrete}} = 150\text{pcf} \)
- Concrete: \( f_y = 60\text{ksi} \)

**Soil Properties**

- Soil: \( V_{\text{soil}} = 120\text{pcf} \)

- Soil: \( k_s = 0.33 \)

**Load Factors**

- Live Load: \( LLF = 2.17 \)
- Dead Load: \( DLF = 1.3 \)
- Lateral Earth Pressure: \( EPF = 1.7 \)
- Earth Surcharge: \( ESF = 2.17 \)

**Capacity Reduction Factors**

- Concrete: \( \phi_{cm} = 0.9 \)
- Concrete: \( \phi_{cu} = 0.85 \)

**Codes Referenced**

- ACI 318-89
- ACI 350 R7
- AASHTO LRFD
Roof Slab Loading

Surface area of roof slab  

\[ A_p := \left(11 \text{ ft}^2 + 2 \cdot 5.5 \text{ ft}^2\right) \left(7 \text{ ft}^2 + 2 \cdot 5.5 \text{ ft}^2\right) = 98.94 \text{ ft}^2 \]

Critical depth of fill above the roof slab  

\[ H := 13.4 \text{ ft} \]

Load distribution from the two wheels will overlap at the bottom slab level, so the total live load will be uniformly distributed over the slab area. Impact load is considered. Per AASHTO 2012.4.6.2.10.4

Wheel load  

\[ W_{pl} := 16 \text{kips} \]

Live Load  

\[ w_L := \frac{2 \cdot W_{pl} \cdot LLF}{A_p} = 0.70 \cdot \text{kfsf} \]

Dead Load  

\[ w_D := r_L \cdot V_{cr} \cdot \text{DLF} + H \cdot V_{cr} \cdot \text{ESF} = 3.65 \cdot \text{kfsf} \]

Bending

Ultimate Moment  

\[ M_u := \left(w_L + w_D\right) \frac{(7 \text{ ft})^2}{8} = 26.67 \cdot \text{kips ft} \]

Effective Depth  

\[ d_{eff} := r_L - 2 \text{ in} - 0.4375 \text{ in} = 7.56 \text{ in} \]

Width considered  

\[ b := 1 \text{ ft} \]

Area of steel provided #7 @ 9"  

\[ A_{st} := 0.80 \frac{\text{in}^2}{\text{ft}} \]

\[ a := \frac{A_{st} \cdot 1 \text{ ft} \cdot f_y}{0.85 \cdot f_c \cdot b} \]

\[ a = 0.94 \text{ in} \]

Moment capacity  

\[ M_c := \left[A_{st} \cdot f_y \left(d_{eff} - \frac{a}{2}\right)\right] \cdot p_{im} = 25.53 \frac{\text{kips ft}}{\text{ft}} \]

\[ M_c < M_u \text{ Additional reinforcement required.} \]

Shear  

\[ w_u := w_L + w_D = 0.03 \text{ ksi} \]

\[ V_u := w_u \left(\frac{7 \text{ ft}}{2} - d_{eff}\right) = 12.49 \frac{\text{kips}}{\text{ft}} \]

Shear capacity  

\[ V_c := p_{im} \cdot 2 \sqrt{\frac{f_c}{\text{ksi}}} \cdot 12 \frac{d_{eff}}{\text{in}} \]

\[ V_c := \frac{V_c}{1000} \frac{\text{kips}}{\text{ft}} = 10.91 \frac{\text{kips}}{\text{ft}} \]

\[ V_c < V_u \text{ Additional reinforcement required.} \]
3. VAULT 3 – WALL EVALUATION
**Lateral Earth Pressures:**

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<th>Layer</th>
<th>Elev</th>
<th>H</th>
<th>γ</th>
<th>$k_a$</th>
<th>C</th>
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<th>Water Pressure</th>
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**Active Pressure:**

$$\sigma_a = \gamma H k_a - 2C\sqrt{k_a}$$

**Passive Pressure:**

$$\sigma_p = \gamma H k_p + 2C\sqrt{k_p}$$

**NOTES:**

1. Soil profile and parameters as used in the original design by Rotondo Precast dated 1996-11-14.
2. Sand Properties: $\phi = 30^\circ$, $\gamma = 120$ pcf
Net Lateral Pressures on Vault Wall

Pressures (psf)

Elevation (ft)

3000  2000  1000  0  -1000  -2000  -3000

24.0

19.0

14.0

9.0

4.0

(1075) 610
643
920
(1646) 946

## unfactored

(##) factored
Equivalent frame method is used to evaluate the corner moments in walls. Stiffness values used are as per the design calculations by Rotondo Precast dated 11/14/1996.
Uniform load on the wall due to lateral earth pressure:

$$w = \frac{1.015 + 1.646}{2} = 1.36 \text{ ksf}$$

Fixed end moment:

$$M_F = \frac{wL^2}{12}$$

Area of steel provided:

$$6 \text{ @ } 10\text{"} = 0.5302 \text{ in}^2/\text{ft}$$

Whitney stress block:

$$a = \frac{A_s b_y}{0.85 b_c} = \frac{0.5302 \text{ in}^2 \times 60 \text{ ksf}}{0.85 \times 5 \text{ ksf} \times 12\text{"}}$$

$$= 0.624\"$$

Moment Capacity:

$$M_{hn} = 0.9 \times 0.5302 \times 60 \left(5.5 - \frac{0.624}{2}\right)$$

$$= 12.4 \text{ ksf/ft} > M_u = 11.71 \text{ ksf/ft}$$
Factored loads:

\[ M_u = \frac{W_u l^2}{8} \]

\[ = 1.36 \times 11^2 / 8 = 20.57 \text{ k-ft/lb} \]

\[ d_{ef} = 5.5'' \]

\[ \text{Area of steel} = \# G @ 10'' = 0.5302 \text{ in}^2/\text{lb} \]

Whitney Stress Block, \( a = \frac{A_s l y}{k b^3} = \frac{0.5302 \times 60}{0.85 \times 5 \times 12} = 0.62'' \)

Moment Capacity \( \phi M_n = 0.9 A_s l y \left( \frac{d - a}{2} \right) = 12.4 \text{ k-ft/lb} \)

\[ \phi M_n \leq M_u \]

Shear Force \( V_u = 1.36 \left( \frac{11 - 5.5}{12} \right) = 10.33 \text{ k/lb} \)

Shear Stress \( \tau_u = \frac{V_u}{bd_{ef}} = 0.16 \text{ ksi} \)

Allowable shear stress \( \tau_a = 0.85 \times 2 \sqrt{f'c} \)

\[ \tau_a = 0.12 \text{ ksl} \]

\[ \tau_a \leq \tau_u \]
Factored loads

\[ M_u = \frac{w_u l^2}{8} = \frac{1.36 \times 7^2}{8} = 8.33 \text{ k-ft/ft} \]

\[ c_{efb} = 5.625'' \]

\[ \text{Shear stud} = \# 4 @ 10'' = 0.24 \text{ in}^2/\text{ft} \]

Whitney stress block:
\[ a = \frac{A_{fly}}{0.85 b c_{efb}} = \frac{0.24 \times 60}{0.85 \times 5 \times 12} = 0.28'' \]

Moment capacity:
\[ \phi M_n = 0.9 \times 0.24 \times 60 \left( 5.625 - 0.24 \right) \]
\[ = 6 \text{ k-ft/ft} \]

\[ \phi M_n < M_u \]

Shear force:
\[ V_u = 1.36 \left( \frac{7}{3} - \frac{5.625}{12} \right) = 4.12 \text{ k-lbf} \]

Shear stress:
\[ \tau_u = \frac{V_u}{bd} = 0.06 \text{ ksi} \]

Allowable shear stress:
\[ \tau_a = 0.85 \times 2 \sqrt{f_c} = 0.12 \text{ ksi} \]

\[ \tau_a > \tau_u \]
4. VAULT 3 – FLOOR SLAB EVALUATION
Floor Slab Loading

Surface area of floor slab \( A_p := (11\text{ ft} + 2 \cdot s_t) \cdot (7\text{ ft} + 2 \cdot s_t) = 98.94\text{ ft}^2 \)

Critical depth of fill above the floor slab \( h := 13.4\text{ ft} \)

Load distribution from the two wheels will overlap at the bottom slab level, so the total live load will be uniformly distributed over the slab area. No impact load since more than 8' fill available.

Live Load \( w_L := \frac{2Wh \cdot LLF}{A_p} = 0.70\text{ ksf} \) Per AASHTO 2012 4.6.2.10.4

Dead Load = weight of structure and earth cover

Volume of the structure \( Vol := (11\text{ ft} + 2 \cdot s_t) \cdot (7\text{ ft} + 2 \cdot s_t) \cdot (7\text{ ft} + r_L + f_t) - 11\text{ ft} \cdot 7\text{ ft} \cdot 7\text{ ft} = 300.94\text{ ft}^3 \)

\[ w_D = \frac{(Vol \cdot V_{concrete \cdot DLF} + H \cdot A_p \cdot V_{source \cdot ESF})}{A_p} = 0.03\text{ ksf} \]

Bending

Ultimate Moment \( M_u := \left( w_L - w_D \right) \frac{(7\text{ ft})^2}{8} = 29.30\text{ kip \cdot ft} \)

Effective Depth \( d_{eff} := f_t - 1\text{ in} - .375\text{ in} = 6.50\text{ in} \)

Width considered \( b := 1\text{ ft} \)

Area of steel provided \( A_{st} := 0.53\text{ in}^2 \)

\[ a := \frac{A_{st} \cdot 1\text{ ft} \cdot f_y}{0.85 \cdot f_c \cdot b} \quad a = 0.94\text{ in} \]

Moment capacity \( M_c := \left[ A_{st} \cdot f_y \left( d_{eff} - \frac{a}{2} \right) \right] \cdot p_{fm} = 14.76\text{ kip \cdot ft} \)

\( M_c < M_u \) Additional reinforcement required.

Shear

\( w_u := w_L + w_D = 0.03\text{ ksf} \)

\( V_u := w_u \left( \frac{7\text{ ft}}{2} - d_{eff} \right) = 14.15\text{ kip} \)

Shear capacity \( V_c := p_{fm} \cdot 2 \cdot \sqrt{\frac{f_y}{psf} \cdot d_{eff} \cdot 12 \text{ in}} \)

\[ V_c := \frac{V_c}{1000} \text{ kip} = 9.38\text{ kip} \quad V_c < V_u \) Additional reinforcement required.