Mr. William Neal, P.E.
Technological Specialist
DTE Energy
One Energy Plaza
Detroit, MI 48226

Subject: Safety Factor Assessment
Monroe Power Plant Ash Basin Facility
Monroe, MI

Dear Mr. Neal:

This letter report presents Geosyntec Consultants’ (Geosyntec’s) safety factor assessment for DTE Electric Company’s (DTE’s) Monroe Power Plant Ash Basin (Ash Basin). The safety factor assessment is required under the United States Environmental Protection Agency (USEPA) Coal Combustion Residual (CCR) Rule (CCR Rule) published on 17 April 2015 (40 CFR Parts 257 and 261). Under the CCR Rule, the Ash Basin is an “existing surface impoundment” and must meet safety factor requirements per §257.73(e)\(^1\) of the CCR Rule.

This letter report presents an executive summary followed by details of the analysis.

EXECUTIVE SUMMARY

A slope stability safety factor assessment was completed in accordance with §257.73(e) of the CCR Rule. The CCR Rule requires that surface impoundments have minimum safety factors for various loading conditions.

The results of the analyses indicate the Ash Basin meets the safety factor requirements per §257.73(e) with the maximum water level operated at a maximum elevation\(^2\) of approximately 609 ft.

\(^1\) §257.73(e) – Periodic Safety Factor Assessments.
\(^2\) Elevations are in NGVD29 datum.
SAFETY FACTOR ASSESSMENT

Requirements of the CCR Rules

Slope stability analyses were conducted to assess whether the Ash Basin meets the safety factor (also referred to as “factor of safety”) requirements of §257.73(e)(1) of the CCR Rule. §257.73(e)(1) requires that:

(i) “The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.

(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.

(iii) The calculated seismic factor of safety must equal or exceed 1.00

(iv) For embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.”

Summary of Method and Analyses

Analyses were conducted to calculate factors of safety (FS) for loading conditions described in §257.73(e)(1)(i) through §257.73(e)(1)(iii) of the CCR Rule. Analysis for liquefaction FS was not conducted per §257.73(e)(1)(iv) of the CCR Rule because the embankment is not considered to be susceptible to liquefaction because of its stiff clayey nature. More information on the liquefaction potential of the Monroe Ash Basin embankment is provided in a subsequent section of this report.

The FS values were calculated with limit equilibrium methods using the computer software program SLIDE 6.0© (by Rocscience), a two-dimensional (2D) slope stability computer program. The FS for potential slip surfaces were evaluated using Spencer’s Method (Spencer, 1967).

Monroe Ash Basin Embankment and Subsurface Conditions

The embankment was constructed using on-site soils that were excavated within the footprint of the Ash Basin. The embankment material is characterized as stiff to very stiff, consisting of lean clay with some sand and a trace of gravel. The embankment was constructed using standard engineering and construction methods including compaction of each lift to a specified minimum compaction level based on maximum dry density and optimum moisture content of the soil material. The embankment material is considered to be relatively uniform
based on the results of a number of field investigations and laboratory testing programs conducted since construction was completed. The embankment has a maximum height of approximately 46 ft and side slopes ranging from 2 horizontal to 1 vertical (2H:1V) to 2.5H:1V.

The subsurface soil conditions at the site (below the embankment) consist of an approximately 30- to 50-ft thick stiff to hard silty clay layer with trace to some sand and gravel that generally gets progressively stiffer with depth. For the analysis, this soil is called “natural soil”. The bedrock below this soil unit is characterized primarily as dolomite with occasional interbedded shale.

There are two phreatic surfaces at the site below the embankment: (i) the upper phreatic surface was observed in soil units at depths ranging from 10 to 40 ft below natural ground, and (ii) the bedrock phreatic surface which is at or above the ground surface (artesian condition).

**Cross Sections Selected for Analyses**

Analyses were conducted on four cross sections that were deemed potentially “critical” based on embankment height and steepness of the outer slopes. The cross section at Station 58+75 was analyzed for the northern part of the embankment, the cross section at Station 75+50 was analyzed for the western part of the embankment, the cross section at Station 133+00 was analyzed for the southern part of the embankment, and the cross section at Station 164+00 was analyzed for the eastern part of the embankment. Cross section locations are provided on Figure 1.

**Engineering Parameters**

Shear strength parameters of the embankment and the native soil were evaluated using consolidated-undrained triaxial compression (CU) test (ASTM D 4767) results. Twenty-three CU tests were performed on soil samples obtained from the embankment. Twenty were sampled utilizing thin-walled (Shelby) tubes (ASTM D 1587) and the remaining three were reconstituted using compaction methods in ASTM D 1557 (modified Proctor). Eight CU tests were performed on native soil samples obtained from Shelby tube samples. Sample locations are provided on Figure 1.

Geosyntec developed effective shear strength parameters (i.e., soil friction angle $\phi'$ and soil cohesion intercept $c'$) using the CU test results. Geosyntec used the maximum obliquity approach to define soil failure. With this approach, shear strength parameters are evaluated
for the stress condition corresponding to the maximum ratio of major principal effective stress ($\sigma'_1$) to minor principal effective stress ($\sigma'_3$).

The shear strength versus the mean effective stress ($q$ vs. $p'$) from the maximum obliquity approach was plotted for the embankment and the native soil on Figures 2 and 3, respectively. Geosyntec selected the effective friction angle and effective cohesion intercept as computed from the slope and intercept of the best fit linear relationship of the data. The selected effective friction angles are 34$^\circ$ and 37$^\circ$ for the embankment and native soil, respectively. The selected effective cohesion intercept values are 165 psf and 90 psf for the embankment and native soil, respectively.\(^3\)

Unit weights used in the analyses are based on the samples collected as part of various field investigation studies that were conducted since 2009.

**Water Level in the Ash Basin and Phreatic Surfaces Used for Analyses**

The water level in the Ash Basin has been between approximately 607 ft and 611 ft since the Ash Basin started operating. Based on the operation records, the water level has been mostly 610.5 ft and 611 ft since the beginning of 2011. In 2015, DTE lowered the water level to a maximum elevation of 609 ft and will maintain that maximum elevation for the remainder of the operating life of the Ash Basin, which is estimated to be in 2023.

Five vibrating wire piezometers were installed in the embankment at Station 133+00 in August 2011 to obtain pore pressure information to be used to estimate the existing phreatic surface for a global stability assessment of the Ash Basin (Geosyntec, 2012a). The piezometers were installed at five different locations within a transverse section of the embankment (see Figure 4). The pore pressures obtained from the piezometer readings along with the Ash Basin water level are presented as piezometric elevations on Figure 5.

Considering that the Ash Basin has been holding sluice water for the last approximately 40 years and the measured piezometric elevations have been near constant for the last approximately two and a half years of measurement history, it is reasonable to assume that the phreatic surface in the embankment has reached a steady state condition. Therefore, it is reasonable to assume that piezometric levels will remain approximately at current values or decrease during the remaining operating life of the Ash Basin as a result of the decrease in

\(^3\) For the seismic loading condition, drained strength parameters were selected for the analysis because undrained shear strengths yielded higher FS values.
Ash Basin operating water level. Consequently, the existing phreatic surface as measured in piezometers is representative for the safety factor assessment of the long-term condition.

The Ash Basin has received a “Significant Hazard Potential” classification per §257.73(a)(2). Based on this classification, the water level for the “maximum surcharge pool loading condition” must be estimated based on the 1,000-yr flood event. However, the analysis was conducted based on the more conservative probable maximum flood (PMF).

For the analysis considering the maximum storage pool loading condition (per §257.73(e)(1)(i)) and seismic condition (per §257.73(e)(1)(iii)), the phreatic surface was established by linearly connecting the piezometric elevations between piezometers, and upstream and downstream boundary conditions. The phreatic surface was first established for Station 133+00 where piezometers are located, and then applied to other cross sections. Because the embankment geometry is similar and the embankment material is relatively homogenous over the entire embankment, it is reasonable (for this type of evaluation) to assume hydraulic characteristics and phreatic surfaces are similar along the length of the embankment.

For the analysis considering the maximum surcharge pool loading condition (per §257.73(e)(1)(ii)), the same phreatic surface as for the maximum storage pool loading condition was used, except that the water level in the Ash Basin was raised to elevation 612.3 ft as a result of the Probable Maximum Flood. It is assumed that the increase in water level from 609 ft to 612.3 ft will not create a considerable change in the phreatic surface within the embankment because the water level will return to elevation 609 ft in approximately six days. Because of the low hydraulic conductivity of the embankment materials, pore pressure changes that could affect the calculated stability of the embankment would not be expected to occur in six days.

Artesian conditions in the bedrock exhibits approximately 35 ft of pressure head (Geosyntec, 2012b). The effect of pressure head was assessed by analyzing potential slip surfaces that extend 30 to 50 ft below ground level. The analysis results indicated that these potential slip surfaces exhibit higher FS values than for those reported subsequently. Therefore, artesian pressures were not considered in the analyses.

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4 A separate letter is provided for the hazard potential classification for the Monroe Ash Basin, which is considered to be “Significant Hazard”.
5 Provided as part of a separate letter for the Hydraulic Capacity Assessment.
6 Provided as part of a separate letter for the Hydraulic Capacity Assessment.
Seismic Coefficient for Analysis

The peak horizontal acceleration was selected based on the maps published by USGS (2010). A peak horizontal acceleration at the hard rock (with a 2% probability of exceedance in 50 years) of 0.04g was selected from the map. For analysis, a seismic coefficient of 0.04 was used. The use of a seismic coefficient for slope stability analysis based on the peak horizontal acceleration (without any reduction) is conservative.

Liquefaction Potential of the Monroe Ash Basin Embankment

The Ash Basin embankment was constructed using the onsite clayey native soil. These soils may be susceptible to strength loss during a seismic event. A method proposed by Bray and Sancio (2006) uses the results of laboratory investigations on silts and clays to define a range of soil index parameters for which a silt or clay may be susceptible to strength loss. The results provided on Figure 6 indicate that the Monroe Ash Basin embankment is not susceptible to strength loss, and therefore not susceptible to liquefaction. Results provided on Figure 6 are from index soil data (see Geosyntec, 2012b).

Analysis Results and Conclusion

The analysis results are summarized in Table 1 and provided on Figures 8 through 20.
Table 1. Analysis Summary.

<table>
<thead>
<tr>
<th>Station #</th>
<th>Station 75+50</th>
<th>Station 75+50*</th>
<th>Station 133+00</th>
<th>Station 164+00</th>
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<td>Maximum Surcharge Pool Loading Condition Per §257.73(e)(1)(ii)</td>
<td>Seismic Loading Condition Per §257.73(e)(1)(iii)</td>
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</tr>
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<td>SF = 1.50</td>
<td>SF = 1.40</td>
<td>SF = 1.00</td>
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* Maximum storage pool loading condition for Station 75+50 was analyzed considering Navarre Drain was flowing both full and dry.

Based on the results of slope stability analyses, the Ash Basin meets the safety factor assessment required per §257.73(e) of the CCR Rule.

CONCLUSIONS AND RECOMMENDATIONS

The results indicate that the Ash Basin embankment meet the FS criteria of §257.73(e).

The maximum storage pool operating level is to be maintained at or below elevation 609 ft as this is a key factor in the analysis. If this water level cannot be maintained, or there are structural/geometrical changes to the embankment, analyses need to be conducted to assess compliance with the CCR Rule. However, no structural changes to the embankment and discharge structure are required at this time to meet FS criteria.

QUALIFICATIONS OF LICENSED PROFESSIONAL ENGINEER

John Seymour is a qualified licensed professional engineer with over 30 years of experience in civil and geotechnical engineering associated with earthen structures and dams.
CERTIFICATION

I, John Seymour, am a qualified licensed professional engineer in Michigan have evaluated the Ash Basin and hereby certify that the results of the safety factor assessment meet the requirements of 40 CFR 257.73(e).

Certified by:

[Signature]

John Seymour, P.E.
Michigan License Number 620103356
Senior Principal

Date 10/7/2016

Attachments: Figures 1 through 19

Copies to: Mark Green (DTE)
REFERENCES


FIGURES
Figure Shear Strength Envelope and Parameters for Ash Basin Embankment

Monroe Ash Basin Safety Factor Assessment
September, 2016

Shear Strength Envelope and Parameters for Ash Basin Embankment

\[ \phi' = 34^\circ \quad c' = 165 \text{ psf} \]

\[ \tan \psi = \sin \phi' \]

\[ d = c' \times \cos \phi' \]

where:

\[ p' = \left( \sigma'_1 + \sigma'_3 \right) / 2 \]
\[ q = \left( \sigma'_1 - \sigma'_3 \right) / 2 \]

\( \sigma'_1 \): Major principal effective stress

\( \sigma'_3 \): Minor principal effective stress

\( \psi \)
where:

\[ p' = \frac{\sigma'_1 + \sigma'_3}{2} \]

\[ q = \frac{\sigma'_1 - \sigma'_3}{2} \]

\[ \phi' = 37^\circ \quad c' = 90 \text{ psf} \]

\[ \tan \psi = \sin \phi' \]

\[ d = c' \times \cos \phi' \]

\[ \sigma'_1 \quad \text{Major principal effective stress} \]

\[ \sigma'_3 \quad \text{Minor principal effective stress} \]
Piezometer Locations at Station 133+00
Figure
Piezometric Elevation at Piezometer Locations at Station 133+00

Monroe Ash Basin Safety Factor Assessment September, 2016

Notes:
- Piezometric elevations at Piezometer (Piez.) 1 & 2 locations are not reported beyond October 29, 2013 due to likely piezometer malfunctions.
Figure 6

Evaluation of Liquefaction Potential of Embankment

PI = Plasticity Index
LL = Liquid Limit
$w_c$ = water content
<table>
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<tr>
<th>Material Name</th>
<th>Color</th>
<th>Unit Weight (lbs/ft³)</th>
<th>Strength Type</th>
<th>Cohesion (psf)</th>
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<td>Embankment</td>
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**Monroe Ash Basin Safety Factor Assessment**

**Analysis Description**
- **Project**: Monroe Ash Basin Safety Factor Assessment
- **Analysis By**: YK
- **Date**: 6/15/2015, 1:34:40 PM
- **Scale**: 1:300
- **File Name**: station58_75_static.slim

**Long Term Stability**

**Client**: Geosyntec Consultants

**Analysis**

- **Elev. 609 ft**
- **Embankment**
- **Native Soil**
### Material Properties

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<th>Strength Type</th>
<th>Cohesion (psf)</th>
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**Analysis Description**

Maximum Surcharge Pool Loading

**Client**

Geosyntec Consultants

**Analysis By**

YK

**Date**

6/15/2015, 1:34:40 PM

**Scale**

1:300

**File Name**

station58_75_static_MSPL.slim

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**FIGURE 8**
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Analysis Description

Seismic Loading

Analysis By: YK
Date: 6/15/2015, 1:34:40 PM
Scale: 1:300

Client: Geosyntec Consultants
File Name: station58_75_static_seismic.slim

FIGURE 9
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**Elev. 612.3 ft**

**Embankment**

**Native Soil**

---

**Monroe Ash Basin Safety Factor Assessment**

**Analysis Description**: Maximum Surcharge Pool Loading

**Analysis By**: YK

**Client**: Geosyntec Consultants

**Date**: 6/17/2015, 11:38:55 AM

**Scale**: 1:300

**File Name**: station75_50_static_MSPL.slim

**FIGURE 12**
### Material Properties

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- **Embankment**: Mohr-Coulomb, Cohesion = 165 psf, Phi = 34 deg
- **Native Soil**: Mohr-Coulomb, Cohesion = 90 psf, Phi = 37 deg

---

**Figure 13**

- **Monroe Ash Basin Safety Factor Assessment**
- **Seismic Loading**
- Analysis By: YK
- Date: 6/17/2015, 11:38:55 AM
- Scale: 1:300
- File Name: station75_50_static_seismic.slim
- Client: Geosyntec Consultants

---

**Analysis**

1. **Embankment**: Mohr-Coulomb, Cohesion = 165 psf, Phi = 34 deg
2. **Native Soil**: Mohr-Coulomb, Cohesion = 90 psf, Phi = 37 deg

---

**Seismic Loading**

- **Embankment**: Mohr-Coulomb
- **Native Soil**: Mohr-Coulomb

---

**Geosyntec Consultants**

- **Engineers**
- **Scientists**
- **Experts**

---

**FIGURE 13**
### Material Properties

<table>
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### Analysis Description

**Project:** Monroe Ash Basin Safety Factor Assessment

**Analysis Description:** Long-term Stability

**Analysis By:** YK

**Client:** Geosyntec Consultants

**Date:** 6/12/2015, 1:25:17 PM

**Scale:** 1:400

**File Name:** Station133_static.slem

**FIGURE 14**
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Monroe Ash Basin Safety Factor Assessment

Seismic Loading

Analysis Description: Seismic Loading

Analysis By: YK

Date: 6/12/2015, 1:25:17 PM

Scale: 1:400

File Name: Station133_static_seismic.slim

Project: Monroe Ash Basin Safety Factor Assessment

Client: Geosyntec Consultants

FIGURE 16
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Monroe Ash Basin Safety Factor Assessment

Analysis Description: Maximum Surcharge Pool Loading

Analysis By: YK
Client: Geosyntec Consultants
Date: 6/17/2015, 3:20:58 PM
Scale: 1:300
File Name: station 164_static_MSPL.slim

FIGURE 18
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Elev. 609 ft

**Analysis Description**

Seismic Loading

**Analysis By**

YK

**Client**

Geosyntec Consultants

**Date**

6/17/2015, 3:20:58 PM

**Scale**

1:300

**File Name**

station 164_seismic.slim

**FIGURE 19**